WATER TREATMENT ANALYSIS GUIDE

WILKE MORRISON: MRRSAR010

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Author signature	Signed by candidate
Student Name	Wilke Morrison
Student Number	MRRSAR010
Title	Mr

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STUDENT NAME: WILKE MORRISON
STUDENT NUMBER: MRRSAR010
2/22 Randolph Avenue
Parkside, 5063
Adelaide, Australia
T +61 427 101 576
E wilke.morrison@gmail.com

SYNOPSIS

The treatment of water up to potable/drinkable standards is a complex process, with many variables and parameters impacting on each other. In South Africa drinking water delivered to consumers must meet the requirements as recorded in the South African National Standards (SANS).

Today, more so than ever, there are a number water sources that can be exploited and treated to provide safe drinking water, namely; surface water (dams and rivers), sea water, ground water and treated wastewater. The focus of this dissertation is on surface water; however, reference is made in the first sections with regards to sea water and ground water.

The first step in designing a treatment process begins with analysis of the raw water source. Unfortunately, there is not a one size fits all approach and it is left up to the process engineer to find the correct method of investigation. This can be a daunting task, especially if lacking in experience and available information. The first part of this dissertation focusses on just that. It prescribes the method of sampling and aims to provide the reader with context on when to and what to test for. It goes further to suggest how the results may influence the process design and how certain contaminants can be removed. It also draws the attention to the sampling timeframe required, to obtain representative information, encompassing fluctuations in water quality.

The second part of this dissertation describe the methods for designing a conventional water treatment system, comprising; aeration, coagulation, flocculation, dissolved air floatation, sedimentation, filtration and disinfection. It also comments on the water quality that warrants certain process steps to assist the process engineer in choosing the correct configuration. For most steps the design approach of two or more technologies are presented. This allows the process engineer to consider which technology best suits the application at hand. The design procedures are programmed into an, excel based, software model, which permits quick and easy design. A brief description of how the software model can be used is also covered. The results given by the software model is validated through a set of examples, appended to this document.

Ultimately it is concluded that although this dissertation provides a guide for designing a treatment process it is not an encompassing tool that considers all the intricacies involved. That is, there are too many factors involved and considerations required, and cannot all be captured in one dissertation such as this. As such, it is finally recommended that any design attempts should be conducted by a suitably qualified and experienced process engineer that may use this dissertation to augment their design development.

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LIST OF ABBREVIATIONS AND SYMBOLS

AC	Activated carbon
Atm	Atmosphere
DAF	Dissolved air flotation
DAFF	Dissolved air flotation filtration
DBPs	Disinfectant by-products
DO	Dissolved oxygen
DOC	Dissolved organic carbon
DWA	Department of Water Affairs
E.coli	Escherichia coli
EOM	Extracellular organic matter
FAC	Free available chlorine
GAC	Granular activated carbon
HFC	Horizontal flow clarifier
HMS	Hydrolyzing metal salts
HPC	Heterotrophic plate count
IC	Inorganic carbon
kgO_2/h	Kilogram oxygen per hour
kg/m^3	Kilogram per cubic meter
kPa	Kilopascal
kW	Kilowatts
log	Logarithmic
m	Meter
m^3	Cubic meter
m^2	Square meter
m^3Air/h	Cubic meter of air per hour
mbar	Millibar
MEng	Master's of engineering
meq/l	Milliequivalents per litre
mg C/l	Milligram carbon per litre
mg/l	Milligram per litre
mg/meq	Milligram per milliequivalents
mg.min/l	Milligram minutes per litre
mgO_2/l	Milligram oxygen per litre
m/h	Meter per hour
min	minutes
ml	millilitre

mm	Millimetre
m^3/mh	Cubic meter per meter hour
$m^3/m.s$	Cubic meters per meter seconds
m^3O_2/h	Cubic meter of oxygen per hour
Mol	Moles
m^3/s	Cubic meter per second
m/s	Meter per second
m/s^2	Meters per second squared
mS/m	Millisiemens per meter
m^2/s	Square meter per second
MW	Molecular weight
nm	Nanometer
No	Number
NOM	Natural organic material
Ns/m^2	Newton seconds per square meter
NTU	Nephelometric turbidity units
Pa	Pascal
PAC	Powdered activated carbon
pH	Potential hydrogen
PHMS	Prehydrolyzed metal salts
POC	Particulate organic carbon
Pt-Co	Platinum cobalt
RO	Reverse osmosis
s^{-1}	Per second
s	Second
SANS	South African National Standards
SDI	Silt density index
SG	Specific gravity
SSFs	Slow sand filters
SUVA	Specific ultraviolet absorbance
SWTR	Surface Water Treatment Rule
TDS	Total dissolved solids
THM	Trihalomethanes
THMFP	Trihalomethanes formation potential
TOC	Total organic carbon
TSS	Total suspended solids
TWL	Top water level
UF	Ultrafiltration

US EPA	United States Environmental Protection Agency
UV	Ultraviolet
^{o}	Degrees
^{o}C	Degree Celsius
µg/l	Microgram per litre
µm	Micron
µS/cm	Microsiemens per centimetre
0/00	Parts salt per thousand parts of seawater
VBA	Visual Basic Application

1 INTRODUCTION

This coursework master's dissertation (60 credits) has been developed as part of the author's Master's of Engineering Degree (MEng) in Water Quality Engineering at the University of Cape Town (UCT).

The author currently works in the field of water and wastewater treatment, as a consulting engineer, with eight (8) year's relevant experience. The latter four (4) years have been mainly focussed on potable water treatment and rendering it safe for human consumption.

During this time the author received some exposure to the chemical- and physical water treatment processes, however, it was felt that in-depth knowledge on the matter was lacking. Therefore, the focus of this dissertation pertains to potable water treatment only and its emphasis is on the following aspects:

- Raw water quality: What should one test for in the raw water, when and why should you test for it and how can it be removed or used to inform the process design.
- Process design: A user friendly excel based software model has been developed by the author, that enables the user to perform a process design that should be able to adequately treat most surface waters (from rivers and dams).

1.1 Water Treatment Challenges

Water is a precious and limited resource that plays a vital role in sustaining life on earth. Also, 60% of an adult human body consists of water and if not consumed in sufficient amounts, can lead to dehydration, and ultimately result in fatality.

However, water does not only fulfil a critical survival need. It is also used in many other applications. For example, in the municipal sphere water is used for various other purposes, such as bathing, brushing of teeth, washing of clothes, transporting excrement (in the form of wastewater), preparing food, watering of gardens, to name but a few. In the industrial sector water can be used for cooling, food and drink production, washing etc. The list goes on and on.

In most cases water in its natural form is not suitable for use in any of the above-mentioned applications and must be treated first before it is considered appropriate and safe for distribution to consumers. As such, the goal of water treatment is to take water from a raw water source, this can include; the ocean, a wastewater treatment works (in the case of re-use), aquifers, dams or rivers, and to treat the water so that it complies with local regulations and standards. In South Africa drinking/potable water must comply with SANS 241-1 (South African National Standards, 2015:1-14), and the maximum allowable limit of each determinant is listed in Tables 1 and 2 under Section 4 thereof.

There are many water treatment process configurations available and there is a vast selection of technologies offered for each process step. It is often difficult to establish the right process configuration based on the physical and chemical characteristics of the water. The next challenge is to choose the right technology for each process step. Some technologies are very specific to the type of water they can treat, and it is not one size fits all. Further, other technologies are sensitive to conditions such as flow variations through the water treatment works.

1.2 Application

This dissertation has been developed to provide a tool that could guide and assist the design engineer in making some critical process decisions.

Establishing the parameters to test for in the water is the first challenge. When designing a treatment process, it is mostly done with the aim of reducing the concentration of some specific determinants from the raw to the treated water. The list of determinants and the allowable concentrations are typically governed by some standard or regulation and often design engineers limit the testing regime to these determinants. Unfortunately, in most cases this does not provide sufficient information to inform an encompassing process design. On the other hand, although more information leads to better decisionmaking and a more soundly based design it is not considered practical to test for every possible determinant.

Section 2 of this dissertation provides a list of determinants and describes when and why the designer should consider testing for them. Possible technologies that can be used to treat/remove these determinants are also suggested.

In Section 3 the process design software model is discussed. This model includes all the typical process steps, normally, required for treating surface water. Two (2) different technologies have been programmed into the model for some process steps. Where applicable, the designer can extract the information and use his/her costing model to compare the two (2) technologies from a financial point of view.

2 Water Quality Assessment

The design of any water treatment process starts off with establishing the quality of the raw water. It is also important to consider the risk of possible future contaminants, which may result from natural causes or human activities, such as land development in and around the catchment area.

Apart from and preferably before implementing a dedicated sampling and testing programme, the design engineer should consider what information may already be available, such as:

- Water quality data from an appropriate governing body. For example, in South Africa the Department of Water Affairs (DWA) has various sampling stations at water sources around the country. This information is often not very comprehensive; however, it may provide long-term indications on some determinants.
- Operational data from an existing water treatment works, abstracting water from the same source.

The data obtained from these sources may inform which contaminants to include or exclude from the comprehensive monitoring programme.

Water quality data should ideally be sampled over an extended period (a couple of years) so that any time related variation in the water quality can be established and more specifically, so that the impact of seasonal variation can be identified. It is appreciated that, in the context of South Africa, such information is not always available, and that the implementation of a water treatment process can’t be postponed for several years to allow for water quality testing. However, it is recommended that a minimum sampling and testing period of six (6) months, preferably overlapping summer and winter, be employed to at least get some indication of the impact of seasonal variation. The recommended method for sampling is shown in Table 1.

Table 1: Recommended sampling method

Test Description	Volume of Sample	Sampling Recommendations
Physical, aesthetic, operational, chemical determinants and algae testing	Minimum 5 litres	1. Make sure the container is cleaned thoroughly. 2. Do not touch inside of or around the edges of the entrance/bottle neck of the container. 3. Make sure the sample is taken in a location where the sediments are undisturbed. 4. Only open the container at depth (at least 0.1 m) below the water surface to avoid air entrapment.
Microbiological testing	Minimum 10 litres	
Treatability test	Minimum 5 litres	

The sections below investigate and discuss the different contaminants that can form part of a water quality sampling and testing programme. The aim of these sections is to provide the reader with a point of reference from which to make decisions in establishing a comprehensive monitoring programme. A sheet ("WQ Data") has been setup in the software model where the test results can be captured.

Acidity and alkalinity

Alkalinity refers to water's capability to neutralize an acid or absorb hydrogen ions without a significant change in pH (Goswami, 2015:Section 305.22). It is the area under the acid added pH titration curve between two defined pH values, which is the same as the integral of the buffer capacity curve between two pH values. In other words when an acid is added to a well buffered water, the alkalinity concentration is high and the concentration of free H^+ ions, and consequently the pH, will remain relatively unchanged because the added acid is absorbed by the weak acid/base species present. Alkalinity in municipal/potable waters is mostly provided by carbonate (CO_3^{2-}) and bicarbonate (HCO_3^-) ions of the dissolved inorganic carbon (IC) system, as well as hydroxyl (OH^-) ions and traces of nitrate (NO_3^-) (Goswani, 2015:Section 305.22). Acidity is the opposite of alkalinity and is water's ability to neutralise bases due to the presence of the IC system's carbonic acid (H_2CO_3 - dissolved CO_2) and HCO_3^-.

Typically, the alkalinity of water is measured by titrating from the sample pH to a pH around 4.5 (Loewenthal and Marais, 1976). However, when the pH is lower than 4.5 it becomes difficult to accurately measure and it becomes more appropriate to measure the acidity by titrating with a base, rather than an acid.

High alkalinity waters can be neutralised by adding an acid, e.g. sulphuric acid (H_2SO_4). Acidic waters can be stabilised via various stabilisation processes, most notably by adding hydrated lime ($Ca(OH)_2$) or bringing it into contact with limestone ($CaCO_3$).

Aluminium

According to SANS 241-1 (South African National Standards, 2015:7-9), the maximum allowable limit for aluminium (Al) is 300 µg/l. Further, it has been suggested by some that Al can cause dementia associated with kidney dialysis as well as Alzheimer's disease (Post, Atherholt and Cohn, 2011:Section 2.6.1).

It is important to determine what fraction of Al is present in dissolved form. This can be done by passing water through a 0.45 µm filter and then measuring the resulting Al concentration. The dissolved Al represents the fraction that will be more difficult to remove through conventional treatment processes. However, at the right pH and temperature aluminium hydroxide ($Al(OH)_3$) is formed that can precipitate out of solution (Letterman, Amirtharajah and O'Melia, 1999:6.18). The remaining fraction of Al that is suspended in water can be removed by conventional treatment process (coagulation, flocculation, sedimentation and filtration).

Ammonia (free and saline)

Depending on the water pH, ammonia (NH_3) or ammonium (NH_4) will be the dominant species and one easily converts to the other as pH fluctuates (Seneye, 2018). It is therefore important to measure for both to get an idea of the maximum possible NH_3 concentration.

NH_3 is toxic and according to SANS 241-1 (South African National Standards, 2015:7-9) 1.5 mg/l is the maximum allowable limit. Further, NH_3 consumes chlorine to form chloramines. Chloramines is often applied to waters that are prone to taste and odour issues (Cohn, Cox and Berger, 1999:2.56). In addition, it worth noting that NH_4 negatively impacts on biological filtration (Mouchet, 1992:166)

NH_3 can be removed through reverse osmosis (RO) filtration, or chemical oxidation through breakpoint chlorination (Singer and Reckhow, 1999:12.15).

Elements contributing to salinity

The elements listed in Table 2 all contribute to salinity when dissolved in water. Typically, these elements can be removed by conventional treatment processes when in suspended form. However, they may trigger the need for specialised treatment when dissolved. As such, and in the interest of keeping sampling cost down, it is recommended that these elements only be tested for when the overall salinity (i.e. the total dissolved solids) is above the SANS 241-1 (South African National Standards, 2015:7-9) limit of 1200 mg/l. Further, the overall salinity concentration can be used to calculate the oxygen saturation concentration (refer Section 3.1.3) (Benson and Krause, cited in Rounds, 2011:A4)

Table 2: Elements contributing to salinity

Element	Risk	Limit set bySANS 241-1 (South African National Standards, 2015:7-9)	Treatment process
Antimony (Sb)	Chronic health (South African National Standards 241-1, 2015:7-9)	1.5 µg/l	RO filtration (Logsdon, Hess and Horsley, 1999:3.6-3.7)
Barium (Ba)		700 µg/l	RO filtration and ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7)
Bromine (Br)	Chronic and acute health risks Bromide (Br-) consumes free chlorine and leads to formation of disinfection by-products (DBPs) (Singer and Reckhow, 1999:12.16) Interferes with and destabilises chloramination (Trofe cited in Haas, 1999:14:12) Affects ozonation to form brominated DBPs (Singer and Reckhow, 1999:12.15)	N/A	RO filtration, however, in seawater it reacts with free chlorine to form hypobromous acid (HOBr) that may pass through RO filters (Post, Atherholt and Cohn, 2011:Section 2.8.4)
Cadmium (Cd)	Chronic and acute health risks (South African National Standards, 2015:7-9)	3 µg/l	RO filtration and ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7) Will get some removal with granular activated carbon (GAC) at higher pH (Huang, cited in Summers, Knappe and Snoeyik, 2011:Section 14.5.3)

Element	Risk	Limit set bySANS 241-1 (South African National Standards, 2015:7-9)	Treatment process
Chromium (Cr)[1]		50 µg/l	RO filtration, ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7) and granular activated carbon (GAC) (Summers, Knappe and Snoeyink, 2011: Section 14.5.3)
Cobalt (Co)		N/A	Ion exchange (Rengaraj, et al, 2002:185)
Nickel (Ni)		70 µg/l	RO filtration and ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7)
Selenium (Se)		40 µg/l	RO filtration and ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7)
Strontium (Sr)		N/A	Ion exchange (Clifford, Sorg and Ghurye, 2011:Section 12.2) and RO filtration (Richards, Richards and Schäfer, 2010:189)
Vanadium (V)			RO filtration (Richards, Richards and Schäfer, 2010:189)

[1]Only measure when there is high risk of industrial pollution.

Arsenic

Arsenic (As) is toxic to humans. It is listed as a chronic health risk by SANS 241-1 (South African National Standards, 2015:7-9), which also prescribe a maximum concentration of 10 µg/l.

Depending on its prevailing form, As may be removed by conventional treatment process such as coagulation, Dissolved Air Flotation (DAF), sedimentation and filtration, or through more specialised processes such as ion exchange or RO filtration (Logsdon, Hess and Horsley, 1999:3.6-3.7)

Calcium and magnesium

The hardness of water is determined by the sum of divalent and trivalent cations and because magnesium (Mg^{2+}) and Calcium (Ca^{2+}) are typically much higher in concentration than others, they are said to make up the total hardness (Post, Atherholt and Cohn, 2011:Section 2.6.9). Hardness is typically expressed as calcium carbonate ($CaCO_3$). Water is generally considered as hard when the equivalent concentration of $CaCO_3$ is above 150 mg/l and soft at concentrations lower than 75 mg/l (Sawyer and McCarty, cited in Benefield and Morgan, 1999:10.15).

Hard water tends to scale and precipitate on pipes and water transport system surfaces rather than absorb minerals from surrounding components such as concrete or steel pipes. The opposite is true for soft waters. The scaling of hard waters can be a nuisance, e.g. when scaling occurs to the extent that pipes become blocked. To counter this an acid (e.g. H_2SO_4) can be added to the water to lower the precipitation potential of calcium and magnesium carbonates and phosphates. Soft waters on the other hand can cause damage to calcium based and steel infrastructure. Typically, a stabilising agent (e.g. $Ca(OH)_2$ or $CaCO_3$) is added to prevent this (Loewenthal, Wiechers and Marais. 1986).

Further, it is worth noting that water containing appreciable amounts of Ca^{2+} will better resist changes in pH when a coagulant is added, often negating the requirement for pH-correction, where Ca^{2+} is physically added to the water, prior to coagulation. Where there is considerable risk of scaling, the need for an acid can be replaced by overdosing a coagulant chemical to consume some of the hardness of the water.

Chloride and sulphate

Water containing high concentrations of chloride (Cl^-) and sulphate (SO_4^{2-}) tend to corrode metallic pipes (Schock, 1999:17.43). Further, relative concentrations of Cl^- and SO_4^{2-} to HCO_3^- is often associated with pitting of copper tubing (Ferguson, von Franqué, and Shock; Cruse and Pomeroy, cited in Schock, 1999:17.43). When consumed in large quantities SO_4^{2-} may have a laxative effect that could lead to acute diarrhea and ultimately dehydration (Post, Atherholt and Cohn, 2011:Section 2.6.18). In this regard the SANS 241-1 (South African National Standards, 2015:7-9) limit for SO_4^{2-} is 500 mg/l and in order to mitigate aesthetic risk, the upper limit for SO_4^{2-} and Cl^- have been set at 250 mg/l and 300 mg/l, respectively.

Both can typically be removed by RO filtration (Logsdon, Hess and Horsley, 1999:3.6-3.7) when present in dissolved form. However, SO_4^{2-} can also be removed by nanofiltration (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Colour

A yellowish-brown colour in water is largely as a result of the humic matter fraction of natural organic matter (NOM) (Edzwald and Tobiason, 2011:Section 3.5.2). NOM in water result from leachates from natural organic materials (e.g. roots, humus, peat, bark and leaves). The measurement of colour in water is given in mg/l platinum-cobalt (Pt-Co). Pt-Co provides an indication of the “yellowness” of the water and as such gives a good indication of the humic fraction of the NOM concentration.

SANS 7887 (South African National Standards, 2005:iii) notes that the yellowish-brown colour may also be attributed to iron and clay particles in the water and colour can either be measured as true or apparent colour. Apparent colour results from dissolved and undissolved particles in the water. According to SANS 7887 (South African National Standards, 2005:iii) true colour is due to dissolved substances only and is measured after the water is filtered through a 0.45 μm membrane.

Dissolved particles contributing to apparent colour will most likely be removed by conventional treatment processes, e.g. DAF. Therefore, it is recommended that only true colour be measured as this represents the particles that will not be easily removed. True colour therefore provides an indication of the NOM that may pass through the process and form DBPs during disinfection (Singer and Reckhow, 1999:12.26).

Colour may also render water aesthetically unpleasant to consumers and a maximum concentration of 15 mg/l Pt-Co is allowed by SANS 241-1 (South African National Standards, 2015:7-9), mainly for this reason.

If true colour is measured in appreciable amounts, then it can be removed by ultrafiltration (UF) or reverse osmosis (RO) (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Conductivity, corrosion and total dissolved solids

Conductivity can broadly be defined as water’s ability to pass an electrical current and is measured in Siemens per meter (mS/m). It is commonly known that electrical current is defined as the movement of charge, i.e. ions and/or electrons. When a current is established through the flow of electrons it’s termed electronic conduction, and when resulting from the movement of ions it’s called ionic conduction.

Solid materials like metals typically induce electronic conduction because of the high number of free electrons. In water however, electrical current typically results from ionic conduction and a high concentration of ions (a.k.a. ionic strength) increase the conductivity (Schock, 1999:17.42).

Pure or distilled water has low conductivity, however, when for example a salt like sodium chloride (NaCl) is added it dissociates to form Na^+ and Cl^- ions, and consequently a rise in conductivity is noticed. It’s therefore obvious why sea (or salt) water is a relatively good conductor. When encountering metal compounds, the positively charged ions in water move closer to the contact surface to pick up electrons and as a result a small electrical current as well as corrosion sets in.

Total dissolved solids (TDS) can include some small amounts of dissolved organic matter, however, it can generally be defined as the sum of anions (negatively charged) and cations (positively charged) in water. Yet it does not provide specific insight to the composition, nature or relationship between ions.

From the above it is clear that corrosion, conductivity and TDS are all linked. Typically, only conductivity and TDS are measured, and they can therefore be used as indicators for estimating whether water is likely to be corrosive. It is also worth noting that conductivity can be used to calculate the salinity of water (Weiss, cited in cited in Rounds, 2011:A3).

The ions that make up TDS include those polyvalent metallic cations that contribute to total hardness (further described below). These as well as other inorganic salts make up the largest fraction of TDS, and in appreciable amounts give water a salty or even brackish taste.

Elevated conductivity or TDS levels are not necessarily considered a direct health risk. Yet it provides indication of the water’s affinity to corrode and absorb unwanted materials (e.g. lead) which could have dramatic health implications for the consumer.

Generally, the levels of TDS found in drinking water sources have no direct aesthetic effect. Indirectly it could lead to corrosion of surrounding materials resulting in discolouration of water. It can also interfere with the chemical reaction of washing detergents resulting in staining of clothes or other product.

TDS represent those particles in water that are not easily removed through conventional treatment processes, however they may be reduced through RO filtration (Logsdon, Hess and Horsley, 1999:3.6-3.7). This may then result in less conductive and corrosive water.

The SANS 241-1 (South African National Standards, 2015:7-9) upper limit for conductivity and TDS is 170 mS/m and 1200 mg/l respectively.

Copper

Copper (Cu) has nutritional value and a diet containing insufficient Cu leads to nervous system degeneration, skeletal defects, anaemia and reproductive abnormalities. Excessive quantities of Cu may cause damage to liver and kidney systems and result in acute effects, such as gastrointestinal disturbances (Post, Atherholt and Cohn, 2011:Section 2.6.7). SANS 241-1 (South African National Standards, 2015:7-9) have set the upper limit for Cu at 2000 mg/l to reduce chronic health risks. Cu can be detrimental to biological organisms that are present in biofiltration systems and that are responsible for treating water.

The corrosion of brass, copper fixtures and copper pipes adds copper to water. Other smaller contributors of copper include weathered rock formations and industrial contamination (Cohn, Cox and Berger, 1999:2.27).

Cu can be removed from water via RO filtration (Richards, Richards and Schäfer, 2010:189) or ion exchange (Gottlieb and Watkins, 2012:Section 14.1).

Cyanide

The maximum concentration of Cyanide (CN^-) allowed for by SANS 241-1 (South African National Standards, 2015:7-9) is 200 µg/l. CN^- is toxic to humans and attacks the heart, respiratory system and central nervous system (Department of Health, 2004). The major source of CN^- in water is from industrial pollutants and a risk assessment should dictate the need for its measurement.

The recommended method for removing CN^- is through chemical oxidation (Logsdon, Hess and Horsley, 1999:3.6-3.7) and not withstanding this, some CN^- will be removed via chlorine disinfection. The reaction between chlorine gas (Cl_2) and free cyanide such as sodium cyanide (NaCN) results in the formation of cyanogen chloride (CNCl) as shown below.

$$NaCN + Cl_2 \rightarrow CNCl + NaCl$$

This is an instantaneous reaction and almost completely independent of pH. At a pH between 8.5 – 11, an oxygen reduction potential (ORP) of between 250 – 400 mV, and in the presence of an alkali, such as sodium hydroxide (NaOH), CNCl will be converted to the more stable cyanate (NaCNO) as shown below.

$$CNCl + 2NaOH \rightarrow NaCNO + NaCl + H_2O$$

At a pH between 8.5 – 9.0 the above reaction takes 10 to 30 minutes for 100% conversion of NaCN to NaCNO and at a pH between 10 – 11 it takes 5 to 7 minutes. Further, at the lower pH range (8.5 – 9.0) and an ORP between 300 mV to 600 mV the NaCNO will react with NaOH and be removed as follows:

$$2NaCNO + 4NaOH + 3Cl_2 \rightarrow 6NaCl + 2CO_2 + N_2 + 2H_2O$$

DOC, UV absorbance and THMFP

Natural organic matter (NOM), resulting from living and decaying vegetation, make up the largest fraction of total organic carbon (TOC) in most water supplies. Humic and fulvic acids is an important fraction of NOM. When the TOC concentration is around a few mg/l the observed yellow-brown hue, that's almost tea like, is a result of humic acid. Humic acid may also impart a musty type smell to water. Fulvic acid on the other hand, gives water a darker colour and is less aromatic. In the treatment process these acids react with chlorine to form DBPs (Hroncich, 1999: 4.49). At the normal pH range of natural waters, they carry a significant negative charge, which impacts on coagulation by consuming significant amounts of the coagulant chemical.

These natural occurring organics normally don't pose any health risks, except for some high-molecular-weight natural hydrocarbons (Post, Atherholt and Cohn, 2011:Section 2.7). Conversely DBPs have been the focus of many studies with evidence supporting its link to cancer and reproductive/developmental issues, resulting from continuous exposure to low levels of DBPs. Equally, however, there are also findings against this. Currently the effects of DBPs are still being debated.

Total organic carbon (TOC) comprise both particulate (POC) and dissolved organic carbon (DOC), and typically in surface waters DOC makes up 90 to 99 percent of the TOC. In groundwaters the fraction of POC is nearly zero and the TOC is essentially equal to the DOC (Edzwald and Tobiason, 2011: Section 3.5.3). DOC is defined as the fraction of organic carbon that passes through a 0.45 µm filter. However, this sometimes include some very small particulate colloidal particles that can get through. Nonetheless industry accepts this portion as completely dissolved. Since DOC is easier to measure and fundamentally reveals the encompassing organic carbon concentration it may not be required to measure TOC.

Edzwald, Becker and Wattier (cited in Edzwald and Tobiason, 2011: Section 3.5.4) illustrated that the ultraviolet (UV) absorbance at 254 nm (UV_{254}) can be used to estimate which fraction of TOC will contribute to the formation of DBPs, such as trihalomethanes (THM). This is because organic compounds with conjugated double bonds C=C tend to absorb more light (at 254 nm) than other types of organics and these same compounds are attacked by and chemically react with chlorine when they donate electrons through their double bonds, to form DBPs. This fraction of TOC may also typically be removed via coagulation and where its largely driving coagulant demand UV_{254} can be used for setting dose rates. This is typically done by setting a target UV_{254} downstream of coagulation. In the same fashion it can also be used to test the effectiveness of activated carbon or membrane processes at removing organics.

As mentioned above the organic compounds in question are electron donors and as such they become a source of oxidant demand. As such for oxidation processes, UV_{254} can provide an indication of increased chemical demand that may affect operating cost.

Some (Edzwald, Becker and Wattier cited in Edzwald and Tobiason, 2011: Section 3.5.5) have developed the specific ultraviolet absorbance (SUVA) at 254 nm:

- as indicator of the NOM composition;
- to estimate chemical demand; and
- to quantify the fraction of TOC that can be removed by coagulation.

SUVA values can be derived as follows and is expressed in units of m^{-1} absorbance per mg/l of DOC:

$$SUVA = \frac{(UV_{254}\ in\ cm^{-1}) \times 100\,\frac{m^{-1}}{cm^{-1}}}{(DOC\ in\ mg/l)}$$

What the different SUVA values reveal is recorded in Table 3 (Edzwald and Tobiason, 2011:Section 3.5.5) . It is important to note when high fractions of organic carbon are removed it can make up a substantial portion of the treatment process residuals (i.e. sludge) that must be managed on or off-site.

Table 3: Characterization of NOM and TOC removals for SUVA values (Edzwald and Tobiason, 2011:Section 3.5.5)

SUVA	Composition	Effects	TOC Removals by coagulation
> 4	High fraction of aquatic humic matter High aromatic and hydrophobic character High molecular weight (MW)	High UV absorbance High oxidant and chlorine demand	60–80% Higher end for waters with high TOC
2–4	Mixture of aquatic humic and non-humic matter Mixture of aromatic and aliphatic character Mixture of low to high MW	Medium UV absorbance Medium oxidant and chlorine demand	40–60% Higher end for waters with high TOC

< 2	High fraction of non-humic matter High aliphatic and low Hydrophobic character low MW	Low UV absorbance Low oxidant and chlorine demand	<20–40% Higher end for waters with high TOC

To estimate the concentration of DBPs such as trihalomethanes (THM), resulting from the remaining TOC (after treatment), the relationship between TOC and the trihalomethane formation potential (THMFP), developed by Chapra, Canale, and Amy (cited in Letterman, Amirtharajah and O'Melia, 1999:6.4) can now be used as follows:

$$THMFP = 43.78TOC^{1.248}$$

Where TOC is in mg C/l and THMFP is in µg/l.

Dissolved oxygen

The effects, advantages and disadvantages of dissolved oxygen (DO) in water is discussed under Section 3.1. Also, it may be of interest to note, the organisms that exist in biological filtration systems require a certain concentration of DO to perform oxidation-reduction reactions (Mouchet, 1992:165).

Fluoride

Fluoride (F) occurs naturally in many water supplies (Post, Atherholt and Cohn, 2011:Section 2.6.8), but increased concentrations have been noticed where pesticides are captured by runoff water over agricultural land.

In some countries, like the USA, F is added, in low concentrations, to municipal drinking water to strengthen the teeth and increase the bone density of consumers. The risk is however that an overdose of F, resulting from faulty equipment, could lead to "diarrhoea, nausea, headache, abdominal pain and dizziness" (Centers for Disease Control and Prevention, cited in Post, Atherholt and Cohn, 2011:Section 2.6.8). Continuous over consumption of F may result in skeletal fluorosis, which causes bones to harden and become brittle (Post, Atherholt and Cohn, 2011:Section 2.6.8).

SANS 241-1 (South African National Standards, 2015:7-9) demands a F concentration of less than 1.5 mg/l. Excess F concentration can be reduced by RO filtrations (Richards, Richards and Schäfer, 2010:189) or lime softening (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Hydrogen sulphide

Hydrogen sulphide (H_2S) is typically found in groundwaters and distribution networks due to the actions of sulphur reducing bacteria on sulphates (Mallevialle and Suffet, cited in Cohn, Cox and Berger 1999:2.70), most notably desulfovibrio desulfuricans. H_2S causes an unpleasant rotten egg smell that can be detected at concentrations as low as ≤ 0.1 µg/l. It also accelerates corrosion (American Water Works Association; Singley, et al, cited in Schock 1999:17.43), leads to fouling of filter membranes and is toxic to the organisms associated with biological filtration (Mouchet, 1992:164).

As mentioned in Section 3.1.2 below, H_2S can be removed by air stripping. In distribution networks a well-maintained chlorine residual will limit the growth of H_2S causing bacteria (Post, Atherholt and Cohn, 2011:Section 2.10).

Iron

Iron (Fe) may be present in surface waters, but it is typically more common in groundwater as a result of water that has percolated through Fe-containing soils and rocks. Too much Iron in water is associated with both health and aesthetic risks (South African National Standards 241-1, 2015:7-9). However, considering the nutritional requirement of iron for both adult men (10 to 12 mg per day) and woman (10 to 15 mg per day) (National Research Council (NRC), cited in Cohn, Cox and Berger 1999:2.29), it is unlikely that the concentrations found in drinking water will lead to considerable health issues. Nevertheless, SANS 241-1 (South African National Standards, 2015:7-9) require an iron concentration of less than 2 mg/l to reduce chronic health risks. Further, iron at certain concentrations may cause discolouration of household items such as laundry and further lead to a metallic taste (Post, Atherholt and Cohn, 2011:Section 2.6.10). The SANS 241-1 (South African National Standards, 2015:7-9) limit for aesthetically pleasing water is 300 µg/l.

Iron may be suspended or dissolved in water. Some concentrations of suspended iron may be removed by coagulation, but this needs to be established by upfront testing. Dissolved iron present as ferrous (Fe^{2+}) can be oxidised, either chemically (Singer and Reckhow, 1999:1) or biologically (Mouchet, 1992:160), to Fe^{3+}, which will naturally precipitate out of solution as iron(III) hydroxide $(FeOH)_3$ and can then be extracted by conventional treatment processes.

Lead

Corroded pipes, constructed from Lead (Pb) based material is the main contributor of Pb to drinking water (Post, Atherholt and Cohn, 2011:Section 2.11). As such, Pb may only become prevalent in the water distribution network, and post treatment, due to old lead containing pipes. Yet, Pb may be found in water bodies that are exposed to pollutants coming from industry. Pb exposure has been associated with a number of health complications, including anaemia, kidney damage and delayed neurological and physical development (Cohn, Cox and Berger, 1999:2.29). The maximum concentration allowed for by SANS 241-1 (South African National Standards, 2015:7-9) is 10 µg/l.

Because Pb generally enters the water after treatment it is suggested by some (Logsdon, Hess and Horsley, 1999:3.6-3.7) that its concentration be controlled by corrosion control treatment of the distribution system. If Pb is present in source water then Ion exchange may be employed as a removal step (Gottlieb and Watkins, 2012:Section 14.1). However, at a pH of above seven Pb could be present in a non-ionic colloidal form and render ion exchange ineffective. In this form Pb will precipitate out of solution and can be filtered out using a submicron filter. (Keller, 2005:28).

Manganese

Like Fe, manganese (Mn) is typically more often found in groundwater due to natural weathering of Mn containing rocks and soils. It may also come from man-made sources such as agriculture, discarded batteries or steel alloy production (Post, Atherholt and Cohn, 2011:Section 2.6.12). Mn is an essential nutrient and 2 to 5 mg/day is recommended as safe for adults. For infants the recommended intake is lower at 0.3 to 0.6 mg/day (National Research Council, cited in Cohn, Cox and Berger 1999:2.30). The negative effects of excess manganese in the human body relates to respiratory tract and brain functions. More specifically it may result in lung embolism, bronchitis, forgetfulness and nerve damage. It may even lead to symptoms similar to Parkinson disease (Post, Atherholt and Cohn, 2011:Section 2.6.12). SANS 241-1 (South African National Standards, 2015:7-9) prescribe a maximum concentration of 400 µg/l from a health point of view.

The aesthetic target by SANS 241-1 (South African National Standards, 2015:7-9) is 100 µg/l. An aesthetic target is set to ovoid bad taste and the black staining of laundry. Also, when present in high concentrations Mn is oxidised by chlorine into a precipitate that ends up as sediment in pipe networks. When dislodged this sediment causes water discolouration. The aesthetic target reduces the risk of sediment build-up in pipes.

Mn can either be present in suspended or dissolved form. A 0.45 µm filter may be used to measure the dissolved fraction of Mn. Similar to Fe, some Mn could be removed through coagulation, but this ought to be confirmed via testing. Like Fe, the dissolved Mn in the form of manganous (Mn^{2+}) can be chemically oxidised (Singer and Reckhow, 1999:12.1) and will precipitate out of solution, and be captured by downstream sedimentation or filtration. Alternatively, Mn^{2+} can be biologically oxidised (Mouchet, 1992:165) and removed. Another option is removal through ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Mercury

Mercury (Hg) that enter a water body can either come from natural sources, or from man-made products and activities such as pesticides, mining, industrial activities and burning of fossil fuels. Hg is primarily present as inorganic Hg in water and methylmercury (organic, CH_3Hg^+) in sediments. The inorganic Hg in water is sometimes transformed to methylmercury by bacteria. Inorganic Hg is not readily absorbed into the human body and therefore not as toxic as methylmercury. The latter is considered neurotoxic and can lead to motor and mental disfunctions, and even death (Cohn, Cox and Berger, 1999:2.30). The upper limit allowed by SANS 241-1 (South African National Standards, 2015:7-9) is 6 µg/l.

Mercury is not typically found in surface water; however, sampling and testing should be considered where there is a risk of man-made pollutants.

Hg in its inorganic form can be removed through lime softening (high pH) or RO filtration (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Nitrate and Nitrite

Nitrate (NO_3^-) occurs naturally in water and elevated concentrations may be due to wastewater discharge, fertilizers and feedlots (Cohn, Cox and Berger, 1999:2.31). NO_3^- in groundwater may indicate exposure to contamination sources such as septic tanks.

Nitrite (NO_2^-) is not typically present in surface waters, however, NO_3^- may be reduced to NO_2^-, particularly in the presence of ammonia (Cohn, Cox and Berger, 1999:2.31). The opposite may also occur where NO_2^- is oxidised to NO_3^-. It is therefore important to obtain concentrations for both in order to judge whether there is a risk of contamination.

Increased intake of NO_3^- may lead to a condition called methemoglobinemia, which is caused by elevated levels of methemoglobin in the bloodstream. Methemoglobin is formed from and is a type of haemoglobin (it in fact replaces the haemoglobin cell) that is unable to transfer oxygen to body tissues.

NO_2^- resulting from denitrification of NO_3^- in the body increases the formation of methemoglobin. In healthy adults the formation of methemoglobin is reversed by a protective enzyme (NADH-cytochrome b5 reductase) Infants, younger than 6 months however have lower NADH-cytochrome b5 reductase levels and therefore have limited ability to fight the formation of methemoglobin.

To avoid acute health problems SANS 241-1 (South African National Standards, 2015:7-9) has set the maximum allowable concentration for NO_3^- and NO_2^- at 1 mg/l. In Europe biological processes are commonly used to remove NO_3^- (Rittmann and Huck, cited in Rittmann et al, 2012:Section 17.3.2.1). RO filtration, comprising a special high rejection membrane, or ion exchange (Schoeman and Steyn, 2002:18) are alternatives to biological processes.

Potential hydrogen

The free hydrogen (H^+) ion activity in water is measured potentiometrically as pH and expressed by the negative log of the hydrogen ion activity as in the equation below:

$$pH = -\log f_m[H^+]$$

$[H^+] = Molar\ concentration\ of\ H^+$

$f_m = Monovalent\ ion\ activity\ coefficient$

It is important to note that the coefficient shown above is affected by ionic strength and temperature. pH effects the water treatment process in many ways and because of its simple measurement is often used as a control parameter for the various water treatment processes. In the sections below, it is explained how pH effects aeration, coagulation and disinfection. It is therefore important not only to know what the pH of the raw water is, but also how it tracks through the various process steps. Further, the pH range recommended by SANS 241-1 (South African National Standards, 2015:7-9) is between 5 and 9.7 for the final water exiting the treatment plant.

A certain pH can be targeted using various chemicals. To reduce the pH (specifically to get within the acceptable range for coagulation) one may resort to an acid, such as HCl or H_2SO_4. Various chemicals can be used to increase the pH, like sodium hydroxide (NaOH), sodium carbonate (Na_2CO_3), $Ca(OH)_2$ or $CaCO_3$.

Orthophosphate

Water sources that receive significant volumes of final effluent from waste water treatment plants, which don’t include phosphate removal or that are operated ineffectively may contain high concentrations of orthophosphates (PO_4^{3-}). The same may be true where the catchment area is largely covered by agricultural land and where fertilisers are prominently used.

Phosphates is one of the main food sources for algae and thus provides a surrogate measure for algae potential. It may be removed from water through limestone stabilisation, where the phosphates tend to absorb onto $CaCO_3$ (Mustafa et al, 2007:286).

Potassium

Potassium (K) adds to the salinity of the water and may typically be present in high concentrations in sea water or brackish water. K in the water can be reduced through RO filtration (Richards, Richards and Schäfer, 2010:189); however, high concentrations of K are likely to foul the membranes. Mitigation measures to reduce the fouling risk should be employed when RO filtration is considered, which may include ion exchange.

Redox Potential

The redox potential of the water provides an indication of the tendency of chemical species to either be reduced or oxidised. This is important, to establish whether metals and other chemical compounds can easily be oxidised by aeration or other chemicals during the treatment process.

In the case of ground water treatment and where biofiltration is considered, for Fe and Mn removal, the redox potential will provide insight into whether these metals will be oxidised, by aeration, before the water reaches the filters. In waters conforming to a high redox potential the Fe and Mn (more specifically Fe^{2+} and Mn^{2+}) will likely be oxidised by the oxygen added in the upstream aeration step, rather than the biological organisms living in the filters. This oxidation product causes complications with the filters, often leading to filter blinding of the media or blocking (due to precipitation) of the underdrainage system.

Silt Density Index

The silt density index (SDI) is an index used to predict the likelihood of fowling of RO membranes (Taylor and Wiesner, 1999:11.44). To determine the SDI a constant pressure of 206.8 kPa is maintained when water is sent through a 0.45 µm filter. The percentage drop of the flow for every minute is recorded. The average percentage drop is then determined over a period of typically 15 minutes. The SDI test should be done within 4 hours of sampling and the sample should be well shaken.

The SDI number will dictate whether pre-treatment is required and what type of RO filtration system (e.g. spiral-wound RO or fibre RO) is most appropriate.

Silica

Silica oxide (SiO_2) is often present in surface water due to erosion of soils during storm events (Edzwald and Tobiason, 2011:Section 3.4.2), however they are typically at low concentrations. In brackish water, however, they are found to be more prominent. Further, it is worth noting that SiO_2 in water could impede chemical oxidation of Fe. At present the most effective removal method is through RO filtration, yet SiO_2 at high concentrations tend to foul the membranes. Current research suggests that using electrodialysis reversal (EDR) may be a better way of removing Silica in brackish surface waters (Duranceau, cited in Duranceau and Taylor, 2011:11.7.3).

Sodium

Sodium (Na) is a natural constituent present in source waters. Increased Na concentrations in the blood stream may lead to hypertension (high blood pressure) (Cohn, Cox and Berger, 1999:2.33). SANS 241-1 (South African National Standards, 2015:7-9) allows a maximum concentration of 200 mg/l to meet aesthetic requirements. At low concentrations RO filtration (Richards, Richards and Schäfer, 2010:189) is typically employed to reduce Na levels.

Total Carbonate (IC) Species

As mentioned earlier it becomes difficult to measure alkalinity at low pH ranges. This is because alkalinity is said to result mostly from HCO_3^- and CO_3^{2-} ions and from Figure 1 (Gehm and Bregman, cited in Benefield and Morgan, 1999:10.10) it is evident that at a low pH they make up a small fraction of the total IC (carbonate) species - C_T = the sum of H_2CO_3, HCO_3^- and CO_3^{2-} concentrations.

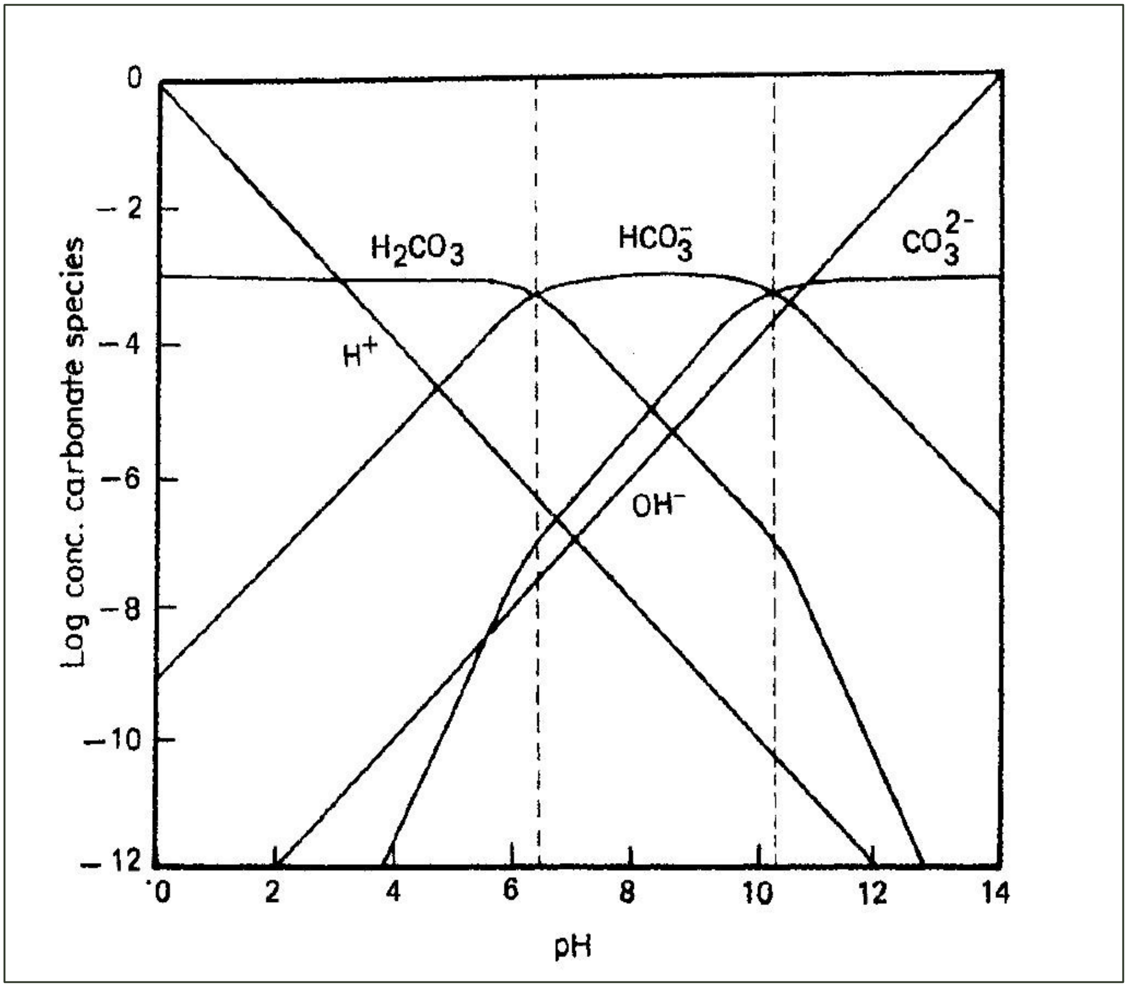


Figure 1: Concentration distribution diagram for carbonic acid (Gehm and Bregman, cited in Benefield and Morgan, 1999:10.10)

The measurement of total IC species (C_T) at a pH of less than 5 will reveal mostly the concentration of carbonic acid (H_2CO_3). With this, the effect on the alkalinity when adding a stabilising agent (e.g. $Ca(OH)_2$) can be determined. When $Ca(OH)_2$ is added to water it releases OH^- ions which drives up the pH and the total IC species move to the right on the above log scale (Benefield and Morgan, 1999:10.9). Between a pH of about 6.2 and 10.1 HCO_3^- is dominant species and largely makes up the alkalinity of the water, the rest is made up by the CO_3^{2-} ions. It is worth noting that as the pH increases the potential for scaling increases as well. This is due to the ionic product of the [Ca^{2+}], [CO_3^{2-}] and [Mg^{2+}] molar concentrations exceeding the solubility product of $CaCO_3$ and $MgCO_3$, which then precipitate out of solution (Loewenthal and Marais, 1976).

Total Hardness

Water is said to be hard when it tends to scale, if it precipitates soap and makes it difficult to lather (Post, Atherholt and Cohn, 2011:Section 2.10.4), or when it needs extra detergent for proper cleaning.

Hardness is caused by any polyvalent metallic cations present in the water. The two main contributors in natural water are the divalent cations Ca^{2+} and Mg^{2+} (Edzwald and Tobiason, 2011:Section 3.2.3), and any contributions by other metals (e.g. Sr^{2+}, Fe^{2+} and Mn^{2+}) is often deemed negligible. However, in groundwaters containing high concentrations of Fe^{2+} and Mn^{2+} it is best to include them in the total hardness calculations.

Total hardness is normally expressed as mg/l of $CaCO_3$ (Benefield and Morgan, 1999:10.14), The measured concentrations (in mg/l) of Ca^{2+} and Mg^{2+} can be converted to mg/l $CaCO_3$ by first determining the corresponding milliequivalents per litre (meq/l). For example, the molar mass per meq for Ca^{2+} is 20.05 mg/meq (the molar mass of 40.01 mg is divided by the charge, in the case of Ca = 2) and the meq/l is derived from dividing the measured concentration, say 82 mg/l, by the mg/meq, which in this case will equal 4 meq/l. Now the meq/l is multiplied by the mg/meq of $CaCO_3$ (50 mg/meq) to give a total of 200 mg/l $CaCO_3$. The same is done for Mg^{2+} and the sum of $CaCO_3$ in mg/l is the measured hardness. Table 4 (Sawyer and McCarty, cited in Benefield and Morgan, 1999:10.15) provide a means of linking the $CaCO_3$ concentration to a hardness description.

Table 4: Hardness classification scale (Sawyer and McCarty, cited in Benefield and Morgan, 1999:10.15)

Mg/l as $CaCO_3$	Hardness description
0 – 75	Soft
75 – 150	Moderately hard
150 – 300	Hard
>300	Very hard

Turbidity and Total Suspended Solids

Total suspended solids (TSS) comprises the total suspended or colloidal particles in the water. These represent small particles that will not naturally settle out by gravity.

To measure turbidity a beam of light is passed through a water sample and the intensity of light scattered at 90^o, caused by particulate material, is measured in nephelometric turbidity units (NTU) (Edzwald and Tobiason, 2011:Section 3.4.4). Turbidity measurement encompasses a wide range of particles, from various sources, including TSS. As such, turbidity is often used as a substitute measure to estimate the TSS. Some have attempted to develop direct correlations between the two and suggested a ratio of between 1 to 2 (Kawamura, 2000:385). It is however suggested to also measure the TSS as it will form a large component of the process residuals (sludge) that will drive the design of sludge handling infrastructure.

TSS can typically be removed and turbidity reduced through the conventional treatment process. SANS 241-1 (South African National Standards, 2015:7-9) suggest a maximum of 1 NTU to meet operational requirements and 5 NTU to meet aesthetic requirements. 1 NTU is required after filtration and before disinfection as particles contributing to turbidity tend to adsorb onto and encase pathogens, protecting them from disinfection (Mohapatra and Narayankar, 2013:194). These particles also react with chlorine and hence increases the chlorine demand.

Zinc

Zinc (Zn) is commonly found in source waters but may also find its way into finished drinking water as a leachate from galvanised metal pipes (Cohn, Cox and Berger, 1999:2.33). Although Zn has some nutritional value, and 15 mg per day is recommended for adult males and 12 mg per day for adult females, excess consumption may cause health concerns (National Research Council, cited in Cohn, Cox and Berger, 1999:2.33). It has been suggested that Zn consumption in the order of 40 mg/l over a long period of time could lead to “muscular weakness and pain, irritability, and nausea” (Cantilli, Abernathy, Donohue; Greger, cited in Cohn, Cox and Berger, 1999:2.33). Zn is also detrimental to biological filtration and could compromise the complete process if present in high concentrations (Mouchet, 1992:164).

SANS 241-1 (South African National Standards, 2015:7-9) nominate a maximum concentration of 5 mg/l. Zn can be removed through a number of processes, including lime softening, RO filtration and ion exchange (Logsdon, Hess and Horsley, 1999:3.6-3.7).

Faecal coliforms

Faecal coliforms refer to a specific group of bacteria. They are found in the intestines of animals and humans and excreted through faeces. It is often used to determine if the source water is exposed to sewage contamination. Escherichia coli (E.coli) is a type of faecal coliform (Cohn, Cox and Berger, 1999:2.15) and therefore included in the overall measurement. SANS 241-1 (South African National Standards, 2015:7-9) require a zero count per 100 ml test sample and a well operated conventional treatment plant will remove most faecal coliforms before those remaining are deactivated through disinfection.

Heterotrophic plate count

The heterotrophic plate count (HPC) is used to determine the concentration of heterotrophic bacteria in the water (American Public Health Association et al., cited in Cohn, Cox and Berger, 1999:2.15). Heterotrophic bacteria refer to those who use organic carbon for energy and growth (Bartram, et al, cited in Post, Atherholt and Cohn, 2011:Section 2.3.5). In other words, it is an encompassing measurement method that covers most bacteria likely to be found in source waters.

SANS 241-1 (South African National Standards, 2015:7-9) stipulate a count of 1000 per ml, which is normally achievable through conventional treatment.

Total coliforms bacteria

Total coliforms include faecal coliforms (Cohn, Cox and Berger, 1999:2.15) as well as other bacteria from the coliform group, which are more widely distributed in the environment. Conventional treatment is typically employed to meet the SANS 241-1 (South African National Standards, 2015:7-9) requirement of 10 counts per 100 ml.

Cryptosporidium

Cryptosporidium (Crypto) falls under the protozoa group and infect their hosts by forming thick-walled zygotes (oocysts) in their intestines. Crypto leads to cryptosporidiosis (a diarrheal disease), which is mainly contracted when water containing oocysts are ingested. Farm animals, especially young cattle, and humans are the two main sources of crypto in water (Post, Atherholt and Cohn, 2011:Section 2.2.3). In South Africa however, it has been suggested that baboons may be a source of crypto, especially in the Cape mountains.

As mentioned below typical methods of chlorine dosing (e.g. Cl_2 or $Ca(ClO)_2$) has little to no effect on crypto. Suitable alternative methods include UV, chlorine dioxide or ozone. SANS 241-1 (South African National Standards, 2015:7-9) tolerate a zero count per 10 litres of water.

Giardia

Giardia causes diarrheal disease, and acts like crypto in that it forms thick-walled cysts in the intestine of its host. Giardia infects humans and a number of animals (Post, Atherholt and Cohn, 2011:Section 2.2.3) before being excreted in the faeces. This suggests that both animals and humans are contributors of giardia in source waters.

A large portion of Giardia (up to 3 log) can be removed through conventional treatment processes (refer Table 13) before the remaining fraction is deactivated by disinfection (chlorine dosing, UV or ozone). This will typically meet the SANS 241-1 (South African National Standards, 2015:7-9) requirement of zero count per 10 litres.

Chlorophyll-a, Total Algal Cells and Algal Types

Algal growth in dams is associated with nutrients in the water, and when the water is clear, resulting in high underwater light intensity, and increased water temperature. Elevated nutrient levels will lead to increased algal growth unless growth is hindered by deficient light or low water temperature.

Algae is sometimes not only a nuisance at a treatment plant but can often compromise a complete process. It can clog up screens and weirs, resulting in increased hydraulic losses and reduced flow through the plant. It can also hinder the settling of floc particles that may lead to the carryover from the sedimentation process and blinding of filters. In addition, algae may contribute to increased coagulant and chlorine demand, the formation of disinfection by-products (DBPs) that may have a carcinogenic risk, pH fluctuations, taste and odour problems and the release of algal toxins.

Measuring for total algal cells is typically done for surface waters but may be somewhat of a hit and miss. This is because the sampler may only collect the sample close to the water surface and the type of algae present may proliferate deeper down. Also, at the time of sampling the algae may not be in bloom and motile algae move up and down in the water column over the day.

Algae produce chlorophyll during photosynthesis (Cohn, Cox and Berger, 1999:2.13). One is more likely to pick-up the presence of chlorophyll-a in the water and based on the concentration be able to judge the likelihood of present or future algal blooms. Typically, a specialist is consulted in this regard.

Lastly, the type of algae should be identified to inform suitable treatment, e.g. blue-green algae (cyanobacteria) can lead to the production of taste and odour chemicals (Juttner and Watson, cited in Post, Atherholt and Cohn, 2011:Section 2.10.1) and here geosmin is the most common type. This compound is produced by the algae and released when it is killed off. It has a very strong earthy taste and odour. Humans can detect this at very low levels.

Activated carbon can be used to remove geosmin, through adsorption, and can be added as granular activated carbon (GAC) or powdered activated carbon (PAC). When the adsorbate comes into contact with the surface of the activated carbon (AC) particles it is held in place by forces such as “hydrogen bonds, dipole-dipole interactions and van der Waal’s forces” (Summers, Knappe and Snoeyink, 2011:Section 14.3.1).

Algae tend to float or stay in suspension due to a lower density than water. Dissolved air flotation (DAF) (Gregory and Edzwald, 2011:Section 9.6.5) or dissolved air flotation filtration (DAFF) is commonly used to remove algae and may be paired with AC treatment where taste and odour are of concern.

3 Treatment Processes

This section describes the treatment processes that is usually associated with a conventional water treatment works, including:

- aeration and air stripping;
- coagulation and flocculation;
- dissolved air floatation (DAF);
- sedimentation;
- filtration; and
- disinfection.

Each of the above processes is broken up into individual sections. Each section begins by giving a brief overview of the process and the mechanics involved. Thereafter, recommendation is given on the water quality parameters that can be addressed through the process. Further, the approach for designing the applicable process component is explained and how it’s been built into the software model described.

Lastly, possible future improvements to the software model are listed that may or may not be included in future versions.

3.1 Aeration and Air Stripping

3.1.1 Process Description

Aeration generally refers to the transfer of gas into water (adsorption), and in water treatment specifically, the aim is to introduce oxygen into the water. In gas-transfer theory an air- and water film is believed to exist at the air-water interface and these films are said to limit the rate of gas transfer. It is further held that film resistance is influenced by temperature and turbulence (Dyksen, Raczko and Savino, 2012:Section 6.2.3).

Aeration occurs when water is brought into close contact with air and oxygen dissolves into the water. In the natural environment, given sufficient time, a state of equilibrium will be reached between the concentration of oxygen in water and air. A relatively long time-period is typically required to achieve this equilibrium. The function of aeration methods in water treatment applications is to speed up this natural process (Dyksen, Raczko and Savino, 2012:Section 6.2.3). This is achieved by increasing the air-water interface, where thin sheets or drops of water is allowed to fall through the air. Other methods include diffused aeration where small bubbles of air is introduced at the bottom and left to rise through a water column (Minnesota Rural Water Association et al., 1994:Aeartion 1). The efficiency of an aeration system is determined by the size of the surface area of the air-water interface. Aeration is further enhanced by creating turbulent conditions, which reduces the liquid-film thickness, and subsequently the film resistance (Dyksen, Raczko and Savino, 2012:Section 6.2.3).

Air stripping is the process where volatile gases are removed from the water and transferred into atmosphere (Hand, Hokanson and Crittenden, 2011:Section 6.1) (desorption). It occurs due to the scrubbing caused by aeration turbulence in which the partial pressure of the volatile compound in the water is lower than in the air. As water falls through the air the gas film is agitated and gases are released from the water to the surrounding air.

3.1.2 Water Quality

Air stripping (and by default aeration) should be considered for water that contains high concentrations of CO_2. This is typical of surface waters abstracted at deep locations, and results from a combination of respiration of microscopic animals (Minnesota Rural Water Association et al., 1994: Aeration 2) and a lack of photosynthesis by plant organisms. Volatile taste- and odour-causing substances, caused by oils and algae by-products, can also be removed by aeration and air stripping. However, it’s worth noting that aeration is only partially effective at removing oils, since they are much less volatile than gases (Minnesota Rural Water Association et al., 1994: Aeration 3).

Fe^{2+} and Mn^{2+} are both oxidised by dissolved oxygen to respectively form insoluble $Fe(OH)_3$ and MnO_2 precipitates, which can be removed through subsequent filtration and/or sedimentation (Singer and Reckhow, 1999:12.25).

Like CO_2, H_2S can also be removed by air stripping, however, with H_2S the process is dependent on the pH of the water. Due to the low pK_s value (~7) of the H_2S/HS^- dissociation, effective H_2S air stripping is achieved at a pH of 7 or less. At higher pH conditions the H_2S is ionized as HS^-, preventing removal.

Methane (CH_4) can also be removed by aeration and air stripping. CH_4 is added to water by the decomposition of organic matter in water. It can also be found in ground water that are near or exposed to natural-gas deposits (Minnesota Rural Water Association et al., 1994: Aeration 3).

Care must be taken not to over aerate the water. Water that contains a high concentration of dissolved oxygen may cause corrosion, air binding in filters and/or algal growth (Minnesota Rural Water Association et al., 1994:Aeration 6).

3.1.3 Design Approach

There are many aeration techniques and technologies available in water treatment, including, but not limited to, diffused or bubble aeration, surface aeration and spray aeration. The design approach of this section focusses on surface and diffused aeration. For surface aeration the software model includes and allows for the design of a cascading weir system, a surface aeration technique. Allowance has also been made in the model to design a diffused aeration system that could either be a standalone arrangement or used to enhance the cascading weir aeration.

3.1.3.1 Surface Aeration

A cascading weir system comprises a set of weirs and pools, installed in series along the water flow path. Water overflows the first weir, creating a water jet, and drops into the downstream pool, it then overflows the second weir and drops into the second downstream pool and so on. The turbulence caused in the pools, along with the air that gets entrained in the water jet and between the water jet and the receiving water surface, allows for oxygen to be dissolved into the water. At the same time the scrubbing caused by the water jet as it passes through air allows for air stripping to occur, whereby CO_2 and, at the right pH, H_2S are released into the surrounding air.

The cascading weir design part of the software model allows the user to design the system with either horizontally placed rectangular weirs or v-notch weirs, up to a maximum of 5 weirs in series. The design of the horizontally placed weirs system is based on equations developed by Kim and Walters (2001). The design of the v-notch weir system is based equations developed by Baylar, Hanbay and Ozpolat (2008). The first design requirement is to determine the saturation concentration, which as the name suggests refers to the concentration of oxygen at which the water becomes saturated. In other words, it is the maximum concentration of oxygen that can be dissolved into the water. The saturation concentration is based on formulas found in literature (Weiss; Benson and Krause, cited in Rounds, 2011:A3 -A4) and is influenced by temperature, pressure and salinity as can be seen below.

$$u = exp\left(11.8571 - \frac{3840.7}{T} - \frac{216961}{T^2}\right)$$

$$\emptyset_o = \frac{975}{10^6} - \frac{1.426 \times T}{10^5} + \frac{6.436 \times T^2}{10^8}$$

$$F_p = \left(\frac{(P-u)(1-\emptyset_o \times P)}{(1-u)(1-\emptyset_o)}\right)$$

$$S = 5.572 \times 10^{-4} \times SC + 2.02 \times 10^{-9} \times SC^2$$

$$F_s = exp\left[-S \times \left(0.017674 - \frac{10.754}{T} + \frac{2140.7}{T^2}\right)\right]$$

$$DO_o = exp\left(-139.34411 + \frac{1.575701 \times 10^5}{T} - \frac{6.642308 \times 10^7}{T^2} + \frac{1.243800 \times 10^{10}}{T^3} - \frac{8.621949 \times 10^{11}}{T^4}\right)$$

$$C_s = DO_o \times F_S \times F_P$$

$u = Vapour\ pressure\ (atm)$

$T = Temperature\ (Kelvin)$

$\emptyset_o = Virial\ coefficient\ of\ oxygen$

$F_p = Pressure\ factor$

$P = Barometic\ pressure\ (atm)$

$S = Salinity\ \left({}^{0}/_{00}\right)$

$SC = Electrical\ conductance\ \left({}^{\mu S}/_{cm}\right)$

$F_s = Salinity\ factor$

$DO_0 = Baseline\ dissolved\ oxygen\ \left({}^{mgO_2}/_{l}\right)$

$C_s = Saturation\ concentration\ \left({}^{mgO_2}/_{l}\right)$

Now that the saturation concentration is calculated, it is possible to determine the oxygen concentration in the first pool, i.e. downstream of the first weir. The method for calculating this oxygen concentration depends on the number of weirs in the system but first the X, Y, V and Z variables must be defined as follows:

$$X = \frac{C_{d1} - C_{u1}}{C_s - C_{u1}}$$

$$Y = C_{d1} + \frac{(C_{d1} - C_{u1}) \times (C_s - C_{d1})}{C_s - C_{u1}}$$

$$V = X \times [C_s - Y - (C_s - Y) \times X]$$

$$Z = Y + (C_s - Y) \times X$$

$C_{d1} = Oxygen\ concentration\ downstream\ of\ 1^{st}\ weir\ \left({}^{mgO_2}/_{l}\right)$

$C_{u1} = Oxygen\ concentration\ upstream\ of\ 1^{st}\ weir\ \left({}^{mgO_2}/_{l}\right)$

The appropriate mathematical equation can now be linked to the system with the respective number of weirs, and thereafter, where required, a goal-seek function in the software model determines the oxygen concentration downstream of the first weir.

1-weir system $C_{d1} = C_{dz}$

2-weir system $C_{d1} = C_{dz} - Y$

3-weir system $C_{d1} = C_{dz} - Y - (C_s - Y) \times X$

4-weir system $C_{d1} = C_{dz} - V + Z$

5-weir system $C_{d1} = X - \dfrac{C_{dz} - V + Z}{C_s - V + Z}$

$C_{dz} = Final\ oxygen\ concentration\ \left({}^{mgO_2}/_{l}\right)$

The required fall height from one weir to the next, i.e. the distance between the top water level (TWL) of the upstream chamber and downstream one can now be determined for either horizontally placed rectangular weirs or v-notch weirs.

$$E = \frac{C_{d1} - C_{u1}}{C_s - C_{u1}}$$

$$f = 1.0 + 0.02103 \times (T - 20) + \frac{8.261 \times (T - 20)^2}{10^5}$$

$$E_{20} = 1 - (1 - E)^{\frac{1}{f}}$$

For v-notch weirs:

$$h = \left(\frac{[(1 - E_{20})^{-1} - 1] \times Q^{0.28} \times (sin\frac{\emptyset}{2})^{0.206}}{0.149}\right)^{\frac{1}{1.341}}$$

For horizontally placed rectangular weirs:

$$r_{20} = \frac{1}{1 - E_{20}}$$

$$q_j = \frac{Q}{2b_j}$$

$$h = \left[\frac{2^{0.11325} \times q_j^{0.2265} \times (r_{20} - 1)}{0.453 \times 0.667^{0.475} \times g^{0.11325}}\right]^{\frac{1}{1.45675}}$$

$E = Aeration\ efficiency\ at\ T$

$f = Temperature\ correction\ factor$

$E_{20} = Aeration\ efficiency\ at\ 20°C$

$h = fall\ height\ (m)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

$\emptyset = V - notch\ angle\ (°)$

$r_{20} = Oxygen\ deficit\ ratio\ at\ 20°C$

$q_j = Jet\ discharge\ per\ unit\ jet\ perimeter\ \left({}^{m^3}/_{m.s}\right)$

$b_j = Jet\ perimeter\ (m)$

$g = Gravity\ constant\ \left({}^{m}/_{s^2}\right)$

Ultimately the total fall height can be calculated.

$h_{tot} = h \times\ no\ of\ weirs$

$h_{tot} = Total\ fall\ height\ (m)$

As part of the design process the software model will alert the user if the desired oxygen concentration is higher than the saturation concentration and will instruct the user to choose a lower design value.

For calculating the fall height required over horizontal weirs it is assumed that the jet perimeter, used to determine the jet discharge per unit jet perimeter, where the water jet hits the downstream water, is equal to two (2) times the width of the weir. However, in reality this may differ depending on the height that the water drops from one chamber to the next. Future improvements of this section of the software model may therefore include more accurate determination of the jet perimeter.

Other additions to the software model can also include a review of the final oxygen concentration based on the minimum and maximum plant flowrates, as well as an increase in the allowable number of weirs.

3.1.3.2 Diffused Aeration

The design of a diffused aeration system comprising a compressor and an air receiver has been incorporated into the software model. The compressor units will stop and start as required to maintain a certain air pressure in the air receiver. From the air receiver, air is discharged to the point of release via a sparge manifold or diffuser unit. Pressure is required to overcome the friction losses inherent to the discharge pipework and equipment, as well as the static head resulting from the water depth. It is worth noting that a diffused aeration system can be used in conjunction with a surface aeration system or a standalone configuration.

To determine the compressor motor size, one must first calculate the free air demand, which can then be used together with the desired pressure in the air receiver and motor efficiency to derive the kilowatts (kW) required.

$$O_2 = \frac{Q \times 3600 \times 1000 \times C}{10^6}$$

$$n = \frac{O_2 \times 1000}{32}$$

$$V = \frac{nRT}{P}$$

$$FAD = \frac{V}{DE \times OC_{air}}$$

$$P_c = \frac{P_{increase} \times FAD}{\eta \times 3600 \times 1000}$$

$O_2 = Mass\ oxygen\ demand\ \left({}^{kgO_2}/_{h}\right)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

$C = Oxygen\ concentration\ \left({}^{mgO_2}/_{l}\right)$

$V = Oxygen\ volume\ \left({}^{m^3O_2}/_{h}\right)$

$n = Number\ of\ moles\ (mol)$

$R = Gas\ constant$

$T = Temperature\ (Kelvin)$

$P = Pressure\ (Pa)$

$DE = Dissolution\ efficiency\ (\%)$

$FAD = Free\ air\ demand\ \left({}^{m^3Air}/_{h}\right)$

$OC_{air} = Percentage\ of\ air\ comprising\ oxygen\ (\%)$

$P_c = Compressor\ motor\ size\ (kW)$

$P_{increase} = Pressure\ increase\ (mbar)$

$\eta = Motor\ Efficiency\ (\%)$

Using some of the parameters calculated above the volume of the air receiver can now be determined.

$$V = t \times \frac{FAD \times P_a}{(P_1 - P_2) \times 60}$$

$V = Volume\ of air\ receiver\ (m^3)$

$t = Time\ for\ pressure\ drop\ to\ occur\ (min)$

$P_a = Absolute\ atmosphere\ pressure\ (atm)$

$P_1 = Initial\ Receiver\ Pressure\ (atm)$

$P_2 = final\ receiver\ pressure\ (atm)$

The review section the software model will indicate if the dissolved oxygen required by the user exceeds the saturation concentration. This may lead to supersaturation, resulting in complications downstream.

The software model assumes that 21% of air consists of oxygen and a dissolution efficiency of 30%. Also, sizing the compressor motor does not consider the friction losses in the pipework between the compressor and the receiver, and this could be included in future versions of the software model.

3.2 Coagulation and Flocculation

3.2.1 Coagulation Process Description

In water treatment, coagulation can be described as the process that increases the tendency of small particles to attach to one another (to form bigger particles), as well as to the media in a filter bed (Letterman and Yiacoumi, 2011:Section 8.1). The coagulation process consists of four sequential steps, namely; "coagulant transformation, uptake of adsorbed species, particle destabilization, and interparticle collisions" (Letterman and Yiacoumi, 2011:Section 8.1.1). The first three steps occur immediately after the chemical disperses during rapid mixing. Interparticle collisions commence during rapid mixing and continues in the flocculation process (Letterman and Yiacoumi, 2011:Section 8.1.1) (described in more detail below).

A solution is said to be electrostatically stabilized when particles are resistant to aggregation because of electrostatic repulsive forces that keep the particles at a distance from one another. Most particles in water comprise a negative surface charge (Niehof and Loeb; Hunter and Liss, cited in Letterman and Yiacoumi, 2011:Section 8.3.1), and when two particles with the same charge approach each other, the electrostatic repulsive forces keep them from aggregating. The magnitude of the repulsive forces are a function of the particle's net surface charge and the distance between the particles, whereby the force increases with a higher net surface charge and a decrease in the distance between the particles.

On the other hand, attractive forces, called the London-van der Waals forces, exist between all particles, which are essentially independent of the water composition and depend on the type and number of atoms in the particles and in the water (Letterman and Yiacoumi, 2011:Section 8.3.1). The London-van der Waals forces increase with a decrease in the distance between two particles. Aggregation of particles typically occur when the attractive forces exceed the repulsive forces, and consequently the solution is termed destabilized.

Coagulation in water treatment employ coagulant chemicals to effect surface charge neutralization in order to destabilize colloidal particles in the water. Typical coagulant chemicals include chloride or sulfate salts that contain the metal ions Fe^{3+} or Al^{3+}, respectively (Letterman and Yiacoumi, 2011:Section 8.5.1), and these are commonly referred to as hydrolyzing metal salts (HMS). Alternatively, prehydrolyzed metal salts (PHMS), including polyaluminum chloride, which is made with aluminum chloride (Letterman and Yiacoumi, 2011:Section 8.5.1), are used.

When these chemicals are added to water they form hydrolysis products. The chemistry behind the formation of these products is complex and not completely understood, however, some researchers assume that in the case of aluminum sulphate ($Al_2(SO_4)_3$), commonly referred to as alum, one resulting hydrolysis product is $Al(OH)^{2+}$ (Bertsch and Parker, cited in Letterman and Yiacoumi, 2011:Section 8.5.1). This mononuclear hydrolysis product adsorbs onto the surface of the colloidal particle and neutralises its negative surface charge, and consequently its electrostatic repulsive force. When particles now collide the London-van der Waals forces dominate, and the particles start to aggregate to form flocs.

A cationic (comprising mostly positive sites) polyelectrolyte, often termed a flocculent-aid or floc-aid, is sometimes added to the water along with a HMS, and facilitates destabilization through interparticle bridging (Letterman and Yiacoumi, 2011:Section 8.4.3). The floc-aid comprises long thin molecules and when it encounters a negatively charged particle a portion of its surface area adsorb onto the particle surface. The remaining portion stretches out into the solution and interparticle bridging occurs when it adsorbs onto other particles as well (Letterman and Yiacoumi, 2011:Section 8.4.3).

Rapid mixing (sometimes called flash mixing) of the chemicals into the water is a critical step for effective coagulation. In the case of metal salts, the purpose is to quickly disperse the chemical across the process water before metal hydroxide precipitates are formed (Letterman and Yiacoumi, 2011:Section 8.6.1). It is therefore important that the dosing point be located closely upstream of the rapid mix structure. Further, a properly designed rapid mixing system will help ensure the coagulant is uniformly distributed throughout the particles in the water.

3.2.2 Flocculation Process Description

Flocculation refers to the physical process of creating an optimal environment for interparticle contacts and formation of particle aggregates. It involves a gentle mixing stage that follows the rapid mixing step described above. During flocculation aggregates are formed that can be removed in downstream processes, e.g. sedimentation and filtration (Letterman and Yiacoumi, 2011:Section 8.6.2).

In water treatment flocculation, the relative motion and collisions between particles can be described through the following mechanisms (Letterman and Yiacoumi, 2011:Section 8.6.3):

- *Brownian Diffusion*: Particles collide with water molecules which causes them to move about randomly and to come in contact with each other to form aggregates.
- *Transport in Laminar Shear*: Particles, that are close enough together, come into contact with one another (in a laminar flow field) when they are located across points with varying fluid velocities.
- *Differential Settling*: Particles with higher settling velocities tend to catch-up with other, slower particles, in the vertical direction, which causes the particles to collide and attach to one another.
- *Turbulent Transport*: Currents caused by the fluctuating motion of a fluid in turbulent flow result in the formation of eddies. These eddies have varying velocity gradients that promote interparticle collisions and consequently the formation of flocs.

3.2.3 Water Quality

Coagulation and flocculation is a prerequisite for sedimentation and flotation. It is also required for effective high rate granular media filtration. Whenever the water quality dictates the need for any of these processes it must be preceded by a properly designed and well operated coagulation and flocculation system.

Proper floc formation is often difficult to achieve in low turbidity waters (i.e. low solids concentration) because of the low concentration of particles that can collide to form bigger aggregates. For such waters, consideration must be given to add particles to the water so that aggregation can occur. In this regard bentonite is often the chemical of choice.

Serious consideration must be given to implement coagulation and flocculation where high concentrations of natural organic material (NOM) prevail. These process steps are very effective at removing significant fractions of NOM from the water (Letterman and Yiacoumi, 2011:Section 8.2.1).

3.2.4 Coagulation Design Approach

The first step in designing a coagulation system is determining which coagulant chemicals to use and how much thereof, i.e. the dosing rate, is needed. Many models have been developed to predict the best suited coagulant as well as estimating the required dosage. However, calculations pertaining to these models have not been included in this dissertation. Instead it is recommended that a treatability test be performed with at least the most common of coagulants, i.e. chloride and sulphate salts, as well as with a PHMS that is readily available or already in use by the water supply authority at other treatment works.

The second step is to design a proper rapid mix system that will allow for quick and uniform dispersion of the coagulant chemicals. There are various types of rapid mix technologies available in water treatment, ranging from hydraulic mixing, diffusion mixing, in-line static- or mechanical mixing, surface mechanical mixing, pipe diffusion to mechanical flash mixing. The software model, however, only allows for the design of a hydraulic- and pump diffusion mixing system.

3.2.4.1 Hydraulic Mixing System

Hydraulic mixing devices, include, but are not limited to, venturi meters, parshall flumes, and cascading weirs. This dissertation focusses on the design of the last mentioned, because of all the devices considered this one best accommodates fluctuations in the hydraulic feed. This system comprises a level weir that is installed perpendicular to the flow in a channel or chamber and is followed downstream by a defined, and specifically sized, chamber. The water level upstream of the weir is higher than the downstream water level because of the hydraulic headloss created across the weir. A coagulant chemical is dosed at the point where the water plunges from the higher to the lower level, which creates a mixing effect that disperses the chemical.

The mixing energy, created by the weir, is quantified in terms of the velocity gradient and multiplying it with the mixing time results in a certain Gt value. A hydraulic mixing system is designed based on specific velocity gradient and Gt values.

In designing a cascading weir system, one must first determine the Gt value and the downstream chamber volume, based on a desired or chosen velocity gradient.

$$Gt_{des} = G_{des} \times t_{des}$$

$$V_{chamber} = Q \times t_{des}$$

$Gt_{des} = Gt\ value\ at\ design\ flowrate$

$G_{des} = Velocity\ gradient\ at\ design\ flowrate\ (s^{-1})$

$t_{des} = Retention\ time\ at\ design\ flowrate\ (s)$

$V_{chamber} = Volume\ of\ chamber\ downstream\ of\ weir\ (m^3)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

With the volume of the downstream chamber known, the power (Kawamura, 2000:88) and required headloss across the weir can now be determined.

$$P = G_{des}^2 \times \mu \times V_{chamber}$$

$$h = \frac{P}{g \times \rho \times Q}$$

$P = Power\ (Watts)$

$\mu = Absolute\ viscosity\ \left({}^{Ns}/_{m^2}\right)$

$h = headloss\ across\ weir\ (m)$

$g = Gravity\ constant\ \left({}^{m}/_{s^2}\right)$

$\rho = Water\ density\ \left({}^{kg}/_{m^3}\right)$

Finally, the position of the weir, relative to the downstream water level, is determined. The software model first establishes whether the weir must be a submerged or elevated weir. Following, it applies the appropriate formula for determining the weir level/vertical position.

For elevated weirs:

$$h_{el} = h - \left(\frac{3 \times Q}{C_D \times \sqrt{2 \times g} \times B \times 2}\right)^{\frac{2}{3}}$$

$h_{el} = Weir\ level\ above\ downsteam\ water\ level\ (m)$

$C_D = Discharge\ coefficient$

$B = Weir\ width\ (m)$

For submerged weirs:

$$h_{sub} = \frac{Q - \frac{2}{3} \times C_D \times B \times \sqrt{2 \times g} \times h^{\frac{3}{2}}}{C_D \times B \times \sqrt{2 \times g \times h}}$$

$h_{sub} = Weir\ level\ below\ downstream\ water\ level\ (m)$

Once the cascading weir system is designed it must be checked against the minimum and maximum flowrates, and whether the velocity gradient and Gt values remain within the allowable ranges. For minimum flow conditions the maximum retention time is calculated first.

$$t_{max} = \frac{V_{Chamber}}{Q_{min}}$$

$$t_{max} = Rention\ time\ at\ minimum\ flowrate\ (s)$$

$$Q_{min} = Minimum\ flowrate\ \left({}^{m^3}\!/_{s}\right)$$

If the weir was determined to be elevated, then the software model uses the equation below to determine the minimum headloss.

$$h_{min} = h_{el} + \left(\frac{3 \times Q_{min}}{C_D \times \sqrt{2\ \times g} \times B \times 2}\right)^{\frac{2}{3}}$$

$$h_{min} = Headloss\ across\ weir\ at\ minimum\ flowrate\ (m)$$

If the weir was determined to be submerged, then the submerged weir equation is set to zero and a goal seek function is used to solve for the minimum headloss.

$$0 = Q_{min} - \left(\frac{2}{3} \times C_D \times B \times \sqrt{2\ \times g} \times h_{min}^{\frac{3}{2}} + C_D \times B \times h_{sub} \times \sqrt{2 \times g \times h_{min}}\right)$$

The minimum headloss can now be used for deriving the minimum velocity gradient and Gt value.

$$P_{min} = h_{min} \times Q_{min} \times g \times \rho$$

$$P_{min} = Power\ at\ minimum\ flowrate\ (Watts)$$

$$G_{min} = \sqrt{\frac{P_{min}}{\mu \times V_{chamber}}}$$

$$G_{min} = Velocity\ gradient\ at\ mimimum\ flowrate\ (s^{-1})$$

$$Gt_{min} = G_{min} \times t_{max}$$

$$Gt_{min} = Gt\ value\ at\ minimum\ flowrate$$

The same process is followed for maximum flowrate conditions and the software model alerts the user if any of the ranges indicated in Table 5 are exceeded.

Table 5: Design ranges for hydraulic mixing

Description	Minimum	Maximum	Reference
Mixing time (s)	1	5	Kawamura (2000:88)
Velocity gradient (s^{-1})	600	1000	Delphos and Letterman (2012:Section 7.4.3.1)
Gt	300	1600	Kawamura (2000:80)

It is worth noting that the discharge coefficient is fixed at a value of 0.6, which is typically used in this type of sharp-crested weir system. However, to obtain a more accurate number the following equation may be applied:

$$C_D = 0.602 + 0.083 \times \frac{h_o}{P}$$

$$h_o = Head\ over\ the\ weir\ (m)$$

$$P = Height\ of\ weir\ above\ the\ channel\ invert\ (m)$$

The above formula may be incorporated in future versions of the software model. Further, it is assumed that the mixing time is equal to the retention time of the downstream chamber, which in practice, and specifically for low flows and large chambers, may not always be the case, resulting in distorted theoretical Gt values. Additional investigation and research is required to determine the actual mixing time and it could be based on the retention time of a hydraulic jump formed downstream of the weir. This may be added to future versions of the design model.

3.2.4.2 Pump Diffusion Mixing System

In pump diffusion mixing, a pump system is used to pump a relatively small stream of water against the plant flow direction. The pumped water is forced through a specially designed nozzle to create a cone like spray pattern against the current of the main treatment stream. This creates a mixing effect and with the coagulant injected just upstream (almost at the nozzle release point) allows for proper and uniform chemical dispersion.

The biggest advantage with this system is that its degree of mixing can be adjusted to account for plant flow variation. Also, there is minimum headloss across the system and it consumes less power than, for example, a mechanical flash mixing system. In addition, when retrofitting an existing works, it can be installed relatively easily within an existing treatment train, as minimum civil infrastructure is required.

This dissertation's approach for designing a pump diffusion system is based on methods developed by Kawamura (2000:94-96) and commences with establishing a retention time, based on a chosen velocity gradient and average Gt value.

$$t_{des} = \frac{Gt_{des}}{G_{des}}$$

$t_{des} = Retention\ time\ at\ design\ flowrate\ (s)$

$Gt_{des} = Gt\ value\ at\ design\ flowrate$

$G_{des} = Velocity\ gradient\ at\ design\ flowrate\ (s^{-1})$

Using the retention time, the volume of the mixing zone can be determined. This volume then fixes the required pipe size, as well as the mixing zone length.

$$V_{mix} = Q \times t_{des}$$

$$D = \left(\frac{V_{mix} \times 4}{1.5 \times \pi}\right)^{\frac{1}{3}}$$

$$L_{mix} = 1.5 \times D$$

$V_{mix} = Volume\ of\ mixing\ zone\ (m^3)$

$Q = Design\ flowrate\ \left({}^{m^3}/{}_{s}\right)$

$D = Pipe\ Diameter\ (m)$

$L_{mix} = Mixing\ zone\ length\ (m)$

Ultimately the pump motor size can be established from the pump efficiency and duty point, i.e. the total head the pump must overcome and the flowrate. Where the pump flowrate is based on a chosen percentage of the overall plant flow and the total head comprise the losses through the delivery pipework and the nozzle, as well as the dynamic headloss.

$$Q_{pump} = r \times Q$$

$$P = G_{des}^2 \times \mu \times V_{mix}$$

$$h_{dyn} = \frac{P}{Q_{pump} \times g \times \rho}$$

$$d_o = 2 \times \sqrt{\frac{Q_{pump}}{C_D \times \pi \times \sqrt{2 \times g \times h_{dyn}}}}$$

$$h_{tot} = h_{nozzle} + h_{pipe} + h_{dyn}$$

$$P_{h(kW)} = \frac{Q_{pump} \times g \times \rho \times h_{tot}}{1000 \times \eta}$$

$Q_{pump} = Pump\ flowrate\ \left({}^{m^3}/{}_{s}\right)$

$r = Percentage\ of\ design\ flowrate\ (\%)$

$P = Water\ horsepower\ (Watts)$

$\mu = Absolute\ viscosity\ \left({}^{Ns}/{}_{m^2}\right)$

$d_o = Nozzle\ oriface\ diameter\ (m)$

$C_D = Discharge\ coefficient$

$h_{dyn} = Dynamic\ headloss\ (m)$

$g = Gravity\ constant\ \left({}^{m}/{}_{s^2}\right)$

$\rho = Water\ density\ \left({}^{kg}/{}_{m^3}\right)$

$h_{tot} = Pump\ head\ duty\ point\ (m)$

$h_{nozzle} = Nozzle\ headloss\ (m)$

$h_{pipe} = Delivery\ pipework\ headloss\ (m)$

$P_{h(kW)} = Pump\ Motor\ Size\ (kW)$

$\eta = Pump\ motor\ efficiency\ (\%)$

Before selecting a pump, it is important to consider the pump's duty range for achieving the desired mixing between the minimum and maximum plant flowrates. An appropriately selected pump will operate efficiently across all duty points. The duty point associated with minimum plant flow, and the corresponding minimum velocity gradient and Gt values are calculated as follows.

$$Q_{pump,min} = r_{min} \times Q_{min}$$

$$h_{dyn,min} = \frac{Q_{pump,min}^2}{2 \times g \times \left(C_D \times \pi \times \left(\frac{d_o}{2}\right)^2\right)^2}$$

$$h_{nozzle,min} = \frac{h_{nozzle} \times Q_{pump,min}}{Q_{pump}}$$

$$h_{p,min} = \frac{h_{pipe} \times Q_{pump,min}}{Q_{pump}}$$

$$h_{tot,min} = h_{nozzle,min} + h_{p,min} + h_{dyn,min}$$

$$P_{min} = Q_{pump,min} \times \rho \times g \times h_{dyn,min}$$

$$G_{min} = \sqrt{\frac{P_{min}}{\mu \times V_{mix}}}$$

$$t_{max} = \frac{V_{mix}}{Q_{min}}$$

$$Gt_{min} = G_{min} \times t_{max}$$

$Q_{pump,min} = Pump\ flow\ at\ minimum\ flowrate\ \left(m^3/_s\right)$

$r_{min} = \ Portion\ of\ minimum\ flowrate\ (\%)$

$Q_{min} = Minimum\ flowrate\ \left(m^3/_s\right)$

$h_{dyn,min} = Dynamic\ headloss\ at\ minimum\ flowrate\ (m)$

$h_{nozzle,min} = Nozzle\ headloss\ at\ minimum\ flowrate\ (m)$

$h_{p,min} = Delivery\ pipework\ headloss\ at\ min\ flowrate\ (m)$

$h_{tot,min} = Pump\ head\ duty\ point\ at\ min\ flowrate\ (m)$

$P_{min} = Power\ at\ minimum\ flowrate\ (Watts)$

$G_{min} = Velocity\ gradient\ at\ mimimum\ flowrate\ (s^{-1})$

$t_{max} = Rention\ time\ at\ minimum\ flowrate\ (s)$

$Gt_{min} = Gt\ value\ at\ minimum\ flowrate$

The same approach is followed for the calculating the above values for the maximum plant flowrate and the resulting Gt values are tested against the allowable range of between 400 and 1600 (Kawamura, 2000:88).

Further, it is assumed that the increase/decrease of losses through the nozzle and delivery pipework is directly proportional to flow. Future improvements to the software model may therefore include:

- deriving the discharge coefficient from first principle equations;
- linking the losses through the nozzle to information from specific suppliers; and
- calculating the headloss through the delivery pipework.

Further the acceptable ranges for the retention time and velocity gradient should also be investigated and built into model, as well as the acceptable flow velocity through the nozzle orifice.

3.2.5 Flocculation Design Approach

In water treatment there exist mainly two types of flocculation systems – those that rely on mechanical mixing and those that rely on flow hydraulics. Both types include several stages and the focus of design is to taper the velocity gradient, and consequently the Gt value, from high to low through these stages. This must be done whilst keeping within the acceptable design ranges for the various flow conditions.

It should further be noted that the acceptable velocity gradient and Gt values described below are based on a conventional treatment process, which include sedimentation followed by filtration. The formation of large flocs with a higher specific gravity (SG) are required for settling, and the smaller, more stable flocs that reach the filters are removed when they are adsorbed onto the filter media. In plants where sedimentation is omitted, and flocculated water is put directly onto the filters a smaller and stronger/stable floc is desired that is less prone to hydraulic forces shearing off parts of it that can pass through a filter bed. For direct filtration plants the retention time is reduced, to about 10 minutes, to prevent the formation of large unstable flocs.

3.2.5.1 Hydraulic Systems

Baffle channel flocculation systems rely on flow hydraulics. The two main configurations are round-the-end baffled channel type and over- and under-baffled channel type. The first mentioned is found more commonly as it is less susceptible to sediment accumulation upstream of the baffle wall, and was therefore applied in the software model

Round-the-end baffled channel type flocculators comprise a series of channels, each equipped with appropriately spaced baffle walls that protrude into the normal direction of flow to create a snake like flow pattern. The spacing and slit opening (the space between the end of the baffle wall and adjacent side of the channel) of the baffle walls are what drives the headloss across the system, which induces a specific velocity gradient. A series of channels with the same baffle wall spacing and slit openings are said to make up a flocculation stage, typically comprising a minimum of two channels. The baffle wall spacing and slit openings are increased from the upstream to the downstream stages so that the velocity gradient tapers off. The design approach discussed below is based on methods developed by Kawamura (2000:134-136).

The volume and retention time of each stage is based on the plant flowrate, the number of flocculation systems, the number of stages, and the retention time of each system.

$$V_{st} = \frac{Q \times t_{sys} \times 60}{n_{st} \times n_{sys}}$$

$$t_{st} = \frac{V_{st} \times n_{sys}}{Q}$$

$V_{st} = Active\ volume\ of\ stage\ (m^3)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

$t_{sys} = System\ retention\ time\ at\ design\ flowrate\ (min)$

$n_{st} = Number\ of\ stages\ (No)$

$n_{sys} = \text{Number of systems}\ (No)$

$t_{st} = Retention\ time\ of\ stage\ at\ design\ flowrate\ (s)$

The corresponding Gt value and required headloss across each stage can then be determined based on a chosen velocity gradient.

$$Gt_{st} = G_{st} \times t_{st}$$

$$h_{st} = \frac{G_{st}^2 \times \mathrm{v} \times V_{st} \times n_{sys}}{g \times Q}$$

$Gt_{st} = Gt\ value\ of\ stage\ at\ design\ flowrate$

$G_{st} = Velocity\ gradient\ of\ stage\ at\ des\ flowrate\ (s^{-1})$

$h_{st} = Headloss\ across\ stage\ at\ design\ flow\ rate\ (m)$

$\mathrm{v} = Kinematic\ viscosity\ \left({}^{m^2}/_{s}\right)$

$g = Gravity\ constant\ \left({}^{m}/_{s^2}\right)$

As mentioned above there should ideally be a minimum of two (2) channels per stage, but it is also good practice to have a minimum of six (6) channels per system. As such, the software model will derive the number of channels as follows to adhere to these criteria.

$$n_{ch} = n_{st} \times 3, if\ n_{st} = 2$$

$$n_{ch} = n_{st} \times 2, if\ n_{st} > 2$$

$n_{ch} = Number\ of\ channels\ (No)$

Now the baffle spacing and slit opening can be derived from the total number of channels and the chosen number of compartments, which is defined as the area between two baffle walls.

$$S_{baf} = \frac{l_{act} \times n_{ch}}{n_{st} \times n_{com}}$$

$$v = \sqrt{\frac{h_{st} \times 2 \times g}{n_{com} \times 1.5}}$$

$$S_{slit} = \frac{Q}{n_{sys} \times v \times \left(h_{out} + h_{down} + \frac{h_{st}}{2}\right)}$$

$l_{act} = Length\ of\ active\ storage\ (m)$

$h_{out} = \text{Water depth at outlet}\ (m)$

$S_{baf} = Baffle\ spacing\ (m)$

$n_{com} = Number\ of\ compartments\ in\ stage\ (No)$

$v = Flow\ vel\ through\ slit\ at\ design\ flow\ \left({}^{m}/_{s}\right)$

$S_{slit} = Slit\ opening\ (m)$

$h_{down} = Headloss\ across\ downstream\ stages\ (m)$

Hydraulic flocculation systems are typically sensitive to flow variation and therefore several checks and balances are required to ensure the system doesn't fail under the envisaged maximum and minimum flowrates. Parameters that are calculated and checked in the software model include the velocity gradient, retention time, Gt value and flow velocities through the slits for maximum and minimum flowrates.

$$t_{st,max} = \frac{V_{st} \times n_{act,sys}}{Q_{min}}$$

$$v_{min} = \frac{Q_{min}}{n_{act,sys} \times \left(h_{out} + h_{down} + \frac{h_{st}}{2}\right) \times S_{slit}}$$

$$h_{st,min} = \frac{n_{com} \times v_{min}^2 \times 1.5}{2 \times g}$$

$$G_{st,min} = \sqrt{\frac{h_{st,min} \times g}{t_{st,max} \times \mathrm{v} \times 10^{-6}}}$$

$$Gt_{st,min} = G_{st,min} \times t_{st,max}$$

$t_{st,max} = Retention\ time\ of\ stage\ at\ min\ flowrate\ (s)$

$n_{act,sys} = Number\ of\ active\ systems\ (No)$

$Q_{min} = Minimum\ flowrate\ \left({m^3}/{s}\right)$

$v_{min} = Flow\ vel\ through\ slit\ at\ min\ flowrate\ ({m}/{s})$

$h_{st,min} = Headloss\ of\ stage\ at\ min\ flowrate\ (m)$

$G_{st,min} = Vel\ gradient\ of\ stage\ at\ min\ flowrate\ (s^{-1})$

$Gt_{st,min} = Gt\ value\ of\ stage\ at\ min\ flowrate$

As can be seen above the parameters under minimum flow conditions are defined by the number of active systems. This allows the user to isolate one or more system(s) to meet the design criteria. A properly designed system will meet all the parameters as listed in Table 6.

Table 6: Round-the-end baffled channel design parameters

Description	Range	Reference/comments
Velocity gradient (s^{-1})	10 – 50	Kawamura (2000:118,121)
Retention time (minutes)	30 – 45	
Gt value	30,000 – 200,000	
Flow velocity (m/s)	> 0.9144	
Baffle spacing	> 0.75	For easy cleaning of channel

The above hydraulic flocculation model is designed based on the assumption that the headloss through a slit opening is based on a fixed k-value of 1.5 (2 x 90° bends of k-value = 0.75 each). Future improvements to the model could therefore include deriving a more accurate k-value.

3.2.5.2 Mechanical Flocculation Systems

Mechanical flocculation systems are made up of a number of tanks, placed in series so that water flows from one to the next. Each tank is fitted with one or more mechanical mixers that rotate at a certain frequency to induce a slow and steady mixing effect. The mixer shafts can either be installed in the vertical or the horizontal position. This dissertation however only considers flocculation systems with vertical shaft mixers.

Typically, all tanks that are part of a system are the same size and each tank represents a stage in the system, linked to a specific velocity gradient. The velocity gradient is tapered off through the stages in the direction of flow by reducing the rotational speed of the downstream mixer.

The tanks are separated by baffle walls with carefully spaced circular openings, or orifices. These baffle walls can be designed using the same approach as described in Section 3.3 below.

A mechanical flocculation system can be designed by first choosing the number of stages and then the velocity gradient linked to each stage. Subsequently the volume and retention time of each tank can be determined, which result in a certain Gt value. Once the Gt value is fixed it can be used to calculate the mixer's motor size.

$$V_{st} = \frac{Q \times t_{sys} \times 60}{n_{sys} \times n_{st}}$$

$$t_{st} = \frac{t_{sys} \times 60}{n_{st}}$$

$$Gt_{st} = G_{st} \times t_{st}$$

$$P_{kW} = \frac{G_{st}^2 \times \mu \times V_{st}}{1000 \times \eta \times n_{mix}}$$

$V_{st} = Active\ volume\ of\ stage\ (m^3)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

$t_{sys} = System\ retention\ time\ at\ design\ flowrate\ (s)$

$n_{sys} = \text{Number of systems}\ (No)$

$n_{st} = Number\ of\ stages\ (No)$

$t_{st} = Retention\ time\ of\ stage\ at\ design\ flowrate\ (s)$

$Gt_{st} = Gt\ value\ of\ stage\ at\ design\ flowrate$

$G_{st} = Velocity\ gradient\ of\ stage\ at\ des\ flowrate\ (s^{-1})$

$P_{kW} = Mixer\ motor\ size\ (kW)$

$\mu = Absolute\ viscosity\ \left({}^{Ns}/_{m^2}\right)$

$\eta = Mixer\ motor\ efficiency\ (\%)$

$n_{mix} = Number\ of\ mixers\ per\ stage\ (No)$

To account for flow variation the number of active systems as well as the mixer speeds can be adjusted, until all the design parameters as listed in Table 7 are satisfied. The equations below indicate how changing these two parameters affect the system under minimum flow conditions. Similarly, the software model performs the same analysis under maximum flow conditions. However, the latter is based on the actual number of systems as it is assumed during maximum flow periods all systems will be active.

$$P_{kW,min} = P_{kW} \times r_{min}$$

$$G_{st,min} = \sqrt{\frac{P_{kW,min} \times 1000 \times n_{mix}}{\mu \times V_{st}}}$$

$$t_{st,max} = \frac{V_{st} \times n_{act,sys}}{Q_{min}}$$

$$Gt_{st,min} = G_{st,min} \times t_{st,max}$$

$P_{kW,min} = Mixer\ motor\ power\ at\ \min flowrate\ \ (kW)$

$r_{min} = \ Pecrtentage\ of\ mixer\ motor\ power\ (\%)$

$G_{st,min} = Velocity\ grad\ of\ stage\ at\ min\ flowrate\ (s^{-1})$

$t_{st,max} = Retention\ time\ of\ stage\ at\ min\ flowrate\ (s)$

$n_{act,sys} = Number\ of\ active\ systems\ (No)$

$Q_{min} = Minimum\ flowrate\ \left({}^{m^3}/_{s}\right)$

$Gt_{st,min} = Gt\ value\ of\ stage\ at\ min\ flowrate$

Table 7: Mechanical flocculation design parameters

Description	**Range**	**Reference/comments**
Velocity gradient (s^{-1})	10 – 70	Kawamura (2000:121)
Retention time (minutes)	20 – 40	
GT value	12,000 – 168,000	

Future improvements of the software model may include:

- design of the baffle walls between the various stages; and
- deriving the tank geometry based on best practice length to width to depth ratios.

3.3 Dissolved Air Flotation

3.3.1 Process Description

Dissolved air flotation (DAF), like sedimentation, is also a solid-water separation (clarification) process (Gregory and Edzwald, 2011:Section 9). It involves the injection of a small stream of air saturated water, called the recycle flow, into the main process stream. The recycle flow can be abstracted directly after of the DAF system or further down in the process, for example downstream of filtration (Gregory and Edzwald, 2011:Section 9.4.3). Once abstracted the water is pumped through a pressure vessel, called a saturator, where air gets injected and mixed into the water. The air is typically supplied by a set of compressors or blowers.

The DAF system consists of two zones namely, a contact zone and a separation zone (Gregory and Edzwald, 2011:Section 9.5.2). The contact zone is located upstream of the separation zone. The air saturated water is injected at the bottom of the contact zone through a set of special nozzles or valves to form a curtain of microbubbles along the width of the DAF tank. The mixture of bubbles and water is commonly referred to as white water, given its milky like appearance (Gregory and Edzwald, 2011:Section 9.4.3).

Like sedimentation DAF is also preceded by coagulation and flocculation (Gregory and Edzwald, 2011:Section 9.4.3), though in the case of DAF the design parameters for these process steps are slightly different, mainly to achieve a different floc size. Floc particles, from the flocculation process step, enter the contact zone where they collide with and attach to the microbubbles to form floc-bubble aggregates. These floc-bubble aggregates typically have a lower density than water, which, causes them to float to the surface (Dowbiggin and Breese, 2012:Section 8.5.2) at a specific floc-bubble rise velocity.

Water exits the DAF system near the bottom of the tank and as such a continuous downward flowrate, called the hydraulic loading rate, is introduced. It is thus clear to see that conversely to sedimentation, which relies on settling and overflow for solid-water separation, the DAF system relies on flotation and underflow withdrawal (subnatant) to clarify the water. And in the case of DAF, solid-water separation is typically achieved in the separation zone when the floc-bubble rise velocity exceeds the hydraulic loading rate (Gregory and Edzwald, 2011:Section 9.5.4).

3.3.2 Water Quality

DAF is suitable for treating low-turbidity and low-alkalinity coloured waters (Zabel and Melbourne, cited in Gregory and Edzwald, 2011:Section 9.4.1), because the floc particles formed after flocculation of such waters are typically small and have a low settling rate. Therefore, suitably sized microbubbles that attach to these particles can allow them to float to the surface.

DAF is also effective at removing algae. Some algae species tend to float by themselves and are therefore easily scraped off at the surface of the DAF tank. Other algae species have low densities and will float easily once a bubble attachment is formed.

3.3.3 Design Approach

In this section the design approach of the DAF system is discussed. The various design steps and equations, defined below, are based on those developed by Gregory and Edzwald (2011:Section 9.5).

The floc-bubble rise velocity is calculated from the following equations.

$$N_{max} = \left(\frac{d_f}{d_b}\right)^2$$

$$N_{ab} = \frac{N_{max}}{2}\ , N_{ab} \geq 1$$

$$d_{pb} = \left[d_f^3 + N_{ab} \times (d_b)^3\right]^{\frac{1}{3}}$$

$N_{max} = Maximum\ number\ of\ bubbles\ per\ floc\ (No)$

$d_f = Floc\ diameter\ (\mu m)$

$d_b = Bubble\ diameter\ (\mu m)$

$N_{ab} = Actual\ number\ of\ bubbles\ per\ floc\ (No)$

$$\rho_{pb} = \frac{\rho_p \times d_p^3 + N_{ab} \times (\rho_b \times d_b^3)}{d_p^3 + N_{ab} \times d_b^3}$$

$$v_{fb} = \frac{g \times (\rho_w - \rho_{pb}) \times (d_{pb} \times 10^{-6})^2 \times 3600}{18 \times \mu}$$

$d_{pb} =$ *Floc-bubble diameter* (μm)

$\rho_{pb} =$ *Floc-bubble density* $\left(\frac{kg}{m^3}\right)$

$\rho_p =$ *Floc density* $\left(\frac{kg}{m^3}\right)$

$\rho_b =$ *Bubble density* $\left(\frac{kg}{m^3}\right)$

$v_{fb} =$ *Floc-bubble rise velocity* $\left(\frac{m}{h}\right)$

$\rho_w =$ *Water density* $\left(\frac{kg}{m^3}\right)$

$\mu =$ *Absolute viscosity* $\left(\frac{10^{-3} N.s}{m^2}\right)$

The area, as well as the length and width of the separation zone is determined based the following equations.

$$Q_r = Q \times R_r$$

$$Q_{tot} = \frac{Q + Q_r}{n}$$

$$A_{sz} = \frac{Q_{tot} \times 3600}{v_{sz-hl}}$$

$$L_{sz} = \sqrt{A_{sz} \times R_{l/w}}$$

$$W_{sz} = \frac{L_{sz}}{R_{l/w}}$$

$Q_r = Recycle\ flow\ \left(\frac{m^3}{s}\right)$

$Q = Design\ flow\ \left(\frac{m^3}{s}\right)$

$R_r = Recycle\ ratio\ (\%)$

$Q_{tot} = Total\ inflow\ per\ tank\ \left(\frac{m^3}{s}\right)$

$n = Number\ of\ tanks\ (No)$

$A_{sz} = Separation\ zone\ area\ (m^2)$

$v_{sz-hl} = Separation\ zone\ hydraulic\ loading\ rate\ \left(\frac{m}{h}\right)$

$L_{sz} = Separation\ zone\ length\ (m)$

$R_{l/w} = Length\ to\ width\ ratio$

$W_{sz} = Separation\ zone\ width\ (m)$

The volume and dimensions of the contact zone is calculated, using the following equations.

$$V_{cz} = Q_{tot} \times 60 \times t_{cz}$$

$$L_{cz} = \frac{V_{cz}}{W_{sz} \times d}$$

$V_{cz} = Contact\ zone\ volume\ (m^3)$

$t_{cz} = Contact\ zone\ retention\ time\ (min)$

$L_{cz} = Contact\ zone\ length\ (m)$

$d = Tank\ depth\ (m)$

Lastly, the software model calculates the hydraulic loading rate of the contact zone using the following equations:

$$A_{cz} = L_{cz} \times W_{cz}$$

$$v_{cz-hl} = \frac{Q_{tot} \times 3600}{A_{cz}}$$

$A_{cz} = Contact\ zone\ area\ (m^2)$

$v_{cz-hl} = Contact\ zone\ hydraulic\ loading\ rate\ \left(\frac{m}{h}\right)$

The following assumptions and fixed values are built into the DAF section of the software model:

- A minimum of one bubble will attach to a floc particle, i.e. the actual number of bubbles per floc has been set to always default to a minimum of 1 if the calculated value is less than 1.5.

- The mean density of all floc particles is 1100 kg/m^3 (Gregory and Edzwald, 2011:Section 9.5.2).
- The density and absolute viscosity of the water, as well as the bubble density varies based on the temperature, but a constant pressure of 1atm is assumed.

The software model will establish if the following design requirements have been met:

- The floc-bubble rise velocity must be larger than the hydraulic loading rate (Gregory and Edzwald, 2011:Section 9.5.4) .
- The length of the contact zone must be larger than 1 meter for easy maintenance.
- The contact zone hydraulic loading rate must be less than 200 m/h (Gregory and Edzwald, 2011:Section 9.6.1).

It is foreseen that the software model be improved in future by adding the following:

- Approximate sizing of the recycle pumps and compressors/blowers.
- Approximate sizing of the saturator
- Allowing for variance in the water and air density, as well as the absolute viscosity of water based on the prevailing pressure conditions.

3.4 Sedimentation

3.4.1 Process Description

Sedimentation is a solid-water separation process (Gregory and Edzwald, 2011:Section 9), whereby a suspension is divided into a clarified liquid and resulting suspension with an increased concentration (Kawamura, 2000:139). In this dissertation the sedimentation process is separated into two classifications, namely grit chambers and sedimentation tanks, and the criteria of each is based on the "specific gravity, quantity and size" (Kawamura, 2000:139) of the particles to be separated from the liquid.

3.4.2 Grit Chambers

Grit chambers are typically considered when water is taken from a river that is prone to episodes of flash floods and should be installed near the intake (Kawamura, 2000:139). The purpose of the grit chamber is to remove nonflocculable particles such as sand and silt (Kawamura, 2000:141). This suggests that particles are removed through Class I settling.

Class I settling refers to the settling of nonflocculable particles, which are "unhindered by the presence of other settling particles" (Rich, 1961:82). Therefore, the settling characteristics is a function of the properties of the individual particles and that of the surrounding fluid (Rich, 1961:82).

Class I settling can further be described by considering an ideal settling basin that comprises a rectangular horizontal flow basin, divided into four zones as shown in Figure 2.

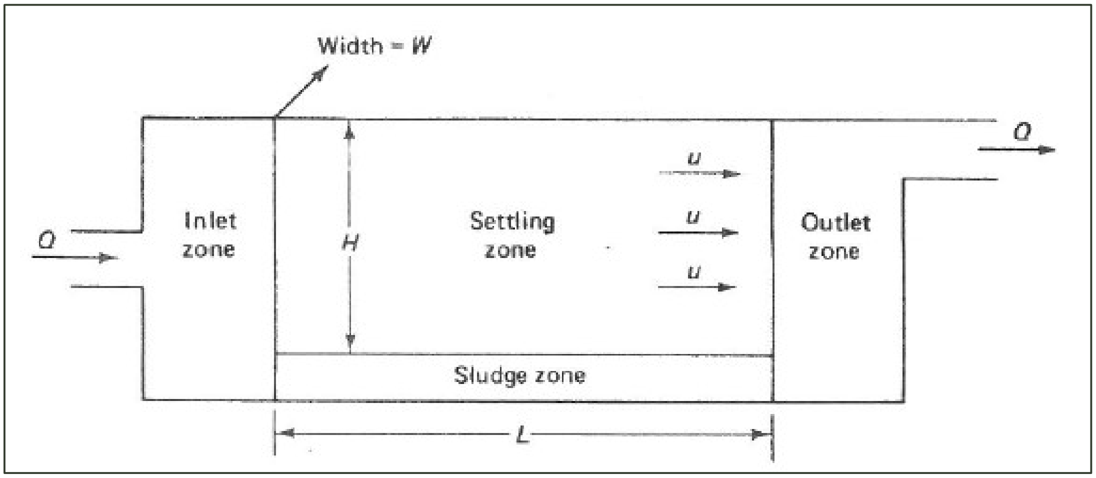


Figure 2: Schematic diagram of a rectangular horizontal flow settling basin (Sundstrom and Klei, 1979:191)

Suspended particles are evenly dispersed in the inlet zone so that the concentration of each size particle is uniform across the vertical cross-section at the settling zone inlet. In the settling zone the particles settle along a straight line, governed by the sum of their horizontal (*u*) and vertical (v_0) velocity vectors, as shown in Figure 3 (Sundstrom and Klei, 1979:191). The horizontal velocity is governed by the horizontal fluid velocity and is considered to be the same for all particles. The length of the settling and sludge zone (*L*) is determined by the particles with the lowest vertical velocity that must be removed, and that enters the settling zone at the top (*H*). Those particles with equal to or greater settling velocities will therefore also reach the bottom inside the settling zone. Particles with lower settling velocities (v_s) will reach the bottom of the settling zone, provided they enter at a low enough depth (*h*). The portion (F_x) of these particles that will be removed can be expressed as follows:

$$F_x = \frac{h}{H} = \frac{v_s t_0}{v_0 t_0} = \frac{v_s A}{Q} \qquad Where: \quad v_0 = \frac{H}{t_0} = \frac{Hu}{L} = \frac{HuW}{LW} = \frac{Q}{A}$$

From the above equation it can be seen that Class I settling is independent from the tank depth. Whereas a decrease in tank depth will result in an increase in the horizontal fluid velocity, which in turn is balanced by the shorter distance the particles must travel to reach the bottom of the settling zone. It is however important to note that at a certain point the increase in horizontal fluid velocity may cause settled sludge to shear away from the sludge zone, causing them to be resuspended (Sundstrom and Klei, 1979:195).

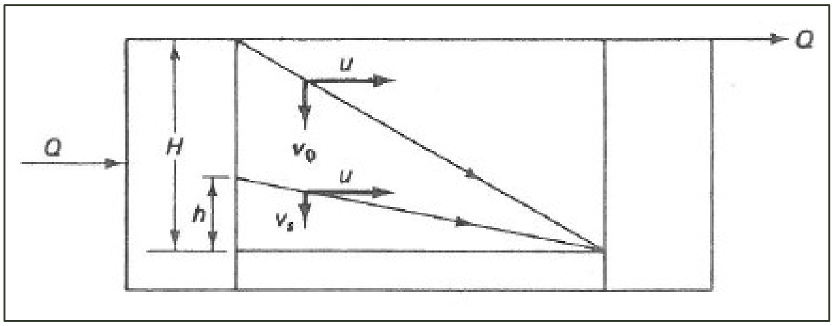


Figure 3: Settling paths of discrete particles in a rectangular basin (Sundstrom and Klei, 1979:191)

3.4.3 Sedimentation Tanks

The purpose of the sedimentation process is to remove settleable solids via gravitational settling, and for sedimentation tanks proper upstream coagulation and flocculation required (Kawamura, 2000:139). As such, sedimentation tanks are subject to Class II settling. However, depending on the properties of the particles that enter the tank, Class I settling will also occur to some extent.

In Class II settling, particles collide and coalesce to form larger particles. The settling velocity is determined by the concentration and properties of the individual and grouped together particles. It is further influenced by the tendency of the particles to absorb onto one another, based on their surface charge, and the surrounding flow characteristics, which causes them to collide. Under quiescent conditions collisions mainly occur because of heavier particles, with higher settling velocities, catching up and colliding with lighter and slower particles on their way down. Each time a collision is made the weight and settling velocity of the parent particle (Rich, 1961:91) increases and consequently its tendency to collide with slower particles further down. In contrast to Class I settling, Class II settling is therefore dependent on the depth of the basin. Under normal conditions, velocity gradients between various zones exist within the sedimentation tank. This causes further collisions and conglomeration between particles.

Laboratory testing and results can be used to establish sedimentation tank design parameters as well as predict the percentage of particles that will be removed. Consider a suspension that is placed in a column and allowed to settle under quiescent conditions. A sample is taken at specific time intervals (t) and depths (Z), and the fraction (X) of particles, based on the original suspension concentration, that have settled to said depth during said time is determined. These fractions are plotted against time and depth, and then connected by drawing in isoconcentration lines, as shown in Figure 4. These lines also show how the settling velocities increase with depth.

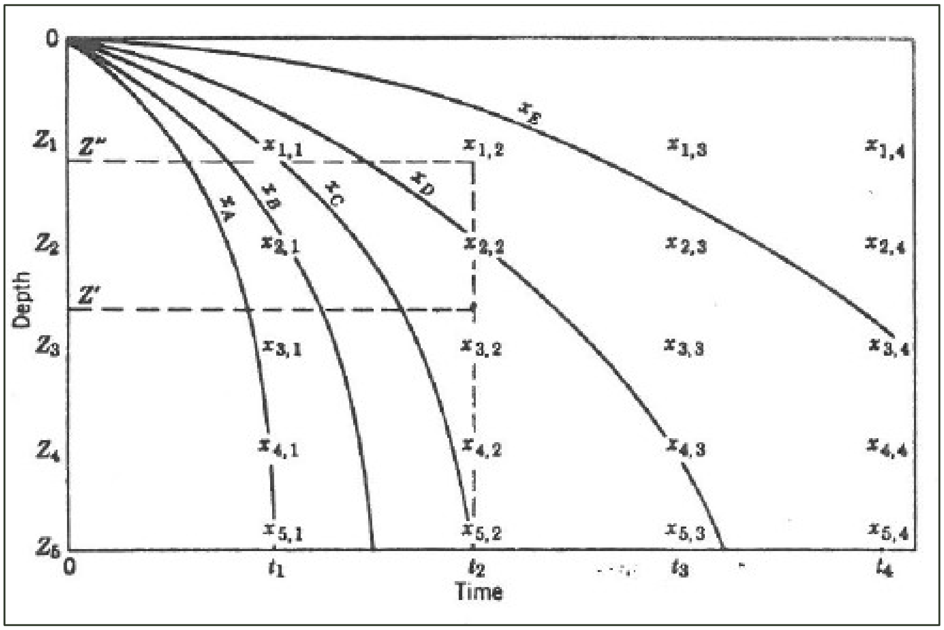


Figure 4: Fractional removal of flocculating particles - a function of time and depth (Rich, 1961:92)

Now, for example, to estimate the concentration (X_T) of particles that is removed in a basin with depth Z_5 at time t_2 it can be seen that all particles making up concentration X_C will have a settling velocity greater than or equal to Z_5/t_2 and will thus be removed. Further, the portion of concentrations X_D and X_E that will be removed are included in the equation (Rich, 1961: 93) below as follows:

$$X_T = X_C + \frac{Z'}{Z_5}(X_D - X_C) + \frac{Z''}{Z_5}(X_E - X_D)$$

Sedimentation is implemented upstream of the filtration process step to reduce the solids loading on the filters, which leads to longer filter runs and less backwashing.

Flocculated water enters the sedimentation tank on the one side and settled water is withdrawn from the opposite end. This reduces the chance of short circuiting. The inlet system, separating the inlet zone from the settling zone, may comprise a perforated wall, commonly referred to as a baffle wall. The outlet- or withdrawal system typically involve an overflow weir/launder, located at the water surface. As water enters the tank the water level rises until it reaches the launder. The rate at which the water rises is referred to as the surface loading or overflow rate.

Continuous and effective removal of settled material, generally termed sludge, is critical (Kawamura, 2000:144). Failing to do so will result in a build-up of sludge, which reduces the effective depth of the basin and consequently increase the horizontal fluid velocity. As described in Section 3.4.2, this in turn may cause the settled particles to resuspend.

3.4.4 Water Quality

Sedimentation is essential for treating high turbidity waters, which as described in Section 2 is commonly associated with a high suspended solids concentration.

Care must be taken in attempting treatment of highly coloured water by means of sedimentation. Flocs formed from this type of water are typically small and have low settling velocities (Gregory and Edzwald, 2011:Section 9.4.1). This results in oversized sedimentation tanks that rarely achieve the desired effluent quality.

Flow short circuiting may occur because of density flow in the sedimentation system. Density flow happens when there is a significant difference between the density of the new influent water and the density of the water which is already within the tank. The difference in densities may be caused by a variance in the temperature between the two waters or one containing a higher concentration of suspended solids than the other. This is typically caused by switching from one water source to another (Kawamura, 2000:145). It is thus important to consider the difference in temperature and concentration of suspended solids between the two sources.

Grit chambers may be added, upstream of the coagulation and flocculation process if there is a risk of the water containing high concentrations of nonflocculable, discrete particles such as sand and silt (Kawamura, 2000: 140). These particles typically have high settling velocities and as a result the grit chamber can be designed based on much higher surface loading rates than sedimentation tanks.

3.4.5 Design Approach

Conventional sedimentation processes have been in existence since the early 1900s, however, more recently it has seen many advancements and modifications, mainly to increase the surface loading rate without compromising the effluent water quality, or in fact to improve the effluent water quality. The different types of sedimentation tanks available today range from:

- the more traditional horizontal flow clarifier (HFC) with or without settler modules;
- reactor clarifiers that comes with an inhouse flocculation step, and that relies on upflow and a sludge blanket to retain flocs; to
- high-speed ballasted settling process that incorporates the use of microsand or other high-density material, termed a ballast, to latch onto floc particles and pull them to the bottom of the tank.

The software model allows for designing of the first type of sedimentation tank listed above, as well as for designing a grit chamber. The development of the design procedure of each is described in detail below.

3.4.5.1 Grit Chamber and Horizontal Flow Clarifier (HFC)

The approach for designing a grit chamber and HFC is based on the theory of Class I settling, generally accepted particle settling velocities (Kawamura, 2000:143) and methods developed by Kawamura (2000:173-178). This is because laboratory results, relating to Class I & II settling, are rarely available.

The design starts off by selecting the number of tanks, the length, width and water depth of each tank, as well as the smallest sized particle that is to be removed. Thereafter several parameters are calculated, initially using the maximum plant flowrate, to establish if the designed system meets all performance and design requirements.

Firstly, the sludge zone length is calculated as indicated below. The sludge zone length is the horizontal distance that a particle, with a specific settling velocity, will travel as it settles from the water surface to the bottom of the tank.

$$v_{hor} = \frac{Q_{max}}{W \times H \times n}$$

$$L_s = \frac{K \times SF \times H \times v_{hor}}{v_o}$$

$v_{hor} = Mean\ horizontal\ flow\ velocity\ \left({}^{m}/_{s}\right)$

$Q_{max} = Maximum\ flowrate\ \left({}^{m^3}/_{s}\right)$

$W = Tank\ width\ (m)$

$H = Water\ depth\ (m)$

$n = Number\ of\ tanks\ (No)$

$L_s = Sludge\ zone\ length\ (m)$

$K = Constant$

$SF = Safety\ factor$

$v_o = Particle\ settling\ velocity\ \left({}^{m}/_{h}\right)$

Following, the surface loading rate, as defined above, and the tank retention time are calculated. The retention time is the theoretical time that it takes for the water to pass through the tank, in other words the time it takes to completely replace the volume of the tank. The retention time is of significance in so far that, in theory, flocculation continues to some degree in the tank, therefore, larger particles continue to form over time, which improves performance.

$$t_{ret} = \frac{H \times W \times L \times n}{Q_{max} \times 60}$$

$$v_s = \frac{Q_{max} \times 3600}{W \times L \times n}$$

$t_{ret} = Water\ retention\ time\ (min)$

$L = Tank\ length\ (m)$

$v_s = Surface\ loading\ rate\ \left({}^{m}/_{h}\right)$

Going forward, and to ensure that all design criteria are met, the length to width and length to water depth ratios are calculated as indicated below.

$$r_{l/_w} = \frac{L \times (n_{bw} + 1)}{W}$$

$$r_{l/_h} = \frac{L}{H}$$

$r_{l/_w} = Length\ to\ width\ ratio$

$n_{bw} = Number\ of\ longitudinal\ baffle\ walls\ (No)$

$r_{l/_h} = Length\ to\ depth\ ratio$

Finally, the Reynolds- and Froude Numbers are calculated. These numbers provide an indication of the flow characteristics and system performance.

$$P = (H \times (n_{bw} + 1) \times 2) + W$$

$$R = \frac{a}{P}$$

$$Re = \frac{v_{hor} \times R}{\nu}$$

$$Fr = \frac{v_{hor}^2}{g \times R}$$

$P = Wetted\ perimeter\ (m)$

$R = Hydraulic\ radius\ (m)$

$a = Internal\ tank\ area\ (m^2)$

$Re = Reynolds\ number$

$\nu = Kinematic\ viscosity\ \left({}^{m^2}/_{s}\right)$

$Fr = Froude\ number$

Diffuser walls, placed at the tank inlet, allow for uniform distribution of the entering water. It also assists with preventing density flows, whereby a gentle vertical mixing effect is created upstream of the diffuser wall that reduces the difference in temperature and density between the incoming and present water in the tank. The diffuser walls are designed as follows:

$$s_{c-c} = s_p + d_p$$

$s_{c-c} = Port\ centre\ to\ centre\ spacing\ (m)$

$$n_{p,r} = \frac{\frac{W}{n_{bw}+1} - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,c} = \frac{H - s_p \times 2 - d_p}{s_{c-c}}$$

$$v_p = \frac{Q_{max}}{n \times (n_{bw}+1) \times n_p \times \pi \times \left(\frac{d_p}{2}\right)^2}$$

$s_p = Spacing\ between\ ports\ (m)$

$d_p = Port\ diameter\ (m)$

$n_{p,r} = Number\ of\ ports\ per\ row\ (No)$

$n_{p,c} = Number\ of\ ports\ per\ column\ (No)$

$v_p = Flow\ vel\ through\ ports\ at\ \max flowrate\ \left({}^{m}/_{s}\right)$

$n_p = Total\ number\ of\ ports\ (No)$

The particles that need be removed by grit chambers typically comprise a high SG, resulting in high settling velocities, and therefore are less susceptible to carryover caused by high launder loading rates. For this reason, it is not deemed necessary to determine a specific launder length for grit chambers and the length can be accepted to be equal to the width of the chamber. Launder loading rates should however be considered for HFCs, where localised velocities may pull up and carry over low SG floc particles. To that effect, the launder length is calculated as follows:

$$l_{launder} = \frac{Q_{max} \times 3600}{n \times v_{launder} \times 2 \times n_{launder}}$$

$l_{launder} = Launder\ length\ (m)$

$v_{launder} = Launder\ loading\ rate\ \left({}^{m^3}/_{mh}\right)$

$n_{launder} = Number\ of\ launders\ (No)$

After the system is designed the Reynolds- and Froude Numbers are determined for the design- and minimum flowrates, using the formulas indicated above. However, under minimum flowrate conditions they are based on the number of active tanks, giving the user the option of testing the effect of isolating one or more of the tanks.

The applicable resulting values, as determined above, are compared against the ranges indicated in Table 8.

Table 8: Sedimentation process design requirements

Description	Grit Chamber	HFC	Reference
Tank length (m)	> Sludge zone length		Gregory and Edzwald, 2011:Section 9.2.1
Mean flow velocity (m/s)	< 0.0625	< 0.0117	Kawamura, 2000:171
Detention time (min)	> 10.5	> 165	
Surface loading rate (m/h)	< Particle settling rate		Gregory and Edzwald, 2011:Section 9.2.1
Length to width ratio	> 6	> 4	Kawamura, 2000:171
Water depth to width ratio	> 8	> 15	
Reynolds number	< 86 200	< 20 000	
Froude number	> 10^{-5}		
Flow velocity through diffuser wall ports (m/s)	< 0.25		
Launder length	N/A	< Tank length	-

The following assumptions and fixed values are built into the software model:

- The constant (K) is fixed at 1.5 (Kawamura, 2000:173).
- The particle settling rate is based on those developed by Kawamura (2000:143).

- Each launder is mounted somewhere at an offset from the sidewalls, resulting in two overflow sides.

Future improvements to the software model may include the following:

- Testing for additional parameters (incl. mean flow velocity, detention time, surface loading rate, flow velocity through diffuser wall ports and launder loading rate) under minimum flowrate conditions to ensure all design requirements are met when one or more systems are isolated.
- Restricting the tank width and number of baffle walls to conform to supplier specific sludge scraper system lengths.
- Incorporation and application of laboratory test results and Class II settling theory for the design of sedimentation tanks.

3.4.5.2 Horizontal Flow Clarifier (HFC) with Settler Modules

HFCs can be designed to include settler modules, which allows for an increased surface loading rate, hence resulting in a smaller tank footprint. These settler modules cover only a portion of the tank surface area, typically comprise either inclined plates or tube settlers and can be configured to allow for counter-current, co-current, or crossflow sedimentation. This dissertation considers the design of a tube settler system, installed such to promote counter-current settling.

Floc particles entering the bottom of the tube settlers are forced in the inclined direction of flow. As the particle settles it lands on the downstream side of the settler. From here it either rolls down to the bottom of the tank, if its SG is high enough, or it stays stagnant until encountering other flocs to form particles that eventually rolls down the side of the tube.

The design approach for these types of systems are based on methods developed by Kawamura (2000:181-183). Firstly, the area covered by the tube settlers, based on the maximum plant flowrate is considered.

$$A_{set} = \frac{Q_{max} \times d_{set}}{n \times v_o \times SF \times (h_{set} \times \cos\alpha + d_{set} \times \cos^2\alpha)}$$

$A_{set} = Area\ covered\ by\ settler\ modules\ (m^2)$

$Q_{max} = Maximum\ flowrate\ \left({}^{m^3}/_{s}\right)$

$d_{set} = Settler\ module\ internal\ dimension\ (m)$

$n = Number\ of\ tanks\ (No)$

$v_o = Particle\ settling\ velocity\ \left({}^{m}/_{h}\right)$

$h_{set} = Settler\ module\ vertical\ height\ (m)$

$\alpha = Settler\ module\ incline\ (^{o})$

This area along with a chosen tank width can now be used to determine the length of tank that is covered by the tube settlers. From here the tank length can be determined based on the percentage of surface area that is to be covered by the settlers. Once fixed it is important to test the length to width ratios against the specific design criteria, listed below.

$$L_{set} = \frac{A_{set}}{W}$$

$$L = \frac{L_{set}}{r_{set}}$$

$$r_{l/_w} = \frac{L \times (n_{bw} + 1)}{W}$$

$L_{set} = Length\ of\ tank\ covered\ by\ settler\ modules\ (m)$

$W = Tank\ width\ (m)$

$L = Tank\ length\ (m)$

$r_{set} = Portion\ of\ tank\ covered\ by\ settler\ modules\ (\%)$

$r_{l/_w} = Length\ to\ width\ ratio$

$n_{bw} = Number\ of\ longitudinal\ baffle\ walls\ (No)$

Going forward it is important to determine the surface loading rate of the area covered by the settler modules as well as their volumetric retention. These parameters are calculated as shown below.

$$v_s = \frac{Q_{max} \times 3600}{n \times A_{set}}$$

$$v_{set} = \frac{Q_{max}}{n \times A_{set} \times \sin\alpha}$$

$$l_{set} = \frac{h_{set}}{\sin\alpha}$$

$$t_{set} = \frac{l_{set}}{v_{set} \times 60}$$

$v_s = Surface\ loading\ rate\ \left({}^{m}/_{h}\right)$

$v_{set} = Flow\ velocity\ in\ settler\ module\ ({}^{m}/_{s})$

$l_{set} = Settler\ module\ length\ (m)$

$t_{set} = Settler\ module\ retention\ time\ (min)$

The Reynolds- and Froude Numbers that are of significance are those based on the settler module profile and flow characteristics, and are calculated as indicated below.

$$a_{set} = d_{set}^2$$

$$P_{set} = 4 \times d_{set}$$

$$R_{set} = \frac{a_{set}}{P_{set}}$$

$$Re = \frac{v_{set} \times R_{set}}{\nu}$$

$$Fr = \frac{v_{set}^2}{g \times R_{set}}$$

$a_{set} = Settler\ module\ internal\ area\ (m^2)$

$P_{set} = Settler\ module\ wetted\ perimeter\ (m)$

$R_{set} = Settler\ module\ hydraulic\ radius\ (m)$

$Re = Reynolds\ number$

$\nu = Kinematic\ viscosity\ \left({}^{m^2}/_{s}\right)$

$Fr = Froude\ number$

The design of diffuser walls and launder lengths, as described above, remain the same, whether it be for HFCs with or without settler modules.

Finally, the Reynolds and Froude Numbers are calculated for the design- and minimum flowrates, with allowances made to isolate one or more systems for minimum flow. In reviewing the design, the software model will establish if the design requirements, as indicated in Table 9 have been met.

Table 9: Sedimentation process design requirements (continued)

Description	Settler Modules	Reference
Length to width ratio	> 4	Kawamura 2000:171-172
Surface loading rate (m/h)	< 5.65	
Detention time (min)	> 4	
Flow velocity in settler module (m/s)	< 0.0025	
Reynolds number*	< 50	
Froude number*	> 10^{-5}	
Flow velocity through diffuser wall ports (m/s)	< 0.25	
Launder length	< Tank length	

* Also tested for average and minimum flowrates

The assumptions, fixed values and future improvements of this section include those listed above for grit chamber and HFC design. Additional improvements may include:

- additional functionality to allow for design of other types of settler modules, e.g. inclined plates; and
- designing and testing for the approaching flow velocity.

3.5 Filtration

3.5.1 Process Description

Filtration is the final polishing step in a conventional water treatment process. The two main types of filtration systems include the following:

- Granular media filtration: Pre-treated water flows through a granular media bed, which is responsible for the removal of particles.
- Membrane filtration: Membranes comprise a mesh of small openings that allows the passing of water but retains undesired particles. Particles are removed by means of size exclusion, i.e. the pore openings are smaller than the particles.

The focus of this dissertation is on granular media filtration and more specifically on slow sand filters and monomedia (a single media type) rapid sand- and high-rate gravity filters. Literature and everyday terminology often don’t distinguish well between rapid sand filters and high-rate filters and the two are often used interchangeably to refer to the same thing. It is therefore important to note at this point the difference between the two, as shown in Table 10.

Table 10: Rapid sand filters vs high-rate filters

Description	Rapid sand filters	High-rate filters	Reference
Filtration rates (m/h)	5 to 7.5	12.5 to 30	Kawamura (2000:234)
Media depth (m)	0.6 to 0.75	0.8 to 2	
Media effective size (mm)	0.45 to 0.65	0.8 to 2	

It is worth noting that there is a growing interest in high-rate filters due to its higher filtration rates and consequently smaller footprint, resulting in lower capital cost.

Granular media filters rely on two main particle removal mechanisms, which can be classified as follows:

- Depth filtration: Particles deposit within the pore spaces due to attachment onto the media surface or previously deposited particles. Consequenlty, the particles that are removed by the filter may be much smaller than the characteristic dimensions of the pore spaces (Tobiason et al., 2011:Section 10.1.1).
- Cake filtration: A cake layer is formed on top of the media by previously deposited particles, which excludes new particles transported by the water based on their size, i.e. the pore sizes in the cake layer is smaller than the excluded particles (Tobiason et al., 2011:Section 10.1.1).

3.5.1.1 Rapid sand- and high-rate filters

Rapid sand and high-rate filters mainly rely on depth filtration to clarify the water; although, cake filtration does occur to a lesser extent at localised areas throughout the media bed (Tobiason et al., 2011:Section 10.1.1). Water that has passed through the media is collected below in an underdrain system, from where it exits the filter and ends up in a filtered water reservoir, often termed a clearwell. The underdrain system is typically designed to exclude the media, whilst allowing the water to flow through.

At the start of a filter run, when the filter bed is clean, i.e. little to no deposits reside in the media, poor effluent quality is often observed. This is because the lack of particles in the media result in large pores, which is a function of the media properties (e.g. effective size etc.), that allows the particles to pass through the filter unhindered. The filter is said to undergo ripening (Castro, Mysore and Chapman, 2012:Section 9.3.1), when the size of the pores decreases because of more and more particles becoming enmeshed, until the desired filtered water quality is achieved.

Particles that are deposited within the pore spaces of the media are held in equilibrium with the shearing force exerted by the water that aims to tear them away and pull them deeper into the media (Tobiason et al., 2011:Section 10.4.1). As more deposits arrive the pores through which the water passes become smaller, resulting in higher localised flow velocity and consequential higher shear forces. The increase in deposits also results in larger particles sizes, which causes particle instability and parts thereof are easily sheared away. The result is that particles or parts of particles are pulled deeper and deeper into the filter as the filter runtime goes on. The burden of removal therefore "passes deeper and deeper into the filter" (Eliassen; Stanley; Ling, cited in Tobiason et al., 2011:Section 10.4.1). This happens until the particles reach the bottom of the media and ends up in the filtered water, below the underdrain system. This phenomenon is referred to as breakthrough (Tobiason et al., 2011:Section 10.4.1).

As the flow velocity through the pores increase the hydraulic headloss across the bed also increases (Castro, Mysore and Chapman, 2012:Section 9.2.6). This happens until the headloss through the bed equals the hydraulic head available for flow to occur by gravity.

When breakthrough occurs, or the available hydraulic head has been spent it signals that a backwash is needed to remove the particle deposits in the filter. A backwash sequence typically includes an air scour step followed by a rinse step. During the air scour and rinse step, air and water is sent through the filter media, in the flow direction opposite to filtration. Improved media cleansing is achieved when a step comprising a combination of air and water flow is added to the backwash sequence. The air for backwashing is provided by a set of blowers or compressors and the water is drawn from the clearwell by a set of pumps and pumped upwards through the filter media.

3.5.1.2 Slow sand filters

Slow sand filters (SSFs) rely on cake filtration and biological activity (Tobiason et al., 2011:Section 10.1.1) to remove particles from the water. A layer, called the schmutzdecke, which is largely organic in origin (Huisman and Wood, 1974:20), builds up on the surface of the filter bed. This layer typically consists of bacteria, protozoa, rotifers, copepods, roundworms, flatworks and oligochaetes (Cleasby and Logsdon, 1999:8.75).

The schmutzdekke removes inert suspended particles, through physical straining and breaks down some organic components, through biological action (Cleasby and Logsdon, 1999:8.75). It also reduces the raw water turbidity (Kawamura, 2000:281).

Water that has passed through the schmutzdeke enter a maze of pores created by the sand. Each time the water enters a new pore it slows down and consequently these pores act as millions of minute sedimentation tanks, which allow the particles that have passed through the schmutzdeke layer to settle onto the nearest sand grain. Once contact is made the particles are held attached to the sand grain by the operation of electrical forces (Huisman and Wood, 1974:21-22).

Like rapid sand and high-rate filters the hydraulic headloss across the SSF also increases over time. However, this increase in headloss is mainly caused by the schmutzdeke layer, which become so dense and its pores so small that, even at the low filtration rates to which SSFs are subjected to, a noticeable increase in headloss is observed. SSFs do not require backwashing to clean the deep parts of the media, and rather it is considered sufficient to manually remove the schmutzdeke layer, typically every 45 to 60 days (Collins, Youngstrom and Broder, 2012:Section 10.2.1). According to Kawamura (2000:282) it may take up to 1 week for the schmutzdecke to re-establish itself after cleaning. However, others (Cullen and Letterman, cited in Cleasby and Logsdon, 1999:8.75) suggest a period anywhere between 6 hours and 2 weeks.

3.5.2 Water Quality

Filtration is a prerequisite for treating any surface water up to potable drinking standards. However, it is often difficult to select the appropriate filtration technology due to a myriad of variables that influence the process. Thereafter the next challenge is to consider direct filtration or a more conventional process that includes sedimentation prior to filtration. Table 11 has been developed from two sources, to assist with the selection process.

Table 11: Raw water quality for filter selection

Parameter	SSF	Direct filtration	Conventional system
Turbidity (NTU)	< 5***	< 20*	< 3000*
Color (apparent)	< 18.5***	< 20*	< 100**
Alkalinity (mg/l)	< 200*	< 200*	< 500*
Hardness as $CaCO_3$ (mg/l)	< 150*	< 150*	< 700*
Iron (mg/l)	< 0.3***	< 0.3***	< 2*
Manganese (mg/l)	< 0.05***	< 0.05***	< 0.5*
TOC (mg/l)	< 2*	< 2.5*	< 7*
Algae (ASU/ml) <u>or</u>	-	< 1000*	< 10 000*
Chlorophyll-a (µg/l)	< 5***	-	-
Giardia (count/100 l)	< 3*	< 3*	< 20*
Cryptosporidium (count/100 l)	< 1*	< 1*	< 10*
Coliform (count/ml)	< 1000*	< 1000*	< 1000 000*

*Kawamura, 2000:40
**Castro, Mysore and Chapman, 2012:Section 9.2.2
***Tobiason et al, 2011:Section 10.7.1 & 10.9.4

It is worth noting that SSFs are not very effective at removing color, in fact, according to Tobiason et al. (2011:Section 10.9.4) SSFs typically only achieve 25% removal. The upper limit for color has therefore been set at 18.5 mg Pt-Co/l, which is 25% higher than the maximum color allowed by SANS 241-1 (South African National Standards, 2015:7-9) for drinking water.

3.5.3 Design Approach

The design approach described in this section includes the design of SSFs as well as a basic and high-level design of a high-rate filter system. Others (fellow MEng student and work colleague, L. Ingle) are developing a more comprehensive tool that can be used to perform a detailed design of the latter two. The filtration section of the software model developed as part of this dissertation can however be used for a quick analysis of a filter system at the feasibility stage of a project.

3.5.3.1 Slow Sand Filters

The total filtration area is based on the chosen filtration rate and the plant treatment capacity. The filtration area per filter is determined by dividing the total filtration area by the chosen number of filters. The length and width are then calculated using the filter area and chosen length to width ratio.

$$A = \frac{Q \times 3600}{v \times n}$$

$$L = \sqrt{A \times r_{l/w}}$$

$$W = \frac{L}{r_{l/w}}$$

$A = Filtration\ area\ (m^2)$

$Q = Design\ flowrate\ \left(m^3/s\right)$

$v = Filtration\ rate\ \left(m/h\right)$

$n = Number\ of\ filters\ (No)$

$L = Filter\ length\ (m)$

$r_{l/w} = Length\ to\ width\ ratio$

$W = Filter\ width\ (m)$

The height/depth of the filter is made up of various components, some comprising a wide range of acceptable values. For example, the media depth is, to a large extent, based on the desired number of intervals of scraping off the schmutzdekke (which will inevitably include a couple of centimetres of media layer) before replenishment of the media layer is required. As such, the media depth may be driven by factors such as the remoteness of the works or the availability of media. It is therefore important for the designer to consider the balance between appropriate values for the components contributing to the overall filter height and the associated construction cost. The filter height consists of the items listed below.

$$H = h_{med} + h_{free} + h_{super} + h_{gravel} + h_{u,d}$$

$H = Filter\ height\ (m)$

$h_{med} = Media\ depth\ (m)$

$h_{free} = Freeboard\ height\ (m)$

$h_{super} = Supernatant\ depth\ (m)$

$h_{gravel} = Gravel\ layer\ depth\ (m)$

$h_{u,d} = Underdrain\ system\ height\ (m)$

As mentioned above SSFs require removal of the schmutzdekke from time to time, and during that time the filter is taken out of operation. Filters are also taken out of operation when maintenance is required or when faulty equipment must be replaced. This combined with the fluctuations in inflow means that filters often operate under a range of flow conditions, and it should be checked if the filtration rate is within the acceptable range for the considered worst-case scenario.

$$v_{max} = \frac{Q_{max} \times 3600}{A \times n_{on}}$$

$v_{max} = Maximum\ filtration\ rate\ \left({}^{m}/_{h}\right)$

$Q_{max} = Maximum\ flowrate\ \left({}^{m^3}/_{s}\right)$

$n_{on} = Number\ of\ always\ online\ filters\ (No)$

The software model will alert the user if the allowable maximum filtration rate has been exceeded based on the increased flowrate and the number of filters that has been taken offline.

The following fixed values/assumptions have been built into the SSF section of the software model:

- The depth of the gravel layer is based on depths indicated by Huisman Wood (1974:58-59):
  - Layer 1 = 60 mm
  - Layer 2 = 60 mm
  - Layer 3 = 60 mm
  - Layer 4 = 120 mm
- The height of the underdrain system is based on:
  - 50 mm for a perforated pipe system
  - 160 mm for a brick and precast concrete panel system (Huisman and Wood, 1974:59)

Future additions to the SSF design tool could include headloss calculations through the filter media and underdrain system, specifically to identify the risk of high headloss variance across the bed, which may lead to short-circuiting.

3.5.3.2 Rapid sand- and high-rate filters

The filtration area for high-rate filters is determined in the same way as for SSFs. However, the flooring system is typically made-up of a configuration of supplier specific panels, which must be taken into account. The resulting filtration area is therefore a function of the round number of panels required. As a result, the last mentioned is always greater than the theoretical filtration area required, and an optimised design aims to reduce this difference by as much as possible.

It should also be noted that a filter bed is considered to consist of one cell when the backwash channel is mounted on the side and two cells when the channel is constructed in the middle. The resulting filtration area, filtration rate and filter dimensions are calculated as shown below.

$$A_{in} = \frac{Q \times 3600}{v \times n}$$

$$W_{fil} = \sqrt{\frac{A}{r_{l/w}}}$$

$$W_{in} = W_{fil} + BC_{width} + BCW_{width} \times n_{BCW}$$

$$L_{fil} = W_{fil} \times r_{l/w}$$

$$n_{p,w} = \frac{W_{fil}}{l_{panel}}, (rounded)$$

$$n_{p,w,c} = \frac{n_{p,w}}{n_{cells}}, (rounded)$$

$$n_{p,l} = \frac{L_{fil}}{w_{panel}}, (rounded)$$

$$W_{res} = l_{panel} \times n_{p,w} + w_{anc}$$

$$L_{res} = w_{panel} \times n_{p,l} + l_{anc}$$

$$A_{res} = W_{res} \times L_{res}$$

$$v_{res} = \frac{Q \times 3600}{A_{res} \times n}$$

$$W = W_{res} + BC_{width} + BCW_{width} \times n_{BCW}$$

$$L = L_{res}$$

$A_{in} = Initial\ filtration\ area\ (m^2)$

$Q = Design\ flowrate\ \left({}^{m^3}/_{s}\right)$

$v = Target\ filtration\ rate\ \left({}^{m}/_{h}\right)$

$n = Number\ of\ filters\ (No)$

$W_{fil} = Initial\ width\ of\ filtration\ area\ (m)$

$r_{l/w} = Length\ to\ width\ ratio$

$W_{in} = Initial\ filter\ width\ (m)$

$BC_{width} = Backwash\ channel\ width\ (m)$

$BCW_{wid} = Backwash\ channel\ wall\ width\ (m)$

$n_{BCW} = Number\ of\ backwash\ channel\ walls\ (No)$

$L_{fil} = Initial\ length\ of\ filtration\ area\ (m)$

$n_{p,w} = Number\ of\ panels\ along\ filter\ width\ (No)$

$l_{panel} = Panel\ length\ (m)$

$n_{p,w,c} = Number\ of\ panels\ along\ cell\ width\ (No)$

$n_{cells} = Number\ of\ cells\ (No)$

$n_{p,l} = Number\ of\ panels\ along\ filter\ length\ (No)$

$w_{panel} = Panel\ width\ (m)$

$W_{res} = Resulting\ width\ of\ filtration\ area\ (m)$

$w_{anc} = Ancillary\ width\ contributing\ components\ (m)$

$L_{res} = Resulting\ length\ of\ filtration\ area\ (m)$

$l_{anc} = Ancillary\ length\ contributing\ components\ (m)$

$A_{res} = Resulting\ filtration\ area\ (m^2)$

$v_{res} = Resulting\ filtration\ rate\ \left({}^{m}/_{h}\right)$

$W = Resulting\ filter\ width\ (m)$

$L = Resuting\ filter\ length\ (m)$

The filter height comprises many components and again it is up to the designer to make some reasonable assumptions when deciding on some of the values. The items that determine the filter height are listed below.

$$H = h_{u,d} + h_{clog} + h_{clean} + h_{free} + h_m + h_{anc}$$

$H = Filter\ height\ (m)$

$h_{u,d} = Underdrain\ system\ height\ (m)$

$h_{clog} = Available\ clogging\ head\ (m)$

$h_{clean} = Headloss\ through\ clean\ filter\ bed\ (m)$

$h_{free} = Freeboard\ height\ (m)$

$h_m = Media\ depth\ (m)$

$h_{anc} = Ancillary\ height\ components\ (m)$

Like with SSFs the number of filters in operation continuously changes as filters are backwashed and others taken offline to perform maintenance or when equipment must be replaced. At the same time the inflow to the filters varies during the day. The resulting filtration rate is calculated the same way as for SSFs.

$$v_{max} = \frac{Q_{max} \times 3600}{A_{res} \times n_{on}}$$

$v_{max} = Maximum\ filtration\ rate\ \left({}^{m}/_{h}\right)$

$Q_{max} = Maximum\ flowrate\ \left({}^{m^3}/_{s}\right)$

$n_{on} = Number\ of\ always\ online\ filters\ (No)$

Effective backwashing is a critical component of successful filter operation. The backwashing procedure considered for the software model comprise the three steps described in Section 3.5.1.1. The required air and water flowrates are determined so that a collapse-pulsing mechanism is achieved.

During a collapse-pulsing backwash the air bubble that passes through the media expands until the weight of the media causes it to collapse. High local water velocities occur near the bubble surface as it collapses, which causes media particles to collide violently and abrade one another, thereby loosening floc particles that have adsorbed onto the media. The bubble reforms immediately further up until it collapses again. This process continues until the bubble reaches the top of the bed where it bursts to atmosphere. With the floc and media particles separated the filter is easily washed clean during the rinse step. Figure 5 provides a graphical representation of the ideal backwash combinations (the hatched zone) to achieve a collapse-pulsing backwash.

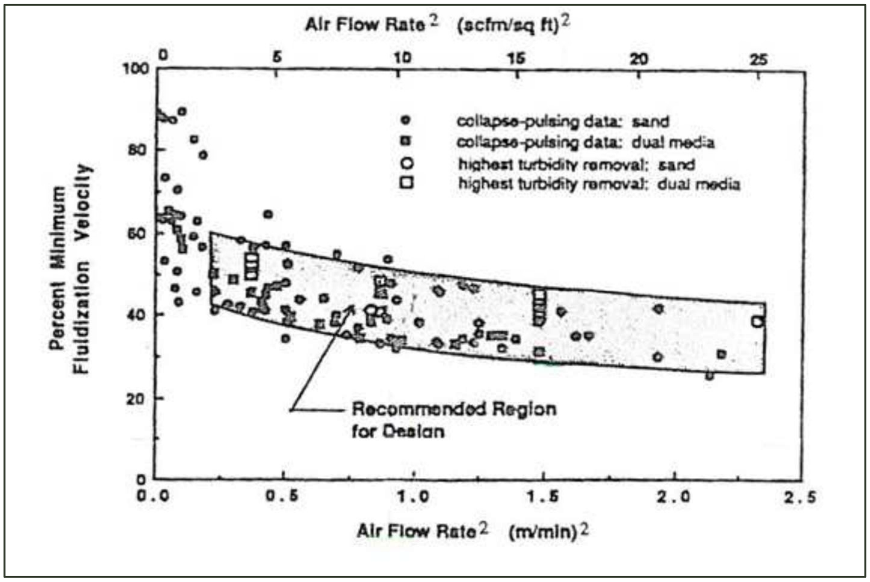


Figure 5: Recommended air and water backwashing rates (Amirtharajah, 1993:209)

From the above Figure 5 it is noted that the backwash water flowrate is a function of the minimum fluidization velocity, which can be described as the flowrate required to fluidize the media bed. The method for designing a collapse-pulsing backwash system is indicated below and has been developed based on methods developed by Cleasby and Logsdon (1999:8.8, 8.15).

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

$$G_a = \frac{\left(\frac{d_{90}}{1000}\right)^3 \times \rho \times (\rho_s - \rho) \times g}{\mu^2}$$

$d_{90} = 90\%\ passing\ diameter\ (mm)$

$d_{10} = Effective\ size\ (mm)$

$UC = Uniformity\ coefficient$

$$\frac{V_{mf}}{3600} = \frac{\mu \times \sqrt{33.7^2 + 0.0408 \times G_a} - 33.7 \times \mu}{\rho \times \frac{d_{90}}{1000}}$$

$$v_{bw1} = \left[43.5 - 8.5 \times \left(\frac{v_a}{60}\right)^2\right] \times \frac{v_{mf}}{100}$$

$$Q_{bw1} = v_{bw1} \times A_{res}$$

$$Q_{bw2} = Q_{bw1} \times r_{p,2}$$

$$v_{bw2} = \frac{Q_{bw2}}{A_{res}}$$

$G_a = Galileo\ number$

$\rho = Water\ density\ \left({}^{kg}/_{m^3}\right)$

$\rho_s = Media\ bulk\ density\ \left({}^{kg}/_{m^3}\right)$

$g = Gravity\ constant\ \left({}^{m}/_{s^2}\right)$

$\mu = Absolute\ viscosity\ \left({}^{Ns}/_{m^2}\right)$

$v_{mf} = Minimum\ fluidisation\ velocity\ \left({}^{m}/_{h}\right)$

$v_{bw1} = Duty\ 1\ backwash\ rate\ \left({}^{m}/_{h}\right)$

$v_a = Air\ scour\ rate\ \left({}^{m}/_{h}\right)$

$Q_{bw1} = Duty\ 1\ backwash\ flowrate\ \left({}^{m^3}/_{s}\right)$

$Q_{bw2} = Duty\ 2\ backwash\ flowrate\ \left({}^{m^3}/_{s}\right)$

$r_{p,2} = Pump(s)\ operating\ \%\ for\ duty\ 2\ (\%)$

$v_{bw2} = Duty\ 2\ backwash\ rate\ \left({}^{m}/_{h}\right)$

The backwash flowrates together with the headloss experienced by the blower and pump can now be used to establish theoretical motor sizes, as shown below. It is however recommended to obtain information from specialist suppliers when performing a detail design.

$$h_w = \left(\frac{3 \times Q_{bw2}}{3600 \times 2 \times l_w \times \sqrt{2 \times g} \times C_D}\right)^{\frac{2}{3}}$$

$$P_{B(kW)} = \frac{h_{air} \times 9804 \times v_a \times A_{res}}{\eta_{blower} \times 3600 \times 1000}$$

$$P_{P(kW)} = \frac{Q_{bw2} \times \rho \times g \times (h_w + h_{bw})}{\eta_{pump} \times 3600 \times 1000}$$

$h_w = Backwash\ weir\ overflow\ depth\ (m)$

$l_w = Backwash\ weir\ length\ (m)$

$C_D = Discharge\ coefficient$

$P_{B(kW)} = Blower\ motor\ size\ (kW)$

$h_{air} = Air\ headloss\ (m)$

$\eta_{blower} = Blower\ efficiency\ (\%)$

$P_{P(kW)} = Pump\ motor\ size\ (kW)$

$h_{bw} = Backwash\ water\ headloss\ (m)$

$\eta_{pump} = Pump\ efficiency\ (\%)$

Finally, the clearwell is sized based on the volume of backwash water required to wash the filter clean as well as the number of backwash volumes to be retained in the clearwell.

$$V_c = A_{res} \times \left[\left(5 \times h_{bc} + 4 \times (h_m \times \varepsilon_0 + h_w)\right) - 0.2\right] \times n_{bw}$$

$V_c = Clearwell\ volme\ (m^3)$

$h_{bc} = B/w\ channel\ height\ above\ media\ (m)$

$\varepsilon_0 = Fixed\ loose\ bed\ porosity$

$n_{bw} = Number\ of\ backwashe\ storages\ (No)$

Like SSFs the software model will alert the user if the allowable maximum filtration rate has been exceeded based on the increased flowrate and the number of filters that has been taken offline.

There are many variables taken into consideration when designing a rapid sand- and high-rate filter system. These variables can readily be obtained from literature, suppliers or other experienced designers. Most of these variables have been fixed in the software model and reflect typical values that are generally used. The fixed values include those listed below and those contained in Table 12:

- The backwash channel height above the media is 0.9 metres.
- The headloss through a clean filter bed is 0.5 m.
- The freeboard above the top water level is 0.5 m.
- The backwash channel internal width is 0.8 metres and the walls are 0.25 metres wide.
- The effective size is set to 1.00 mm (Kawamura, 2000:234).
- The uniformity coefficient is fixed at 1.4 (Kawamura, 2000:234).
- The media bulk density is 2650 kg/m^3 (Cleasby and Logsdon, 1999:8.10).
- The fixed loose-bed porosity is 0.44 (Cleasby and Logsdon, 1999:8.10).
- The discharge coefficient is fixed at a value of 0.616.
- Four (4) bed volumes of water will be required to rinse the filter clean (Cleasby and Logsdon, 1999:8.63).

Table 12: Underdrain system dimensions

Underdrain Type	Height (m)	Ancillary width contributing components (m)	Ancillary length contributing components (m)	Ancillary height contributing components (m)
Monolithic System	0.95	0	0	0
Dual-Parallel Lateral System	0.25	0.1	0.1	0.02

As mentioned above this dissertation only provides a tool for a high-level filter design and others have come up with more detailed software tools. As such, this section of the software model will not be developed further in future. Instead, it will be investigated how best to incorporate those alternative sub-tools.

3.6 Disinfection

3.6.1 Process Description

Processes like sedimentation, flotation and filtration facilitate the physical removal of harmful pathogens. However, the purpose of the disinfection process is to inactivate pathogenic microorganisms that have passed through these process steps. In other words, it is the last line of defence. The different types of disinfection include:

- Chlorine (Cl): Chlorine may be introduced into water in the form of chlorine gas (Cl_2), sodium hypochlorite (NaOCl) or calcium hypochlorite ($Ca(ClO)_2$). When added to water these sources of chlorine are hydrolysed and hypochlorous acid (HOCl) and hypochlorite ions (OCl^-) are formed. Both are responsible for oxidation and deactivation of pathogenic microorganisms.
- Ozone (O_3): When added to water "ozone reacts with hydroxide ions (OH^-) to form hydroxyl- and organic radicals" (Haas, 1999:14.18). Organic material present in the water is then oxidised, and microorganisms deactivated, by these radicals.
- Ultraviolet (UV): The UV light transmitted in the water causes damage to the genome of microorganisms, such as viruses, bacteria and parasites. This leaves the organisms incapable of replicating, and therefore they cannot cause infection (Water Environment Federation, 2018:Section 17.5.1). When the UV "dose" (lamp power emitted) is insufficient, some regeneration/repair may occur further downstream, which may lead to reactivation (Water Environment Federation, 2018:Section 17.5.4.4). UV disinfection systems should therefore

always be accompanied with chlorine disinfection, which allows for residual chlorine that can pre-empt the reactivation process.

The focus of this dissertation is on disinfection with chlorine. As mentioned above when Cl_2 is added to water it quickly disassociates to HOCl and OCl^-. Free available chlorine (FAC) refers to the sum of the concentrations of Cl_2, HOCl and OCl^-, and is expressed in mg/l as Cl_2 (Haas, 1999:14.6).

It can be seen from Figure 6 that at a pH of between 3.5 and 5.5 only HOCl species are present in the water and between 5.5 and 9.5, HOCl and OCl^- are both present. When the pH is above 8 OCl^- become the dominant species. HOCl is the most effective at deactivating unwanted pathogens (Hesby, Burns and Oneby, 2011: Section 11.4.1), and hence the aim is to increase its concentration by ensuring chlorine is dosed at the correct pH.

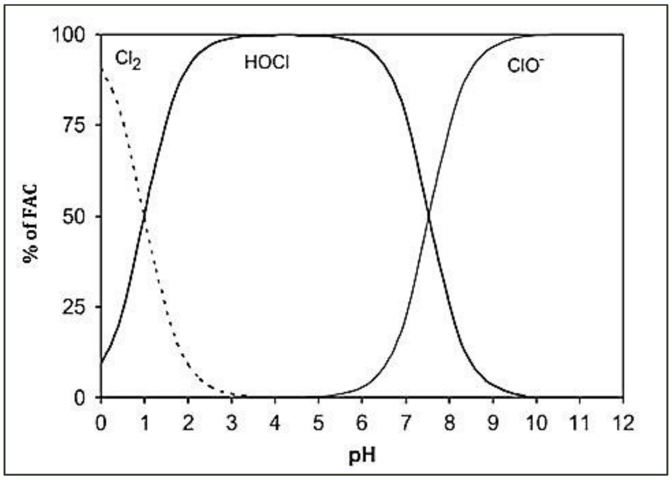


Figure 6: Effect of pH on relative amounts of chlorine, hypochlorite ion and hypochlorous acid (Spahl, 2012:2)

A certain contact time is required between the water and the free available chlorine for sufficient inactivation of harmful pathogens. The contact time is established by a contact tank that allows for a certain residence time at a specific flowrate. The clearwell, described above, often serves the dual purpose of doubling up as a contact tank, as well as retaining filtered water, used for backwashing.

3.6.2 Water Quality

Water that contains coliforms or that may contain coliforms in the future should be disinfected. Generally, coliforms are more rapidly inactivated, even at low chlorine dose concentrations, than certain other microorganisms in the water. Viruses and protozoa cysts (e.g. cryptosporidium oocysts and giardia cysts) are the focus point of these resistant pathogens.

Chlorine disinfection is effective at inactivating viruses and giardia but has little to no effect on cryptosporidium (Nova Scotia Environment, 2012:2). Where cryptosporidium has been found in the water or where it may be expected in the future alternative disinfection techniques such as UV (Linden and Rosenfeldt, 2011:Section 18.1), chlorine dioxide or ozone (Korich et al,. 1990:1426) must be considered.

3.6.3 Design Approach

A chlorine disinfection system is designed to achieve a specific log inactivation of viruses and giardia. The term log inactivation can be expressed as a percentage number and refers to the fraction of pathogenic organisms that has been deactivated. Log inactivation can be converted to percentage as follows:

$$\text{N}\log\text{reduction} = \left(1 - \frac{1}{10^N}\right) \times 100\%\ \text{reduction}$$

The Surface Water Treatment Rule (SWTR) developed by the United States Environmental Protection Agency (US EPA) stipulate a 3-log inactivation and a 4-log inactivation of giardia and viruses, respectively. As noted above other process steps are responsible for removal of giardia and viruses (and even cryptosporidium) and the log removal accredited to these processes are indicated in Table 13 (Nova Scotia Environment, 2012:13-14). The remaining log removal/inactivation required should be achieved through disinfection.

Table 13: Log removal credits (Nova Scotia Environment, 2012:13-14)

Treatment technology	Protozoa Credit		Virus Credit [1]	Individual Filter Turbidity Limit (unless noted otherwise
	Crypto-sporidium [1]	Giardia [1]		
Conventional filtration [2] – includes chemical mixing, coagulation, flocculation, clarification and rapid gravity filtration	3.0-log	3.0-log	2.0-log	• Shall be less than or equal to 0.2 NTU in at least 95% of the measurements made or at least 95% of the time each calendar month. • Shall not exceed 1.0 NTU at any time • Filter-to-waste until below 0.2 NTU – filters shall be capable of directing filtered water to waste immediately following a backwash for a period of time until the filtrate turbidity value is below 0.2 NTU [3].
Direct filtration [2] – includes chemical mixing, coagulation, flocculation, and rapid gravity filtration	2.5-log	2.5-log	1.0-log	
Slow sand filtration	3.0-log	3.0-log	2.0-log	• Shall be less than or equal to 1.0 NTU in at least 95% of the measurements made or at least 95% of the time each calendar month. • Shall not exceed 3.0 NTU at any time • Filter-to-waste - a filter-to-waste feature shall be provided so that the filtered water immediately after filter cleaning is directed into a waste stream [3]

[1] Disinfection shall provide a minimum 0.5-log inactivation for *Giardia* unless a higher log inactivation credit is required. Where disinfection is used to address any shortfall in the log reduction requirements for *Cryptosporidium,* an alternate disinfectant such as UV, chlorine dioxide or ozone shall be required.

[2] Facilities with conventional or direct filtration that achieve 0.15 NTU 95% of the time each calendar month in combined or individual filter effluent are eligible to receive additional log removal credits for protozoa to meet minimum treatment requirements as follows: combined 0.5-log; individual 1.0-log.

[3.] Alternatives that demonstrate an equivalent benefit to filter-to-waste may be considered by NSE on a case-by-case basis for existing facilities. All new facilities shall include a filter-to-waste provision.

The SWTR awards a certain log-reduction number, for giardia and viruses, based on the inactivation number (CT) achieved by the system. The equations below show how the CT number for giardia can be derived from the log-reduction number, pH, temperature and free residual chlorine concentration (the concentration of FAC measured downstream of the contact tank) (Martin, cited in Nova Scotia Environment, 2012:65). It is worth noting that the design is based on the residual chlorine rather than the available chlorine. The reason for this is that it is difficult to apply a number to the available chlorine concentration, because it changes as it reacts with constituents in the water. Further, the free residual chlorine indicates the lowest possible concentration that may prevail throughout the contact tank and therefore using it to calculate the CT number allows for, at least, a conservative approach.

$G_{lr} = 3 - G_{ru}$ $\qquad$ $G_{lr} = Giardia\ removal\ required\ (log)$

$$G_{CT} = 0.2828 \times pH^{2.69} \times Cl^{0.15} \times G_{lr} \times 0.933^{T-5}$$

$G_{ru} = Upstream\ giardia\ removal\ (log)$

$G_{CT} = Giardia\ inactivation\ no.\ (mg.min/l)$

$pH = Potential\ hydrogen$

$Cl = Residual\ chlorine\ concentration\ \left({}^{mg}/_{l}\right)$

$T = Water\ temperature\ (^{o}C)$

The CT number for 4-log inactivation of viruses is based on the water pH and temperature, and can easily be determined by interpolation from Table 14 (Nova Scotia Environment, 2012:73).

Table 14: CT for 4-log inactivation of viruses (Nova Scotia Environment, 2012:73)

Temperature (°C)	pH	
	6 – 9	10
0.5	12	90
5	8	60
10	6	45
15	4	30
20	3	22
25	2	15

The volume of the contact tank can now be calculated using the highest CT number and baffling factor, which is obtained from Table 14 (Nova Scotia Environment, 2012:17).

$$t = \frac{CT_{max}}{Cl \times BF}$$

$$V = t \times 60 \times Q_{max}$$

$t = Contact\ time\ required\ (min)$

$CT_{max} = Max\ inactivation\ no.\ (mg.min/l)$

$BF = Baffling\ factor$

$V = Contact\ tank\ volume\ (m^3)$

$Q_{max} = Maximum\ flowrate\ \left({}^{m^3}/_{s}\right)$

Table 15: Baffling factors (Nova Scotia Environment, 2012:17)

Baffling condition	Baffling factor	Baffling description
Unbaffled (mixed flow)	0.1	• Agitated basin • Very low length-to-width ratio • High inlet and outlet flow velocities • High potential for stagnant zones and short-circuiting
Poor	0.3	• Single or multiple unbaffled inlets and outlets • No intra-basin baffles • Potential for stagnant zones or short-circuiting
Average	0.5	• Baffled inlet or outlet • Some intra-basin baffles

Baffling condition	Baffling factor	Baffling description
Superior	0.7	• Perforated inlet baffle • Serpentine or perforated intra-basin baffles • Outlet weir or perforated launders • Most of tank volume is utilized
Perfect (plug flow)	1	• Length to width ratio greater than or equal to 10:1 • Perforated inlet, outlet and intra-basin baffles

In reviewing the design, the software model will alert the user if the pH value has been exceeded to the extent that Table 14 above cannot be applied and also if the pH is above 7.5, meaning that hypochlorous acid is not the dominant species.

It is anticipated that future version of the software model will include:

- design of the contact tank geometry; and
- designs of other disinfection technologies, e.g. UV disinfection.

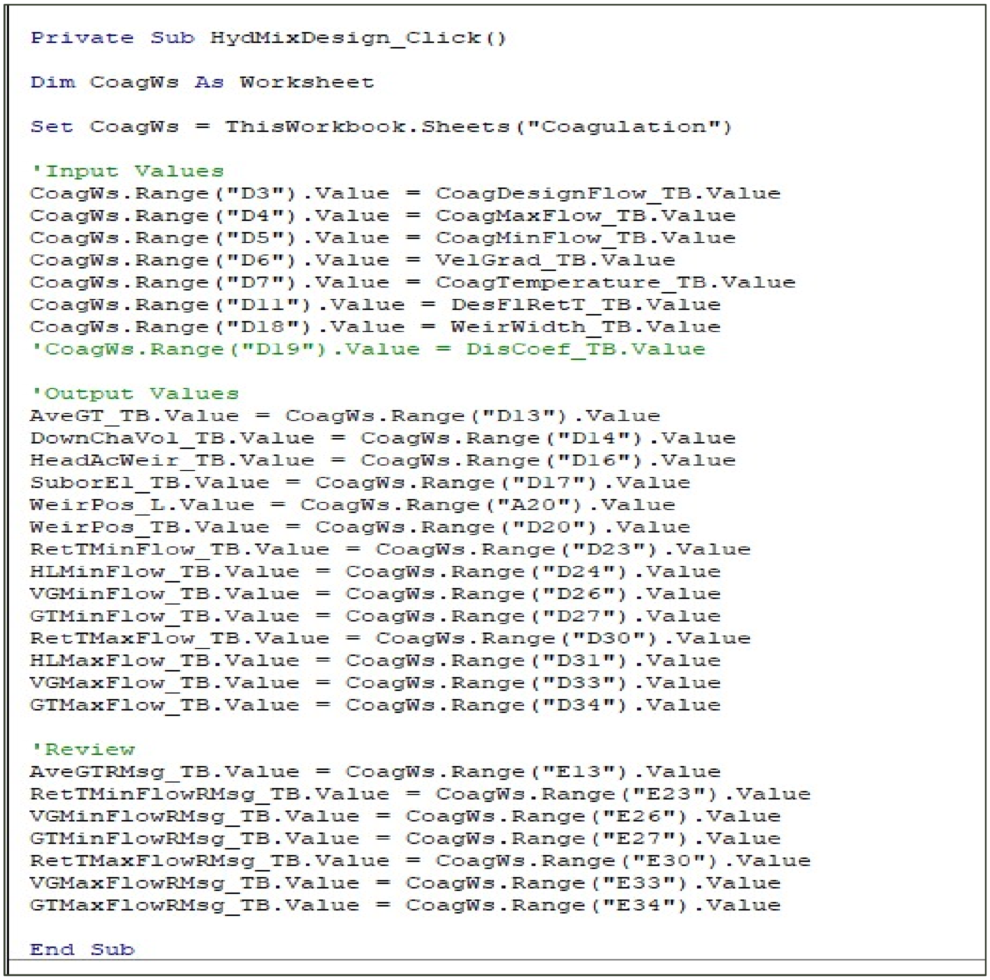
4 Software Model Development, Use and Validation

The software model was developed with the use of Microsoft Excel with the methodology, as described in the design approach under each process, in Section 3 programmed in. For the most part basic excel functionality was used, however, some coding was developed with Microsoft Visual Basic Application (VBA) to assist with some complicated computations. Figure 7 shows an extract from the VBA coding for calculating the oxygen concentration downstream of the first cascading weir in the aeration process.

```
Function CalculateCd1(Cd1 As Double, Cs As Double, Cu1 As Double, Cdz As Double)
Dim X, Y, Z, V As Double
X = (Cd1 - Cu1) / (Cs - Cu1)
Y = ((Cd1 - Cu1) / (Cs - Cu1)) * (Cs - Cd1) + Cd1
V = X * (Cs - ((Cs - Y) * X + Y))
Z = (Cs - Y) * X + Y
'For 1 weir
If Range("NoW") = 1 Then
CalculateCd1 = Cdz - Cd1
'For 2 weirs
ElseIf Range("NoW") = 2 Then
CalculateCd1 = Cdz - Y
'For 3 weirs
ElseIf Range("NoW") = 3 Then
CalculateCd1 = Cdz - ((Cs - Y) * X + Y)
'For 4 weirs
ElseIf Range("NoW") = 4 Then
CalculateCd1 = Cdz - (V + Z)
'For 5 weirs
ElseIf Range("NoW") = 5 Then
CalculateCd1 = X - ((Cdz - (V + Z)) / (Cs - (V + Z)))
Else
CalculateCd1 = 0
End If
End Function
```

Figure 7: Aeration VBA coding

Userforms were also developed using VBA coding. The purpose of these forms is to make the software model more user friendly with the user not having to input or extract values from complicated spreadsheets. It is also to restrict any changes to the calculations running in the background, where ultimately the different sheets can be locked for editing by the user. The VBA coding for the hydraulic coagulation userform is shown in Figure 8.



```
Private Sub HydMixDesign_Click()

Dim CoagWs As Worksheet

Set CoagWs = ThisWorkbook.Sheets("Coagulation")

'Input Values
CoagWs.Range("D3").Value = CoagDesignFlow_TB.Value
CoagWs.Range("D4").Value = CoagMaxFlow_TB.Value
CoagWs.Range("D5").Value = CoagMinFlow_TB.Value
CoagWs.Range("D6").Value = VelGrad_TB.Value
CoagWs.Range("D7").Value = CoagTemperature_TB.Value
CoagWs.Range("D11").Value = DesFlRetT_TB.Value
CoagWs.Range("D18").Value = WeirWidth_TB.Value
'CoagWs.Range("D19").Value = DisCoef_TB.Value

'Output Values
AveGT_TB.Value = CoagWs.Range("D13").Value
DownChaVol_TB.Value = CoagWs.Range("D14").Value
HeadAcWeir_TB.Value = CoagWs.Range("D16").Value
SuborEl_TB.Value = CoagWs.Range("D17").Value
WeirPos_L.Value = CoagWs.Range("A20").Value
WeirPos_TB.Value = CoagWs.Range("D20").Value
RetTMinFlow_TB.Value = CoagWs.Range("D23").Value
HLMinFlow_TB.Value = CoagWs.Range("D24").Value
VGMinFlow_TB.Value = CoagWs.Range("D26").Value
GTMinFlow_TB.Value = CoagWs.Range("D27").Value
RetTMaxFlow_TB.Value = CoagWs.Range("D30").Value
HLMaxFlow_TB.Value = CoagWs.Range("D31").Value
VGMaxFlow_TB.Value = CoagWs.Range("D33").Value
GTMaxFlow_TB.Value = CoagWs.Range("D34").Value

'Review
AveGTRMsg_TB.Value = CoagWs.Range("E13").Value
RetTMinFlowRMsg_TB.Value = CoagWs.Range("E23").Value
VGMinFlowRMsg_TB.Value = CoagWs.Range("E26").Value
GTMinFlowRMsg_TB.Value = CoagWs.Range("E27").Value
RetTMaxFlowRMsg_TB.Value = CoagWs.Range("E30").Value
VGMaxFlowRMsg_TB.Value = CoagWs.Range("E33").Value
GTMaxFlowRMsg_TB.Value = CoagWs.Range("E34").Value

End Sub
```

Figure 8: Hydraulic coagulation userform coding

To use the software model the user should first navigate to the excel sheet titled “Main Page”, as shown in Figure 1 in Annexure B. Now the user can click on any of the buttons in any of the different process steps and the respective userform will come up on the screen. Figure 7 below shows the main page layout and pop up userform for DAF.

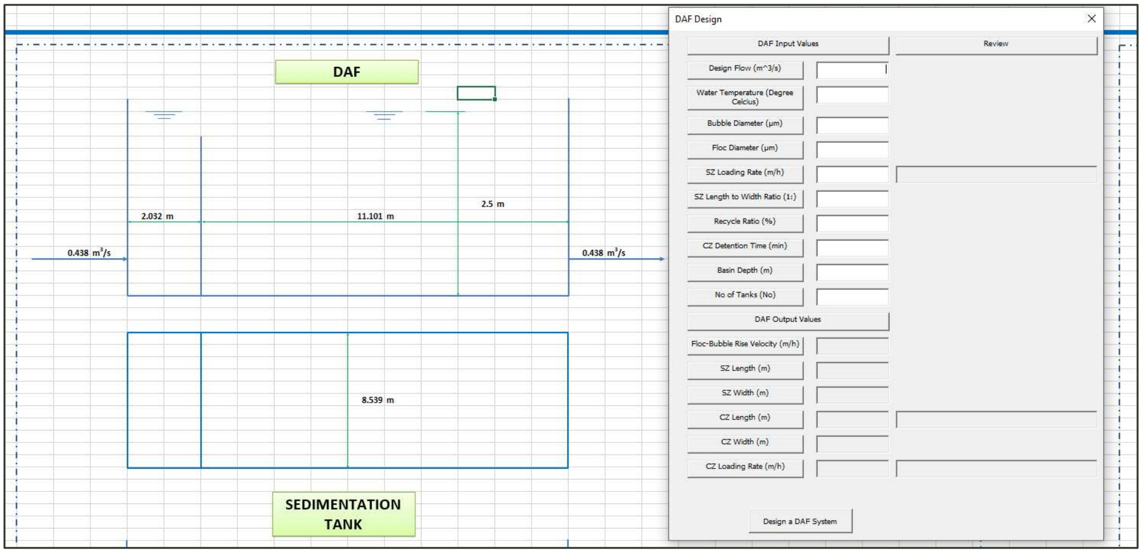
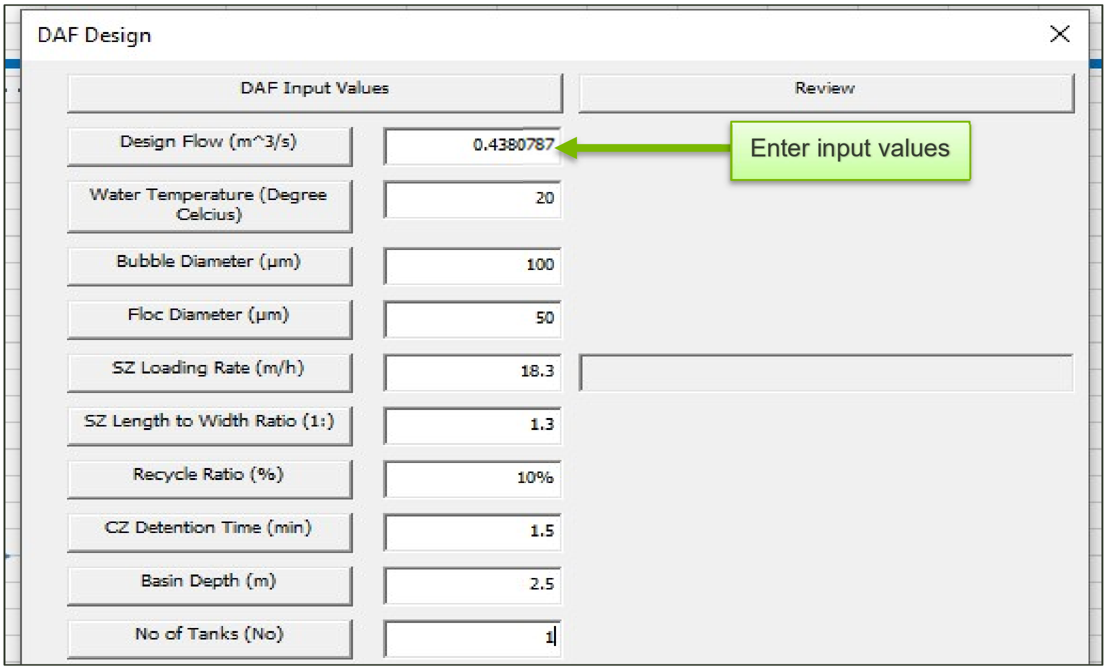


Figure 9: DAF design page

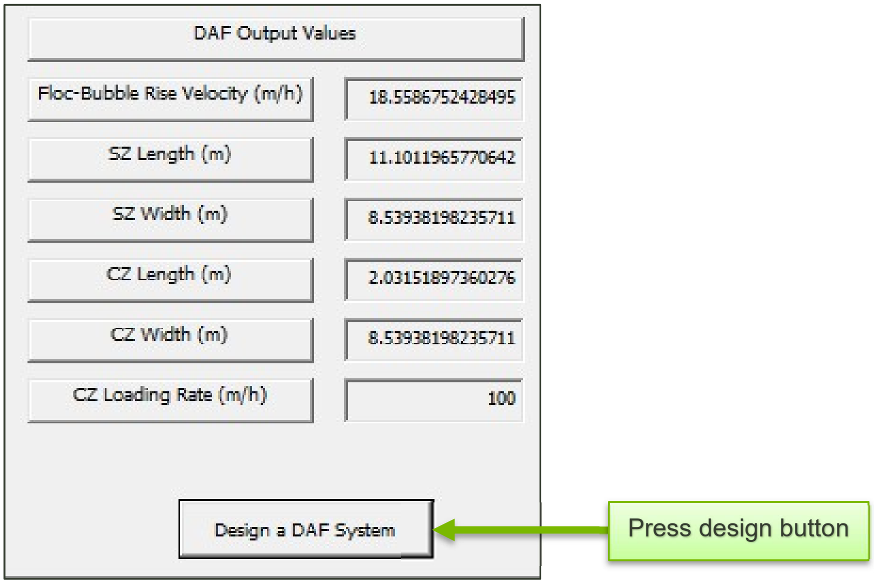
Next, the user can proceed to add values to the input boxes (the boxes with a white background). The ranges of some input values are included in Annexure A. Examples of how this is done are shown in the figures included in Annexure B. Once all the input values have been entered the user can click on the design button (always located at the end of the userform) and the output values will be generated and displayed in the output boxes (the boxes with a grey background). The review boxes will indicate if any design parameters have been exceeded and will display “NO ERROR” if not and a short description if exceeded. If any parameters are exceeded the user should change the input values until no errors are displayed. This is shown in a step by step process below.



Design Flow (m^3/s): 0.4380787
Water Temperature (Degree Celcius): 20
Bubble Diameter (µm): 100
Floc Diameter (µm): 50
SZ Loading Rate (m/h): 18.3
SZ Length to Width Ratio (1:): 1.3
Recycle Ratio (%): 10%
CZ Detention Time (min): 1.5
Basin Depth (m): 2.5
No of Tanks (No): 1

Enter input values

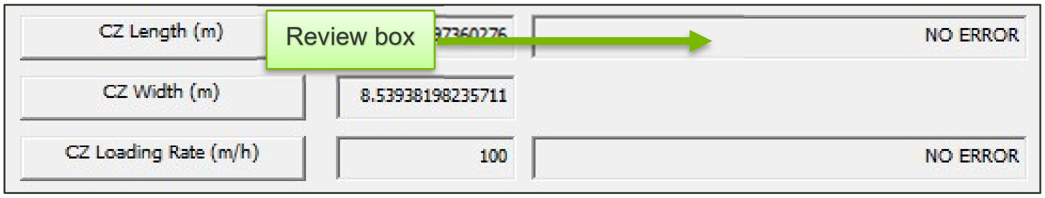
Figure 10: Step 1 - Enter input values


DAF Output Values

Floc-Bubble Rise Velocity (m/h) 18.5586752428495
SZ Length (m) 11.1011965770642
SZ Width (m) 8.53938198235711
CZ Length (m) 2.03151897360276
CZ Width (m) 8.53938198235711
CZ Loading Rate (m/h) 100

Press design button

Figure 11: Step 2 – Press design button


CZ Length (m) 97360276 NO ERROR
CZ Width (m) 8.53938198235711
CZ Loading Rate (m/h) 100 NO ERROR

Review box

Figure 12: Step 3 - Assess review and make changes as required

In an attempt to validate the software model, its output values are compared to systems that have been designed by the author’s colleagues or to systems found in examples from literature. The results of these comparisons are included in Annexure B. The examples in Annexure B may also be useful to the user for further ways on how to use the software model.

5 Conclusions and Recommendations

5.1 Conclusions

This dissertation was developed with the aim of providing engineers and designers with a guide for reviewing raw water quality aspects to aid them in configuring a sampling regime as well as in choosing the correct treatment process. The latter part of the dissertation focusses on development of a tool (software model) for designing some conventional water treatment processes.

Firstly, a list of the potential determinants, likely to be found in the source water, was developed with the help of experienced process engineers. Following, a detailed literature review of each determinant was undertaken with the focus on:

- The likelihood of certain determinants being present in some waters as well as the linkages and interdependencies between them. This is so the list can be tailored for specific conditions to mitigate unnecessary testing.
- The effects on human health, infrastructure and/or treatment processes.
- The influence on certain water characteristics (e.g. DO concentration on pH)
- Possible treatment processes to remove these determinants.
- The determinant’s concentration in treated water quality, as allowed by the SANS.

During this review it became clear there are a myriad of factors which influence when and where to test for certain determinants and that information found in literature is no substitute for practical experience. Rather, the literature information helps to develop the judgement and skills of already experienced designers. However, having a grasp on the linkages between determinants are useful in predicting the likely presence of others, for example chlorophyll-a may indicate there's algae in the water certain times of the year.

An understanding of the adverse effects on human health, infrastructure and/or treatment processes helps with identifying which determinants to target for removal or adjustment. For example, water having a low pH may not necessarily pose any immediate health risk but could indicate water is aggressive to surrounding infrastructure. The same may be true for a high pH water that would negatively impact on treatment processes such as coagulation and disinfection. Another example is that Zn may be found at levels that's acceptable from a health perspective but could still be detrimental to the organisms required for biological filtration.

From studying the different determinants, it became apparent that a knowledge of their effects on certain water characteristics is critical. For instance, it may be admirable to want to add oxygen to water to act as oxidising agent, however, this could reduce the pH and have adverse downstream effects.

Several treatment processes were identified to effect removal of specific determinants. However, from this study it's recognised that these should never be considered in isolation from other processes or determinants. This is emphasized by say wanting to employ RO filtration for reducing TDS concentrations when the water is also high in K that may lead to fouling.

The second part of the dissertation focusses on the design of some conventional water treatment process steps, comprising grit removal, aeration, coagulation, flocculation, sedimentation, filtration and disinfection. During the design development of these processes, it became clear there are many approaches one can take and often requires tailoring by the designer. A good example of this is designing an aeration system where the site may be hydraulically constrained for a cascading weir and the designer may wish to know how much aeration can be achieved and then supplement the balance with diffused aeration.

The end product of this part of the dissertation is a design tool/software model that has been tailored to the authors preferred approach and that may be used by others to design a conventional treatment processes.

5.2 Recommendations

When attempting to design a water treatment process one firstly must establish a suitable water quality sampling and testing programme. This can be done by consulting the information contained in this dissertation as well as additional information found in literature. Such a programme should be determined by a suitably qualified and experienced process engineer who understands the factors that may impact on the programme. What is also important is the duration of the sampling programme and typically a 6-month duration, with maximum cover in seasonal variation, is recommended.

The process design can commence after sufficient water quality information has been obtained and analysed by a qualified and experienced process engineer. If conventional treatment is warranted, then the software model, developed as part of this dissertation can be used. However, it is recommended that the results be compared with the designer's own calculations and design approach. In other words, it should be considered for validation of other designs rather than as a standalone method.

6 References

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ANNEXURE A

Table 1: Aeration design parameters

Aeration			
Section	**Description**	**Range**	**Source/comment**
General Input Values	Pressure (atm)	Around 1 atm	System is typically exposed to atmosphere.
V-Notch Weirs & Level Weirs	No of Weirs in Series (No)	1 – 5	
V-Notch Weirs	Notch Angle (Degrees)	30º - 90º	

Table 2: Coagulation design parameters

Coagulation			
Section	**Description**	**Range**	**Source**
General Input Values	Velocity Gradient (s^-1)	600 – 1000	Delphos and Letterman, 2012:Section 7.4.3.1
Hydraulic Mixing	Design Flow Retention Time (s)	1 – 5	Kawamura, 2000:88
Mechanical Mixing	Percentage of Plant Flowrate (%)	2 – 5	Kawamura, 2000:95

Table 3: Flocculation design parameters

Flocculation			
Section	**Description**	**Range**	**Source/comment**
General Input Values	Retention Time (min)	30 - 45	Kawamura, 2000:121
Hydraulic Flocculation	Water Depth at Outlet (m)	1 - 2	
Hydraulic Flocculation	Length to Width Ratio (1:)	Typically set at around 2	Changed to suit site constraints
Hydraulic/ Mechanical Flocculation	No of Systems (No)	Minimum of 2 to provide redundancy	Good design practise
Hydraulic Flocculation	No of Stages (No)	2 - 5	
Mechanical Flocculation	No of Stages (No)	2 - 6	
Hydraulic Flocculation	Velocity Gradient @ DF (s^-1)	Tapered from 50 to 10	Kawamura, 2000:121
Hydraulic Flocculation	No of Compartments (No)	Staggered from anywhere between 50 to 10	
Mechanical Flocculation	Velocity Gradient @ DF (s^-1)	Tapered from 70 to 10	Kawamura, 2000:121

Table 4: DAF design parameters

Dissolved Air Floatation (DAF)		
Description	**Range**	**Source/comment**
Bubble diameter (µm)	Typically, around 100	Leppinen and Dalziel; Haarhof and Edzwald, cited in Gregory and Edzwald, 2011:Section 9.5.3
Floc diameter (µm)	Typically, around 50	Gregory and Edzwald, 2011: Section 9.5.3
Separation zone hydraulic loading rate (m/h)	6 – 18	Gregory and Edzwald, 2011:Section 9.6.1
Separation zone Length to width Ratio (1:)	1 – 2	
Recycle Ratio (%)	6 – 12	
Contact zone Detention Time (min)	1 – 2.5	
Basin Depth (m)	2 – 3.5	
No of tanks (No)	Minimum of 2 to provide duty/standby configuration	Good design practise

Table 5: Sedimentation design parameters

Sedimentation			
Section	**Description**	**Range**	**Source/comment**
Grit Chamber/ Sedimentation Tank/ Tube Settler	No of Systems (No)	Minimum of 2 to provide redundancy	Good design practise
Grit Chamber	Water Depth (m)	3 – 5	Kawamura, 2000:171
Sedimentation Tank	Water Depth (m)	3 – 4.5	
Tube Settler	Water Depth (m)	3.6 – 4.5	
Grit Chamber/ Sedimentation Tank/ Tube Settler	Baffle Wall Port Diameter (m)	Typically, 0.1 or 0.15	
Grit Chamber/ Sedimentation Tank/ Tube Settler	Baffle Wall Port Spacing (mm)	Min of 100 to allow space for steel reinforcement	*Good design practise*
Sedimentation Tank/ Tube Settler	Launder Load Rate (m^3/mh)	9 - 13	Kawamura, 2000:171
Tube Settler	Tube Settler Height (m)	Supplier dependent, but typically between 0.5 to 0.75	
	Tube Settler Incline (Deg)	Around 60 for self-cleaning	Kawamura, 2000:154

Sedimentation			
Section	**Description**	**Range**	**Source/comment**
	Tube Settler L and W (m)	Supplier dependent, but typically between 0.05 to 0.075	
	% of Tank Covered (%)	Around 75	Kawamura, 2000:182

Table 6: Filtration design parameters

Filtration			
Section	**Description**	**Range**	**Source/comment**
Slow Sand Filters	Filtration Rate (m/h)	0.1 – 0.2	Huisman and Wood, 1974:66
Slow sand Filters / High-Rate Filters	No of Filters (No)	Minimum of 2 to provide redundancy	Good design practise
Slow Sand Filters	Media Depth (m)	1.2 – 1.4	Huisman and Wood, 1974:54
Slow Sand Filters	Length to Width Ratio (1:)	Typically, around 1, but can change to suite site constraints	
Slow Sand Filters	Freeboard (m)	0.2 – 0.3	Huisman and Wood, 1974:52
Slow Sand Filters	Supernatant Water Depth (m)	1 – 1.5	
High-Rate Filters	Filter Panel Length (m)	Refer Table 12 under Section 3.5.3.2	
High-Rate Filters	Filter Panel Width (m)	Refer Table 12 under Section 3.5.3.2	
High-Rate Filters	Available Clogging Head (m)	Typically, around 1.5 – 2.0	
High-Rate Filters	Filtration Rate (m/h)	10 – 25	Kawamura, 2000:234
High-Rate Filters	Media Depth (m)	1.2 – 1.8	Castro, Mysore and Chapman, 2012:Section 9.2.3.4
High-Rate Filters	Length to Width Ratio (1:)	Typically, around 2, but change to suite site constraints and panel sizes	

Filtration			
Section	**Description**	**Range**	**Source/comment**
High-Rate Filters	Air Scour Rate (m/h)	55 – 73	Castro, Mysore and Chapman, 2012:Section 9.2.6.4
High-Rate Filters	Air Headloss (m)	Typically, around 2	

Table 7: Disinfection design parameters

Disinfection		
Description	**Range**	**Source/comment**
Prior Giardia Removal (Log)	Refer Table 13 under Section 3.6.3	
Prior Virus Removal (Log	Refer Table 13 under Section 3.6.3	
Final Chlorine Residual (mg/l)	Typically, around 1 is targeted, but can be as high as 5 where there is significant distance between treatment plant and consumer	
Baffling Factor	Refer Table 15 under Section 3.6.3	

ANNEXURE B

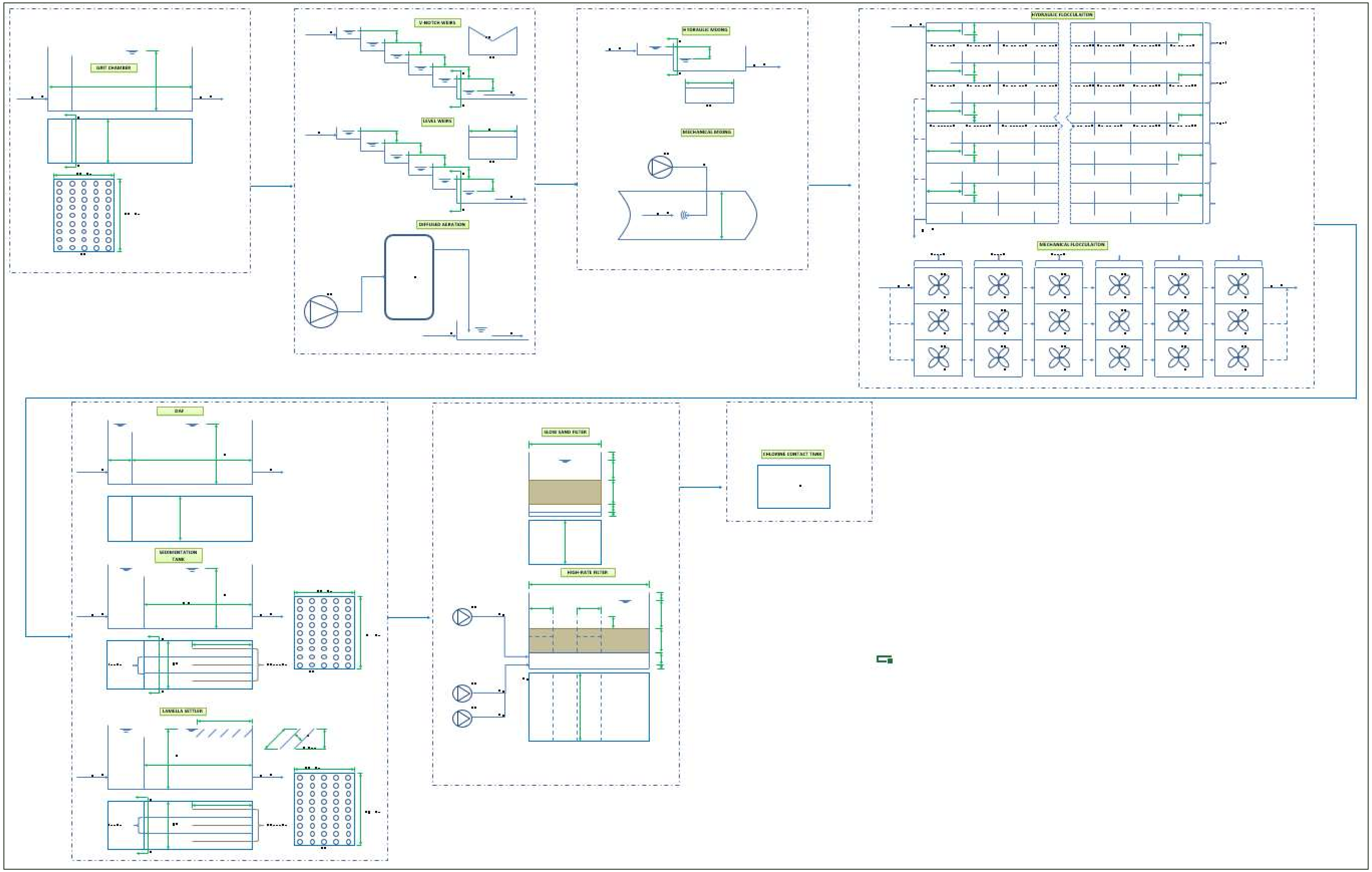


Figure 1: Process flow diagram

Aeration example 1: v-notch weirs

Determine the fall height required over a v-notch weir system to achieve a downstream dissolved oxygen concentration of 3.11 mgO$_2$/l.

The following values are given:

Design flowrate:	0.002 m^3/s
V-notch angle:	45^o
Temperature:	20^{o}C
Measured oxygen concentration:	0 mgO$_2$/l
Measured conductivity:	1000 µS/cm
Pressure at site conditions:	1 atm
Number of weirs:	1

Calculations:

$$u = exp\left(11.8571 - \frac{3840.7}{T} - \frac{216961}{T^2}\right)$$

$$u = exp\left(11.8571 - \frac{3840.7}{20 + 273.15} - \frac{216961}{(20 + 273.15)^2}\right)$$

$$\boldsymbol{u = 0.02307\ atm}$$

$$\emptyset_o = \frac{975}{10^6} - \frac{1.426 \times T}{10^5} + \frac{6.436 \times T^2}{10^8}$$

$$\emptyset_o = \frac{975}{10^6} - \frac{1.426 \times 20}{10^5} + \frac{6.436 \times 20^2}{10^8}$$

$$\boldsymbol{\emptyset_o = 0.0007155}$$

$$F_p = \left(\frac{(P-u)(1-\emptyset_o P)}{(1-u)(1-\emptyset_o)}\right)$$

$$F_p = \left(\frac{(1-0.02307)(1-0.0007155 \times 1)}{(1-0.02307)(1-0.0007155)}\right)$$

$$\boldsymbol{F_p = 1}$$

$$S = 5.572 \times 10^{-4} \times SC + 2.02 \times 10^{-9} \times SC^2$$

$$S = 5.572 \times 10^{-4} \times 1000 + 2.02 \times 10^{-9} \times 1000^2$$

$$\boldsymbol{S = 0.559\ {}^{0}/_{00}}$$

$$F_s = exp\left[-S \times \left(0.017674 - \frac{10.754}{T} + \frac{2140.7}{T^2}\right)\right]$$

$$F_s = exp\left[-0.559 \times \left(0.017674 - \frac{10.754}{(273.15 + 20)} + \frac{2140.7}{(273.15 + 20)^2}\right)\right]$$

$$\boldsymbol{F_s = 0.997}$$

$$DO_o = exp\left(-139.34411 + \frac{1.575701 \times 10^5}{T} - \frac{6.642308 \times 10^7}{T^2} + \frac{1.243800 \times 10^{10}}{T^3} - \frac{8.621949 \times 10^{11}}{T^4}\right)$$

$$DO_o = exp\left(-139.34411 + \frac{1.575701 \times 10^5}{273.15 + 20} - \frac{6.642308 \times 10^7}{(273.15 + 20)^2} + \frac{1.243800 \times 10^{10}}{(273.15 + 20)^3} - \frac{8.621949 \times 10^{11}}{(273.15 + 20)^4}\right)$$

$$\boldsymbol{DO_o = 9.092\ {}^{mgO_2}/_{l}}$$

$$C_s = DO_o \times F_S \times F_P$$

$$C_s = 9.092 \times 0.997 \times 1$$

$$\boldsymbol{C_s = 9.062\ {}^{mgO_2}/_{l}}$$

$$C_{d1} = C_{dz}$$

$$\boldsymbol{C_{d1} = 3.11\ {}^{mgO_2}/_{l}}$$

$$E = \frac{C_{d1} - C_{u1}}{C_s - C_{u1}}$$

$$E = \frac{3.11 - 0}{9.062 - 0}$$

$$\boldsymbol{E = 0.34}$$

$$f = 1.0 + 0.02103 \times (T - 20) + \frac{8.261 \times (T - 20)^2}{10^5}$$

$$f = 1.0 + 0.02103 \times (20 - 20) + \frac{8.261 \times (20 - 20)^2}{10^5}$$

$$\boldsymbol{f = 1.0}$$

$$E_{20} = 1 - (1 - E)^{\frac{1}{f}}$$

$$E_{20} = 1 - (1 - 0.34)^{\frac{1}{1}}$$

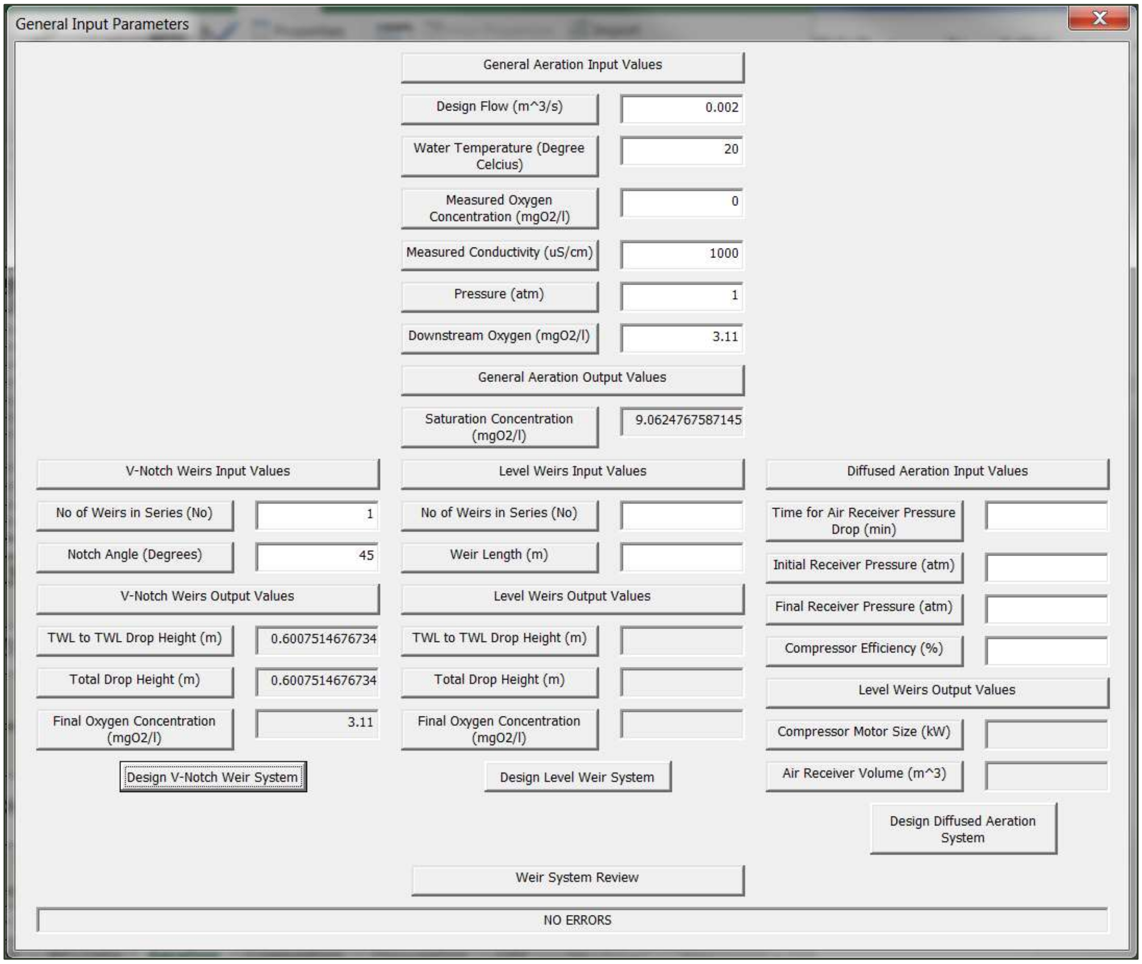
$$\boldsymbol{E_{20} = 0.34}$$

$$h = \left(\frac{[(1 - E_{20})^{-1} - 1] \times Q^{0.28} \times (sin\frac{\emptyset}{2})^{0.206}}{0.149} \right)^{\frac{1}{1.341}}$$

$$h = \left(\frac{[(1 - 0.34)^{-1} - 1] \times 0.002^{0.28} \times (sin\frac{45^{\circ}}{2})^{0.206}}{0.149} \right)^{\frac{1}{1.341}}$$

$$\boldsymbol{h = 0.6\ m}$$

An online calculator developed by the United States Geological Survey (2018) can be used to validate the saturation concentration shown above. Further, the aeration efficiency at 20°C and the fall height matches those calculated by Baylar, Hanbay and Ozpolat (2008:1220). These results are also shown in the Figure 2.



Design Flow (m^3/s): 0.002
Water Temperature (Degree Celcius): 20
Measured Oxygen Concentration (mgO2/l): 0
Measured Conductivity (uS/cm): 1000
Pressure (atm): 1
Downstream Oxygen (mgO2/l): 3.11
Saturation Concentration (mgO2/l): 9.0624767587145

No of Weirs in Series (No): 1
Notch Angle (Degrees): 45
TWL to TWL Drop Height (m): 0.6007514676734
Total Drop Height (m): 0.6007514676734
Final Oxygen Concentration (mgO2/l): 3.11

No of Weirs in Series (No):
Weir Length (m):
TWL to TWL Drop Height (m):
Total Drop Height (m):
Final Oxygen Concentration (mgO2/l):

Time for Air Receiver Pressure Drop (min):
Initial Receiver Pressure (atm):
Final Receiver Pressure (atm):
Compressor Efficiency (%):
Compressor Motor Size (kW):
Air Receiver Volume (m^3):

NO ERRORS

Figure 2: V-Notch weir calculations

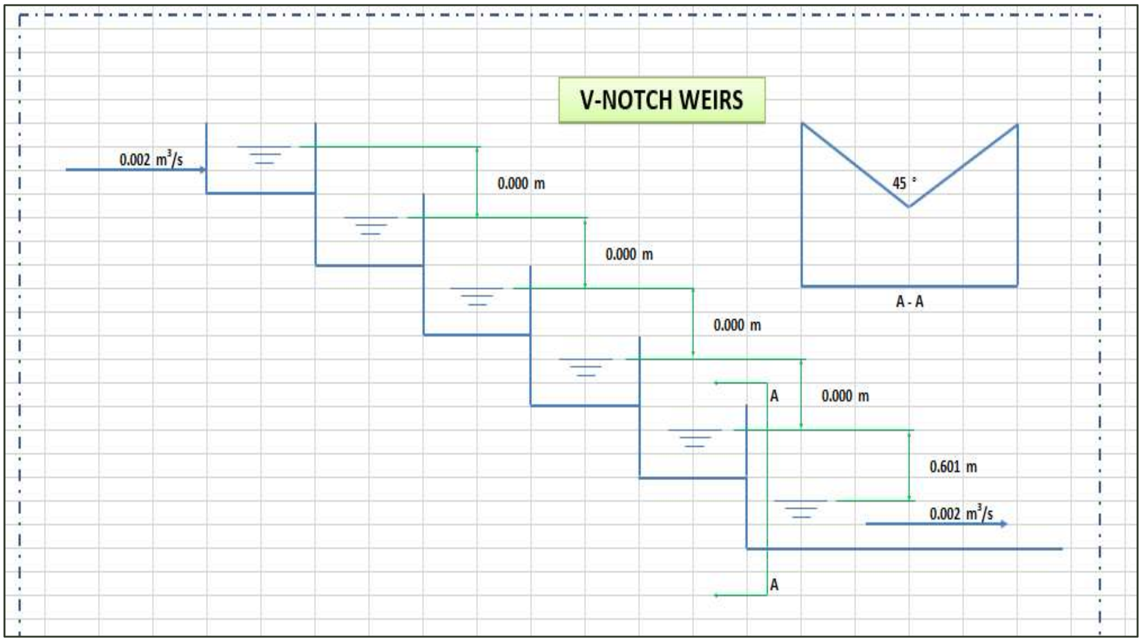


Figure 3: V-Notch weir schematic

Aeration example 2: level rectangular weirs

Determine the fall height required over a horizontally placed rectangular weir system to achieve a downstream dissolved oxygen concentration of 5.1 mgO$_2$/l.

The following values are given:

Design flowrate:	20 Ml/d (0.231 m^3/s)
Weir length:	1 m
Temperature:	20^{o}C
Measured oxygen concentration:	1 mgO$_2$/l
Measured conductivity:	1000 µS/cm
Pressure at site conditions:	1 atm
Number of weirs:	3
Saturation concentration:	Same as for v-notch weirs

Calculations:

$$C_{d1} = C_{dz} - Y - (C_s - Y) \times X$$

Where:

$$X = \frac{C_{d1} - C_{u1}}{C_s - C_{u1}}$$

$$Y = C_{d1} + \frac{(C_{d1} - C_{u1}) \times (C_s - C_{d1})}{C_s - C_{u1}}$$

$$C_{d1} = C_{dz} - \left(C_{d1} + \frac{(C_{d1} - C_{u1}) \times (C_s - C_{d1})}{C_s - C_{u1}}\right) - \left(C_s - \left(C_{d1} + \frac{(C_{d1} - C_{u1}) \times (C_s - C_{d1})}{C_s - C_{u1}}\right)\right) \times \left(\frac{C_{d1} - C_{u1}}{C_s - C_{u1}}\right)$$

$$C_{d1} = 5.1 - \left(C_{d1} + \frac{(C_{d1} - 1) \times (9.06 - C_{d1})}{9.06 - 1}\right) - \left(9.06 - \left(C_{d1} + \frac{(C_{d1} - 1) \times (9.06 - C_{d1})}{9.06 - 1}\right)\right) \times \left(\frac{C_{d1} - 1}{9.06 - 1}\right)$$

$$\boldsymbol{C_{d1} = 2.69\ {}^{mgO_2}/_{l}}$$

$$E = \frac{C_{d1} - C_{u1}}{C_s - C_{u1}}$$

$$E = \frac{2.69 - 1}{9.062 - 1}$$

$$\boldsymbol{E = 0.2099}$$

$$f = 1.0 + 0.02103 \times (T - 20) + \frac{8.261 \times (T - 20)^2}{10^5}$$

$$f = 1.0 + 0.02103 \times (20 - 20) + \frac{8.261 \times (20 - 20)^2}{10^5}$$

$$\boldsymbol{f = 1.0}$$

$$E_{20} = 1 - (1 - E)^{\frac{1}{f}}$$

$$E_{20} = 1 - (1 - 0.2099)^{\frac{1}{1}}$$

$$\boldsymbol{E_{20} = 0.2099}$$

$$r_{20} = \frac{1}{1 - E_{20}}$$

$$r_{20} = \frac{1}{1 - 0.2099}$$

$$\boldsymbol{r_{20} = 1.266}$$

$$q_j = \frac{Q}{2b_j}$$

$$q_j = \frac{0.231}{2 \times 1}$$

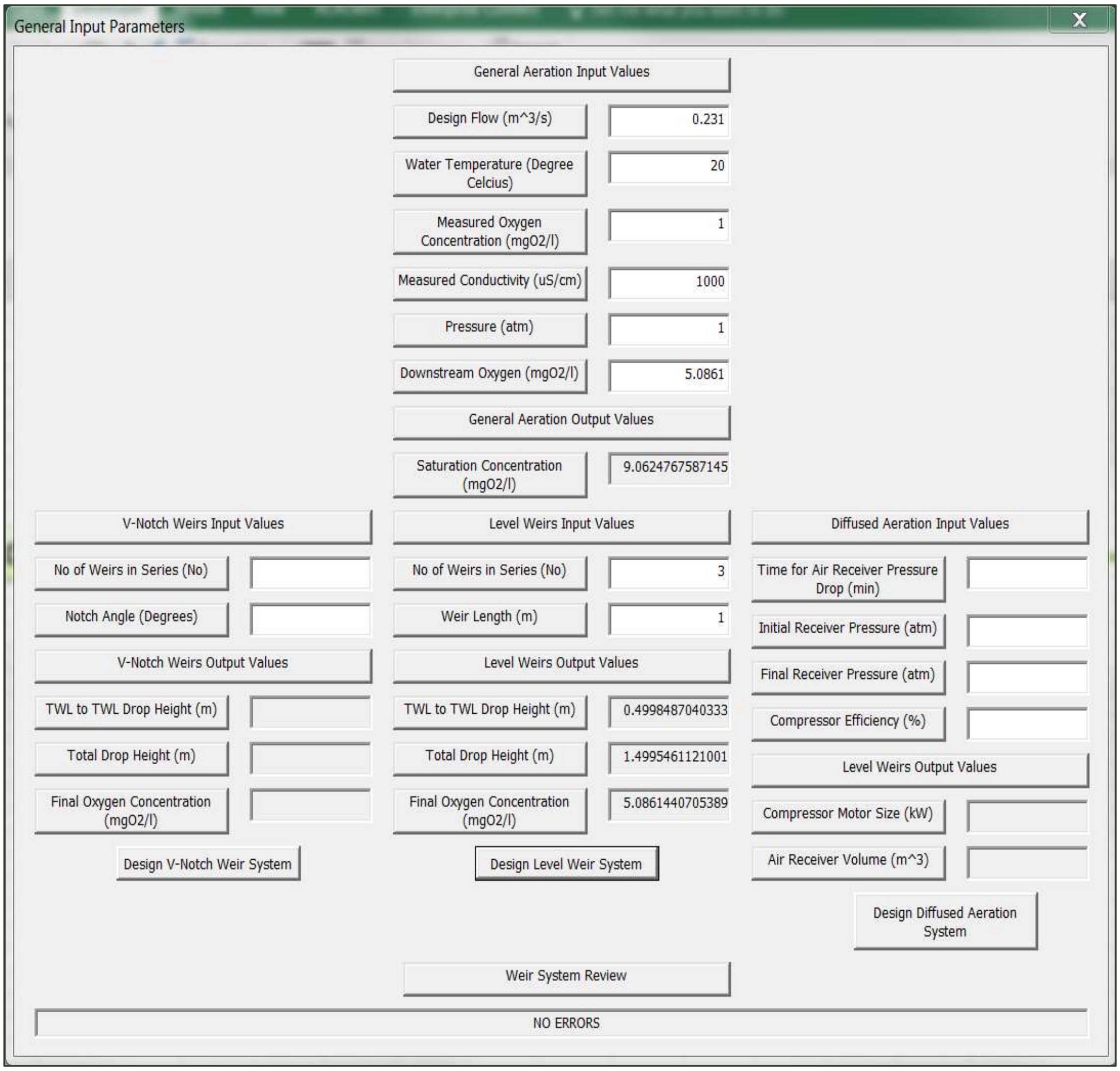
$$\boldsymbol{q_j = 0.1157\ {}^{m^3}/_{m.s}}$$

$$h = \left[\frac{2^{0.11325} \times q_j^{0.2265} \times (r_{20} - 1)}{0.453 \times 0.667^{0.475} \times g^{0.11325}}\right]^{\frac{1}{1.45675}}$$

$$h = \left[\frac{2^{0.11325} \times 0.1157^{0.2265} \times (1.266 - 1)}{0.453 \times 0.667^{0.475} \times 9.81^{0.11325}}\right]^{\frac{1}{1.45675}}$$

$$\boldsymbol{h = 0.5\ m}$$

Others (work colleague, D. Petrie) have used the methods and results above to design a horizontal weir aeration system. The above results are also shown in Figure 4.



General Aeration Input Values

Design Flow (m^3/s) 0.231

Water Temperature (Degree Celcius) 20

Measured Oxygen Concentration (mgO2/l) 1

Measured Conductivity (uS/cm) 1000

Pressure (atm) 1

Downstream Oxygen (mgO2/l) 5.0861

General Aeration Output Values

Saturation Concentration (mgO2/l) 9.0624767587145

Level Weirs Input Values

No of Weirs in Series (No) 3

Weir Length (m) 1

Level Weirs Output Values

TWL to TWL Drop Height (m) 0.4998487040333

Total Drop Height (m) 1.4995461121001

Final Oxygen Concentration (mgO2/l) 5.0861440705389

NO ERRORS

Figure 4: Level rectangular weir calculations

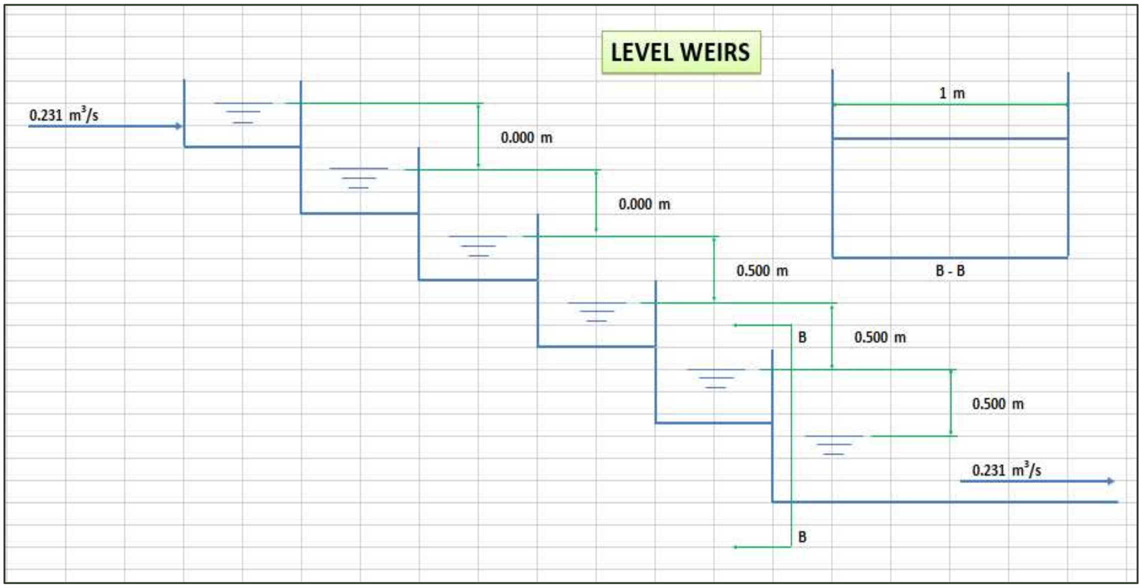


Figure 5: Level rectangular weir schematic

Aeration example 3: diffused aeration

Determine the compressor motor size and air receiver volume of a system based on an oxygen demand of 5 mgO$_2$/l.

The following values are given:

Design flowrate:	20 Ml/d (0.231 m^3/s)
Weir length:	1 m
Temperature:	20°C
Measured oxygen concentration:	1 mgO$_2$/l
Measured conductivity:	1000 µS/cm
Pressure at site conditions:	1 atm
Time for pressure drop to occur:	5 min
Initial receiver pressure	10 atm
Final receiver pressure	8 atm
Motor efficiency	60 %

Calculations:

$$O_2 = \frac{Q \times 3600 \times 1000 \times C}{10^6}$$

$$O_2 = \frac{0.231 \times 3600 \times 1000 \times (5-1)}{10^6}$$

$$\boldsymbol{O_2 = 3.33 \, {}^{kgO_2}/_{h}}$$

$$n = \frac{O_2 \times 1000}{32}$$

$$n = \frac{3.33 \times 1000}{32}$$

$$\boldsymbol{n = 104.06 \, mol}$$

$$V = \frac{nRT}{P}$$

$$V = \frac{104.06 \times 8.314 \times (273.15 + 20)}{1 \times 101325}$$

$$\boldsymbol{V = 2.504\ {}^{m^3O_2}\!/_{h}}$$

$$FAD = \frac{V}{DE \times OC_{air}}$$

$$FAD = \frac{2.504}{0.3 \times 0.21}$$

$$\boldsymbol{FAD = 39.77\ {}^{m^3Air}\!/_{h}}$$

$$P_c = \frac{P_{increase} \times FAD}{\eta \times 3600 \times 1000}$$

$$P_c = \frac{(10-1) \times 1013.25 \times 39.77}{0.6 \times 3600 \times 1000}$$

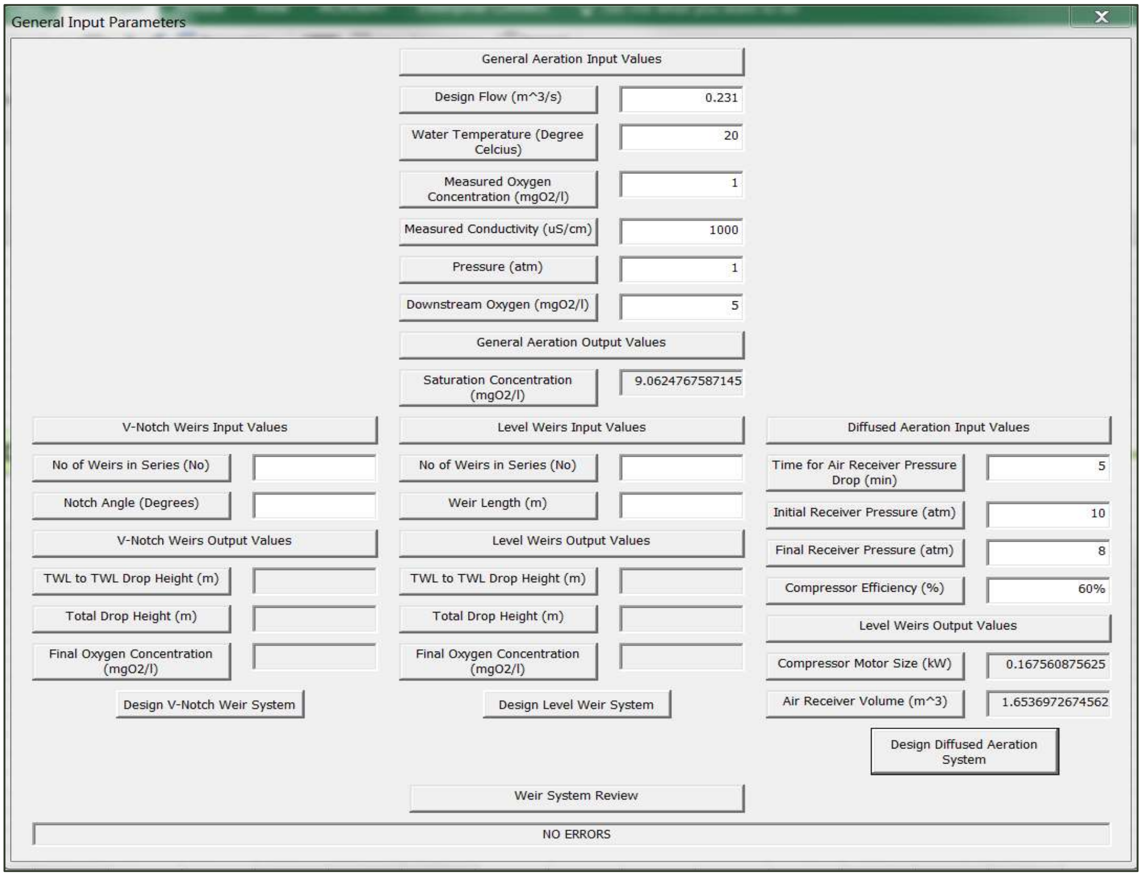
$$\boldsymbol{P_c = 0.168\ kW}$$

$$V = t \times \frac{FAD \times P_a}{(P_1 - P_2) \times 60}$$

$$V = 5 \times \frac{39.77 \times 1}{(10-8) \times 60}$$

$$\boldsymbol{V = 1.66\ m^3}$$

The method for calculating the above free air demand has been used by others (work colleague, D. Petrie) in the design of diffused aeration systems. Exact sizes of the compressor motor and air receiver volume typically requires input from a supplier, however, the above results provide indicative numbers for preliminary sizing. The above results are reflected in Figure 6.



General Input Parameters

General Aeration Input Values

Design Flow (m^3/s): 0.231

Water Temperature (Degree Celcius): 20

Measured Oxygen Concentration (mgO2/l): 1

Measured Conductivity (uS/cm): 1000

Pressure (atm): 1

Downstream Oxygen (mgO2/l): 5

General Aeration Output Values

Saturation Concentration (mgO2/l): 9.0624767587145

Diffused Aeration Input Values

Time for Air Receiver Pressure Drop (min): 5

Initial Receiver Pressure (atm): 10

Final Receiver Pressure (atm): 8

Compressor Efficiency (%): 60%

Level Weirs Output Values

Compressor Motor Size (kW): 0.167560875625

Air Receiver Volume (m^3): 1.6536972674562

NO ERRORS

Figure 6: Diffused aeration calculations

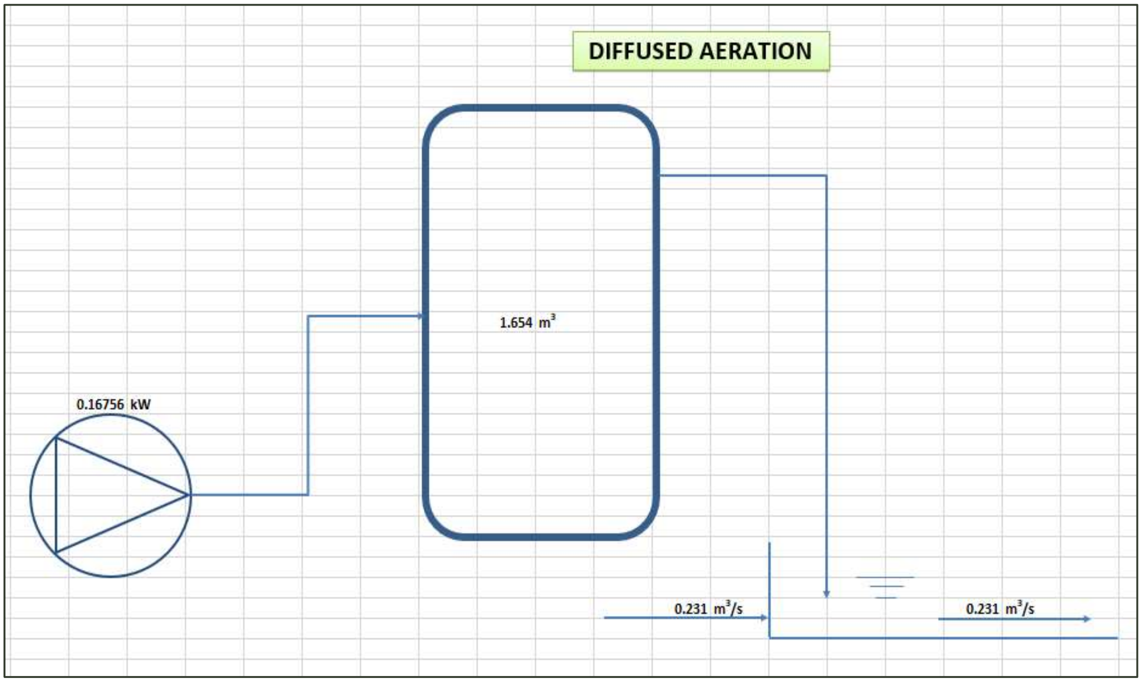


Figure 7: Diffused aeration schematic

Coagulation example 1: hydraulic mixing

Design a hydraulic mixing system to achieve a velocity gradient of 1000 s^{-1} at a design flowrate of 0.1894 m^3/s (~ 16 Ml/d).

The following values are given:

Retention time:	1.375 s
Weir length:	0.3 m
Temperature:	12°C
Absolute viscosity:	1.2349×10^{-3} $N.s/m^2$
Water density:	999.45 kg/m^3

Calculations:

$$Gt_{des} = G_{des} \times t_{des}$$

$$Gt_{des} = 1000 \times 1.375$$

$$\boldsymbol{Gt_{des} = 1375}$$

$$V_{chamber} = Q \times t_{des}$$

$$V_{chamb} \quad = 0.1894 \times 1.375$$

$$\boldsymbol{V_{chamber} = 0.2605\ m^3}$$

$$P = G_{des}^2 \times \mu \times V_{chamber}$$

$$P = 1000^2 \times 1.2349 \times 10^{-3} \times 0.2605$$

$$\boldsymbol{P = 321.69\ Watts}$$

$$h = \frac{P}{g \times \rho \times Q}$$

$$h = \frac{321.69}{9.81 \times 999.45 \times 0.1894}$$

$$\boldsymbol{h = 0.173\ m}$$

$$h_{sub} = \frac{Q - \frac{2}{3} \times C_D \times B \times \sqrt{2 \times g} \times h^{\frac{3}{2}}}{C_D \times B \times \sqrt{2 \times g \times h}}$$

$$h_{sub} = \frac{0.1894 - \frac{2}{3} \times 0.6 \times 0.3 \times \sqrt{2 \times 9.81} \times 0.173^{\frac{3}{2}}}{0.6 \times 0.3 \times \sqrt{2 \times 9.81 \times 0.173}}$$

$$\boldsymbol{h_{sub} = 0.455\ m}$$

Others (work colleague, G du Toit) have used the above methods and results in the design of hydraulic mixing systems.

Minimum and maximum calculations:

$$t_{max} = \frac{V_{Chamber}}{Q_{min}}$$

$$t_{max} = \frac{0.2605}{0.15}$$

$$\boldsymbol{t_{max} = 1.74\ s}$$

$$0 = Q_{min} - \left(\frac{2}{3} \times C_D \times B \times \sqrt{2 \times g} \times h_{min}^{\frac{3}{2}} + C_D \times B \times h_{sub} \times \sqrt{2 \times g \times h_{min}}\right)$$

$$0 = 0.15 - \left(\frac{2}{3} \times 0.6 \times 0.3 \times \sqrt{2 \times 9.81} \times h_{min}^{\frac{3}{2}} + 0.6 \times 0.3 \times 0.455 \times \sqrt{2 \times 9.81 \times h_{min}}\right)$$

$$\boldsymbol{h_{min} = 0.123\ m}$$

$$P_{min} = h_{min} \times Q_{min} \times g \times \rho$$

$$P_{min} = 0.123 \times 0.15 \times 9.81 \times 999.45$$

$$\boldsymbol{P_{min} = 180.205\ Watts}$$

$$G_{min} = \sqrt{\frac{P_{min}}{\mu \times V_{chamber}}}$$

$$G_{min} = \sqrt{\frac{180.205}{1.2349 \times 10^{-3} \times 0.2605}}$$

$$\boldsymbol{G_{min} = 748.8\ s^{-1}}$$

$$Gt_{min} = G_{min} \times t_{max}$$

$$Gt_{min} = 748.8 \times 1.74$$

$$\boldsymbol{Gt_{min} = 1300}$$

$$t_{min} = \frac{V_{Chamb}}{Q_{max}}$$

$$t_{min} = \frac{0.2605}{0.2}$$

$$\boldsymbol{t_{min} = 1.302\ s}$$

$$0 = Q_{max} - \left(\frac{2}{3} \times C_D \times B \times \sqrt{2 \times g} \times h_{max}^{\frac{3}{2}} + C_D \times B \times h_{sub} \times \sqrt{2 \times g \times h_{max}}\right)$$

$$0 = 0.2 - \left(\frac{2}{3} \times 0.6 \times 0.3 \times \sqrt{2 \times 9.81} \times h_{max}^{\frac{3}{2}} + 0.6 \times 0.3 \times 0.455 \times \sqrt{2 \times 9.81 \times h_{max}}\right)$$

$$\boldsymbol{h_{max} = 0.187\ m}$$

$$P_{max} = h_{max} \times Q_{max} \times g \times \rho$$

$$P_{max} = 0.187 \times 0.2 \times 9.81 \times 999.45$$

$$\boldsymbol{P_{max} = 366.64\ Watts}$$

$$G_{max} = \sqrt{\frac{P_{max}}{\mu \times V_{chamber}}}$$

$$G_{max} = \sqrt{\frac{366.64}{1.2349 \times 10^{-3} \times 0.2605}}$$

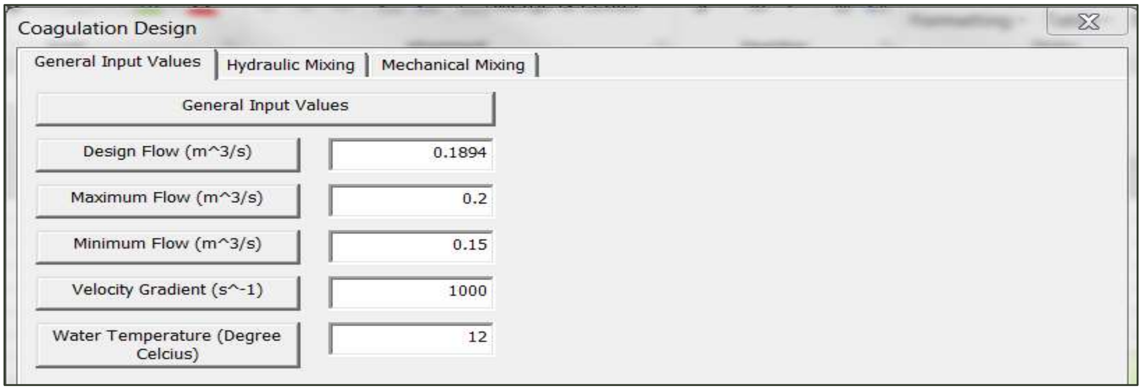
$$\boldsymbol{G_{max} = 1067.8\ s^{-1}}$$

$$Gt_{max} = G_{max} \times t_{min}$$

$$Gt_{max} = 1067.8 \times 1.302$$

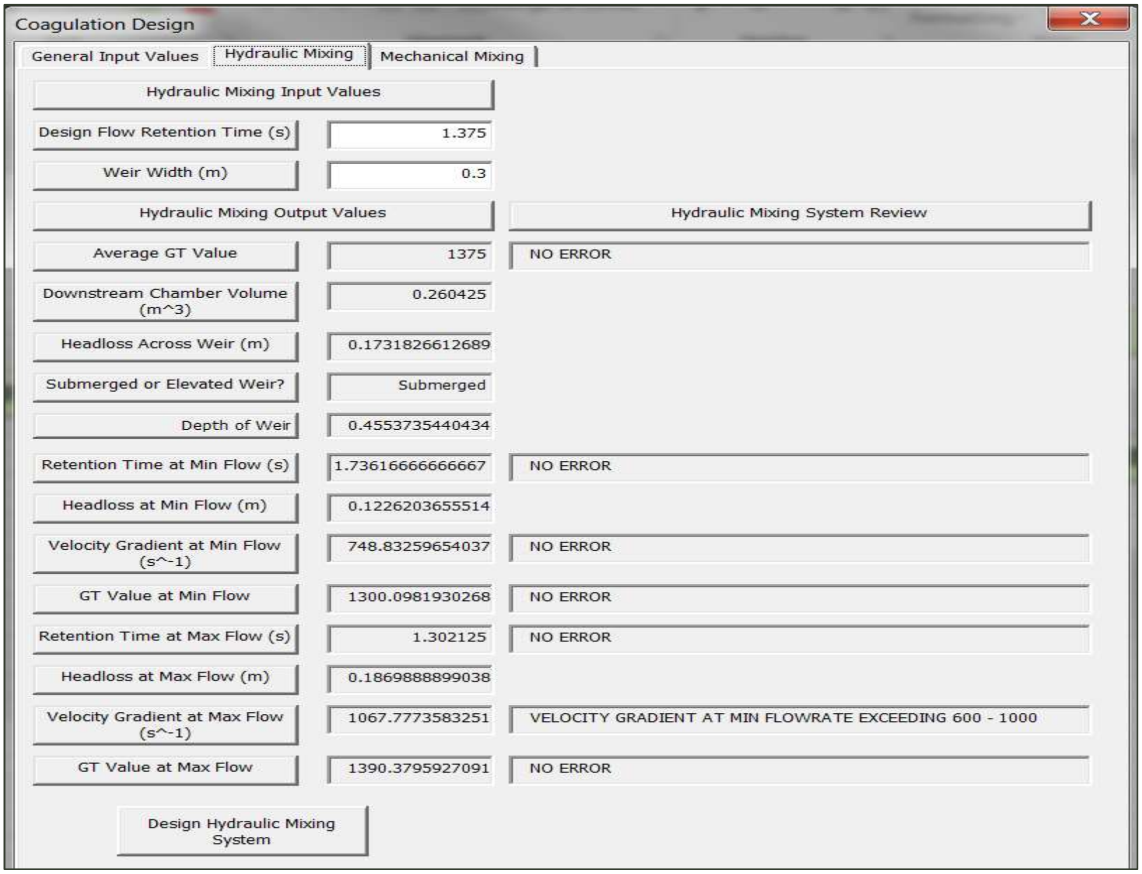
$$\boldsymbol{Gt_{max} = 1390.4}$$

Figure 9 shows the matching output from the software model, where the velocity gradient at the maximum flowrate is marginally exceeded.



Design Flow (m^3/s) 0.1894
Maximum Flow (m^3/s) 0.2
Minimum Flow (m^3/s) 0.15
Velocity Gradient (s^-1) 1000
Water Temperature (Degree Celcius) 12

Figure 8: Hydraulic mixing input values



Coagulation Design

Hydraulic Mixing Input Values		
Design Flow Retention Time (s)	1.375	
Weir Width (m)	0.3	
Hydraulic Mixing Output Values		Hydraulic Mixing System Review
Average GT Value	1375	NO ERROR
Downstream Chamber Volume (m^3)	0.260425	
Headloss Across Weir (m)	0.1731826612689	
Submerged or Elevated Weir?	Submerged	
Depth of Weir	0.4553735440434	
Retention Time at Min Flow (s)	1.73616666666667	NO ERROR
Headloss at Min Flow (m)	0.1226203655514	
Velocity Gradient at Min Flow (s^-1)	748.83259654037	NO ERROR
GT Value at Min Flow	1300.0981930268	NO ERROR
Retention Time at Max Flow (s)	1.302125	NO ERROR
Headloss at Max Flow (m)	0.1869888899038	
Velocity Gradient at Max Flow (s^-1)	1067.7773583251	VELOCITY GRADIENT AT MIN FLOWRATE EXCEEDING 600 - 1000
GT Value at Max Flow	1390.3795927091	NO ERROR

Figure 9: Hydraulic mixing calculations

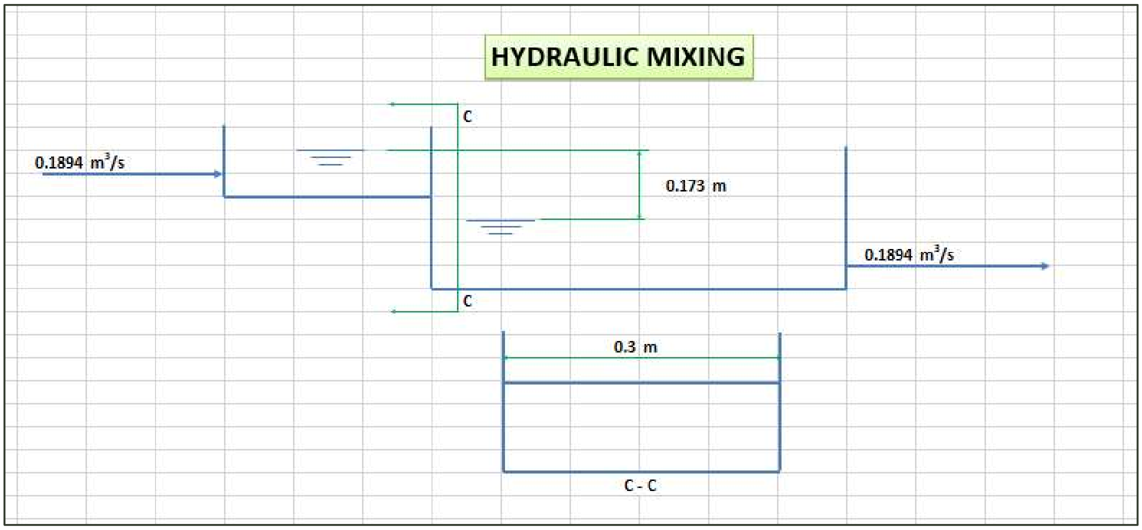


Figure 10: Hydraulic mixing schematic

Coagulation example 2: pump diffusion mixing

Design a pump diffusion mixing system that can achieve a velocity gradient of 750 s^{-1} at a plant design flowrate of 1.5 m^3/s.

<u>The following values are given</u>:

Gt value:	592.5
Percentage of design flowrate:	3%
Water temperature:	~ 12°C
Absolute viscosity:	1.336 x 10^{-3} N.s/m^2 (*temporarily changed in the software model for comparison with example found in literature*)
Water density:	1000 kg/m^3 (*temporarily changed in the software model for comparison with example found in literature*)
Nozzle discharge coefficient:	1
Pump motor efficiency:	85%
Headloss through the nozzle:	4.92 m
Headloss through the pipework:	0.3048 m

Calculations:

$$t_{des} = \frac{Gt_{des}}{G_{des}}$$

$$t_{des} = \frac{592.5}{750}$$

$$\boldsymbol{t_{des} = 0.79}$$

$$V_{mix} = Q \times t_{des}$$

$$V_{mix} = 1.5 \times 0.79$$

$$\boldsymbol{V_{mix} = 1.185}$$

$$D = \left(\frac{V_{mix} \times 4}{1.5 \times \pi}\right)^{\frac{1}{3}}$$

$$D = \left(\frac{1.185 \times 4}{1.5 \times \pi}\right)^{\frac{1}{3}}$$

$$\boldsymbol{D = 1\ m}$$

$$L_{mix} = 1.5 \times D$$

$$L_{mix} = 1.5 \times 1$$

$$\boldsymbol{L_{mix} = 1.5}$$

$$Q_{pump} = r \times Q$$

$$Q_{pump} = 3\% \times 1.5$$

$$\boldsymbol{Q_{pump} = 0.045\ {m^3}/{s}}$$

$$P = G_{des}^2 \times \mu \times V_{mix}$$

$$P = 750^2 \times 1.336 \times 10^{-3} \times 1.185$$

$$\boldsymbol{P = 890\ Watts}$$

$$h_{dyn} = \frac{P}{Q_{pump} \times g \times \rho}$$

$$h_{dyn} = \frac{890}{0.045 \times 9.81 \times 1000}$$

$$\boldsymbol{h_{dyn} = 2.02\ m}$$

$$d_o = 2 \times \sqrt{\frac{Q_{pump}}{C_D \times \pi \times \sqrt{2 \times g \times h_{dyn}}}}$$

$$d_o = 2 \times \sqrt{\frac{0.045}{1 \times \pi \times \sqrt{2 \times 9.81 \times 2.02}}}$$

$$\boldsymbol{d_o = 0.095\ m}$$

$$h_{tot} = h_{nozzle} + h_{pipe} + h_{dyn}$$

$$h_{tot} = 4.92 + 0.3048 + 2.02$$

$$\boldsymbol{h_{tot} = 7.24\ m}$$

$$P_{h(kW)} = \frac{Q_{pump} \times g \times \rho \times h_{tot}}{1000 \times \eta}$$

$$P_{h(kW)} = \frac{0.045 \times 9.81 \times 1000 \times 7.24}{1000 \times 0.85}$$

$$\boldsymbol{P_{h(kW)} = 3.76\ kW}$$

The above nozzle orifice diameter matches the example calculated by Kawamura (2000:94-96). However, in determining the pump size the answers differ somewhat. This is because in the example by Kawamura (2000:94-96) the total pumping head is calculated based on an orifice opening and flowrate (0.044 m^3/s), linked to a specific nozzle type. Whereas, the software model assumes that the nozzle type chosen/constructed will match the orifice opening (0.095 m) and flowrate (0.045 m^3/s) as calculated above.

Minimum and maximum calculations:

$$Q_{pump,min} = r_{min} \times Q_{min}$$

$$Q_{pump,min} = 0.025 \times 1.4$$

$$\boldsymbol{Q_{pump,min} = 0.035\ m^3/_s}$$

$$h_{dyn,min} = \frac{Q_{pump,min}^2}{2 \times g \times \left(C_D \times \pi \times \left(\frac{d_o}{2}\right)^2\right)^2}$$

$$h_{dyn,min} = \frac{0.035^2}{2 \times 9.81 \times \left(1 \times \pi \times \left(\frac{0.095}{2}\right)^2\right)^2}$$

$$\boldsymbol{h_{dyn,min} = 1.22\ m}$$

$$P_{min} = Q_{pump,min} \times \rho \times g \times h_{dyn,min}$$

$$P_{min} = 0.035 \times 1000 \times 9.81 \times 1.22$$

$$\boldsymbol{P_{min} = 419\ Watts}$$

$$G_{min} = \sqrt{\frac{P_{min}}{\mu \times V_{mix}}}$$

$$G_{min} = \sqrt{\frac{419}{1.336 \times 10^{-3} \times 1.185}}$$

$$\boldsymbol{G_{min} = 514.45\ s^{-1}}$$

$$t_{max} = \frac{V_{mix}}{Q_{min}}$$

$$t_{max} = \frac{1.185}{1.4}$$

$$\boldsymbol{t_{max} = 0.85\ s}$$

$$Gt_{min} = G_{min} \times t_{max}$$

$$Gt_{min} = 514.45 \times 0.846$$

$$\boldsymbol{Gt_{min} = 435.4}$$

$$h_{nozzle,min} = \frac{h_{nozzle} \times Q_{pump,min}}{Q_{pump}}$$

$$h_{nozzle,min} = \frac{4.92 \times 0.035}{0.045}$$

$$\boldsymbol{h_{nozzle,min} = 3.83\mathrm{m}}$$

$$h_{p,min} = \frac{h_{pipe} \times Q_{pump,min}}{Q_{pump}}$$

$$h_{p,min} = \frac{0.3048 \times 0.035}{0.045}$$

$$\boldsymbol{h_{p,min} = 0.24\ m}$$

$$h_{tot,min} = h_{nozzle,min} + h_{p,min} + h_{dyn,min}$$

$$h_{tot,min} = 3.83 + 0.24 + 1.22$$

$$\boldsymbol{h_{tot,min} = 5.28\ m}$$

$$Q_{pump,max} = r_{max} \times Q_{max}$$

$$Q_{pump,max} = 0.04 \times 1.6$$

$$\boldsymbol{Q_{pump,max} = 0.064\ {}^{m^3}\!/_{s}}$$

$$h_{dyn,max} = \frac{Q_{pump,max}^2}{2 \times g \times \left(C_D \times \pi \times \left(\frac{d_o}{2}\right)^2\right)^2}$$

$$h_{dyn,max} = \frac{0.064^2}{2 \times 9.81 \times \left(1 \times \pi \times \left(\frac{0.095}{2}\right)^2\right)^2}$$

$$\boldsymbol{h_{dyn,max} = 4.08\ m}$$

$$P_{max} = Q_{pump,max} \times \rho \times g \times h_{dyn,max}$$

$$P_{max} = 0.064 \times 1000 \times 9.81 \times 4.08$$

$$\boldsymbol{P_{max} = 2562\ Watts}$$

$$G_{max} = \sqrt{\frac{P_{max}}{\mu \times V_{max}}}$$

$$G_{max} = \sqrt{\frac{2562}{1.336 \times 10^{-3} \times 1.185}}$$

$$\boldsymbol{G_{max} = 1272.07\ s^{-1}}$$

$$t_{min} = \frac{V_{mix}}{Q_{max}}$$

$$t_{min} = \frac{1.185}{1.6}$$

$$\boldsymbol{t_{min} = 0.741\ s}$$

$$Gt_{max} = G_{max} \times t_{min}$$

$$Gt_{maz} = 1272.07 \times 0.741$$

$$\boldsymbol{Gt_{max} = 942.13}$$

$$h_{nozzle,max} = \frac{h_{nozzle} \times Q_{pump,max}}{Q_{pump}}$$

$$h_{nozzle,max} = \frac{4.92 \times 0.064}{0.045}$$

$$\boldsymbol{h_{nozzle,max} = 6.997\ m}$$

$$h_{p,max} = \frac{h_{pipe} \times Q_{pump,max}}{Q_{pump}}$$

$$h_{p,max} = \frac{0.3048 \times 0.064}{0.045}$$

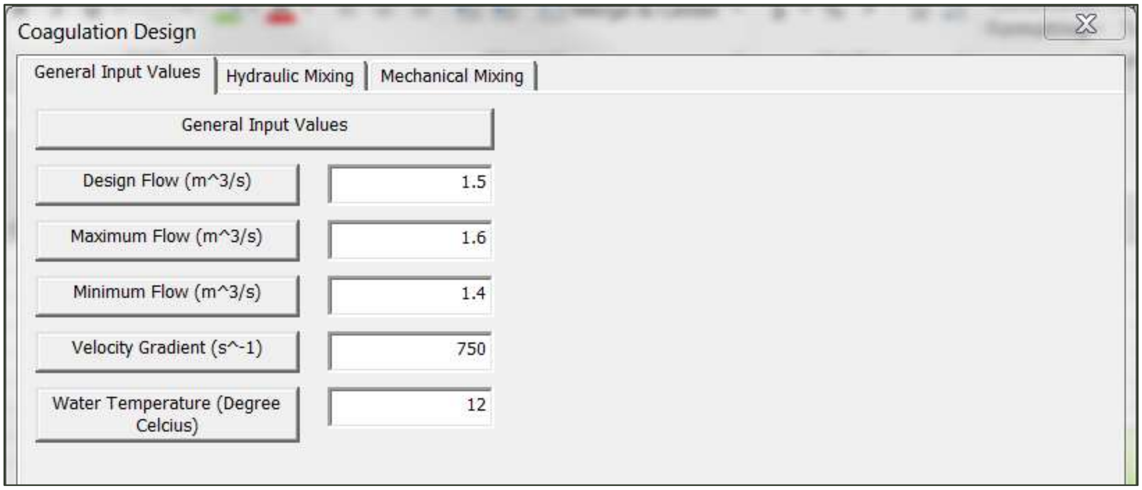
$$\boldsymbol{h_{p,max} = 0.433\ m}$$

$$h_{tot,max} = h_{nozzle,max} + h_{p,max} + h_{dyn,max}$$

$$h_{tot,max} = 6.997 + 0.433 + 4.08$$

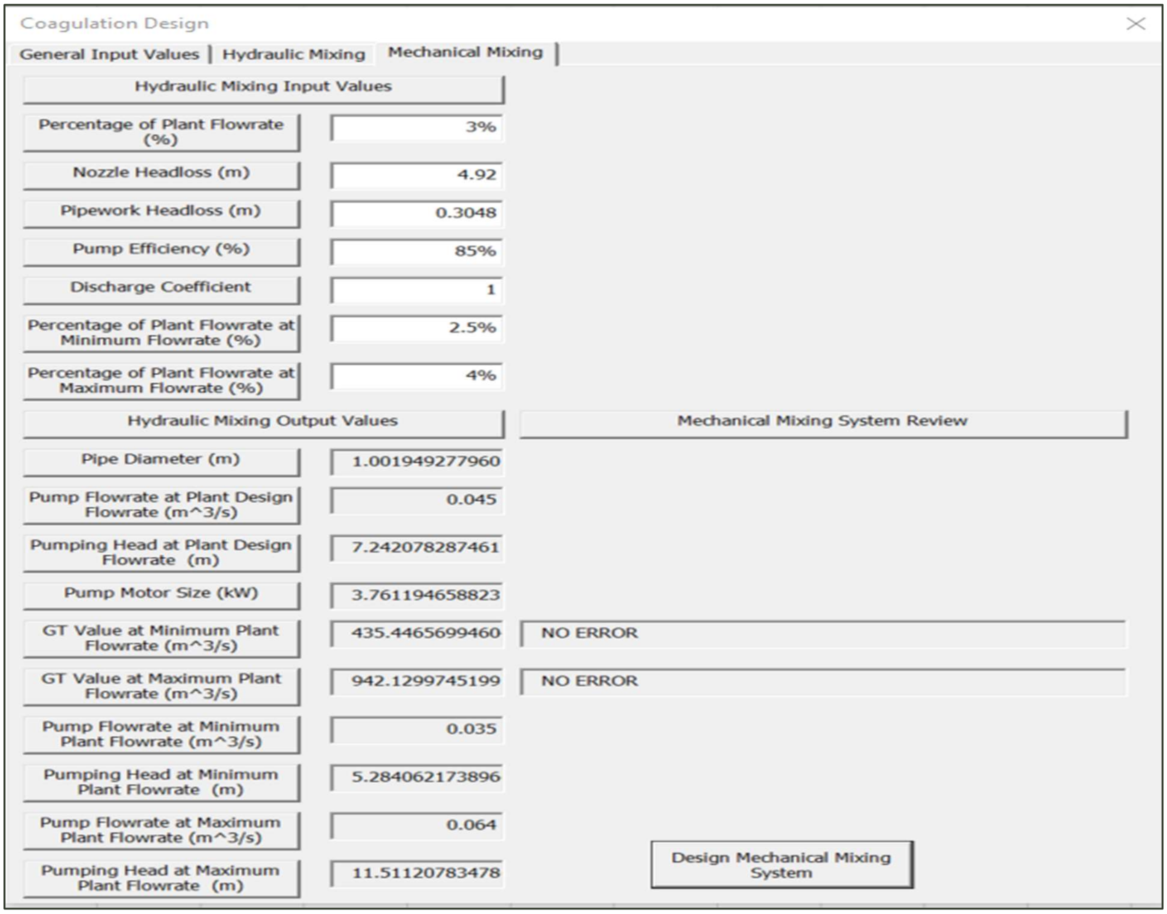
$$\boldsymbol{h_{tot,max} = 11.51\ m}$$

Figure 12 below indicates that the software model gives the same pump size as shown in the manual calculations above, with a marginal difference due to rounding of numbers.



Design Flow (m^3/s) 1.5
Maximum Flow (m^3/s) 1.6
Minimum Flow (m^3/s) 1.4
Velocity Gradient (s^-1) 750
Water Temperature (Degree Celcius) 12

Figure 11: Pump diffusion input values



Hydraulic Mixing Input Values

Percentage of Plant Flowrate (%)	3%
Nozzle Headloss (m)	4.92
Pipework Headloss (m)	0.3048
Pump Efficiency (%)	85%
Discharge Coefficient	1
Percentage of Plant Flowrate at Minimum Flowrate (%)	2.5%
Percentage of Plant Flowrate at Maximum Flowrate (%)	4%

Hydraulic Mixing Output Values

Mechanical Mixing System Review

Pipe Diameter (m)	1.001949277960	
Pump Flowrate at Plant Design Flowrate (m^3/s)	0.045	
Pumping Head at Plant Design Flowrate (m)	7.242078287461	
Pump Motor Size (kW)	3.761194658823	
GT Value at Minimum Plant Flowrate (m^3/s)	435.4465699460	NO ERROR
GT Value at Maximum Plant Flowrate (m^3/s)	942.1299745199	NO ERROR
Pump Flowrate at Minimum Plant Flowrate (m^3/s)	0.035	
Pumping Head at Minimum Plant Flowrate (m)	5.284062173896	
Pump Flowrate at Maximum Plant Flowrate (m^3/s)	0.064	
Pumping Head at Maximum Plant Flowrate (m)	11.51120783478	

Figure 12: Pump diffusion calculations

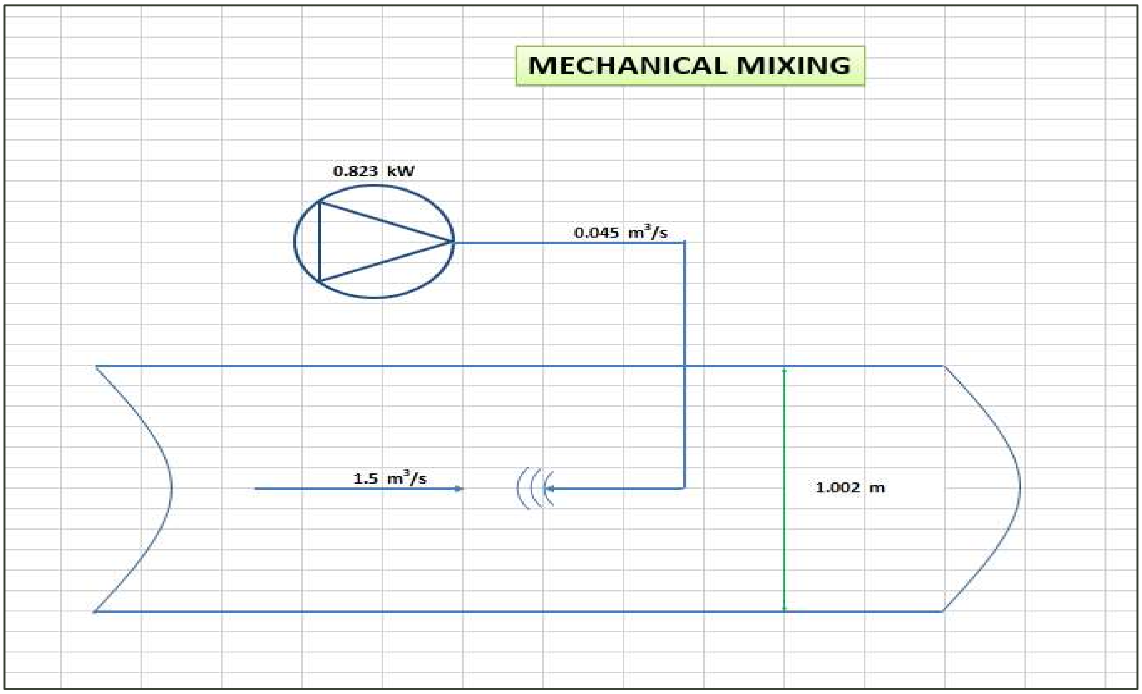


Figure 13: Pump diffusion schematic

Flocculation example 1: hydraulic system

Design a 3-stage hydraulic flocculation system for a plant design flowrate of 1 m^3/s.

<u>The following values are given</u>:

Retention time	25 min
Water temperature:	10^{o}C
Kinematic viscosity:	1.3081 x 10^{-6} m^2/s
Water depth at outlet:	2 m
Active length:	28.4 m
No of systems:	2
Velocity gradient for stage 1	70 s^{-1}
Velocity gradient for stage 2	35 s^{-1}
Velocity gradient for stage 3	20 s^{-1}
Number of compartments for stage 1	20
Number of compartments for stage 2	16
Number of compartments for stage 3	12
No of active systems	2

<u>Calculations:</u>

$$V_{st} = \frac{Q \times t_{sys} \times 60}{n_{st} \times n_{sys}}$$

$$V_{st} = \frac{1 \times 25 \times 60}{3 \times 2}$$

$$\boldsymbol{V_{st} = 250\ m^3}$$

$$t_{st} = \frac{V_{st} \times n_{sys}}{Q}$$

$$t_{st} = \frac{250 \times 2}{1}$$

$$\boldsymbol{t_{st} = 500\ s}$$

$$Gt_{st} = G_{st} \times t_{st}$$

Stage 1

$Gt_{st} = 70 \times 500$

$\boldsymbol{Gt_{st} = 35000}$

Stage 2

$Gt_{st} = 35 \times 500$

$\boldsymbol{Gt_{st} = 17000}$

Stage 3

$Gt_{st} = 20 \times 500$

$\boldsymbol{Gt_{st} = 10000}$

$$h_{st} = \frac{G_{st}^2 \times \mathrm{v} \times V_{st} \times n_{sys}}{g \times Q}$$

Stage 1

$$h_{st} = \frac{70^2 \times 1.3081 \times 10^{-6} \times 250 \times 2}{9.81 \times 1}$$

$\boldsymbol{h_{st} = 0.327}$

Stage 2

$$h_{st} = \frac{35^2 \times 1.3081 \times 10^{-6} \times 250 \times 2}{9.81 \times 1}$$

$\boldsymbol{h_{st} = 0.082}$

Stage 3

$$h_{st} = \frac{20^2 \times 1.3081 \times 10^{-6} \times 250 \times 2}{9.81 \times 1}$$

$\boldsymbol{h_{st} = 0.027}$

$$S_{baf} = \frac{l_{act} \times n_{ch}}{n_{st} \times n_{com}}$$

Stage 1

$$S_{baf} = \frac{28.4 \times 6}{3 \times 20}$$

$\boldsymbol{S_{baf} = 2.84\ m}$

Stage 2

$$S_{baf} = \frac{28.4 \times 6}{3 \times 16}$$

$\boldsymbol{S_{baf} = 3.55\ \mathrm{m}}$

Stage 3

$$S_{baf} = \frac{28.4 \times 6}{3 \times 12}$$

$\boldsymbol{S_{baf} = 4.733\ m}$

$$v = \sqrt{\frac{h_{st} \times 2 \times g}{n_{com} \times 1.5}}$$

Stage 1

$$v = \sqrt{\frac{0.327 \times 2 \times 9.81}{20 \times 1.5}}$$

$\boldsymbol{v = 0.462\ m/s}$

Stage 2

$$v = \sqrt{\frac{0.081 \times 2 \times 9.81}{16 \times 1.5}}$$

$\boldsymbol{v = 0.258\ m/s}$

Stage 3

$$v = \sqrt{\frac{0.027 \times 2 \times 9.81}{12 \times 1.5}}$$

$\boldsymbol{v = 0.17\ m/s}$

$$S_{slit} = \frac{Q}{n_{sys} \times v \times \left(h_{out} + h_{down} + \frac{h_{st}}{2}\right)}$$

Stage 1

$$S_{slit} = \frac{1}{2 \times 0.462 \times \left(2 + 0.081 + 0.027 + \frac{0.327}{2}\right)}$$

$$\boldsymbol{S_{slit} = 0.476\ m}$$

Stage 2

$$S_{slit} = \frac{1}{2 \times 0.258 \times \left(2 + 0.027 + \frac{0.081}{2}\right)}$$

$$\boldsymbol{S_{slit} = 0.936\ m}$$

Stage 3

$$S_{slit} = \frac{1}{2 \times 0.17 \times \left(2 + \frac{0.027}{2}\right)}$$

$$\boldsymbol{S_{slit} = 1.46\ m}$$

The above results compare well with results from the example found in literature (Kawamura, 2000:134-136). The one fundamental difference is the literature example assumes an average water depth across the system (2 m) when calculating the slit opening sizes. However, the software model adds the headloss from the downstream stages to the chosen/given final outlet water depth, to obtain the water depth at the outlet of the stage in question. This is then added to the average headloss across the applicable stage to find the average water depth. Further, the literature example uses a marginally different kinematic viscosity, which adds to the variance in results.

<u>Minimum and maximum calculations:</u>

$$t_{st,max} = \frac{V_{st} \times n_{sys}}{Q_{min}}$$

Stage 1

$$t_{st,max} = \frac{250 \times 2}{0.9}$$

$$\boldsymbol{t_{st,max} = 555.56\ s}$$

Stage 2

$$t_{st,max} = \frac{250 \times 2}{0.9}$$

$$\boldsymbol{t_{st,max} = 555.56\ s}$$

Stage 3

$$t_{st,max} = \frac{250 \times 2}{0.9}$$

$$\boldsymbol{t_{st,max} = 555.56\ s}$$

$$v_{min} = \frac{Q_{min}}{n_{act,sys} \times \left(h_{out} + h_{down} + \frac{h_{st}}{2}\right) \times S_{slit}}$$

Stage 1

$$v_{min} = \frac{0.9}{2 \times \left(2 + 0.027 + 0.082 + \frac{0.327}{2}\right) \times 0.476}$$

$$\boldsymbol{v_{min} = 0.416\ {}^{m}/_{s}}$$

Stage 2

$$v_{min} = \frac{0.9}{2 \times \left(2 + 0.027 + \frac{0.082}{2}\right) \times 0.936}$$

$$\boldsymbol{v_{min} = 0.233\ {}^{m}/_{s}}$$

Stage 3

$$v_{min} = \frac{0.9}{2 \times \left(2 + \frac{0.027}{2}\right) \times 1.457}$$

$$\boldsymbol{v_{min} = 0.153\ {}^{m}/_{s}}$$

$$h_{st,min} = \frac{n_{com} \times v_{min}^2 \times 1.5}{2 \times g}$$

Stage 1

$$h_{st,min} = \frac{20 \times 0.416^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,min} = 0.265\ m}$$

Stage 2

$$h_{st,min} = \frac{16 \times 0.233^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,min} = 0.066\ m}$$

Stage 3

$$h_{st,min} = \frac{12 \times 0.153^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,min} = 0.022\ m}$$

$$G_{st,min} = \sqrt{\frac{h_{st,min} \times g}{t_{st,max} \times \mathrm{v} \times 10^{-6}}}$$

Stage 1

$$G_{st,min} = \sqrt{\frac{0.265 \times 9.81}{555.56 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,min} = 59.77\ s^{-1}}$$

Stage 2

$$G_{st,min} = \sqrt{\frac{0.066 \times 9.81}{555.56 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,min} = 29.88\ s^{-1}}$$

Stage 3

$$G_{st,min} = \sqrt{\frac{0.022 \times 9.81}{555.56 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,min} = 17.076\ s^{-1}}$$

$$Gt_{st,min} = G_{st,min} \times t_{st,max}$$

Stage 1

$$Gt_{st,min} = 59.77 \times 555.56$$

$$\boldsymbol{Gt_{st,min} = 33203.9}$$

Stage 2

$$Gt_{st,min} = 29.88 \times 555.56$$

$$\boldsymbol{Gt_{st,min} = 16601.96}$$

Stage 3

$$Gt_{st,min} = 17.076 \times 555.56$$

$$\boldsymbol{Gt_{st,min} = 9486.83}$$

$$t_{st,min} = \frac{V_{st} \times n_{sys}}{Q_{max}}$$

Stage 1

$$t_{st,max} = \frac{250 \times 2}{1.1}$$

$$\boldsymbol{t_{st,max} = 454.55\ s}$$

Stage 2

$$t_{st,max} = \frac{250 \times 2}{1.1}$$

$$\boldsymbol{t_{st,max} = 454.55\ s}$$

Stage 3

$$t_{st,max} = \frac{250 \times 2}{1.1}$$

$$\boldsymbol{t_{st,max} = 454.55\ s}$$

$$v_{max} = \frac{Q_{max}}{n_{sys} \times \left(h_{out} + h_{down} + \frac{h_{st}}{2}\right) \times S_{slit}}$$

Stage 1

Stage 2

Stage 3

$$v_{max} = \frac{1.1}{2 \times \left(2 + 0.027 + 0.082 + \frac{0.327}{2}\right) \times 0.476}$$

$$\boldsymbol{v_{max} = 0.508\ {}^{m}/_{s}}$$

$$v_{max} = \frac{1.1}{2 \times \left(2 + 0.027 + \frac{0.082}{2}\right) \times 0.936}$$

$$\boldsymbol{v_{max} = 0.284\ {}^{m}/_{s}}$$

$$v_{max} = \frac{1.1}{2 \times \left(2 + \frac{0.027}{2}\right) \times 1.457}$$

$$\boldsymbol{v_{max} = 0.188\ {}^{m}/_{s}}$$

$$h_{st,max} = \frac{n_{com} \times v_{max}^2 \times 1.5}{2 \times g}$$

Stage 1

$$h_{st,max} = \frac{20 \times 0.508^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,max} = 0.395\ m}$$

Stage 2

$$h_{st,max} = \frac{16 \times 0.284^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,max} = 0.099\ m}$$

Stage 3

$$h_{st,max} = \frac{12 \times 0.188^2 \times 1.5}{2 \times 9.81}$$

$$\boldsymbol{h_{st,max} = 0.032\ m}$$

$$G_{st,max} = \sqrt{\frac{h_{st,max} \times g}{t_{min} \times \mathrm{v} \times 10^{-6}}}$$

Stage 1

$$G_{st,max} = \sqrt{\frac{0.395 \times 9.81}{454.55 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,max} = 80.76\ s^{-1}}$$

Stage 2

$$G_{st,max} = \sqrt{\frac{0.099 \times 9.81}{454.55 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,max} = 40.38\ s^{-1}}$$

Stage 3

$$G_{st,max} = \sqrt{\frac{0.032 \times 9.81}{454.55 \times 1.3081 \times 10^{-6}}}$$

$$\boldsymbol{G_{st,max} = 23.07\ s^{-1}}$$

$$Gt_{st,max} = G_{st,max} \times t_{st,min}$$

Stage 1

$$Gt_{st,max} = 80.76 \times 454.55$$

$$\boldsymbol{Gt_{st,max} = 36708.31}$$

Stage 2

$$Gt_{st,max} = 40.38 \times 454.55$$

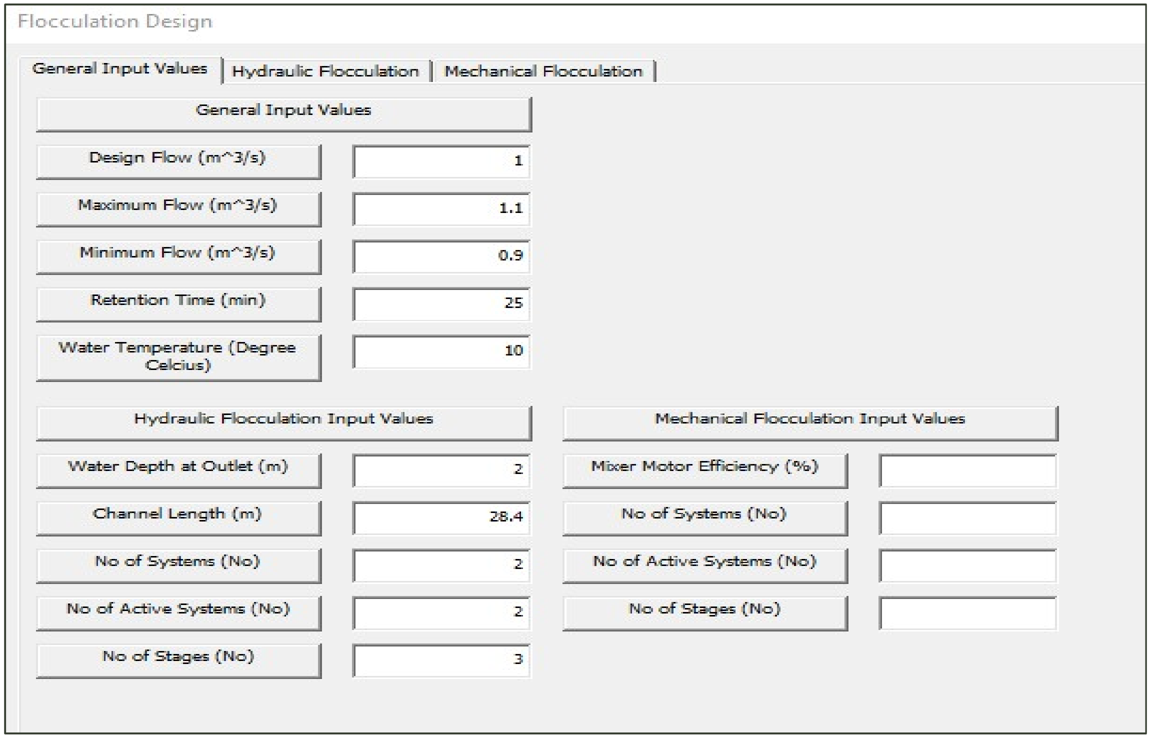
$$\boldsymbol{Gt_{st,max} = 18354.15}$$

Stage 3

$$Gt_{st,max} = 23.07 \times 454.55$$

$$\boldsymbol{Gt_{st,max} = 10488.09}$$

The above results are shown in Figure 15. Several parameters are exceeded, and this is due to the retention time chosen, which is less than the required minimum. Also, the velocity gradient for the 1st stage exceeds the specified maximum value.



General Input Values

Design Flow (m^3/s)	1
Maximum Flow (m^3/s)	1.1
Minimum Flow (m^3/s)	0.9
Retention Time (min)	25
Water Temperature (Degree Celcius)	10

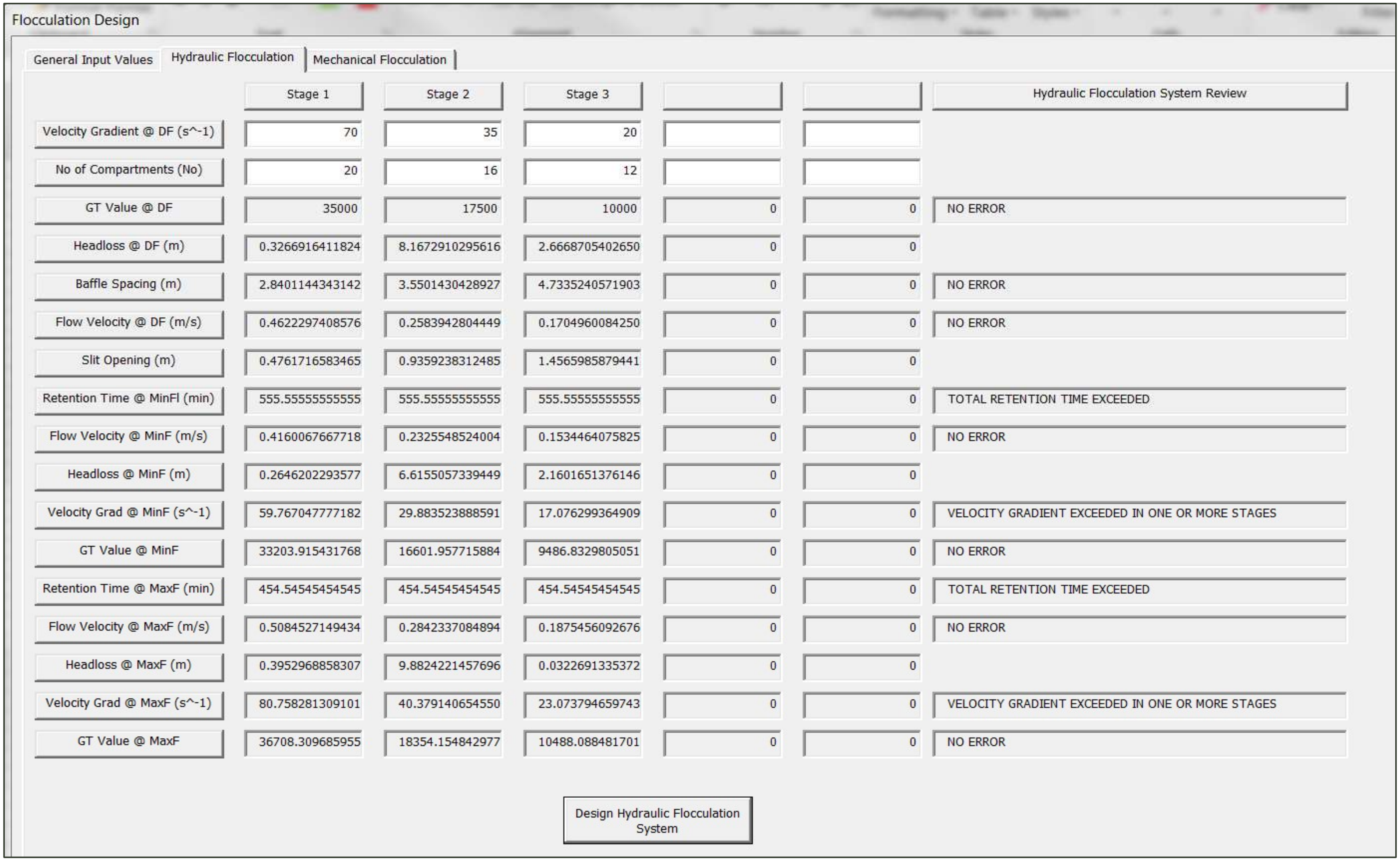
Hydraulic Flocculation Input Values

Water Depth at Outlet (m)	2
Channel Length (m)	28.4
No of Systems (No)	2
No of Active Systems (No)	2
No of Stages (No)	3

Mechanical Flocculation Input Values

Mixer Motor Efficiency (%)	
No of Systems (No)	
No of Active Systems (No)	
No of Stages (No)	

Figure 14: Hydraulic flocculation input values



	Stage 1	Stage 2	Stage 3			Hydraulic Flocculation System Review
Velocity Gradient @ DF (s^-1)	70	35	20			
No of Compartments (No)	20	16	12			
GT Value @ DF	35000	17500	10000	0	0	NO ERROR
Headloss @ DF (m)	0.3266916411824	8.1672910295616	2.6668705402650	0	0	
Baffle Spacing (m)	2.8401144343142	3.5501430428927	4.7335240571903	0	0	NO ERROR
Flow Velocity @ DF (m/s)	0.4622297408576	0.2583942804449	0.1704960084250	0	0	NO ERROR
Slit Opening (m)	0.4761716583465	0.9359238312485	1.4565985879441	0	0	
Retention Time @ MinFl (min)	555.55555555555	555.55555555555	555.55555555555	0	0	TOTAL RETENTION TIME EXCEEDED
Flow Velocity @ MinF (m/s)	0.4160067667718	0.2325548524004	0.1534464075825	0	0	NO ERROR
Headloss @ MinF (m)	0.2646202293577	6.6155057339449	2.1601651376146	0	0	
Velocity Grad @ MinF (s^-1)	59.767047777182	29.883523888591	17.076299364909	0	0	VELOCITY GRADIENT EXCEEDED IN ONE OR MORE STAGES
GT Value @ MinF	33203.915431768	16601.957715884	9486.8329805051	0	0	NO ERROR
Retention Time @ MaxF (min)	454.54545454545	454.54545454545	454.54545454545	0	0	TOTAL RETENTION TIME EXCEEDED
Flow Velocity @ MaxF (m/s)	0.5084527149434	0.2842337084894	0.1875456092676	0	0	NO ERROR
Headloss @ MaxF (m)	0.3952968858307	9.8824221457696	0.0322691335372	0	0	
Velocity Grad @ MaxF (s^-1)	80.758281309101	40.379140654550	23.073794659743	0	0	VELOCITY GRADIENT EXCEEDED IN ONE OR MORE STAGES
GT Value @ MaxF	36708.309685955	18354.154842977	10488.088481701	0	0	NO ERROR

Figure 15: Hydraulic flocculation calculations

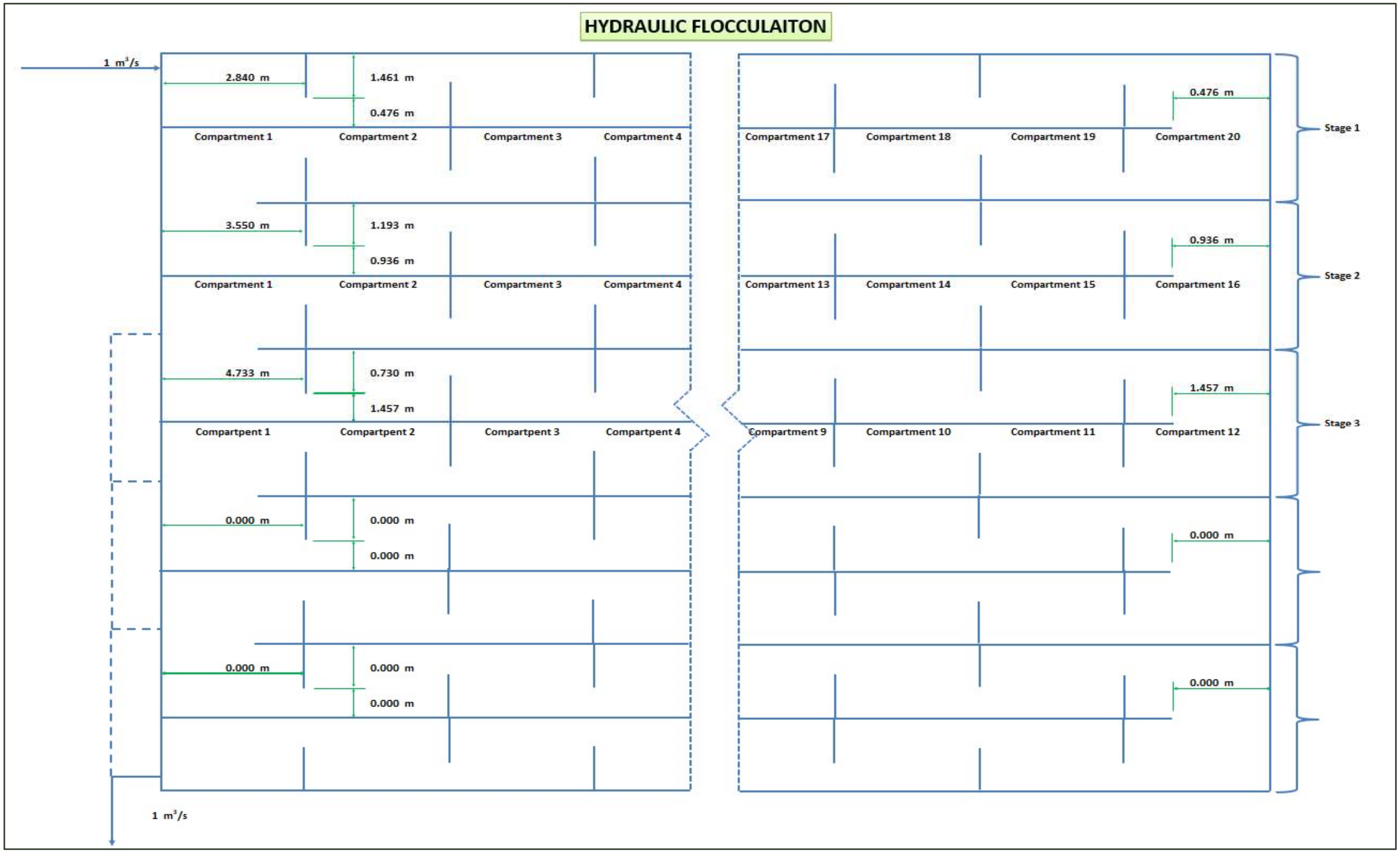


Figure 16: Hydraulic flocculation schematic

Flocculation example 2: mechanical system

Design a 3-stage mechanical flocculation system for a plant design flowrate of 1.32 m^3/s.

The following values are given:

Retention time	20 min
Water temperature:	10^{o}C
Absolute viscosity:	1.3076 x 10^{-3} N.s/m^2
No of systems:	2
Velocity gradient for stage 1	60 s^{-1}
Velocity gradient for stage 2	60 s^{-1}
Velocity gradient for stage 3	30 s^{-1}
Number of mixers for stage 1	2
Number of mixers for stage 2	2
Number of mixers for stage 3	2
Mixer motor efficiency	75%
No of active systems	2

Calculations:

$$V_{st} = \frac{Q \times t_{sys} \times 60}{n_{sys} \times n_{st}}$$

$$V_{st} = \frac{1.32 \times 20 \times 60}{2 \times 3}$$

$$\boldsymbol{V_{st} = 264\ m^3}$$

$$t_{st} = \frac{t_{sys} \times 60}{n_{st}}$$

$$t_{st} = \frac{20 \times 60}{3}$$

$$\boldsymbol{t_{st} = 400\ s}$$

$$Gt_{st} = G_{st} \times t_{st}$$

Stage 1

$$Gt_{st} = 60 \times 400$$

$$\boldsymbol{Gt_{st} = 24000}$$

Stage 2

$$Gt_{st} = 60 \times 400$$

$$\boldsymbol{Gt_{st} = 24000}$$

Stage 3

$$Gt_{st} = 30 \times 400$$

$$\boldsymbol{Gt_{st} = 12000}$$

$$P_{kW} = \frac{G_{st}^2 \times \mu \times V_{st}}{1000 \times \eta \times n_{mix}}$$

Stage 1

$$P_{kW} = \frac{60^2 \times 1.3076 \times 10^{-3} \times 264}{1000 \times 0.75 \times 2}$$

$$\boldsymbol{P_{kW} = 0.828\ kW}$$

Stage 2

$$P_{kW} = \frac{60^2 \times 1.3076 \times 10^{-3} \times 264}{1000 \times 0.75 \times 2}$$

$$\boldsymbol{P_{kW} = 0.828\ kW}$$

Stage 3

$$P_{kW} = \frac{30^2 \times 1.3076 \times 10^{-3} \times 264}{1000 \times 0.75 \times 2}$$

$$\boldsymbol{P_{kW} = 0.207\ kW}$$

The above results compare well with the example from Kawamura (2000:127-129). Marginal differences are attributed to the fact that Kawamura fixes the volume per stage, based on a chosen length and width for each chamber. Also, Kawamura uses a different absolute viscosity.

Minimum and maximum calculations:

$$P_{kW,min} = P_{kW} \times r_{min}$$

Stage 1

$$P_{kW,min} = 0.828 \times 0.6$$

Stage 2

$$P_{kW,min} = 0.828 \times 0.6$$

Stage 3

$$P_{kW,min} = 0.207 \times 0.6$$

$\boldsymbol{P_{kW,min} = 0.497\ kW}$ $\boldsymbol{P_{kW,min} = 0.497\ kW}$ $\boldsymbol{P_{kW,min} = 0.124\ kW}$

$$G_{st,min} = \sqrt{\frac{P_{kW,min} \times 1000 \times n_{mix}}{\mu \times V_{st}}}$$

Stage 1 *Stage 2* *Stage 3*

$$G_{st,min} = \sqrt{\frac{0.497 \times 1000 \times 2}{0.0013076 \times 264}} \qquad G_{st,min} = \sqrt{\frac{0.497 \times 1000 \times 2}{0.0013076 \times 264}} \qquad G_{st,min} = \sqrt{\frac{0.124 \times 1000 \times 2}{0.0013076 \times 264}}$$

$\boldsymbol{G_{st,min} = 53.67\ s^{-1}}$ $\boldsymbol{G_{st,min} = 53.67\ s^{-1}}$ $\boldsymbol{G_{st,min} = 26.83\ s^{-1}}$

$$t_{st,max} = \frac{V_{st} \times n_{act,sys}}{Q_{min}}$$

Stage 1 *Stage 2* *Stage 3*

$$t_{st,max} = \frac{264 \times 2}{1.2} \qquad t_{st,max} = \frac{264 \times 2}{1.2} \qquad t_{st,max} = \frac{264 \times 2}{1.2}$$

$\boldsymbol{t_{st,max} = 440\ s}$ $\boldsymbol{t_{st,max} = 440\ s}$ $\boldsymbol{t_{st,max} = 440\ s}$

$$Gt_{st,min} = G_{st,min} \times t_{st,max}$$

Stage 1 *Stage 2* *Stage 3*

$Gt_{st,min} = 53.67 \times 440$ $Gt_{st,min} = 53.67 \times 440$ $Gt_{st,min} = 26.83 \times 440$

$\boldsymbol{Gt_{st,min} = 23612.88}$ $\boldsymbol{Gt_{st,min} = 23612.88}$ $\boldsymbol{Gt_{st,min} = 11806.44}$

$$P_{kW,max} = P_{kW} \times r_{max}$$

Stage 1 *Stage 2* *Stage 3*

$P_{kW,max} = 0.828 \times 1$ $P_{kW,max} = 0.828 \times 1$ $P_{kW,max} = 0.207 \times 1$

$\boldsymbol{P_{kW,max} = 0.828\ kW}$ $\boldsymbol{P_{kW,max} = 0.828\ kW}$ $\boldsymbol{P_{kW,max} = 0.207\ kW}$

$$G_{st,max} = \sqrt{\frac{P_{kW,max} \times 1000 \times n_{mix}}{\mu \times V_{st}}}$$

Stage 1 *Stage 2* *Stage 3*

$$G_{st,max} = \sqrt{\frac{0.828 \times 1000 \times 2}{0.0013076 \times 264}} \qquad G_{st,max} = \sqrt{\frac{0.828 \times 1000 \times 2}{0.0013076 \times 264}} \qquad G_{st,max} = \sqrt{\frac{0.207 \times 1000 \times 2}{0.0013076 \times 264}}$$

$\boldsymbol{G_{st,max} = 69.282\ s^{-1}}$ $\boldsymbol{G_{st,max} = 69.282\ s^{-1}}$ $\boldsymbol{G_{st,max} = 34.641\ s^{-1}}$

$$t_{st,min} = \frac{V_{st} \times n_{sys}}{Q_{max}}$$

Stage 1 *Stage 2* *Stage 3*

$$t_{st,min} = \frac{264 \times 2}{1.4} \qquad t_{st,min} = \frac{264 \times 2}{1.4} \qquad t_{st,min} = \frac{264 \times 2}{1.4}$$

$\boldsymbol{t_{st,min} = 377.14\ s}$ $\boldsymbol{t_{st,min} = 377.14\ s}$ $\boldsymbol{t_{st,min} = 377.14\ s}$

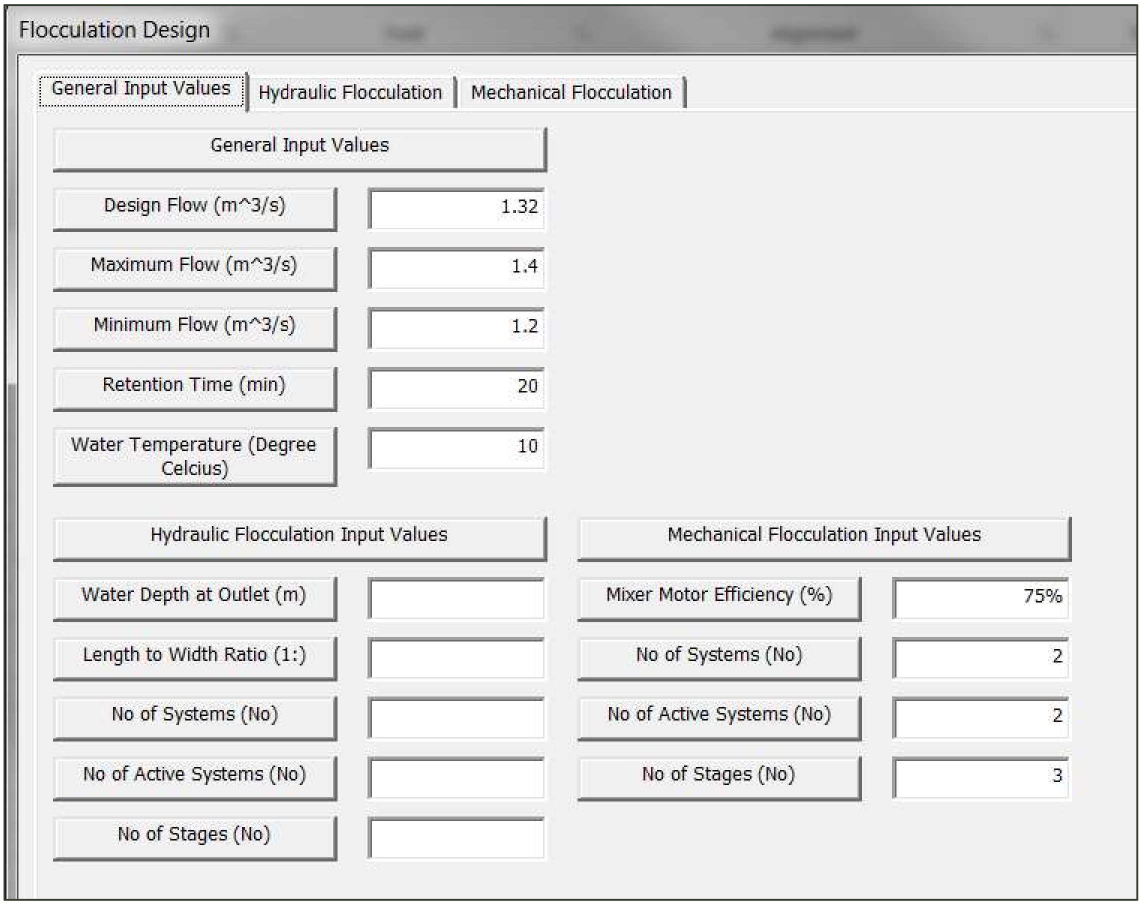
$$Gt_{st,max} = G_{st,max} \times t_{st,min}$$

Stage 1 *Stage 2* *Stage 3*

$Gt_{st,max} = 69.282 \times 377.14$ $Gt_{st,max} = 69.282 \times 377.14$ $Gt_{st,max} = 34.641 \times 377.14$

$\boldsymbol{Gt_{st,max} = 26129.22}$ $\boldsymbol{Gt_{st,max} = 26129.22}$ $\boldsymbol{Gt_{st,max} = 13064.61}$

The results from the software model is shown in Figure 18 below.



General Input Values

Design Flow (m^3/s)	1.32
Maximum Flow (m^3/s)	1.4
Minimum Flow (m^3/s)	1.2
Retention Time (min)	20
Water Temperature (Degree Celcius)	10

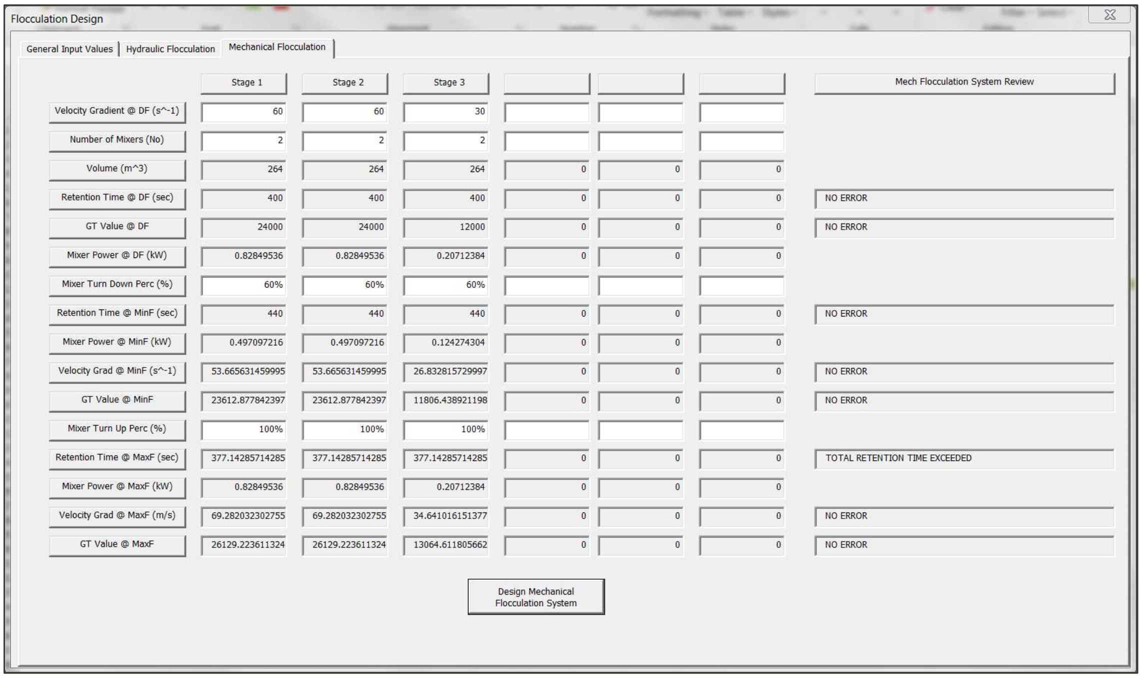
Hydraulic Flocculation Input Values

Water Depth at Outlet (m)	
Length to Width Ratio (1:)	
No of Systems (No)	
No of Active Systems (No)	
No of Stages (No)	

Mechanical Flocculation Input Values

Mixer Motor Efficiency (%)	75%
No of Systems (No)	2
No of Active Systems (No)	2
No of Stages (No)	3

Figure 17: Mechanical flocculation input values



	Stage 1	Stage 2	Stage 3				Mech Flocculation System Review
Velocity Gradient @ DF (s^-1)	60	60	30				
Number of Mixers (No)	2	2	2				
Volume (m^3)	264	264	264	0	0	0	
Retention Time @ DF (sec)	400	400	400	0	0	0	NO ERROR
GT Value @ DF	24000	24000	12000	0	0	0	NO ERROR
Mixer Power @ DF (kW)	0.82849536	0.82849536	0.20712384	0	0	0	
Mixer Turn Down Perc (%)	60%	60%	60%				
Retention Time @ MinF (sec)	440	440	440	0	0	0	NO ERROR
Mixer Power @ MinF (kW)	0.497097216	0.497097216	0.124274304	0	0	0	
Velocity Grad @ MinF (s^-1)	53.665631459995	53.665631459995	26.832815729997	0	0	0	NO ERROR
GT Value @ MinF	23612.877842397	23612.877842397	11806.438921198	0	0	0	NO ERROR
Mixer Turn Up Perc (%)	100%	100%	100%				
Retention Time @ MaxF (sec)	377.14285714285	377.14285714285	377.14285714285	0	0	0	TOTAL RETENTION TIME EXCEEDED
Mixer Power @ MaxF (kW)	0.82849536	0.82849536	0.20712384	0	0	0	
Velocity Grad @ MaxF (m/s)	69.282032302755	69.282032302755	34.641016151377	0	0	0	NO ERROR
GT Value @ MaxF	26129.223611324	26129.223611324	13064.611805662	0	0	0	NO ERROR

Figure 18: Mechanical flocculation calculations

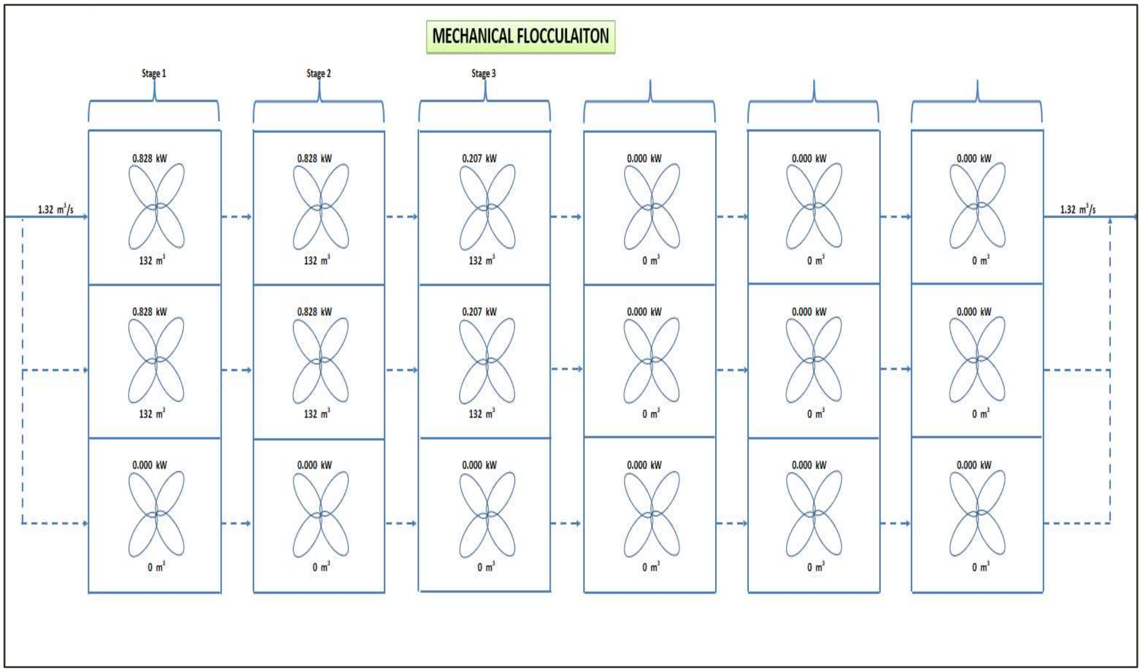


Figure 19: Mechanical flocculation schematic

DAF example

Design a DAF system for a plant design flowrate of 0.438 m^3/s.

<u>The following values are given</u>:

Water temperature:	20^{o}C
Absolute viscosity:	1.0005 x 10^{-3} N.s/m^2
Bubble diameter	100 µm
Floc diameter	50 µm
Separation zone hydraulic loading rate	18.3 m/h
Separation zone length to width ratio	1:1.3
Recycle ratio	10%
Contact zone detention time	1.5 min
Tank depth	2.5
Number of tanks	1

<u>Calculations:</u>

$$N_{max} = \left(\frac{d_f}{d_b}\right)^2$$

$$N_{max} = \left(\frac{50}{100}\right)^2$$

$$\boldsymbol{N_{max} = 0.25}$$

$$N_{ab} = \frac{N_{max}}{2}$$

$$N_{ab} = \frac{0.25}{2}$$

$$\boldsymbol{N_{ab} = 0.125, since\ must\ be\ N_{ab} \geq 1, set\ N_{ab} = 1}$$

$$d_{pb} = \left[d_f^3 + N_{ab} \times (d_b)^3\right]^{\frac{1}{3}}$$

$$d_{pb} = [50^3 + 1 \times (100)^3]^{\frac{1}{3}}$$

$$\boldsymbol{d_{pb} = 104\ \mu m}$$

$$\rho_{pb} = \frac{\rho_p \times d_p^3 + N_{ab} \times (\rho_b \times d_b^3)}{d_p^3 + N_{ab} \times d_b^3}$$

$$\rho_{pb} = \frac{1100 \times 50^3 + 1 \times (1.19 \times 100^3)}{50^3 + 1 \times 100^3}$$

$$\boldsymbol{\rho_{pb} = 123.3\ {}^{kg}/_{m^3}}$$

$$v_{fb} = \frac{g \times (\rho_w - \rho_{pb}) \times (d_{pb} \times 10^{-6})^2 \times 3600}{18 \times \mu}$$

$$v_{fb} = \frac{9.81 \times (998.19 - 123.3) \times (104 \times 10^{-6})^2 \times 3600}{18 \times 1.0005 \times 10^{-3}}$$

$$\boldsymbol{v_{fb} = 18.56\ {}^{m}/_{h}}$$

$$Q_r = Q \times R_r$$

$$Q_r = 0.438 \times 0.1$$

$$\boldsymbol{Q_r = 0.0438\ {}^{m^3}/_{s}}$$

$$Q_{tot} = \frac{Q + Q_r}{n}$$

$$Q_{tot} = \frac{0.438 + 0.0438}{1}$$

$$\boldsymbol{Q_{tot} = 0.482\ {}^{m^3}/_{s}}$$

$$A_{sz} = \frac{Q_{tot} \times 3600}{v_{sz-hl}}$$

$$A_{sz} = \frac{0.482 \times 3600}{18.3}$$

$$\boldsymbol{A_{sz} = 94.8\ m^2}$$

$$L_{sz} = \sqrt{A_{sz} \times R_{l/w}}$$

$$L_{sz} = \sqrt{98.4 \times 1.3}$$

$$\boldsymbol{L_{sz} = 11.1\ m}$$

$$W_{sz} = \frac{L_{sz}}{R_{l/w}}$$

$$W_{sz} = \frac{11.1}{1.3}$$

$$\boldsymbol{W_{sz} = 8.5\ m}$$

$$V_{cz} = Q_{tot} \times 60 \times t_{cz}$$

$$V_{cz} = 0.482 \times 60 \times 1.5$$

$$\boldsymbol{V_{cz} = 43.37\ m^3}$$

$$L_{cz} = \frac{V_{cz}}{W_{sz} \times d}$$

$$L_{cz} = \frac{43.37}{8.5 \times 2.5}$$

$$\boldsymbol{L_{cz} = 2.03\ m}$$

$$A_{cz} = L_{cz} \times W_{cz}$$

$$A_{cz} = 2.03 \times 8.5$$

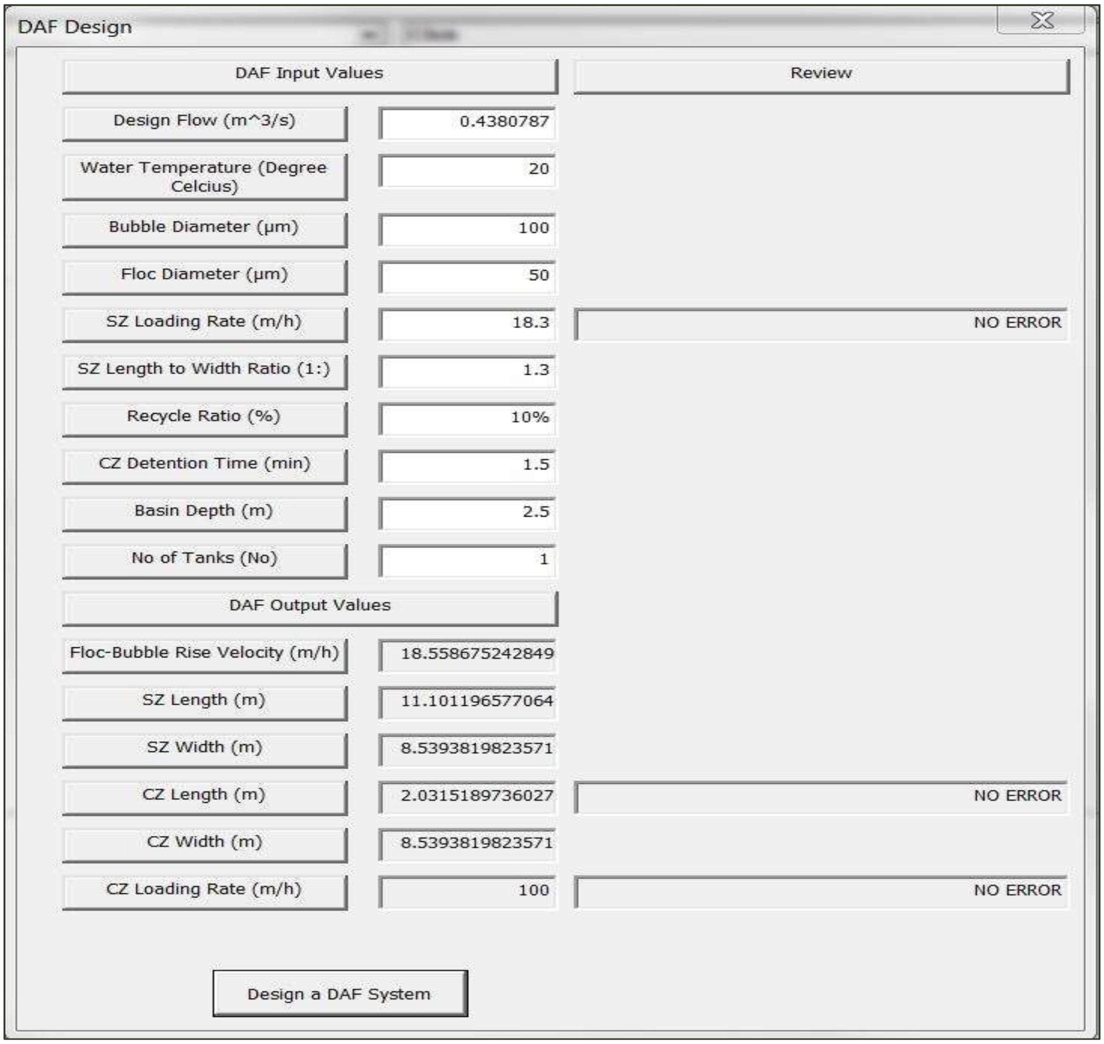
$$\boldsymbol{A_{cz} = 17.35\ m^2}$$

$$v_{cz-hl} = \frac{Q_{tot} \times 3600}{A_{cz}}$$

$$v_{cz-hl} = \frac{0.482 \times 3600}{17.35}$$

$$\boldsymbol{v_{cz-hl} = 100\ {}^{m}/_{h}}$$

The literature example (Gregory and Edzwald, 2011) matches well with the above results. Marginal differences are because of the different absolute viscosity value used. Unfortunately, the literature example does not include the sizing of the contact zone, however, since the above calculation, and as shown in Figure 20, meets all design parameters it is assumed to be correct.



Design Flow (m^3/s)	0.4380787	
Water Temperature (Degree Celcius)	20	
Bubble Diameter (µm)	100	
Floc Diameter (µm)	50	
SZ Loading Rate (m/h)	18.3	NO ERROR
SZ Length to Width Ratio (1:)	1.3	
Recycle Ratio (%)	10%	
CZ Detention Time (min)	1.5	
Basin Depth (m)	2.5	
No of Tanks (No)	1	
Floc-Bubble Rise Velocity (m/h)	18.558675242849	
SZ Length (m)	11.101196577064	
SZ Width (m)	8.5393819823571	
CZ Length (m)	2.0315189736027	NO ERROR
CZ Width (m)	8.5393819823571	
CZ Loading Rate (m/h)	100	NO ERROR

Figure 20: DAF calculations

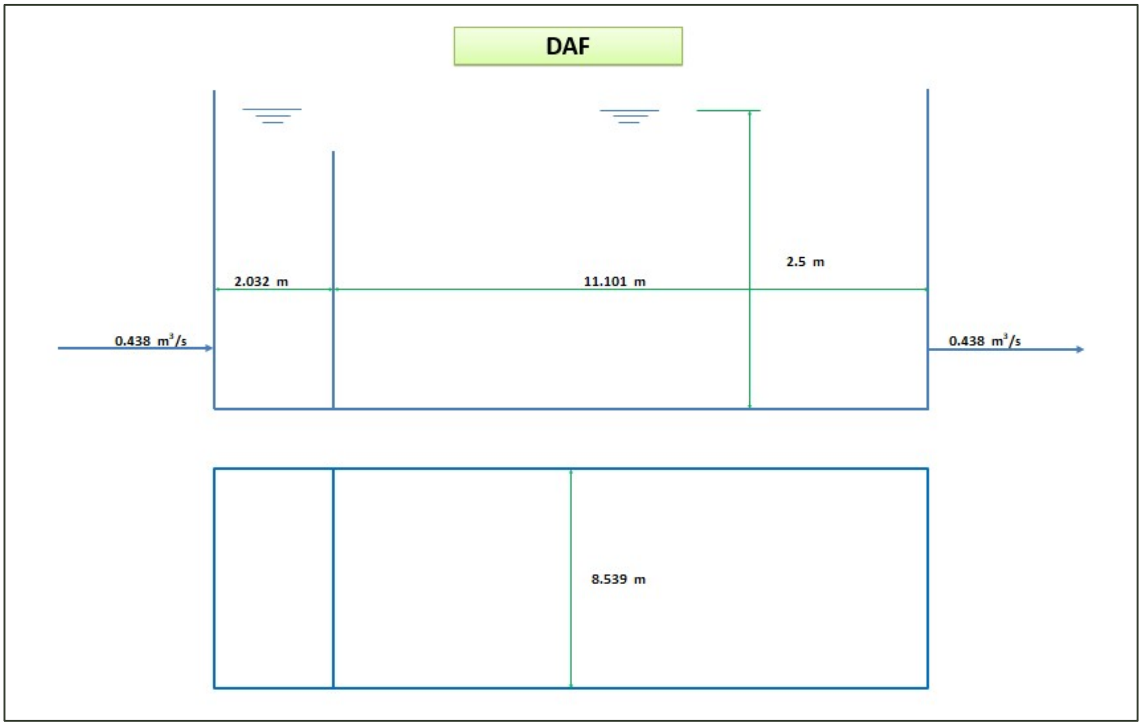


Figure 21: DAF Schematic Sedimentation example 1: grit chamber system

Design a grit chamber system for a plant maximum flowrate of 4.389 m^3/s.

<u>The following values are given</u>:

Tank width:	12.192 m
Water depth:	3.6576 m
Number of tanks:	2
Safety factor:	1
Particle size:	0.1 mm
Particle settling velocity:	0.008055 m/s
Tank length:	33.5 m
Number of longitudinal baffle walls:	0
Water temperature:	10^{o}C
Kinematic viscosity:	1.3081 x 10^{-6} m^2/s

<u>Calculations:</u>

$$v_{hor} = \frac{Q_{max}}{W \times H \times n}$$

$$v_{ho} = \frac{4.389}{12.192 \times 3.6576 \times 2}$$

$$\boldsymbol{v_{hor} = 0.0492\ {}^{m}/_{s}}$$

$$L_s = \frac{K \times SF \times H \times v_{ho}}{v_o}$$

$$L_s = \frac{1.5 \times 1 \times 3.6576 \times 0.0492}{0.008055}$$

$$\boldsymbol{L_s = 33.517\ m}$$

$$t_{ret} = \frac{H \times W \times L \times n}{Q_{max} \times 60}$$

$$t_{ret} = \frac{3.6576 \times 12.192 \times 33.5 \times 2}{4.389 \times 60}$$

$$\boldsymbol{t_{ret} = 11.346\ min}$$

$$v_s = \frac{Q_{max} \times 3600}{W \times L \times n}$$

$$v_s = \frac{4.389 \times 3600}{12.192 \times 33.5 \times 2}$$

$$\boldsymbol{v_s = 19.343\ {}^{m}/_{h}}$$

$$P = (H \times (n_{bw} + 1) \times 2) + W$$

$$P = (3.6576 \times (0 + 1) \times 2) + 12.192$$

$$\boldsymbol{P = 19.507\ m}$$

$$R = \frac{a}{P}$$

$$R = \frac{12.192 \times 3.6576}{19.507}$$

$$\boldsymbol{R = 2.286\ m}$$

$$Re = \frac{v_{ho}\ \ \times R}{\nu}$$

$$Re = \frac{0.0492 \times 2.286}{1.3081 \times 10^{-6}}$$

$$\boldsymbol{Re = 86000}$$

$$Fr = \frac{v_{hor}^2}{g \times R}$$

$$Fr = \frac{0.0492^2}{9.81 \times 2.286}$$

$$\boldsymbol{Fr = 0.000108}$$

The above results compare well with results from the example found in literature (Kawamura, 2000:173-174). Marginal differences are attributed to rounding of answers and the use of a different kinematic viscosity number.

$$s_{c-c} = \ s_p + d_p$$

$$s_{c-c} = \ 0.1 + 0.15$$

$$\boldsymbol{s_{c-c} = \ 0.25\ m}$$

$$n_{p,r} = \frac{\frac{W}{n_{bw}+1} - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,r} = \frac{\frac{12.192}{0+1} - 0.1 \times 2 - 0.15}{0.25}$$

$$\boldsymbol{n_{p,r} \approx 47}$$

$$n_{p,c} = \frac{H - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,c} = \frac{3.6576 - 0.1 \times 2 - 0.15}{0.25}$$

$$\boldsymbol{n_{p,c} \approx 13}$$

$$v_p = \frac{Q_{max}}{n \times (n_{bw} + 1) \times n_p \times \pi \times \left(\frac{d_p}{2}\right)^2}$$

$$v_p = \frac{4.389}{2 \times (0 + 1) \times 13 \times 47 \times \pi \times \left(\frac{0.15}{2}\right)^2}$$

$$v_p = 0.203 \ {}^{m}/{}_{s}$$

Minimum and maximum calculations:

$$v_{hor,des} = \frac{Q}{W \times H \times n}$$

$$v_{hor,des} = \frac{4}{12.192 \times 3.6576 \times 2}$$

$$\boldsymbol{v_{hor,des} = 0.0448 \ {}^{m}/{}_{s}}$$

$$Re_{des} = \frac{v_{hor,des} \times R}{\nu}$$

$$Re_{des} = \frac{0.0448 \times 2.286}{1.3081 \times 10^{-6}}$$

$$\boldsymbol{Re_{des} = 78378}$$

$$Fr_{des} = \frac{v_{hor,des}^2}{g \times R}$$

$$Fr_{des} = \frac{0.0448^2}{9.81 \times 2.286}$$

$$\boldsymbol{Fr_{des} = 8.97 \times 10^{-5}}$$

$$v_{hor,min} = \frac{Q_{min}}{W \times H \times n}$$

$$v_{hor,min} = \frac{3.5}{12.192 \times 3.6576 \times 2}$$

$$\boldsymbol{v_{hor,min} = 0.0392 \ {}^{m}/{}_{s}}$$

$$Re_{min} = \frac{v_{hor,min} \times R}{\nu}$$

$$Re_{min} = \frac{0.0392 \times 2.286}{1.3801 \times 10^{-6}}$$

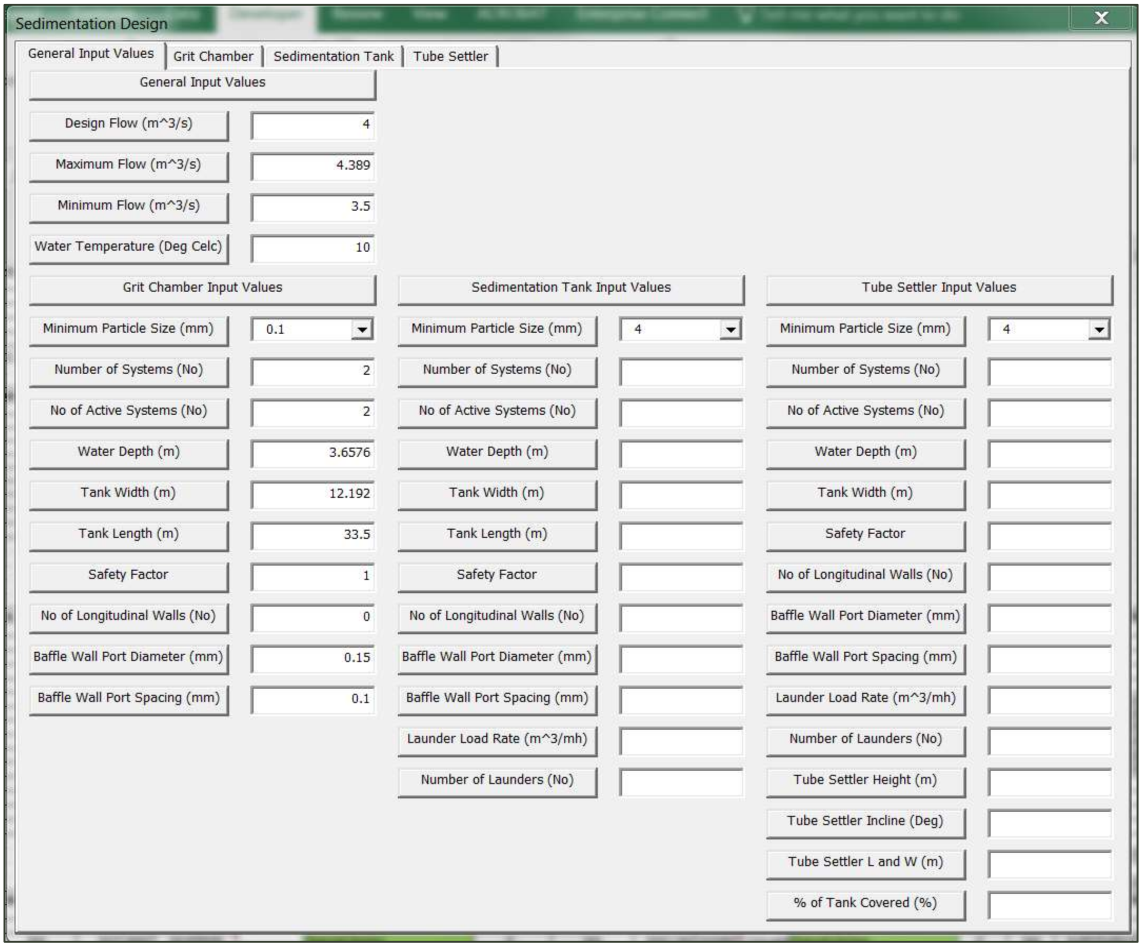
$$\boldsymbol{Re_{min} = 68581}$$

$$Fr_{min} = \frac{v_{hor,min}^2}{g \times R}$$

$$Fr_{min} = \frac{0.0392^2}{9.81 \times 2.286}$$

$$\boldsymbol{Fr_{min} = 6.87 \times 10^{-5}}$$

The above results, with slight differences due to rounding, are shown in Figure 23, where the design does not comply with the required length to width ratio. Further, the chosen length of tank should be longer than the sludge zone length, which is not the case in this example. The mean flow velocity is also a fraction lower than the required minimum.



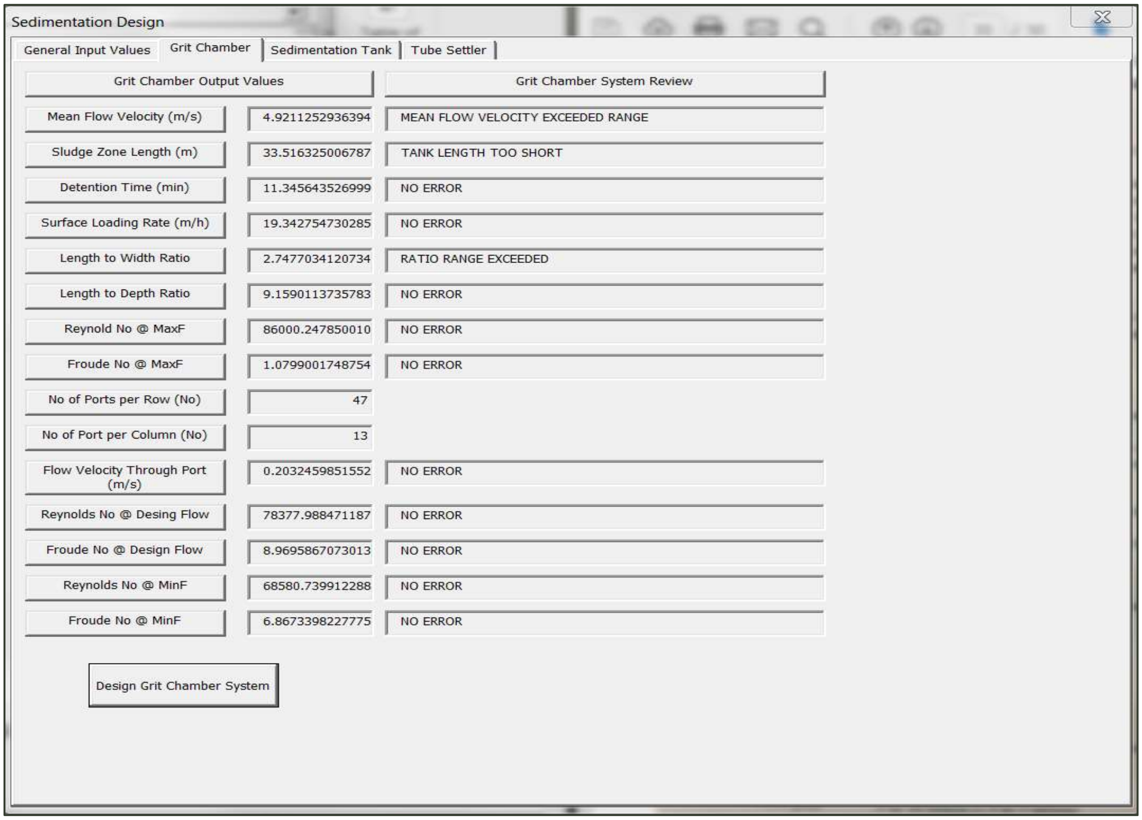
General Input Values	
Design Flow (m^3/s)	4
Maximum Flow (m^3/s)	4.389
Minimum Flow (m^3/s)	3.5
Water Temperature (Deg Celc)	10

Grit Chamber Input Values	
Minimum Particle Size (mm)	0.1
Number of Systems (No)	2
No of Active Systems (No)	2
Water Depth (m)	3.6576
Tank Width (m)	12.192
Tank Length (m)	33.5
Safety Factor	1
No of Longitudinal Walls (No)	0
Baffle Wall Port Diameter (mm)	0.15
Baffle Wall Port Spacing (mm)	0.1

Sedimentation Tank Input Values	
Minimum Particle Size (mm)	4
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Tank Length (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	
Launder Load Rate (m^3/mh)	
Number of Launders (No)	

Tube Settler Input Values	
Minimum Particle Size (mm)	4
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	
Launder Load Rate (m^3/mh)	
Number of Launders (No)	
Tube Settler Height (m)	
Tube Settler Incline (Deg)	
Tube Settler L and W (m)	
% of Tank Covered (%)	

Figure 22: Grit chamber input values



Grit Chamber Output Values		Grit Chamber System Review
Mean Flow Velocity (m/s)	4.9211252936394	MEAN FLOW VELOCITY EXCEEDED RANGE
Sludge Zone Length (m)	33.516325006787	TANK LENGTH TOO SHORT
Detention Time (min)	11.345643526999	NO ERROR
Surface Loading Rate (m/h)	19.342754730285	NO ERROR
Length to Width Ratio	2.7477034120734	RATIO RANGE EXCEEDED
Length to Depth Ratio	9.1590113735783	NO ERROR
Reynold No @ MaxF	86000.247850010	NO ERROR
Froude No @ MaxF	1.0799001748754	NO ERROR
No of Ports per Row (No)	47	
No of Port per Column (No)	13	
Flow Velocity Through Port (m/s)	0.2032459851552	NO ERROR
Reynolds No @ Desing Flow	78377.988471187	NO ERROR
Froude No @ Design Flow	8.9695867073013	NO ERROR
Reynolds No @ MinF	68580.739912288	NO ERROR
Froude No @ MinF	6.8673398227775	NO ERROR

Figure 23: Grit chamber calculations

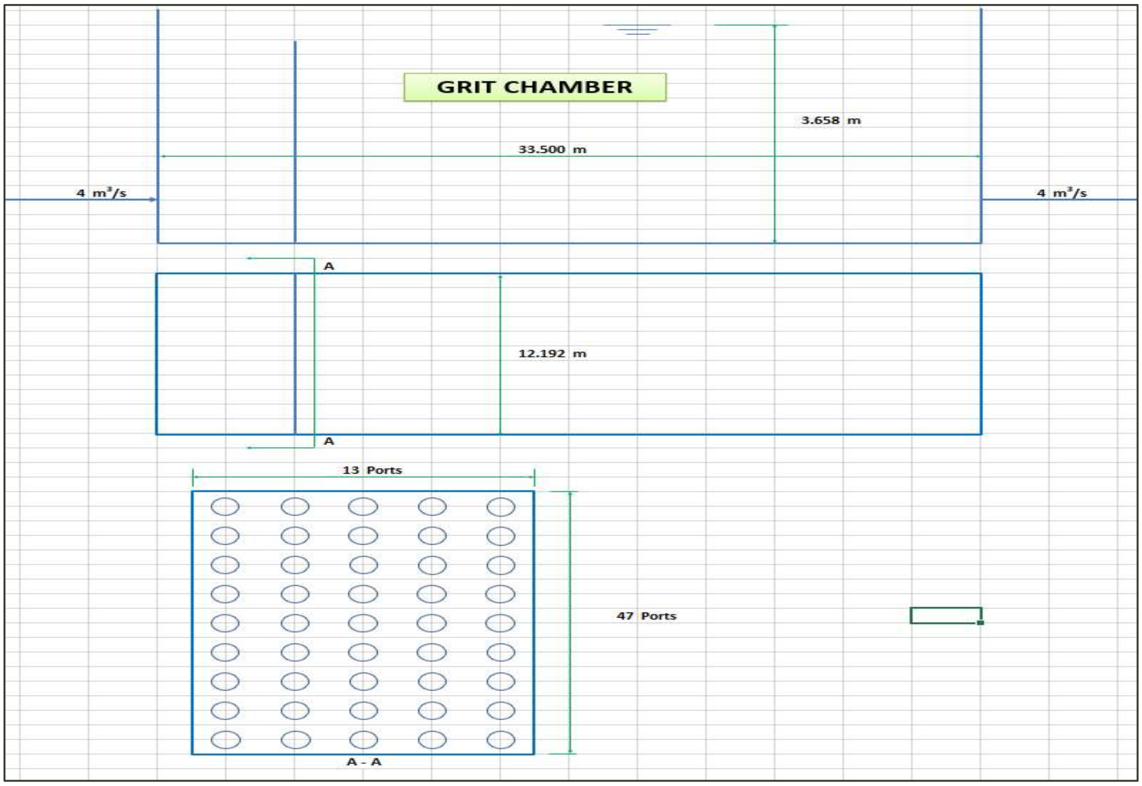


Figure 24: Grit chamber schematic

Sedimentation example 2: horizontal flow clarifier

Design a HFC system for a plant maximum flowrate of 2 m^3/s.

The following values are given:
Tank width: 18 m
Water depth: 4 m
Number of tanks: 2
Safety factor: 1
Particle size: 2 mm
Particle settling velocity: 0.0005278 m/s (*modified from Kawamura (2000:143) for 2 mm alum floc for this example*)
Tank length: 105 m
Number of longitudinal baffle walls: 2
Water temperature: 10^{o}C
Kinematic viscosity: 1.3081 x 10^{-6} m^2/s
Spacing between diffuser wall ports: 0.15 m
Port diameter: 0.12 m
Launder loading rate: 10.8 m^3/mh
Number of launders: 3

Calculations:

$$v_{hor} = \frac{Q_{max}}{W \times H \times n}$$

$$v_{ho} = \frac{2}{18 \times 4 \times 2}$$

$$\boldsymbol{v_{hor} = 0.01389\ {}^{m}/_{s}}$$

$$L_s = \frac{K \times SF \times H \times v_{hor}}{v_o}$$

$$L_s = \frac{1.5 \times 1 \times 4 \times 0.01389}{0.0005278}$$

$$\boldsymbol{L_s = 157.89\ m}$$

$$t_{ret} = \frac{H \times W \times L \times n}{Q_{max} \times 60}$$

$$t_{ret} = \frac{4 \times 18 \times 105 \times 2}{2 \times 60}$$

$$\boldsymbol{t_{ret} = 126\ min}$$

$$v_s = \frac{Q_{max} \times 3600}{W \times L \times n}$$

$$v_s = \frac{2 \times 3600}{18 \times 105 \times 2}$$

$$\boldsymbol{v_s = 1.905\ {}^{m}/_{h}}$$

$$P = (H \times (n_{bw} + 1) \times 2) + W$$

$$P = (4 \times (2 + 1) \times 2) + 18$$

$$\boldsymbol{P = 42\ m}$$

$$R = \frac{a}{P}$$

$$R = \frac{4 \times 18}{42}$$

$$\boldsymbol{R = 1.71\, m}$$

$$Re = \frac{v_{hor} \times R}{\nu}$$

$$Re = \frac{0.0139 \times 1.71}{1.3081 \times 10^{-6}}$$

$$\boldsymbol{Re = 18202}$$

$$Fr = \frac{v_{hor}^2}{g \times R}$$

$$Fr = \frac{0.01389^2}{9.81 \times 1.71}$$

$$\boldsymbol{Fr = 1.15 \times 10^{-6}}$$

$$r_{l/w} = \frac{L \times (n_{bw} + 1)}{W}$$

$$r_{l/w} = \frac{105 \times (2 + 1)}{18}$$

$$\boldsymbol{r_{l/w} = 17.5}$$

$$r_{l/h} = \frac{L}{H}$$

$$r_{l/h} = \frac{105}{4}$$

$$\boldsymbol{r_{l/h} = 26.25}$$

$$s_{c-c} = s_p + d_p$$

$$s_{c-c} = 0.15 + 0.12$$

$$\boldsymbol{s_{c-c} = 0.27}$$

$$n_{p,r} = \frac{\frac{W}{n_{bw}+1} - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,r} = \frac{\frac{18}{2+1} - 0.15 \times 2 - 0.12}{0.27}$$

$$\boldsymbol{n_{p,r} \approx 20}$$

$$n_{p,c} = \frac{H - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,c} = \frac{4 - 0.15 \times 2 - 0.12}{0.27}$$

$$\boldsymbol{n_{p,c} \approx 13}$$

$$l_{launder} = \frac{Q_{max} \times 3600}{n \times v_{launder} \times 2 \times n_{launder}}$$

$$l_{launder} = \frac{2 \times 3600}{2 \times 10.8 \times 2 \times 3}$$

$$\boldsymbol{l_{launder} = 55.56\, m}$$

The example found in literature (Kawamura, 2000:175-180) is based on a specific particle settling rate – 1.9 m/h. As such, and in the interest of comparison the software model was modified for this example to assume that a 2mm floc particle will have the same settling rate, and not as indicated in literature (Kawamura, 2000:143). The difference between the results above and those from literature are attributed to rounding and the difference between the kinematic viscosities used.

Minimum and maximum calculations:

$$v_{hor,des} = \frac{Q}{W \times H \times n}$$

$$v_{hor,des} = \frac{1.95}{18 \times 4 \times 2}$$

$$\boldsymbol{v_{hor,des} = 0.0135\ {}^{m}/_{s}}$$

$$Re_{des} = \frac{v_{hor,des} \times R}{\nu}$$

$$Re_{des} = \frac{0.0135 \times 1.71}{1.3081 \times 10^{-6}}$$

$$\boldsymbol{Re_{des} = 17747}$$

$$Fr_{des} = \frac{v_{hor,des}^2}{g \times R}$$

$$Fr_{des} = \frac{0.0135^2}{9.81 \times 1.71}$$

$$\boldsymbol{Fr_{des} = 1.09 \times 10^{-5}}$$

$$v_{hor,min} = \frac{Q_{min}}{W \times H \times n}$$

$$v_{hor,min} = \frac{1.9}{18 \times 4 \times 2}$$

$$\boldsymbol{v_{hor,min} = 0.0132\ {}^{m}/_{s}}$$

$$Re_{min} = \frac{v_{hor,min} \times R}{\nu}$$

$$Re_{min} = \frac{0.0132 \times 1.71}{1.3801 \times 10^{-6}}$$

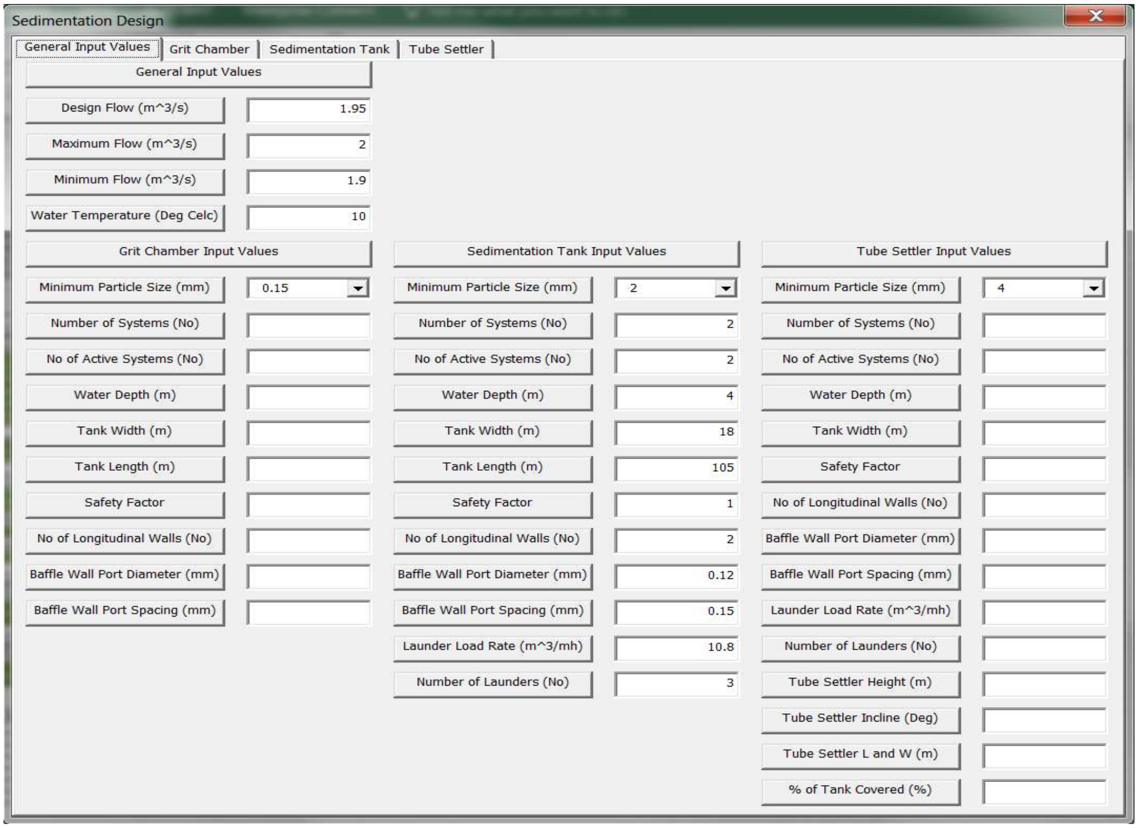
$$\boldsymbol{Re_{min} = 17292}$$

$$Fr_{min} = \frac{v_{hor,min}^2}{g \times R}$$

$$Fr_{min} = \frac{0.0132^2}{9.81 \times 1.71}$$

$$\boldsymbol{Fr_{min} = 1.035 \times 10^{-5}}$$

The above results are reflected in Figure 26. The design does not comply with some design parameters, i.e. the tank length and surface loading rate (only marginally), at the maximum plant flowrate.



Sedimentation Design

General Input Values

Design Flow (m^3/s)	1.95
Maximum Flow (m^3/s)	2
Minimum Flow (m^3/s)	1.9
Water Temperature (Deg Celc)	10

Grit Chamber Input Values

Minimum Particle Size (mm)	0.15
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Tank Length (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	

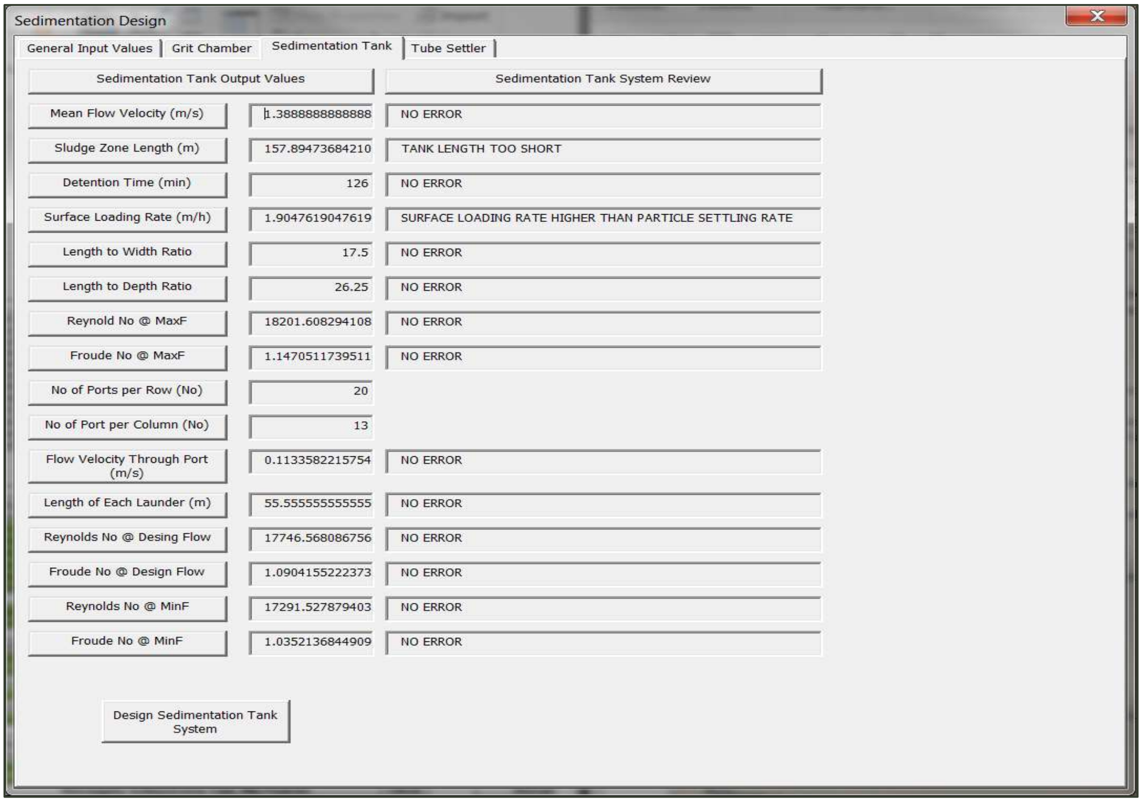
Sedimentation Tank Input Values

Minimum Particle Size (mm)	2
Number of Systems (No)	2
No of Active Systems (No)	2
Water Depth (m)	4
Tank Width (m)	18
Tank Length (m)	105
Safety Factor	1
No of Longitudinal Walls (No)	2
Baffle Wall Port Diameter (mm)	0.12
Baffle Wall Port Spacing (mm)	0.15
Launder Load Rate (m^3/mh)	10.8
Number of Launders (No)	3

Tube Settler Input Values

Minimum Particle Size (mm)	4
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	
Launder Load Rate (m^3/mh)	
Number of Launders (No)	
Tube Settler Height (m)	
Tube Settler Incline (Deg)	
Tube Settler L and W (m)	
% of Tank Covered (%)	

Figure 25: HFC input values



Sedimentation Design

Sedimentation Tank Output Values		Sedimentation Tank System Review
Mean Flow Velocity (m/s)	1.3888888888888	NO ERROR
Sludge Zone Length (m)	157.89473684210	TANK LENGTH TOO SHORT
Detention Time (min)	126	NO ERROR
Surface Loading Rate (m/h)	1.9047619047619	SURFACE LOADING RATE HIGHER THAN PARTICLE SETTLING RATE
Length to Width Ratio	17.5	NO ERROR
Length to Depth Ratio	26.25	NO ERROR
Reynold No @ MaxF	18201.608294108	NO ERROR
Froude No @ MaxF	1.1470511739511	NO ERROR
No of Ports per Row (No)	20	
No of Port per Column (No)	13	
Flow Velocity Through Port (m/s)	0.1133582215754	NO ERROR
Length of Each Launder (m)	55.555555555555	NO ERROR
Reynolds No @ Desing Flow	17746.568086756	NO ERROR
Froude No @ Design Flow	1.0904155222373	NO ERROR
Reynolds No @ MinF	17291.527879403	NO ERROR
Froude No @ MinF	1.0352136844909	NO ERROR

Figure 26: HFC calculations

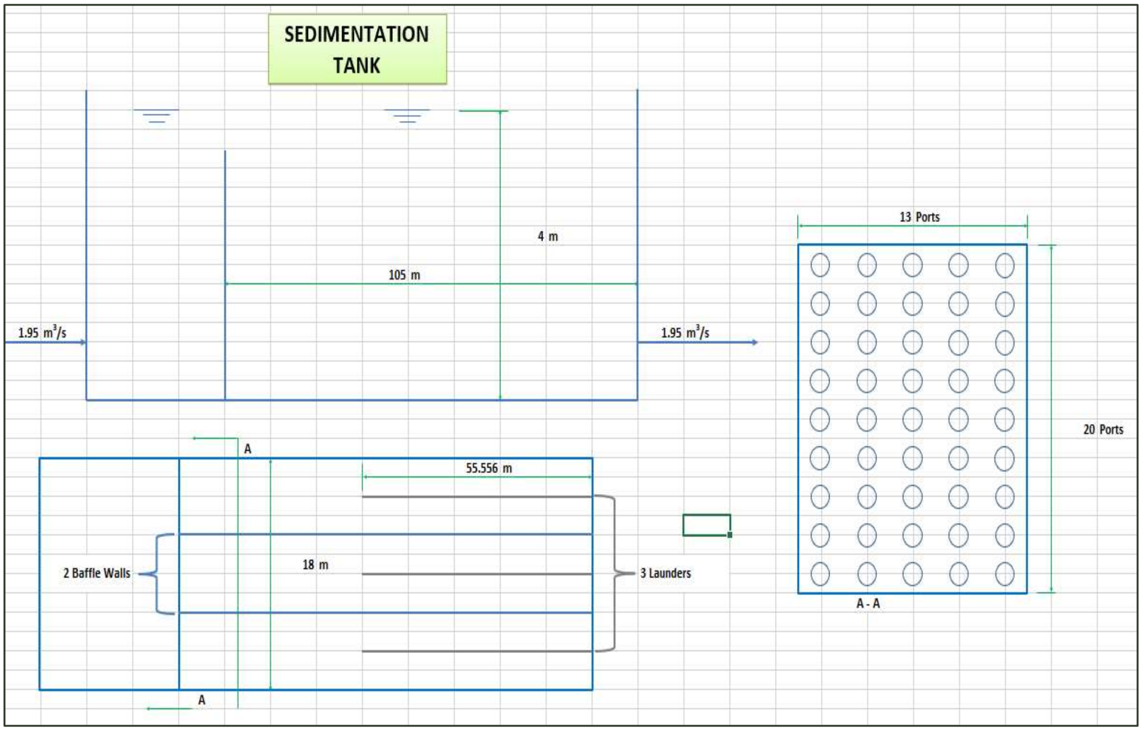


Figure 27: HFC schematic

Sedimentation example 3: HFC with settler modules

Design a sedimentation system comprising two HFCs equipped with settler modules, for a plant maximum flowrate of 2 m³/s.

The following values are given:

Tank width:	18 m
Water depth:	4 m
Number of tanks:	2
Safety factor:	1
Particle size:	1 mm
Particle settling velocity:	0.00025 m/s (*modified from Kawamura (2000:143) for 1 mm alum floc for this example*)
Number of longitudinal baffle walls:	1
Water temperature:	5°C
Kinematic viscosity:	1.5271 x 10^{-6} m^2/s
Spacing between diffuser wall ports:	0.15 m
Port diameter:	0.12 m
Launder loading rate:	7.7625 m^3/mh
Number of launders:	6
Settler module vertical height:	0.55 m
Settler module incline:	60°
Settler module internal dimension:	0.05 m
Portion of tank covered by settler modules:	75%

Calculations:

$$A_{set} = \frac{Q_{max} \times d_{set}}{n \times v_o \times (h_{set} \times \cos\alpha + d_{set} \times \cos^2\alpha)}$$

$$A_{set} = \frac{2 \times 0.05}{2 \times 0.00025 \times (0.55 \times 0.5 + 0.05 \times 0.5^2)}$$

$$\boldsymbol{A_{set} = 695.65\ m^2}$$

$$L_{set} = \frac{A_{set}}{W}$$

$$L_{set} = \frac{695.65}{18}$$

$$\boldsymbol{L_{set} = 38.65\ m}$$

$$L = \frac{L_{set}}{r_{set}}$$

$$L = \frac{38.65}{0.75}$$

$$\boldsymbol{L = 51.53\ m}$$

$$v_s = \frac{Q_{max} \times 3600}{n \times A_{set}}$$

$$v_s = \frac{2 \times 3600}{2 \times 695.65}$$

$$\boldsymbol{v_s = 5.175\ {}^{m}/_{h}}$$

$$v_{set} = \frac{Q_{max}}{n \times A_{set} \times \sin\alpha}$$

$$v_{set} = \frac{2}{2 \times 695.65 \times \sin 60^{o}}$$

$$\boldsymbol{v_{set} = 0.00166\ {}^{m}/_{s}}$$

$$l_{set} = \frac{h_{set}}{\sin\alpha}$$

$$l_{set} = \frac{0.55}{\sin 60^{o}}$$

$$\boldsymbol{l_{set} = 0.635\ m}$$

$$t_{set} = \frac{l_{set}}{v_{set} \times 60}$$

$$t_{set} = \frac{0.635}{0.00166 \times 60}$$

$$\boldsymbol{t_{set} = 6.375\ m}$$

$$a_{set} = d_{set}^2$$

$$a_{set} = 0.05^2$$

$$\boldsymbol{a_{set} = 0.0025\ m^2}$$

$$P_{set} = 4 \times d_{set}$$

$$P_{set} = 4 \times 0.05$$

$$\boldsymbol{P_{set} = 0.2\ m}$$

$$R_{set} = \frac{a_{set}}{P_{set}}$$

$$R_{set} = \frac{0.0025}{0.2}$$

$$\boldsymbol{R_{set} = 0.0125\ m}$$

$$Re = \frac{v_{set} \times R_{set}}{\nu}$$

$$Re = \frac{0.00166 \times 0.0125}{1.5217 \times 10^{-6}}$$

$$\boldsymbol{Re = 13.6}$$

$$Fr = \frac{v_{set}^2}{g \times R_{set}}$$

$$Fr = \frac{0.00166^2}{9.81 \times 0.0125}$$

$$\boldsymbol{Fr = 2.25\ \times 10^{-5}}$$

$$r_{l/_{w}} = \frac{L \times (n_{bw} + 1)}{W}$$

$$r_{l/_{w}} = \frac{51.53 \times (1 + 1)}{18}$$

$$\boldsymbol{r_{l/_{w}} = 5.72}$$

$$l_{launder} = \frac{Q_{max} \times 3600}{n \times v_{launder} \times 2 \times n_{launder}}$$

$$l_{launder} = \frac{2 \times 3600}{2 \times 7.7625 \times 2 \times 6}$$

$$\boldsymbol{l_{launder} = 38.65\ m}$$

The results above and the literature example (Kawamura, 2000:181-184) compares well. The example however is based on a specific particle settling rate – 0.00025 m/s. Therefore, to obtain a good comparison, the settling rate of a 1 mm floc particle in the software model is temporarily set to be the same. Slight differences are noted because the literature example defines/chooses the tank dimensions based on those calculated, whereas the above method incorporates the results directly. Also, the literature example uses a different kinematic viscosity.

$$s_{c-c} = s_p + d_p$$

$$s_{c-c} = 0.15 + 0.12$$

$$\boldsymbol{s_{c-c} = 0.27\ m}$$

$$n_{p,r} = \frac{\frac{W}{n_{bw}+1} - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,r} = \frac{\frac{18}{1+1} - 0.15 \times 2 - 0.12}{0.27}$$

$$\boldsymbol{n_{p,r} \approx 31}$$

$$n_{p,c} = \frac{H - s_p \times 2 - d_p}{s_{c-c}}$$

$$n_{p,c} = \frac{4 - 0.15 \times 2 - 0.12}{0.27}$$

$$\boldsymbol{n_{p,c} \approx 13}$$

$$v_p = \frac{Q_{max}}{n \times (n_{bw} + 1) \times n_p \times \pi \times \left(\frac{d_p}{2}\right)^2}$$

$$v_p = \frac{2}{2 \times (1+1) \times 13 \times 31 \times \pi \times \left(\frac{0.12}{2}\right)^2}$$

$$v_p = 0.11\ {}^{m}/_{s}$$

Minimum and maximum calculations:

$$v_{set,des} = \frac{Q}{n \times A_{set} \times \sin\alpha}$$

$$v_{set,des} = \frac{1.33}{2 \times 695.65 \times \sin 60}$$

$$\boldsymbol{v_{set,des} = 0.0011\ {}^{m}/_{s}}$$

$$Re_{des} = \frac{v_{set,des} \times R_{set}}{\nu}$$

$$Re_{des} = \frac{0.0011 \times 0.0125}{1.5217 \times 10^{-6}}$$

$$\boldsymbol{Re_{des} = 9.067}$$

$$Fr_{des} = \frac{v_{set,des}^2}{g \times R_{set}}$$

$$Fr_{des} = \frac{0.0011^2}{9.81 \times 0.0125}$$

$$\boldsymbol{Fr_{des} = 9.94\ \times 10^{-6}}$$

$$v_{set,min} = \frac{Q_{min}}{1 \times A_{set} \times \sin\alpha}$$

$$v_{set,min} = \frac{1}{1 \times 695.65 \times \sin 60}$$

$$\boldsymbol{v_{set,min} = 0.00166\ {}^{m}/_{s}}$$

$$Re_{min} = \frac{v_{set,min} \times R_{set}}{\nu}$$

$$Re_{min} = \frac{0.00166 \times 0.0125}{1.5217 \times 10^{-6}}$$

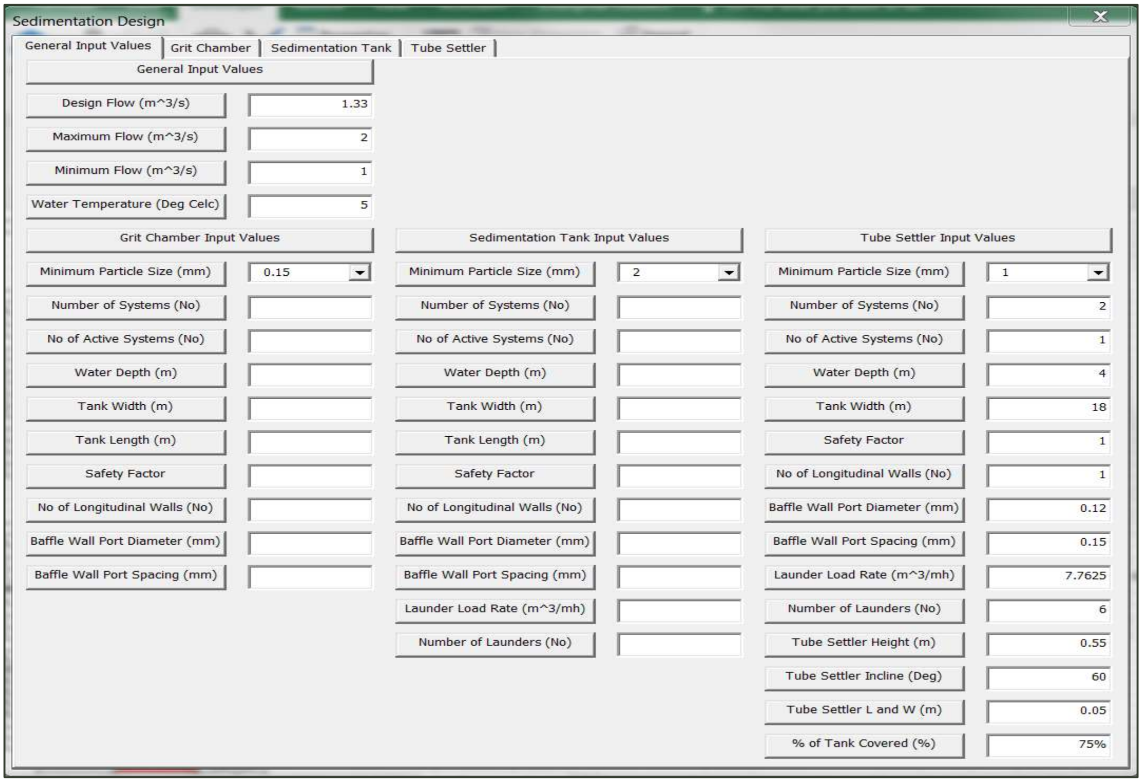
$$\boldsymbol{Re_{min} = 13.64}$$

$$Fr_{min} = \frac{v_{set,min}^2}{g \times R_{set}}$$

$$Fr_{min} = \frac{0.00166^2}{9.81 \times 0.0125}$$

$$\boldsymbol{Fr_{min} = 2.247 \times 10^{-5}}$$

Below in Figure 29 it is shown that the software model produces the same results. All parameters are met, except the Froude Number at the design plant flowrate, being marginally less than the required lower limit, which may be considered adequate.



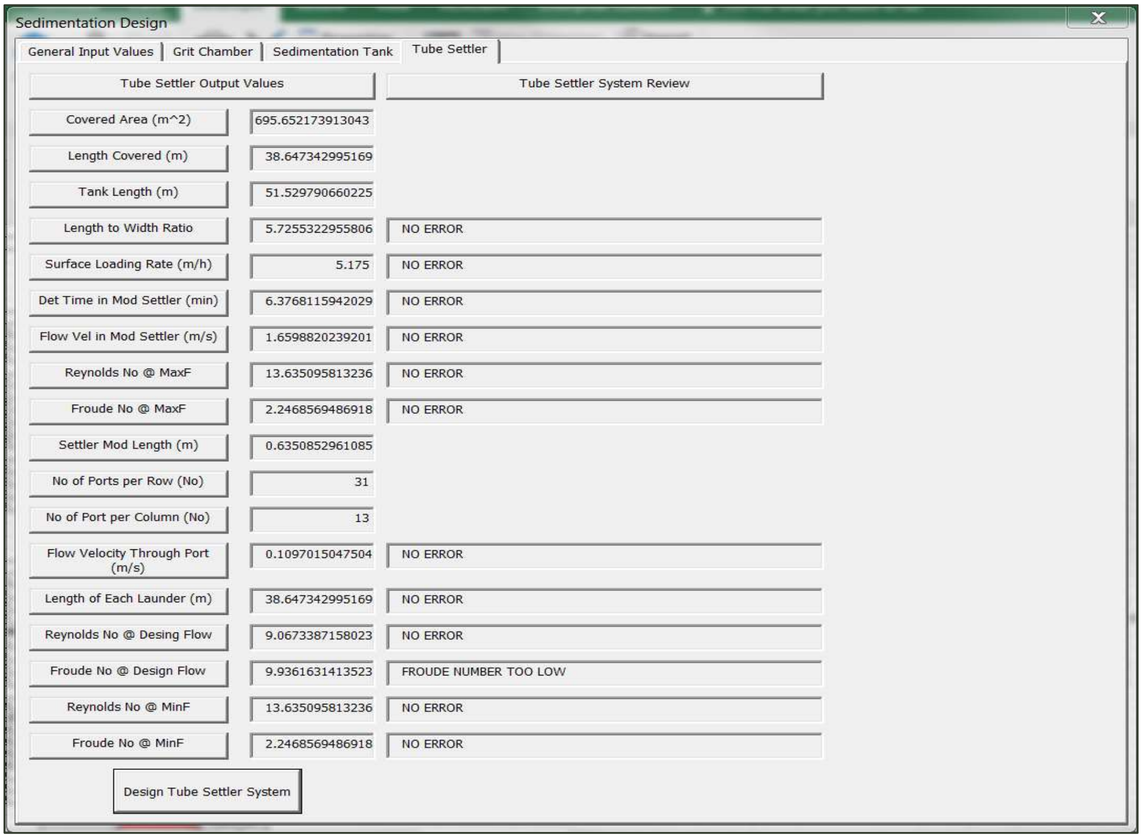
General Input Values	
Design Flow (m^3/s)	1.33
Maximum Flow (m^3/s)	2
Minimum Flow (m^3/s)	1
Water Temperature (Deg Celc)	5

Grit Chamber Input Values	
Minimum Particle Size (mm)	0.15
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Tank Length (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	

Sedimentation Tank Input Values	
Minimum Particle Size (mm)	2
Number of Systems (No)	
No of Active Systems (No)	
Water Depth (m)	
Tank Width (m)	
Tank Length (m)	
Safety Factor	
No of Longitudinal Walls (No)	
Baffle Wall Port Diameter (mm)	
Baffle Wall Port Spacing (mm)	
Launder Load Rate (m^3/mh)	
Number of Launders (No)	

Tube Settler Input Values	
Minimum Particle Size (mm)	1
Number of Systems (No)	2
No of Active Systems (No)	1
Water Depth (m)	4
Tank Width (m)	18
Safety Factor	1
No of Longitudinal Walls (No)	1
Baffle Wall Port Diameter (mm)	0.12
Baffle Wall Port Spacing (mm)	0.15
Launder Load Rate (m^3/mh)	7.7625
Number of Launders (No)	6
Tube Settler Height (m)	0.55
Tube Settler Incline (Deg)	60
Tube Settler L and W (m)	0.05
% of Tank Covered (%)	75%

Figure 28: HFC with settler modules input values



Tube Settler Output Values		Tube Settler System Review
Covered Area (m^2)	695.652173913043	
Length Covered (m)	38.647342995169	
Tank Length (m)	51.529790660225	
Length to Width Ratio	5.7255322955806	NO ERROR
Surface Loading Rate (m/h)	5.175	NO ERROR
Det Time in Mod Settler (min)	6.3768115942029	NO ERROR
Flow Vel in Mod Settler (m/s)	1.6598820239201	NO ERROR
Reynolds No @ MaxF	13.635095813236	NO ERROR
Froude No @ MaxF	2.2468569486918	NO ERROR
Settler Mod Length (m)	0.6350852961085	
No of Ports per Row (No)	31	
No of Port per Column (No)	13	
Flow Velocity Through Port (m/s)	0.1097015047504	NO ERROR
Length of Each Launder (m)	38.647342995169	NO ERROR
Reynolds No @ Desing Flow	9.0673387158023	NO ERROR
Froude No @ Design Flow	9.9361631413523	FROUDE NUMBER TOO LOW
Reynolds No @ MinF	13.635095813236	NO ERROR
Froude No @ MinF	2.2468569486918	NO ERROR

Figure 29: HFC with settler module calculations

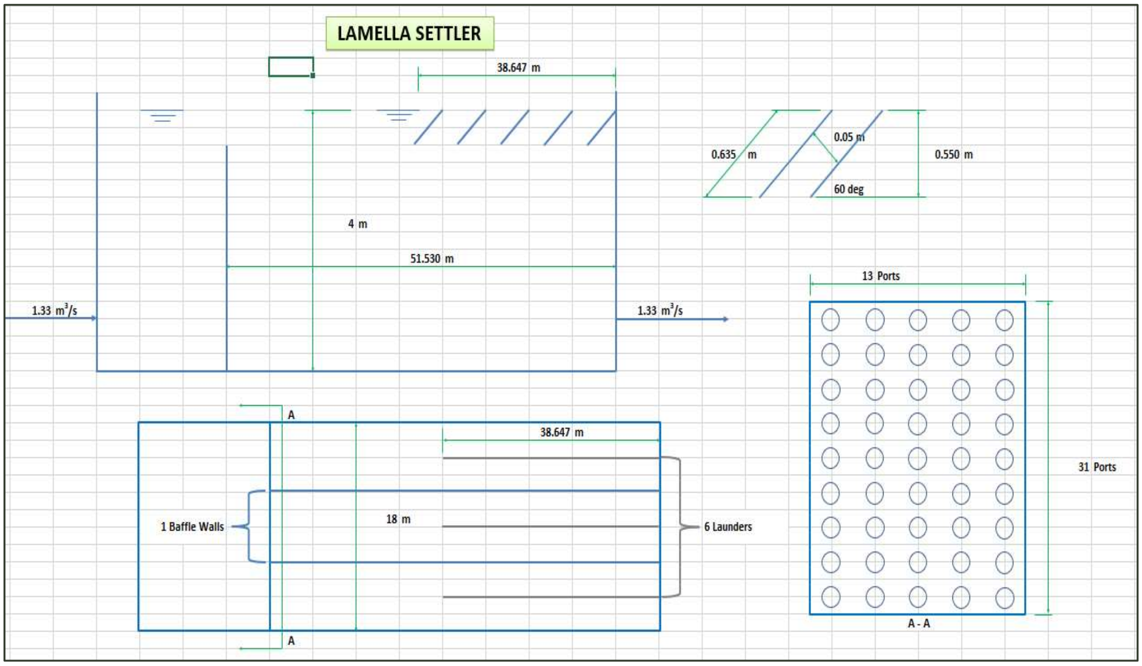


Figure 30: HFC with settler module schematic

Filtration example 1: slow sand filters

Design a slow sand filter system with a treatment capacity of 3 Ml/d (0.0347 m^3/s).

The following values are given:

Filtration rate: 0.1 m/h
Number of filters: 6
Number of online filters: 5
Media depth: 1.25 m
Length to width ratio: 1
Underdrainage system: Perforated pipe system
Freeboard height: 0.3 m
Supernatant depth: 1.25 m
Maximum flowrate: 4.5 Ml/d (0.0521 m^3/s)
Number of always online filters: 5

Calculations:

$$A = \frac{Q \times 3600}{v \times n}$$

$$A = \frac{0.0347 \times 3600}{0.1 \times 6}$$

$$\boldsymbol{A = 208.3\ m^2}$$

$$L = \sqrt{A \times r_{l/w}}$$

$$L = \sqrt{208.3 \times 1}$$

$$\boldsymbol{L = 14.43\ m}$$

$$W = \frac{L}{r_{l/w}}$$

$$W = \frac{14.43}{1}$$

$$\boldsymbol{W = 14.43\ m}$$

$$H = h_{med} + h_{free} + h_{super} + h_{gravel} + h_{u,d}$$

$$H = 1.25 + 0.3 + 1.25 + 0.05 + 0.3$$

$$\boldsymbol{H = 3.15\ m}$$

$$v_{max} = \frac{Q_{max} \times 3600}{A \times n_{on}}$$

$$v_{max} = \frac{0.052 \times 3600}{208.3 \times 5}$$

$$\boldsymbol{v_{max} = 0.18\ {}^{m}/_{h}}$$

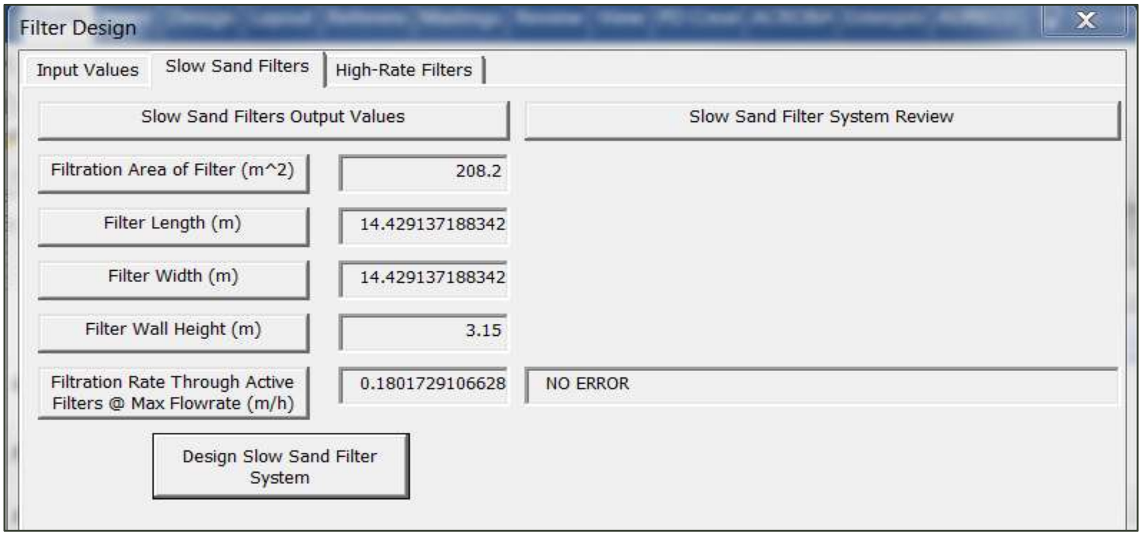
Unfortunately, no examples could be obtained for the design of a slow sand filter. Figure 32 shows the matching results from the software model.

General Input Values

Design Flow (m^3/s)	0.0347
Maximum Flow (m^3/s)	0.0521

Slow Sand Filter Input Values		High-Rate Filter Input Values	
Filtration Rate (m/h)	0.1	Water Temperature (Deg Celc)	
Number of Filters (No)	6	Filter Panel Length (m)	
Number of Online Filters (No)	5	Filter Panel Width (m)	
Media Depth (m)	1.25	Underdrain System	Dual-Parallel Lat
Length to Width Ratio (1:)	1	Available Clogging Head (m)	
Underdrainage System	Pipes	No of Filters (No)	
Freeboard (m)	0.3	No of Online Filters (No)	
Supernatant Water Depth (m)	1.25	Filtration Rate (m/h)	

Figure 31: SSF input values



Filtration Area of Filter (m^2): 208.2
Filter Length (m): 14.429137188342
Filter Width (m): 14.429137188342
Filter Wall Height (m): 3.15
Filtration Rate Through Active Filters @ Max Flowrate (m/h): 0.1801729106628
NO ERROR

Figure 32: SSF calculations

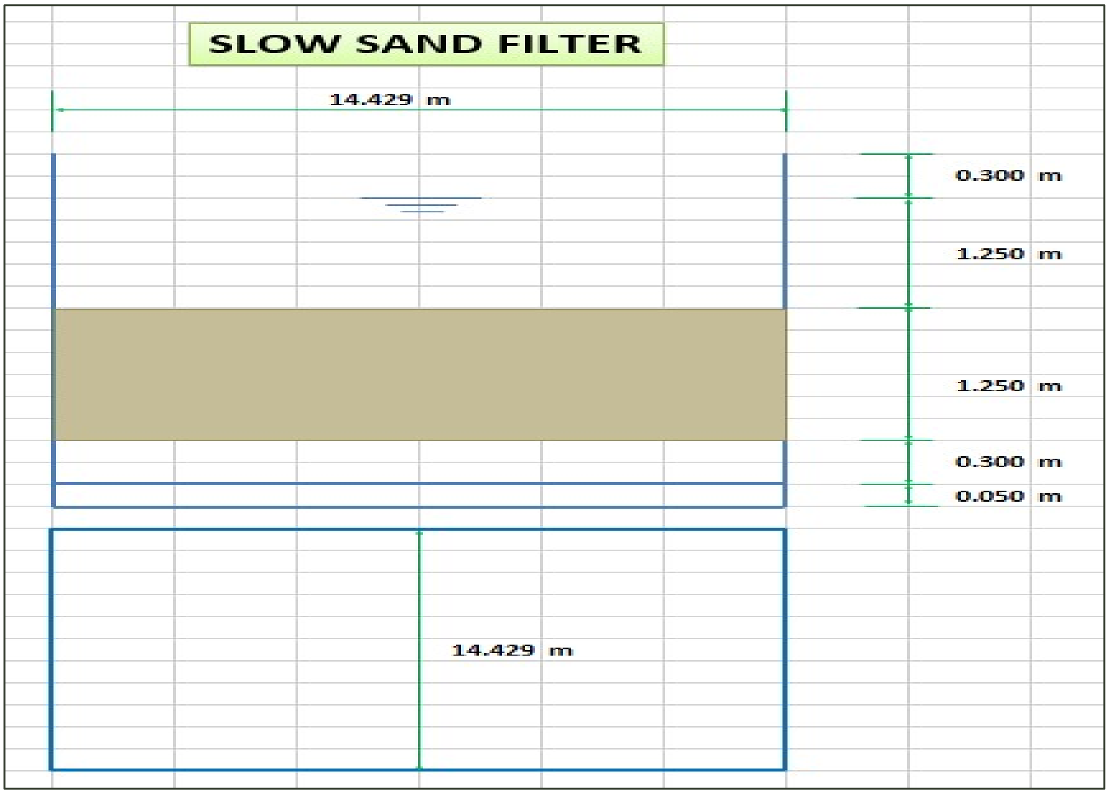


Figure 33: SSF Schematic

Filtration example 2: high-rate filters

Design a high-rate filter system with a treatment capacity of 0.485 m^3/s and a maximum flowrate of 0.5335 m^3/s.

<u>The following values are given</u>:

Water temperature:	20°C
Absolute viscosity:	1.0005×10^{-3} N.s/m^2
Water density:	998.19 kg/m^3
Panel length:	0.61

Panel width: 0.61
Underdrainage system: Monolithic system
Available clogging head: 2 m
No of filters: 4
No of online filters: 3
Target filtration rate: 18 m/h
Media depth: 1.2 m
Length to width ratio: 1.7
Backwash channel position: Side
Air scour rate: 60 m/h
Blower efficiency: 80%
Air headloss: 2 m
Pump efficiency: 80%
Backwash water headloss: 9 m
Number of backwash storages in clearwell: 2
Pumps operating % for duty 2: 150%
Number of always online filters: 3

<u>Calculations:</u>

$$A_{in} = \frac{Q \times 3600}{v \times n}$$

$$A_{in} = \frac{0.485 \times 3600}{18 \times 4}$$

$$\boldsymbol{A_{in} = 24.25\ m^2}$$

$$W_{fil} = \sqrt{\frac{A}{r_{l/w}}}$$

$$W_{fil} = \sqrt{\frac{24.25}{1.7}}$$

$$\boldsymbol{W_{fil} = 3.78\ m}$$

$$W_{in} = W_{fil} + BC_{width} + BCW_{width} \times n_{BCW}$$

$$W_{in} = 3.78 + 0.8 + 0.25 \times 1$$

$$\boldsymbol{W_{in} = 4.83\ m}$$

$$L_{fil} = W \times r_{l/w}$$

$$L_{fil} = 4.83 \times 1.7$$

$$\boldsymbol{L_{fil} = 8.21\ m}$$

$$n_{p,w} = \frac{W_{fil}}{l_{panel}}, (rounded)$$

$$n_{p,w} = \frac{3.78}{0.61}, (rounded)$$

$$\boldsymbol{n_{p,w} \approx 6}$$

$$n_{p,w,c} = \frac{n_{p,w}}{n_{cells}}, (rounded)$$

$$n_{p,w,c} = \frac{6}{1}, (rounded)$$

$$\boldsymbol{n_{p,w,c} = 6}$$

$$n_{p,l} = \frac{L_{fil}}{w_{panel}}, (rounded)$$

$$n_{p,l} = \frac{8.21}{0.61}, (rounded)$$

$$\boldsymbol{n_{p,l} \approx 13}$$

$$W_{res} = l_{panel} \times n_{p,w} + w_{anc}$$

$$W_{res} = 0.61 \times 6 + 0$$

$$\boldsymbol{W_{res} = 3.66\ m}$$

$$L_{res} = w_{panel} \times n_{p,l} + l_{anc}$$

$$L_{res} = 0.61 \times 13 + 0$$

$$\boldsymbol{L_{res} = 7.93\ m}$$

$$A_{res} = W_{res} \times L_{res}$$

$$A_{res} = 3.66 \times 7.93$$

$$\boldsymbol{A_{res} = 29.02\ m^2}$$

$$v_{res} = \frac{Q \times 3600}{A_{res} \times n}$$

$$v_{res} = \frac{0.485 \times 3600}{29 \times 4}$$

$$\boldsymbol{v_{res} = 15.05\ {}^{m}/_{s}}$$

$$W = W_{res} + BC_{width} + BCW_{width} \times n_{BCW}$$

$$W = 3.66 + 0.8 + 0.25 \times 1$$

$$\boldsymbol{W = 4.71\ m}$$

$$L = L_{res}$$

$$\boldsymbol{L = 7.93}$$

$$H = h_{u,d} + h_{clog} + h_{clean} + h_{free} + h_m + h_{anc}$$

$$H = 0.95 + 2 + 0.5 + 0.5 + 1.2 + 0$$

$$\boldsymbol{H = 5.15\ m}$$

$$v_{max} = \frac{Q_{max} \times 3600}{A_{res} \times n_{on}}$$

$$v_{max} = \frac{0.5335 \times 3600}{29 \times 3}$$

$$\boldsymbol{v_{max} = 22.06\ {}^{m}/_{h}}$$

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

$$d_{90} = 1 \times 10^{1.67 \times \log 1.4}$$

$$\boldsymbol{d_{90} = 1.754\ mm}$$

$$G_a = \frac{\left(\frac{d_{90}}{1000}\right)^3 \times \rho \times (\rho_s - \rho) \times g}{\mu^2}$$

$$G_a = \frac{\left(\frac{1.754}{1000}\right)^3 \times 998.19 \times (2650 - 998.19) \times 9.81}{(1.0005 \times 10^{-3})^2}$$

$$\boldsymbol{G_a = 87198.6}$$

$$\frac{V_{mf}}{3600} = \frac{\mu \times \sqrt{33.7^2 + 0.0408 \times G_a} - 33.7 \times \mu}{\rho \times \frac{d_{90}}{1000}}$$

$$\frac{V_{mf}}{3600} = \frac{(1.0005 \times 10^{-3}) \times \sqrt{33.7^2 + 0.0408 \times 87198.6} - 33.7 \times (1.0005 \times 10^{-3})}{998.19 \times \frac{1.754}{1000}}$$

$$\boldsymbol{V_{mf} = 71.6 \ {}^{m}/_{h}}$$

$$v_{bw1} = \left[43.5 - 8.5 \times \left(\frac{v_a}{60}\right)^2\right] \times \frac{v_{mf}}{100}$$

$$v_{bw1} = \left[43.5 - 8.5 \times \left(\frac{60}{60}\right)^2\right] \times \frac{71.6}{100}$$

$$\boldsymbol{v_{bw1} = 25.06 \ {}^{m}/_{h}}$$

$$Q_{bw1} = v_{bw} \ \times A_{res}$$

$$Q_{bw1} = 25.06 \times 29.02$$

$$\boldsymbol{Q_{bw1} = 727.24 \ {}^{m^3}/_{h}}$$

$$Q_{bw2} = Q_{bw1} \times r_{p,2}$$

$$Q_{bw2} = 727.24 \times 150\%$$

$$\boldsymbol{Q_{bw2} = 1090.86 \ {}^{m^3}/_{h}}$$

$$v_{bw2} = \frac{Q_{bw2}}{A_{res}}$$

$$v_{bw2} = \frac{1090.86}{29.02}$$

$$\boldsymbol{v_{bw2} = 37.59 \ {}^{m}/_{h}}$$

$$h_w = \left(\frac{3 \times 1090.86}{3600 \times 2 \times 7.93 \times \sqrt{2 \times 9.81} \times 0.616}\right)^{\frac{2}{3}}$$

$$\boldsymbol{h_w = 0.076 \, m}$$

$$P_{B(kW)} = \frac{h_{air} \times 9804 \times v_a \times A_{res}}{\eta_{blower} \times 3600 \times 1000}$$

$$P_{B(kW)} = \frac{2 \times 9804 \times 60 \times 29.02}{0.8 \times 3600 \times 1000}$$

$$\boldsymbol{P_{B(kW)} = 11.85 \, kW}$$

$$P_{P(kW)} = \frac{Q_{bw2} \times \rho \times g \times (h_w + h_{bw})}{\eta_{pump} \times 3600 \times 1000}$$

$$P_{P(kW)} = \frac{1090.86 \times 998.19 \times 9.81 \times (0.076 + 9)}{0.8 \times 3600 \times 1000}$$

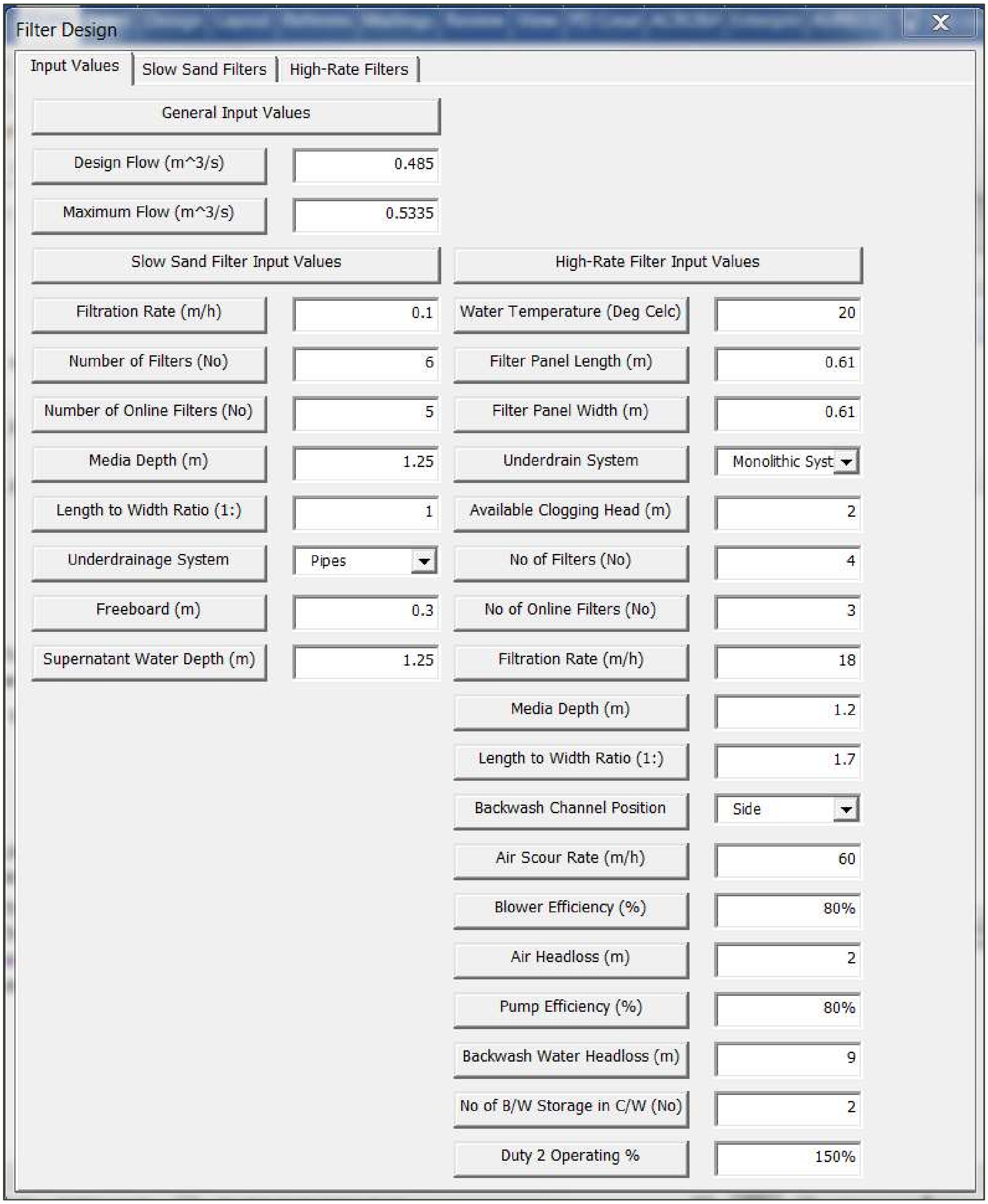
$$\boldsymbol{P_{P(kW)} = 33.66 \, kW}$$

$$V_c = A_{res} \times \left[\left(5 \times h_{bc} + 4 \times (h_m \times \varepsilon_0 + h_w)\right) - 0.2\right] \times n_{bw}$$

$$V_c = 29.02 \times \left[\left(5 \times 0.9 + 4 \times (1.2 \times 0.44 + 0.076)\right) - 0.2\right] \times 2$$

$$\boldsymbol{V_c = 390\ m^3}$$

The above results (excluding the pump motor and blower motor sizing) match that calculated through methods developed by a work colleague (B. Theunissen). Figure 35 shows the results from the software model, matching the above.

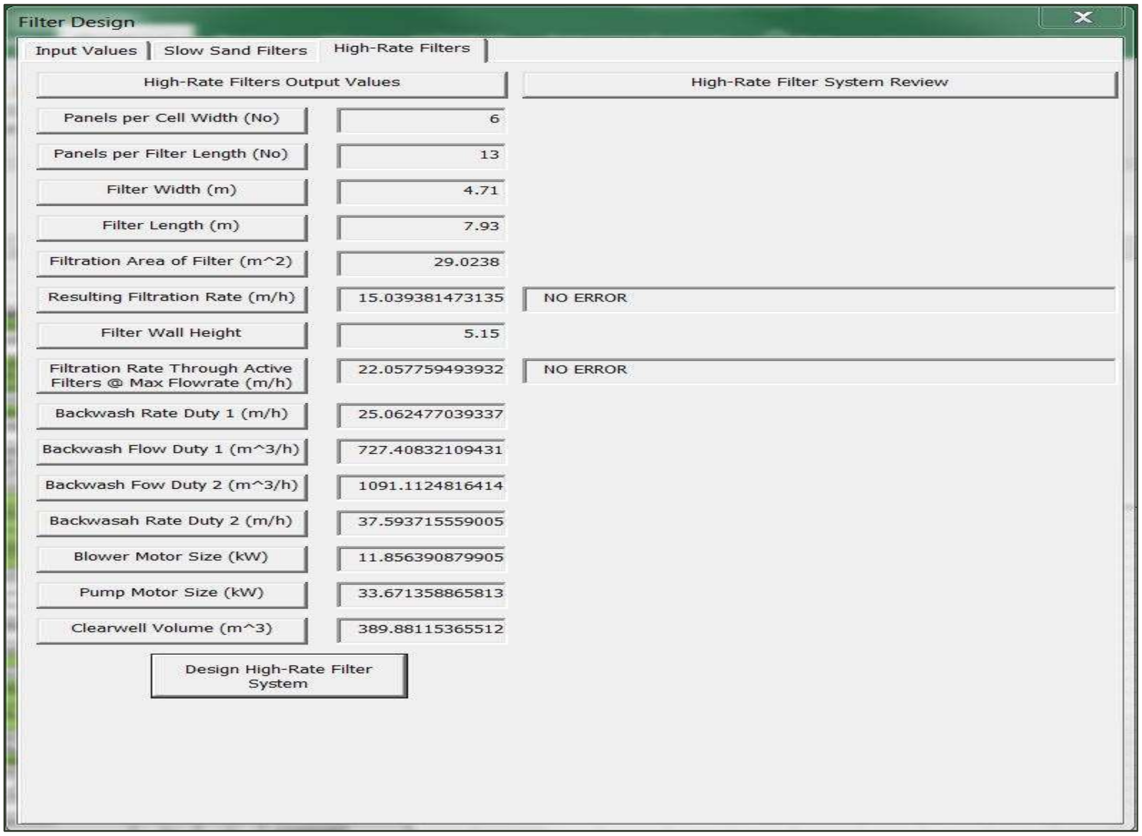


General Input Values	
Design Flow (m^3/s)	0.485
Maximum Flow (m^3/s)	0.5335

Slow Sand Filter Input Values	
Filtration Rate (m/h)	0.1
Number of Filters (No)	6
Number of Online Filters (No)	5
Media Depth (m)	1.25
Length to Width Ratio (1:)	1
Underdrainage System	Pipes
Freeboard (m)	0.3
Supernatant Water Depth (m)	1.25

High-Rate Filter Input Values	
Water Temperature (Deg Celc)	20
Filter Panel Length (m)	0.61
Filter Panel Width (m)	0.61
Underdrain System	Monolithic Syst
Available Clogging Head (m)	2
No of Filters (No)	4
No of Online Filters (No)	3
Filtration Rate (m/h)	18
Media Depth (m)	1.2
Length to Width Ratio (1:)	1.7
Backwash Channel Position	Side
Air Scour Rate (m/h)	60
Blower Efficiency (%)	80%
Air Headloss (m)	2
Pump Efficiency (%)	80%
Backwash Water Headloss (m)	9
No of B/W Storage in C/W (No)	2
Duty 2 Operating %	150%

Figure 34: High-rate filter input values



Panels per Cell Width (No)	6	
Panels per Filter Length (No)	13	
Filter Width (m)	4.71	
Filter Length (m)	7.93	
Filtration Area of Filter (m^2)	29.0238	
Resulting Filtration Rate (m/h)	15.039381473135	NO ERROR
Filter Wall Height	5.15	
Filtration Rate Through Active Filters @ Max Flowrate (m/h)	22.057759493932	NO ERROR
Backwash Rate Duty 1 (m/h)	25.062477039337	
Backwash Flow Duty 1 (m^3/h)	727.40832109431	
Backwash Fow Duty 2 (m^3/h)	1091.1124816414	
Backwasah Rate Duty 2 (m/h)	37.593715559005	
Blower Motor Size (kW)	11.856390879905	
Pump Motor Size (kW)	33.671358865813	
Clearwell Volume (m^3)	389.88115365512	

Figure 35: High-rate filter calculations

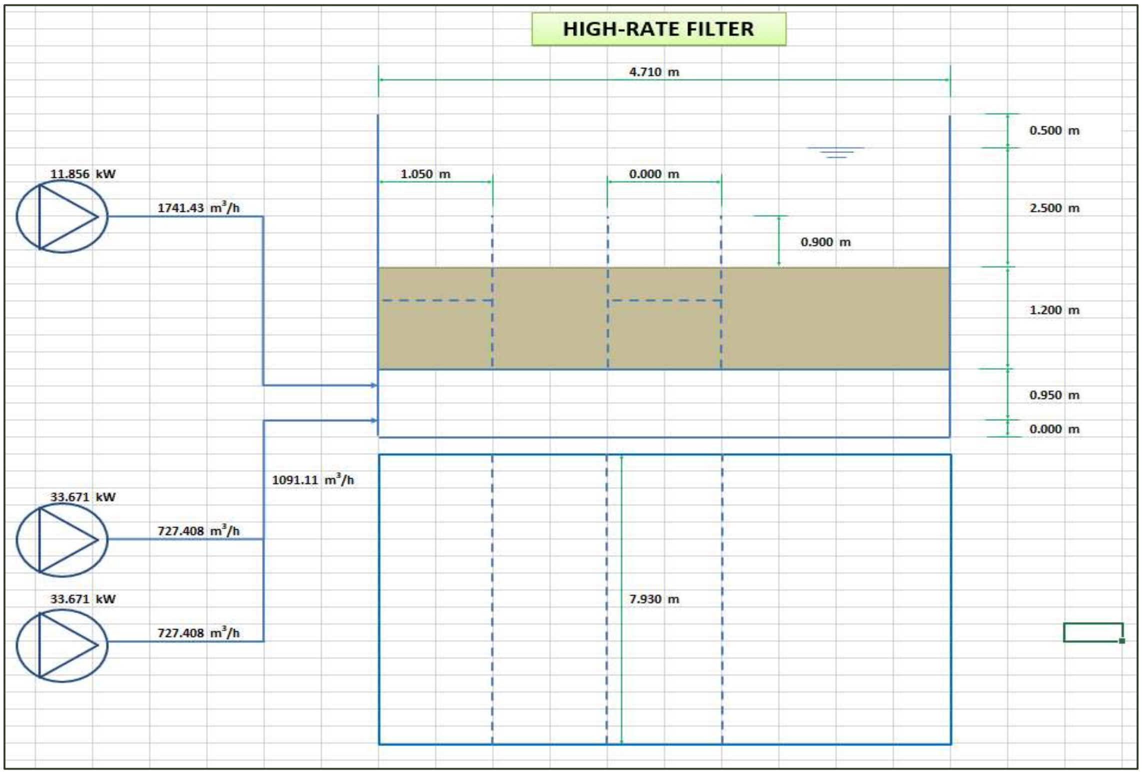


Figure 36: High-rate filter schematic

Disinfection example 1: chlorine disinfection

No comprehensive example could be found to validate the overall disinfection design component of the software model. As such, the validation was done through a combination of different examples and first the equation for calculating the giardia inactivation number (Nova Scotia Environment, 2012:65) is reviewed, based on the following given values:

Maximum flowrate	0.5 m^3/s
Upstream giardia removal	2.5 log
pH	6 and 7
Residual chlorine concentration	0.8 and 1.4
Water temperature	0.5^{o}C

<u>Calculations</u>

$$G_{lr} = 3 - G_{ru}$$

$$G_{lr} = 3 - 2.5$$

$$G_{lr} = 0.5$$

$$G_{CT} = 0.2828 \times pH^{2.69} \times Cl^{0.15} \times G_{lr} \times 0.933^{T-5}$$

$$G_{CT} = 0.2828 \times 6^{2.69} \times 0.8^{0.15} \times 0.5 \times 0.933^{0.5-5}$$

$$\boldsymbol{G_{CT} = 23.15\ ^{mg.min}/_{l}}$$

$$G_{lr} = 3 - G_{ru}$$

$$G_{lr} = 3 - 2.5$$

$$G_{lr} = 0.5$$

$$G_{CT} = 0.2828 \times pH^{2.69} \times Cl^{0.15} \times G_{lr} \times 0.933^{T-5}$$

$$G_{CT} = 0.2828 \times 7^{2.69} \times 1.4^{0.15} \times 0.5 \times 0.933^{0.5-5}$$

$$\boldsymbol{G_{CT} = 38.12\ ^{mg.min}/_{l}}$$

These above results match those determined by the Novia Scotia Environment (2012:65). A CT value for a 4-log removal of viruses can be interpolated from Table 14 as follows:

$$y_2 = \frac{(x_2 - x_1) + (y_3 - y_1)}{x_3 - x_1} + y_1$$

$$y_2 = 8 - \frac{(7-5) + (8-6)}{10-5}$$

$$\boldsymbol{y_2 = 7.2\ ^{mg.min}/_{l}}$$

The above result can also be found in the example by Novia Scotia Environment (2012:87) as well as from the software model.

Using the maximum inactivation number (38.12 mg.min/l) from above results, as well as a baffling factor of 0.7, the volume of the contact tank can be calculated as follows:

$$t = \frac{CT_{max}}{Cl \times BF}$$

$$t = \frac{38.12}{1.4 \times 0.7}$$

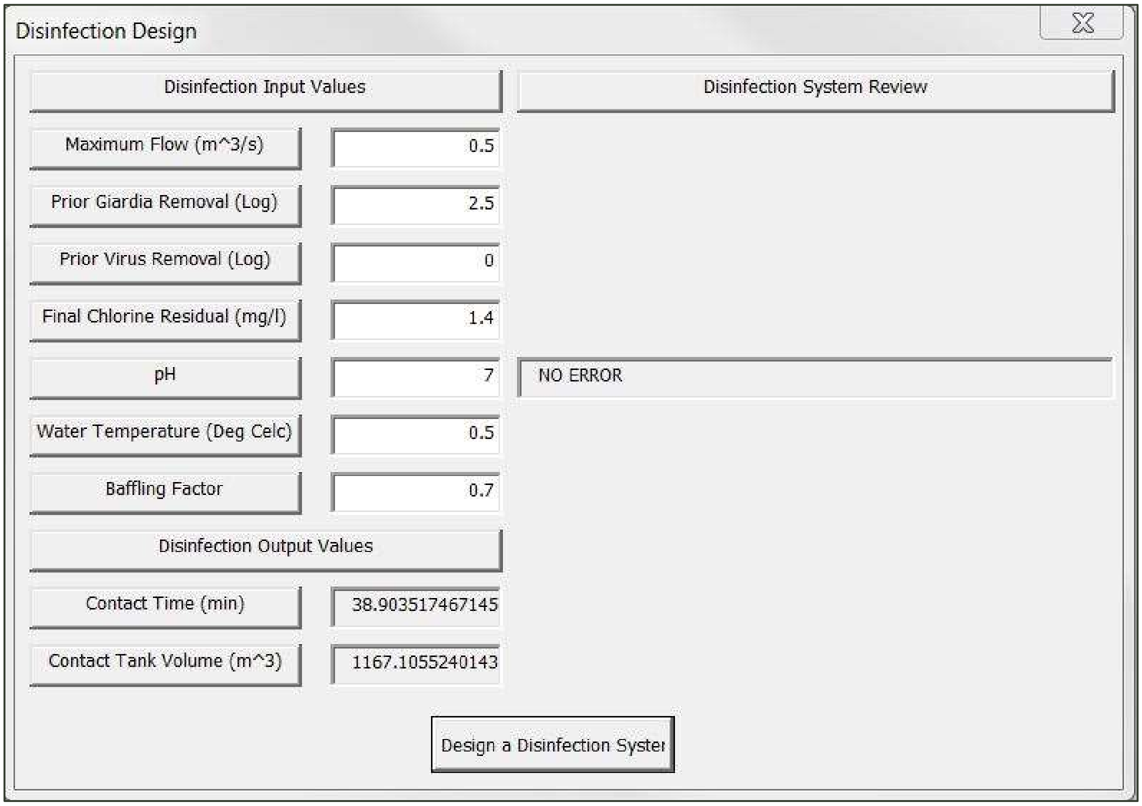
$$\boldsymbol{t = 38.9\ min}$$

$$V = t \times 60 \times Q_{max}$$

$$V = 38.9 \times 60 \times 0.5$$

$$\boldsymbol{V = 1167\ m^3}$$

These results are shown on Figure 37 below.



Maximum Flow (m^3/s)	0.5
Prior Giardia Removal (Log)	2.5
Prior Virus Removal (Log)	0
Final Chlorine Residual (mg/l)	1.4
pH	7
Water Temperature (Deg Celc)	0.5
Baffling Factor	0.7
Contact Time (min)	38.903517467145
Contact Tank Volume (m^3)	1167.1055240143

NO ERROR

Figure 37: Chlorine disinfection calculations

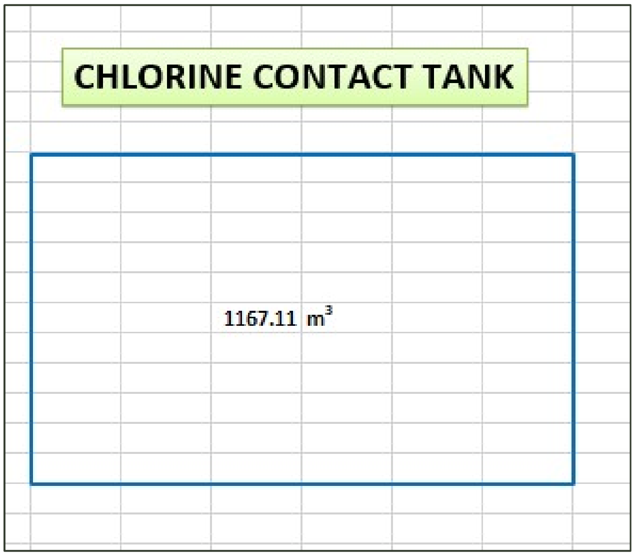


Figure 38: Chlorine disinfection schematic

STUDENT NAME: WILKE MORRISON
STUDENT NUMBER: MRRSAR010
2/22 Randolph Avenue
Parkside, 5063
Adelaide, Australia
C +61 427 101 576
E wilke.morrison@gmail.com