NAMIBIA UNIVERSITY OF SCIENCE AND TECHNOLOGY
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Department of Civil and Environmental Engineering

Designing of an Equalization Process for Improving the Performance of the Gammams Water Care Works, Windhoek, Namibia

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I, Silo Twamanguluka Shino, student number 200822004 hereby declare that the work contained in this thesis, entitled 'Designing of an Equalization Process for Improving the Performance of the Gammams Water Care Works, Windhoek, Namibia' is my own original work and that I have not previously in its entirety or in part submitted it at any university or other higher education institution for the award of a degree.

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ABSTRACT

This thesis examines and presents a first stage assessment of the potential improvement of waste-water treatment plant performance by including an equalization process as part of the treatment train of the Gammams Water Care works in Windhoek, Namibia. The treatment plant's operational objective of achieving compliant final effluent quality on a consistent basis is hampered by influent hydraulic and pollutant load daily diurnal pattern variations oppose to a near uniform condition experienced. These non-uniform hydraulic and pollutant loading conditions impose a negative impact on achieving overall optimal treatment plant performance.

During the study, daily variations in hydraulic and pollutant loading at the Gammams Water Care Works were established and a representative day of a typical week identified for the equalization process attenuation capacity design in order to minimize both flow and pollutant load diurnal pattern variations. Furthermore, a comparative analysis of equalised and unequalised treatment scenarios on overall plant performance was done by applying both numerical and graphical analysis methodologies aided by the STOAT computer software simulation model. This study uniquely made a comparative analysis of the current Gammams Water Care Works treatment train (which currently operates without an equalization step) using the STOAT model, considering both unequalised and equalised process equalization scenarios.

The graphical or Ripple method and a numerical time-step method were employed for equalization process attenuation capacity determination. The numerical time-step method together with STOAT modelling steady state simulations were employed for this 1st approximation step analyses, for initial comparison and impact assessment as well as for identifying the road map for further future detailed extended-time simulation purposes.

The study found that the plant typically experiences diurnal daily influent hydraulic and pollutant load variations similar to other plants receiving mainly domestic wastewater in the early morning hours and between midday and early evenings. For STOAT modelling purposes, current plant influent and subsequent unit process outflows along the treatment train were sampled and selected pollutant concentrations were determined over 4-hour intervals for the representative dry weather weekday mentioned before. The STOAT model calibration was limited to the actual Gammams plant treatment train, unit process sizes and operational criteria employed. Bio-chemical process kinetic algorithms inherent to the STOAT model was not changed due to the software limitations of use and not being allowed. The analysis done of unequalised and equalised scenarios was thereof of a comparative nature.

Based on these unit process outflow test results, pollutant parameters tested were COD, Ammonia, Orthophosphate, TKN, TSS, TDS and TS. The pollutant test results mentioned showed the latter pollutant concentrations reduced in value with sequential treatment train unit processes, confirming pollutant removal and treatment thereof taking place.

In the comparative STOAT model analyses done, only COD, TSS, TKN, Ammonia and Phosphorus were considered in accordance with the model's kinetic algorithm employed.

The study indicates that the introduction of an inline equalization process could potentially reduce the unequalised operation daily diurnal pollutant pattern variations (in relation to the daily average) to subsequent downstream unit processes. For unequalised flow, the minimum and maximum daily diurnal variations for COD amounted to 26.5% and 66%, for Ammonia 42% and 76%, for TSS 78% and 100%, for TKN 30% and 65%, while Phosphorus was 31% and 55%. By introduction of an equalization process, the comparative variations for COD became 10% and 16%, for Ammonia 31% and 7%, for TSS 42% and 28%, for TKN 21% and 14%, and Phosphorus 11% and 10%. Comparatively therefore, equalised flow employed resulted in a diurnal variation of 10 % to 42%, oppose to that of unequalised operation of 26.5% to 100%.

Should equalisation be introduced, the potential improvement in TSS removal efficiency for Primary settlers' amounts to nearly 7% (increasing from 38% to 45%), based on the STOAT model simulation. The combined pollutant removal efficiency here (weighted average ratio of individual pollutant efficiencies to concentration ratio) could potentially improve by nearly 4% (increasing from 16% to 20%).

The combined overall plant pollutant removal efficiency improvement could potentially be as high as approximately 34% (increasing from just below 58% to 92%).

It must however be borne in mind that these efficiencies are based on simulations for a single day selected as representative of typical plant conditions. More extended-time simulations and specific model calibration of plant kinetics would be required for a more accurate and representative efficiency removal outcome. These mentioned results should therefore be seen as indicative of improvement based on a first stage study only and not exact estimates whatsoever.

The study also explored the impact of increased release flows from the equalization basin on required attenuation capacity. It was found that increased release flow result in smaller basin attenuation capacity requirements compared to daily average daily flow release rate (ADF). These smaller attenuation capacities, such in the case of ADF plus 20% and 25%, the pollutant daily diurnal pattern variations increase resulting in subsequently plant final effluent quality not being compliant to that required by regulated effluent standards. However, for increased release flow rates of ADF plus 10% and 15%, the reduced attenuation capacity obtained will not compromise plant final effluent quality.

A phased approach for the provision of increased equalization attenuation capacity with growth in plant influent was also considered in the study. Considering a future 5-year growth in wastewater influent to the plant and associated phased extension of attenuation capacity, it would be viable to employ increased flow release rates from an equalization process basin and achieve reasonable attenuation to allow improved pollutant removal compared to unequalised conditions.

A decision-making tool was developed that allows determination of required equalization process release flow rates taking future growth in inflow into account. This is based on any selected attenuation capacity determined for current 2019 hydraulic loading to the plant.

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List of Acronyms and Abbreviations

ADWF	Average Dry Weather Flow: The average daily flow to the wastewater treatment works during dry periods.
Aerobic	Presence of Oxygen
Anaerobic	Absence of both Oxygen and Nitrate
Anoxic	Absence of Oxygen but with the presence of nitrate
BOD	Biological Oxygen Demand
BNR	Biological Nutrient Removal
COD	Chemical Oxygen Demand
GWCW	Gammams Water Care Works
TSS	Total suspended Solids
TDS	Total Dissolved Solids
TS	Total Solids
N$	Namibia Dollar
=	Equal
%	Percentage
BNR	Biological Nutrient Removal
PSTs	Primary Sedimentation Tanks
WWTP	Wastewater Treatment Plants
STOAT	Sewage Treatment Operation and Analysis over Time

SI Units

m^3/h	Cubic meters per Hour
m	Meters
kg/day	Kilograms per Day
ML/d	Mega-litres per day
m^3/d	Cubic metres per day
m^3/hr	Cubic metres hour
kWh/h	Kilowatts hour per hour

CHEMICAL SYMBOLS AND WATER QAULITY PARAMETERS

N	Nitrogen
NH_4-N	Ammonia
NO_2	Nitrite
NO_3	Nitrate
P	Phosphorous
pH	Potential for Hydrogen
SO_4	Sulphate
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TP	Total Phosphorous

CHAPTER 1

1. INTRODUCTION AND BACKGROUND

1.1 Introduction

The Republic of Namibia is a water-scarce country as highlighted by frequent draught cycles with associated very low rainfall. It is amongst the most arid countries in the world as it is bounded by two deserts, the Namib Desert in the west and the Kalahari Desert in the east whilst more than 80% of the country is desert to semi-desert, (Lahnsteiner & Lempert, 2007). It is against this background, that it is paramount to safeguard our water resources against pollution as well as to ensure their optimal use.

Windhoek is the capital city of Namibia and is located on the central highlands approximately 1600m above sea level (NSA, 2013). The central area of Namibia is characterised by an average annual rainfall of 350-400mm (Midley & Stern, 2014) and potential surface evaporation ranging from 320–340 mm per annum (Department of Water Affairs, 1988). Both the low annual rainfall and the high evaporation rate contribute to the arid climate found in and around the city of Windhoek. The closest perennial rivers, being the Kavango River in the north and the Orange River in the south, are more than 700 km away from the city.

The city of Windhoek mainly depends on the supply of water from three (3) freshwater storage reservoirs, being the Von Bach, Swakopoort and Omatako dams. The Von Bach dam is supplied with water from the Swakoppoort Ephemeral River, the Swakoppoort dam and the Omatako dam. The Omatako dam is supplied with water from Kombat town situated approximately 400 km north of Windhoek through the canal called the Omatako canal. The fresh water supply from the three (3) dam system is augmented by use of underground water supply from boreholes in the south of Windhoek as well as reclaimed water from the Windhoek Reclamation plant (WINGOC).

The Goreangab dam located on the Windhoek city peripheral, have been polluted by upstream catchment development and the exponential growth of the informal settlements outside the city with poor sanitation and leaking sewerage systems within the vicinity of the reservoir (Pazvakawambwa, 2018).

The city of Windhoek has a population of approximately 450 000 inhabitants, with a population growth rate of 4.4% per annum (NSA, 2013). This rapid population growth is caused by rapid industrial, economic and social developments which have triggered an increase in water demand. The increased water demand has caused a rise in wastewater flow, resulting in both hydraulic and pollutants overloading of wastewater treatment plants and consequently increasing the required treatment capacities.

The overloading of wastewater treatment plants causes poor treatment of wastewater, which results in pollution of the receiving water bodies, as well as reduction in the potential reclamation of water to portable standard.

1.2 Background to the Study

The management of wastewater has become a global concern. Wastewater treatment plants with improved efficiency, reliability and control are possible when physical, biological and chemical processes are operated at or near uniform flow and pollutant load conditions (EPA, 1974). The control of the influent load to the treatment plant is of great importance as instances of sludge loss due to poor settling and the effects of toxins in the influent are avoided (Bolmstedt, 2004). The introduction of an equalization processes can assist in control of the flow rate and the pollutant load into wastewater treatment plants (Bolmstedt, 2004).

The Gammams Water Care Works (GWCW) is a domestic wastewater treatment plant for the City of Windhoek, situated approximately 7 km north of the city centre in Wanaheda suburb. It serves the population of Windhoek which is connected to the domestic and light industry sewer system, while the industrial wastewater is diverted to the Ujams Wastewater Treatment Plant.

GWCW is a conventional wastewater treatment plant which involves screening, grit removal, primary treatment in terms of primary sedimentation tanks, and secondary treatment in terms of trickling (Bio-filters) filters and an activated sludge process. Further partial tertiary treatment is achieved using waste stabilization ponds.

In 1959, Gammams Water Care Works was constructed as a conventional biological filtration plant, which was later extended by addition of an Activated Sludge process in 1979. In the year 2001, the plant was further upgraded to accommodate the growth in the population of the city which resulted in an increase in the hydraulic flow to be treated at the plant. Currently, the plant has a design capacity of 26 Megalitres per day with diurnal flow variations ranging between 5 to as high as 45 Mega litres per day (J. Haihambo, personal interview, July 2018). These diurnal patterns of daily flows are due to corresponding variations in daily domestic water use (Ongerth, 1979).

During wet seasons, the primary settling tanks at the Gammams Water Care Works overflow of solids occur over the weir, resulting in poor settling and toxins being discharged into downstream processes (J. Haihambo, personal interview, July 2018).

Within this treatment plant, the primary sludge produced by the primary sedimentation process is anaerobically bio-digested with end-product bio-gas recovered (Tschobanglous, 1991b) and utilised for power generation to supplement the power needs at the plant(J. Haihambo, personal interview, July 2018).

A pre-secondary treatment equalization intervention for the plant is considered a viable option to reduce daily influent load diurnal pattern variations. This is due to a more uniform flow and loading being released to subsequent downstream unit treatment processes. Improved down-stream process efficiencies would potentially result in higher and more consistent effluent quality as well as an increased energy recovery potential.

The influent flow and pollutant load variations in the wastewater treatment plant influent complicate the operations of the biological treatment processes and impede process efficiency (Dold, 1982). Deviations of these hydraulic and pollutant load from a steady state condition cause plant operation problems in areas such as aeration control (due to load rate variations),settling tank overloading (due to flow rate variations) and/or over- or under-aeration which affects settling properties (Dold, 1982).

The equalization process is a method used to overcome the operational problems caused by influent flow rate and pollutant load rate variations, to improve the performance of the downstream treatment facilities (Eddy, 2003).

The equalization process can also potentially reduce plant capital and operational costs. For example: (1) aeration capacity to be provided will be determined essentially by the mean influent COD load instead of the peak load, (2) settling tank areas can be reduced to cope with the mean inflow rate, and not the peak flow rate (Dold, 1982). Since the current Gammams Water Care Works treated effluent serve as a water resource for the New Goreangab Water Reclamation facility (privately operated by WINGOC), the improved effluent quality level and consistency could potentially reduce the cost of further tertiary treatment for reclamation purposes.

The influent diurnal patterns to a particular plant are determined by a number of factors such as population structure; collection sewer layout and length and gradients; climatic and seasonal effects; etc. However, despite the many influencing factors, it was generally found that the combined effect gives rise to influent flow and diurnal load rate patterns that are similar for most plants. Typically, the flow rate reaches a maximum, at some time during the day, of about twice the average daily rate, and a minimum sometime during the night of about half the average rate. The influent COD and TKN concentrations show a similar pattern of behaviour, virtually in phase with the flow variations. As a result, the diurnal cyclic load rate has a huge variation range less than a quarter of the average daily value (Dold, 1982).

1.3 Statement of the Problem

The flow variations experienced at the Gammams Water Care Works range from 420m^3/h to 1519 m^3/h (J. Haihambo, personal communication, July 2018), causes shock loading upon the processes of the treatment system. Shock loads causes high loss of solids in secondary clarifiers resulting in carry over with final plant effluent, sloughing of biological growth or slime of trickling filters as well as upsetting the activated sludge biological process causing inconsistent pollutant removal rates and final effluent quality. This non-uniform hydraulic and pollutant concentration conditions, ranging from 595 mg/L to 1020 mg/L result in poor treatment efficiency and even high plant equipment wear and tear with associated higher refurbishment and maintenance costs. An intervention is therefore required to minimize influent diurnal pattern load variations and achieve near uniform conditions for optimal process operating efficiency and a consistent compliance with regulated effluent quality standards, which can be achieved by the introduction of an equalization process.

1.4 Objectives of the Study

1.4.1 General Objectives

To design an equalization intervention for the Gammams Water Care Works in Windhoek, Namibia.

The specific objectives identified for achieving the general goal are listed below.

1.4.2 Specific Objectives

1. To determine the daily variations in hydraulic and pollutant loading at the Gammams Water Care Works.
2. To design an equalization process.
3. To analyse and draw comparison between equalised and unequalised based overall treatment efficiencies of pollutant and hydraulic loading.

1.4.3 Research questions

The following research questions were identified and need to be answered in order to realise the above objectives:

1. What is the current daily influent diurnal pattern variation in hydraulic and pollutant loadings?
2. What attenuation capacity is required for the equalization process and which methodology can be employed to determine such?
3. To what extent will the treatment process efficiency improve after incorporating an equalization process for both carbonaceous and nitrogenous pollutant load removal?

1.5 Significance/Contribution

This study is crucial in understanding the process of controlling hydraulic and pollutant load passing through the wastewater treatment plant and associated improvement of inherent wastewater resource recovery for power generation at the plant (Gijzen, 2001). The study will enhance researchers' knowledge on achieving consistent treatment process efficiencies and possible improvement in the resource recovery by the introduction of influent hydraulic and pollutant load equalization.

The fact that an improved and more consistent effluent quality is achieved, avoids intermittent non-compliant final point discharges in receiving water bodies that affect community health adversely where indirect reuse of such discharges take place further downstream from discharges.

An equalization intervention has the potential to attenuate wet weather flows to some extent. This in turn will limit the extent of primary settling tank weir overflows and associated operational issues typically experienced during such periods at the Gammams Water Care Works.

The research results will benefit local authorities (municipalities, town and village councils) and even tourism facilities (e.g. lodges & game farms) by quantifying potential process efficiency and effluent quality consistency improvements by the introduction of an equalization process intervention.

In principle, implementing an equalization process in a wastewater treatment plant is aligned with and promotes the principles of sustainable environmental management and natural resource use (EMA, 2007).

CHAPTER 2

2. LITERATURE REVIEW

2.1 Literature Review and/or Theoretical Framework

The industrial wastewater of the City of Windhoek was previously treated employing an oxidation pond system at Ujams. Kgabi and Kalumbu (2017) conducted a study on the impact of treated industrial effluent discharged on the Klein Windhoek river water quality. The study showed poor pond effluent quality in terms of biomonitoring and soil analyses. Due to the environmental pollution caused by the said pond system effluent on the receiving river basin, a new high technology treatment facility (Ujams plant) was commissioned in 2014 to ensure adequate effluent quality is discharged into the river. This treatment plant incorporates an equalization process or buffer tank to reduce influent load variations, as well as neutralize pH variations for improved chemical feed operational control and overall optimal process efficiency.

As reported in EPA Technology Transfer Seminar Publication (EPA, 1974), the city of Fond du Lac in the United States employed a single-stage trickling filter plant in the 1950's to treat combined municipal industrial wastewater. The plant was designed to treat ultimate dry weather flow of approximately 30 Megalitres per day (8mgd) and a BOD loading of approximately 5670 kg/day (12 500 lb/day). However, in the early 1970s the flow reduced to nearly 27 Megalitres per day (7.1mgd) and BOD loading increased to nearly 10 900 kg/day (24 000 lb/day) resulting in pollutant overload. This wide fluctuation in pollutant load resulted in reduced performance of the trickling filters employed. Four (4) abandoned fixed cover digesters were modified and turned into equalization tanks to equalize this pollutant load fluctuation entering the tricking filters.

Similarly, a study of the effects of maintaining a constant flow of wastewater into a plant using an equalization basin was conducted in Newark, New Jersey (LaGrega & J.D. Keenan, 1974). A model developed for facilitating an equalised flow rate released from the equalization process reduced diurnal load variability by approximately 50%. Also, a substantial improvement of suspended solid removal in primary settler was affected.

Furthermore, in Sovanlinna, Finland, a study of the effect of flow equalization and low rate refermentation on the activated sludge process and the biological nutrient removal was conducted on the Pihlajaniemi wastewater treatment plant. This study indicated that the diurnal flow variations were efficiently levelled out and the pollutant matter was transformed into a more accessible form for the Biological Nutrient Removal bacteria when only flow variation was in operation (Mikola, 2013).

In conclusion, most of the wastewater process treatment efficiency problems pertain to non-uniform hydraulic and pollutant wastewater loading conditions caused by fluctuation in the water demand pattern, resulting in a corresponding pattern of wastewater discharge and daily diurnal variation in loading. However, effective measures can be implemented for a successful and optimal functioning wastewater treatment plant by employing an equalization process.

2.2 Historical background on wastewater treatment

wastewater treatment is the removal of impurities (pollutants) present and its removal to specified quality criteria levels before wastewater is discharged into the surface water sources or natural water bodies such as rivers, lakes and oceans (Bigum, 2012). Since pure water is not found in nature, any distinction between clean water and polluted water depends on the type and concentration of impurities found in the water as well as on its intended use. Water is said to be polluted when it contains enough impurities to make it unfit for a particular use.

The existence of wastewater and the need for wastewater treatment is not a new problem. Domestic waste (excreta, urine and grey water and sullage) is a natural part of human life. In parallel to growth in population and increasing urbanization, and with the introduction of the water closets (flush toilets) and centralized wastewater collection systems, challenges to provide adequate treatment for both final safe disposal and avoiding negative environmental impacts has become more extensive and innovative technical and operational interventions an absolute necessity. Apart from wastewater emanating from domestic source, both industrial and storm water (extraneous flow) end up in centralized wastewater collection systems for pre-disposal treatment.

Wastewater treatment and management is of greater importance since untreated wastewater discharged into natural water bodies, constitute a great hazard for the environment and a health risk for human and animal life. The environmental threat is mainly due to overloading of physical and chemical components associated with anthropogenic activities into ground water resources (aquifer), while the health risk is mainly the result of pathogenic contamination (Rupplel & Schlichting, 2011).

Many ancient cities had drainage systems which were mainly intended to carry rainwater away from roofs and pavements. A famous example is the drainage system of ancient Rome (Nathanson & Archis Ambulkar, 2012). This system included many surface conduits that were connected to a large vaulted channel called the Cloaca Maxima (Great Sewer), which carried drained water to the Tiber River. According to Nathanson and Archis, 2012, the Cloaca Maxima was built of stones and on a grand scale and is one of the oldest existing monuments of Roman engineering.

There was little progress in urban drainage systems or sewerage during the middle ages (Nathanson & Archis Ambulkar, 2012). Privy vaults and cesspools were used, but most wastes were simply dumped into gutters to be flushed through the drains during rains and floods. In the early 19th century, the water closets (commonly known as toilets) were installed in houses. These were usually connected to cesspools near where waste was generated and not conveyed for treatment by sewers or sewer systems. Cesspools, located close to living quarters, were seldom emptied and frequently overflowed. Thus, in densely populated areas, the local conditions soon became unbearable and intolerable with apparent threat to public health.

Nathanson and Ambulkar further highlighted that in the middle of 19th Century, the outbreak of cholera was traced in England directly to well-water supplies contaminated with human waste from privy vaults and cesspools. It soon became necessary for all water closets in the larger towns to be connected directly to the storm sewers. This transferred sewage disposed near houses to nearby water bodies, since it was usually believed that the solution to pollution is dilution. When small amounts of sewage were discharged into flowing water bodies, it was believed that a natural stream purification process (called self-purification) would occur to render such discharge safe. However, densely populated communities generated large quantities of sewage, whereby dilution alone could not render such pollution loadings safe. Thus, it became necessary to treat or purify wastewater to a certain degree prior to disposal into water bodies.

In the late 19th and early 20th century, centralized wastewater treatment plants were constructed in the United Kingdom and United States. These centralized wastewater treatment plants had a series of physical, biological and chemical treatment processes allowing treatment for removal of some or most pollutants before it was discharged into the nearby water bodies (Nathanson & Archis Ambulkar, 2012).

In addition, new sewage-collection systems were introduced in the early 1900s for separate storm water and domestic wastewater conveyance to avoid overloads of treatment plants during rainy periods of wet seasons.

In the middle of the 20th century, there were increased public concerns on environmental quality, which resulted in more extensive stringent regulations of wastewater disposal practices. Higher levels of treatment were required to avoid the negative impact of waste disposal on the environment. For example, pre-treatment of industrial wastewater, with the aim of preventing toxic chemicals from interfering with biological processes became a necessity. In fact, wastewater treatment technology advanced to the point where it became possible to remove virtually all pollutants or partial removal from sewage if required. However, such high levels of treatment were not usually justified from a cost point of view.

2.3 Wastewater treatment in developing countries

According to UNICEF/WHO, only 28 % of the population in Sub-Saharan Africa has access to what is considered as a least level of improved sanitation. Globally, the use of basic sanitation services has increased more rapidly than the use of basic drinking water services, at an average of 0.63 percentage points per year between 2000 and 2015 (UNICEF/WHO, 2017). Sixteen of the 24 countries in which at least one person in five has limited sanitation services are found in sub-Saharan Africa. In these countries, the proportion sharing sanitation facilities is largest in urban areas.

Figure 2.1 below gives a visual presentation of sanitation coverage in the countries of the world and highlights the fact that the southern African part of the world suffers from low sanitation coverage except for South Africa and Botswana which have been able to achieve improved basic sanitation services.

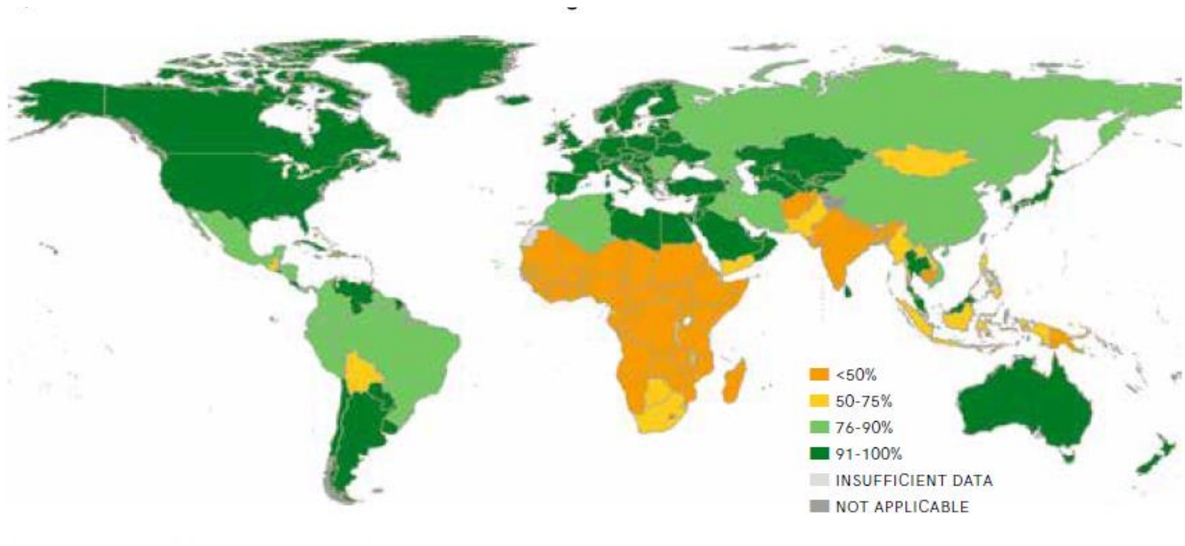


Figure 2. 1: Proportion of national population using the least basic sanitation services. (UNICEF/WHO, 2017)

2.4 Wastewater treatment in Namibia

According to the WHO/UNICEF (2017), Namibia is among the less than 50 % of Sub-Saharan countries with low sanitation coverage facilities while an additional 58 % use shared sanitation facilities (explained as ‘sanitation facilities of an otherwise acceptable type that is shared between two or more households, including public toilets).

Windhoek, the capital city of Namibia, generates an average diurnal flow variation for domestic effluent of about 5 to 45 Mega litres per day (J. Haihambo, personal interview, July, 2018), while the second largest City of Walvisbay produces an average of only about 0.66Mm^3/year (Moyo G, 2012). Approximately 70% of domestic wastewater from Windhoek (Haihambo, 2018), is treated through the processes of primary, secondary and tertiary treatment to achieve effluent of high quality to be reclaimed for portable use and treated to such regulated standards (Moyo G, 2012). The average industrial wastewater produced in Windhoek is nearly 0.3 Mm^3/annum (Lahnsteiner & Lempert, 2007).

Generally, for urban areas in Namibia, wastewater disposal is not a major challenge as all sewer systems are connected to a collective sewer system which then in turn discharges to wastewater treatment plants. Though informal settlements remain a major challenge in Windhoek and generally in other Namibian towns, the city of Windhoek provided communal toilets of which most is connected to the said collector system, but these facilities has become inadequate due to exponential population increase resulting in a practice of open defecation in some areas.

Wastewater disposal is a major challenge in rural areas in Namibia. This is because there are no centralised collective sewer systems in villages due to affordability. Apart from affordability, on-site disposal systems remain a major challenge due to absence of technical know-how of residents in villages. Most of the rural population of Namibia make use of pit latrines.

The government is busy advocating for the reduction or no usage of oxidation ponds, due to the fact that most of the times, these ponds are not maintained and resulting being breeding places for disease causing pathogens adversely affecting public health of communities.

2.5 Characteristics of domestic wastewater

Several types of wastewater are conveyed to a wastewater treatment plant through a centralized collection system, in a given geographic area or community (ISQ, SINTRA, & QUESTOR, 2006). The components found in wastewater flow depends on the type of collection system that is used. The majority flow of wastewater to a conventional wastewater treatment plant consists of domestic wastewater, which refers to 'wastewater discharged from residences and from commercial, institutional and similar facilities' (Metcalf & Eddy 2004). This includes both backwater (mainly faecal matter and urine) which contain pollutant matter and intestinal bacteria (majority harmless but majority more importantly can cause human diseases). It consists of sullage also known as grey water (mainly water from domestic dish washing, food preparation, laundry and bathing).

In addition, the constituents of the domestic wastewater can be divided into physical, chemical and biological parameters which are in many ways interrelated and are all important in the matter of treatment performance, environmental impact, and reuse potential and health aspects.

For the purpose of this study some core aspects of the physical and biological wastewater constituents being the subject of investigation in this study, are as follows:

- Total SOLIDS

These are matters suspended or dissolved in water or wastewater and is related to both specific conductance and turbidity.

The analysis of Total Solids in wastewater treatment is crucial to assess the reuse potential of wastewater and to determine the most suitable type of operations and processes for its treatment (Tchobanoglous, Burton, & Stensel, 2003).

- Total Suspended Solids (TSS)

These are solids suspended in water that can be retained by a filter.

The analysis of Total Suspended Solids (TSS) in wastewater treatment is crucial to assess the reuse potential of wastewater and to determine the most suitable type of operations and processes for its treatment (Tchobanoglous, Burton, & Stensel, 2003).

- Total dissolved solids (TDS)

Total Dissolved Solids (TDS) are solids in water that can pass through a filter (usually with a pore size of 0.45 micrometres). TDS is a measure of the amount of solids dissolved in water.

- Chemical oxygen demand (COD)

This is the amount of oxygen required to oxidise material present in wastewater near or fully completed. COD is a measure of the concentration of the contaminants in the wastewater that can be oxidised by a chemical oxidising agent (Kemira 2003).

- Total Kjeldahl Nitrogen (TKN)

This is the total concentration of pollutant nitrogen and ammonia. The term total nitrogen refers to the sum of the pollutant compounds of nitrogen. When the term Kjeldahl nitrogen is used, it refers to the sum of pollutant nitrogen including pollutant nitrogen from ammonium. Testing for Total Kjeldahl nitrogen (TKN) in wastewater is used as a measure of the nutrients present and the degree of decomposition in wastewater.

- Ammonia

Ammonia is a compound consisting of nitrogen and hydrogen elements with the formula NH_3. Decomposition by bacteria changes the pollutant form of nitrogen to ammonia, and the relative amount of ammonia present in the wastewater is thus an indicator of the age of the wastewater. Nitrogen, as ammonia, is a critical nutrient in biological wastewater treatment. It is utilized by bacteria to produce proteins, including enzymes needed to break down food as well as obtaining energy for metabolic and cell synthesis processes.

- Phosphorus

Just like nitrogen, phosphorus is an essential nutrient for growth of biological life. Raw wastewater normally holds a large fraction of phosphorus, and significantly contributes to eutrophication if disposed untreated into water bodies. Phosphorus is, just like nitrogen, of great interest in relation to reuse purposes, since it constitutes a natural fertilizer resource that can be utilized for irrigation purposes.

2.6 Wastewater treatment processes

The following section reviews the functions of different treatment steps and important design parameters of a conventional wastewater treatment plant and the existing treatment train processes employed at the Gammams Water Care Works.

2.6.1 Primary treatment

Preliminary treatment by screens or grit chambers is usually employed for primary sedimentation. The main objective of this treatment step is to remove a large fraction (50 to 70%) of the total suspended solids in the wastewater. Since suspended solids also contribute to the content of COD in the wastewater, one should expect 25 to 40 % of the total COD to be removed in the process (Metcalf & Eddy 2004). Significant removal of pathogenic organisms is not expected in primary treatment, and up to 1 log unit reduction could be expected (WHO 2006).

2.6.2 Secondary treatment

In general, biological wastewater treatment is based on the principle that microorganisms oxidise dissolved and particulate biodegradable matter into simple end products, which can be removed from the wastewater stream by clarification removal of settled sludge. Such processes can also remove suspended and non-settleable colloidal solids to a certain degree, as they are captured in biological flocs or biofilm. Nutrients such as nitrogen and phosphorus could also be possibly removed either as a part of the solid content or through biological decomposition. As an overview, the main purpose of secondary biological treatment is to remove readily biodegradable pollutants that were not removed from the primary treatment, in combination with further removal of suspended solids and other nutrients (Davis, 2011).

Biological treatment can be achieved either in the presence of oxygen (aerobic processes) or in the absence of oxygen (anaerobic processes). The two main biological treatment processes commonly used in wastewater treatment are; 1) the suspended growth biological treatment, such as an activated sludge process, and 2) the attached growth biological treatment, such as biofilter processes.

- Activated Sludge Process

This process is a biochemical operation performed by a mixed community of microorganisms in an aerobic aquatic environment (Ekama, et al., 1984). The effluent from the primary sedimentation tank enters the aeration tanks where constant aeration takes place. Within the aeration tank, the biomass (aerobic microorganisms, bacterial, protozoa, gases and their cell tissues) is in suspension and aerobic conditions are maintained whereby pollutant matter in sewage is used as food source (substrate) for the biomass for the production of new cells in a process known as synthesis, and pollutant matter gets oxidized.

The microorganisms responsible for treatment are maintained in liquid suspension by the appropriate mixing methods. These processes are operated with positive dissolved oxygen concentration. The activated sludge treatment process configuration employed at the Gammams Water Care works, is known as the modified UCT configuration originally developed by the University of Cape Town. (Haihambo, 2018).

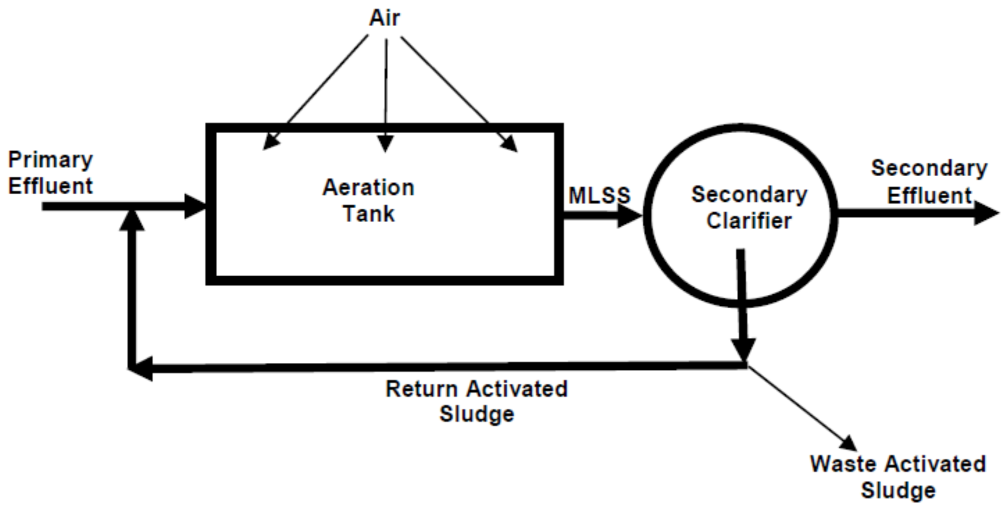


Figure 2. 2: Typical Activated Sludge process (Snyder & Wyant)

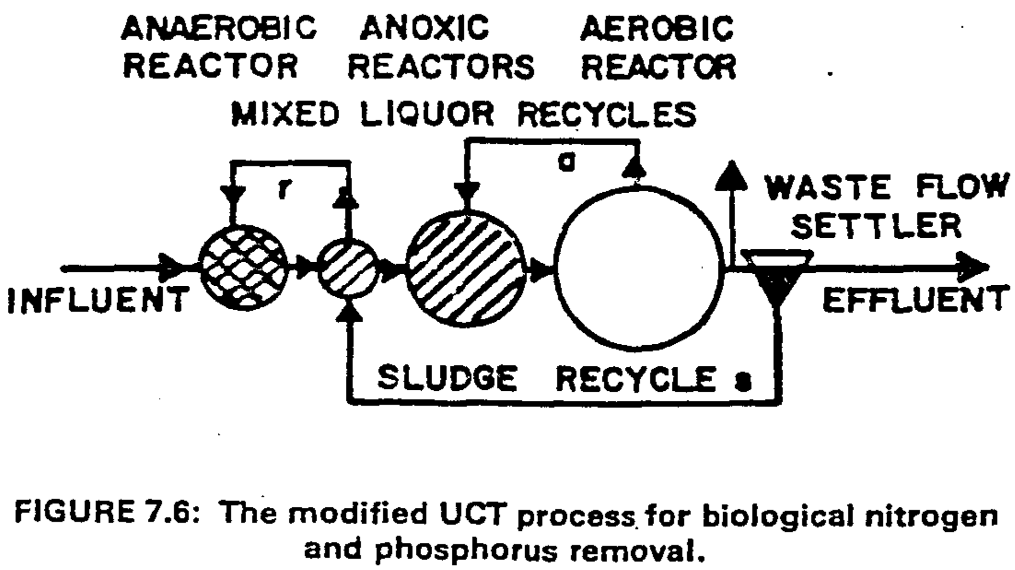


Figure 2. 3: Typical Activated Sludge process: Modified University of Cape Town (UCT) configuration (Tchobanoglous, Burton, & Stensel, 2003)

- Attached growth (or Biofilm) process

In attached growth processes, the microorganisms responsible for the conversion of pollutant material or nutrient are attached to an inert packing material. The pollutant material and nutrients are removed from the wastewater flowing past the attached growth also known as biofilm. Packing materials used in attached growth process include rocks, gravel, slay, sand, redwood and a wide range of plastic and other synthetic materials.

The attached growth process in wastewater treatment can be grouped into three (3) general classes namely: 1) the non-submerged attached growth system; 2) suspended growth processes with fixed-film packing; and 3) the submerged attached growth aerobic processes (Tchobanoglous, Burton, & Stensel, 2003). However, this study focuses on the non-submerged attached growth trickling filter process as employed at the Gammams Water Care Works.

The World Economic Forum (WEF) (as cited in Tchobanoglous et al, 2003) outlined that, trickling filters are often characterised with many disadvantages. These include poorer effluent quality in terms COD and TSS concentration, greater sensitivity to lower temperatures, odour production and uncontrolled solid sloughing events which are related more to the specific process and final humus clarifier designs less than the actual process capabilities. Difficulties to accomplish the biological removal of nitrogen and phosphorus and high turbidity effluent are limiting operational capacity for Trickling filters (Tchobanoglous, Burton, & Stensel, 2003).

2.6.3 Tertiary treatment

Tertiary treatment in wastewater is the final cleaning process that improves the wastewater quality before it is reused, recycled, and/or discharged to the environment. This final treatment removes remaining pollutant compounds, bacteria, viruses, parasites and eutrophication enhancement compounds such as nitrogen and phosphorus.

<u>Biological nitrogen removal</u>

To achieve biological nitrogen removal from wastewater, the processes of nitrification, followed by de-nitrification needs to be employed.

Nitrification is the process in which Nitrosomonas bacteria oxidise ammonia to nitrite and Nitrobacter bacteria in turn oxidise nitrite to nitrate. These microorganisms (bacteria) are autotrophic, which means they derive their carbon source from carbon compounds, such as carbon dioxide and bicarbonate. Environmental conditions of pH, alkalinity, temperature, dissolved oxygen concentration and pollutant loading affect the nitrification process in activated sludge plants as well. The nitrification process is illustrated in figure 2.4 (Metcalf & Eddy 2004):

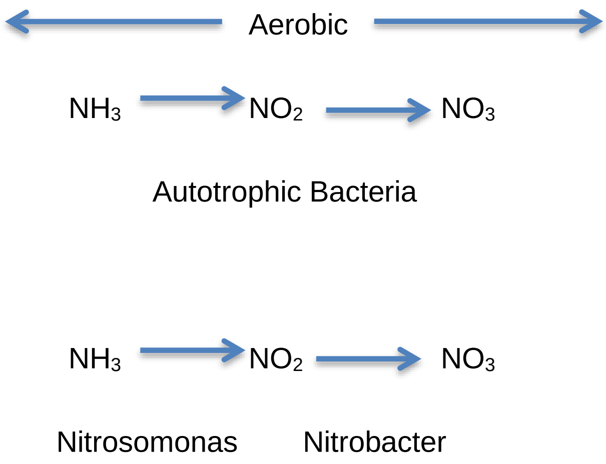


Figure 2. 4: Nitrification process (Snyder & Wyant)

Denitrification is the process in which microorganisms reduce nitrate to nitrite and then nitrite is further reduced to nitrogen gas. Heterotrophic bacteria normally present in activated sludge perform this conversion under anoxic conditions, where no molecular oxygen or dissolved oxygen with sufficient pollutant matter is present. The bacteria derive their oxygen from the oxygen contained in the nitrate. The nitrogen gas produced will be produced in the form of nitric oxide (NO), nitrous oxide (N_2O) or nitrogen gas (N_2) (DEP, 2014).

Furthermore, there are nitrification-denitrification processes. Here the sludge mass normally is subdivided into two reactors zones in series; the first aerated (to nitrify) and the second unaerated (to denitrify). The Modified Ludzack-Ettinger (MLE) configuration is an example of the nitrification-denitrification process. Within this configuration, the first reactor is left unaerated, while the second is aerated. The nitrate generated in the second reactor is transferred to the first via the underflow and an inter-zone (internal) recycle. In the unaerated reactor of the MLE configuration (called the primary anoxic reactor), the denitrification takes place at two simultaneously occurring rates, the first due to the influent readily biodegradable COD, the second due to the particulate COD derived principally from the influent.

Figure 2.5 gives a schematic diagram of the Modified Ludzack-Ettinger nitrification-denitrification activated sludge process.

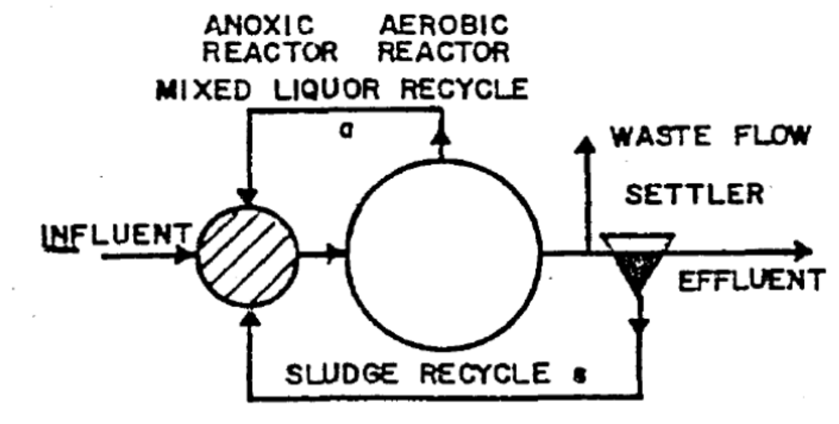


FIGURE 3.2: Modified Ludzack-Ettinger nitrification-denitrification activated sludge process.

Figure 2. 5: Activated sludge process Nitrate removal in Modified Ludzack-Ettinger configuration (Snyder & Wyant)

Biological Phosphorus removal

Along with nitrogen, phosphorus is an essential nutrient for aquatic plant growth. In wastewater, Phosphorus exists in three (3) forms, namely: 1) Orthophosphate; 2) Polyphosphate (P_2O_7); and 3) Pollutant bound phosphorus. Phosphorus discharges to surface waters from wastewater treatment plants (and non-point source run-off) can result in the proliferation of aquatic plants such as algae and others.

Biological phosphorus removal is accomplished by the absorption of phosphates, and pollutant phosphorus in untreated wastewater into bacterial cell tissue and subsequently removing the cell tissue from the wastewater. Absorbing phosphorus into bacterial cell tissue is accomplished by subjecting the bacteria to alternating anaerobic and aerobic environments. In response to the alternating anaerobic and aerobic environments, the bacteria absorb, and store excess phosphorus compared to their normal metabolic requirements under aerobic conditions.

2.7 Impact of Regulations on wastewater Engineering and wastewater Management

During the 19th century, the United States of America's wastewater Engineering and Management Regulations only focused primarily on the removal of colloidal, suspended and floatable materials as well as the treatment of biodegradable materials and pathogenic organisms present in wastewater (Tchobanoglous, Burton, & Stensel, 2003). These regulations were later revised to include aesthetic and environmental concerns, which then became more detailed by including the reduction of Biological Oxygen Demand (BOD), Total suspended Solids (TSS) and removal of pathogenic organisms at a higher level as well as the removal of plant nutrients such as Nitrogen and Phosphorus.

In May 1991, the European Union countries accepted the European Directive 91/271/CEE regarding urban wastewater treatment and environmental water protection (Blöch, 2005). Its main objective was the efficient protection of rivers, lakes and the coastal and marine waters against pollution and eutrophication.

Table 2.7. 1: European wastewater Treatment standards of urban wastewater as laid down in the UWWTD (Blöch, 2005).

a) standard provisions

Parameter	Value (concentration)	Value (% reduction)
Biological Oxygen Demand BOD_5	25 mg/l	70 - 90 %
Chemical Oxygen Demand COD	125 mg/l	75 %

(24 hour average; either concentration or percentage of reduction shall apply)

The Directive provides for mandatory minimum design rules for sewerage systems as well as treatment plants (minimum design requirement = highest maximum weekly average load throughout the year).

b) additional provisions for 'sensitive areas'

Parameter	Value (concentration)	Value (% reduction)
Total nitrogen Plants of 10 000 - 100 000 p.e. Plants >100 000 p.e.	 15 mg/l 10 mg/l	 70 - 80 %
Total phosphorus Plants of 10 000 - 100 000 p.e. Plants >100 000 p.e.	 2 mg/l 1 mg/l	 80 %

(annual averages, either concentration or percentage of reduction shall apply)

Source: European Union legislation on wastewater treatment and nutrients removal, p.3.

Blöch further outlined that, this directive has improved the quality of big European rivers. This was achieved, in terms of BOD levels by a reduction of 20-30%, phosphorus concentrations by 30-40% and Ammonium-Nitrogen (NH_4-N) levels by approximately 40%. Though success has been achieved, Europe has experienced delays in some cases with prevailing discharges of untreated or insufficiently treated wastewater and in such cases legal enforcement measures were applied.

The Republic of Namibia adopted the existing South African wastewater Management Guidelines (461/85) of 1 April 1988, until such time as a proper study has been conducted and new standards have been formulated. These are outlined in Table 2.7.2 below.

Table 2.7. 2: General Standard as laid out in the Government Gazette Regulation R553 of 5 April 1962, in Section 21(1) and 21(2) of the Water Act (Act No 54 of 1956)

Substance/parameter	Special Standard
Faecal Coliform (per 100 mL)	Nil
Chemical Oxygen Demand (mg/L)	75
pH	5.5 - 7.5
Suspended Solids (mg/L)	25
Phosphates (mg/L as P)	2
Nitrogen (mg/L as N)	10

Though Namibia stipulates the above-mentioned standards nationally, discharge of wastewater into water bodies are not strictly monitored, thus no statistical records are available indicating whether the standards are adhered to or not. Furthermore, adequate waste management practices which include the reduction and prevention of waste, as well as wastewater treatment, disposal and recycling remain a challenge (Rupplel & Schlichting, 2011).

In conclusion, effective laws and regulations including legal enforcement measures determine the success of any wastewater management system.

2.8 Controlling the Influent of wastewater to the Treatment plant

Wastewater treatment plants are designed to handle a design pollution load and a design flow. However, these conditions seldom appear in reality especially when influent to the treatment plant are not controlled.

Generally, influent to a wastewater treatment plant exhibits a wide daily diurnal variation in both flow rate and pollution load, and such deviations of these parameters cause plant performance problems (Dold, 1982). Due to these plant performance problems, diurnal variations led to wide interest in the development and the application of control procedures for treatment plant operation, namely in-plant or equalization control (Dold, 1982).

2.9 The In-plant Control

This is a wastewater treatment plant control procedure whereby efforts are made to attenuate variations in influent loading rate, where unit processes are controlled separately. This is to adequately accommodate the cyclic inputs within the treatment plant. This in-plant procedure has high reliance on the operator's participation (Dold, 1982).

An example of in-plant control involves aeration control which was probably developed to save on aeration costs and to improve plant performance using regular Winkler titration results.

The Step-feed procedure is an in-plant control which evolved to accommodate peak loads along the length of a semi-plug flow reactor in such a manner that the aeration capacity could be effectively utilized over the plant. The Step-feed activated sludge process provides the treatment plant with a degree of flexibility. The oxygen requirements are more uniform along the aeration tank, while temporary problems such as sludge bulking and hydraulic overload can be adequately handled (Oswaldo, 1987). The author modelled a comprehensive simulation and design of the Step-feed activated sludge process. It considered an aeration tank divided into a number of stages and secondary settling. The model consists of several interacting "sub-models" that simulate the biodegradation of the pollutant wastes, the thickening of the sludge and clarification of the effluent, with a step-feed plant operating under steady-state conditions.

In practice, many in-plant control procedures have been applied with reasonable success in the past, but with heavy reliance on operator participation.

Currently in the market, in-plant control are based on batch sampling (Al-Dasoqi, Alkhaddar, & Al-Shamma, 2011), where the Sequencing Batch Reactor (SBR) cycles are operated on a fixed time schedule with an assumption of steady-state flow. This assumption is not accurate because the wastewater treatment plant is subjected to flow rate and pollution load variations.

2.10 Equalization Process

Influents to the wastewater treatment plant exhibits a wide daily diurnal pattern variation in terms of hydraulic rate and pollutant concentration and subsequently load rate, which in terms of domestic wastewater is influenced by the water demand return flow (Bolmstedt, 2004). These wide variations cause plant operation problems which results in inefficiency of the plant treatment capacity. The wastewater treatment plant drawbacks are in the areas such as aeration control due to load rate fluctuation, settling tank overloading due to flow rate fluctuation and over/under aeration which affects settling properties in terms of secondary sedimentation (clarification) tanks.

Ongerth (1979), in a comprehensive evaluation of equalization in wastewater treatment, defines equalization as "any facility procedures for minimizing variations in the flow through treatment plants".

Uniform or near uniform pollutant and hydraulic loading has the potential in reducing the operational related cost of the plant by means of reducing the aeration capacity as there will be a more uniform least fluctuating pollutant loading in the influent to be treated. Equalization has a high influence on improving biological treatment efficiency, especially in terms of biological removal of Phosphorus and Nitrogen. Primary settling tanks overall removal efficiency will be improved as well, since only the mean flow rate will be released to PST’s. Furthermore, equalization allows close to uniform hydraulic and pollutant loading conditions which gives a more stable environment for biological treatment organisms for improved stabilization processes.

The introduction of an equalization process and the attenuation of hydraulic and pollutant load variations allows for subsequent downstream processes to take place under the desired uniform conditions mentioned.

Specific benefits accruing from flow equalization in activated sludge treatment plant operation have been identified by a number of authors (La Grega & Keenan, 1974; Wallace, 1968; Spiegel, 1974; Ongerth, 1979) and also outlined by Dold (Dold, 1982):

- Improved performance of primary sedimentation basins and secondary clarifiers.
- Increased capacity of sedimentation and clarification units in existing plants and specification of smaller units for new plants.
- Improved biological process response through a partial reduction in food/micro-organism loading peaks.
- Simplified control of in-plant flow rate dependent operations such as chemical dosing and recycle pumping.
- Lower energy tariff charges by reducing peak power demands for pumping and aeration.
- Lower capital costs by not having to supply the oxygenation capacity to match the peak load requirement.
- Reduction in shock loading effects by discharging recycled concentrated waste streams such as digester supernatant and sludge de-watering filtrate to the equalization basin.
- Steady wastewater flow rates have an effect on the return activated-sludge flow rate and Mixed Liquor Suspended Solids (MLSS) concentrations (Tchobanoglous, Burton, & Stensel, 2003).

2.10.1 Types of Equalization process

2.10.1.1 Constant Volume Mode process

This is an equalization process whereby fixed hold-up volume is provided for the influent flow, and concentration variations are attenuated. The rate of outflow from the equalization tank is always equal to the inflow since the tank is continuously full and flow variations are therefore not reduced. The constant volume mode of operation thus produces some damping of pollutant mass loading variations but does not alleviate the problems of uneven flow rate (Dold, 1982).

2.10.1.2 Variable Volume Mode

In this process mode of operation, the outflow rate from the equalization basin is regulated, allowing the tank attenuation to be varied. Both flow and pollutant loading variations are reduced in the equalization basin.

The two types of physical configuration employed for a variable volume equalization process are:

- In-line equalization

All the flow passes through the equalization basin to achieve more uniform reduced variations in pollutant load and flow rate.

- Off- line equalization

In this case, only over-flow above a predetermined influent flow rate is diverted into the equalization basin. This type of equalization process helps in reducing the pumping requirements, however in the case of the Gammams Water Care Works, it does not serve any benefit considering the fact that sequential unit process treatment is by gravity. In this method of equalization, variations in loading rate can be reduced considerably compared to the plant operating without equalization.

Off-line equalization is commonly used for the capture of the “first flush” from combined collections systems (Manderso, 2018), however this does not apply to the separate system employed in Windhoek and Namibia at large.

2.10.2 Design considerations of flow equalization process

The principal factors that must be considered in the design of equalization process are the location and configuration, volume, basin geometry, mixing and air requirements as well as appurtenances and pumping facilities if required. The equalization unit process is normally located at the end of the preliminary treatment process (after screening and grit removal) and before primary treatment (primary settling tanks).

Equalization design is based on average dry weather flow conditions alone. Wet weather impacts of extraneous flow due to surface water or sub-surface infiltration are not catered for and such being diverted upstream of the equalization basin into a temporary detention facility and returned back into the plant for treatment under dry weather conditions.

There are two basic equalization process configuration options, viz: 1) variable volume; and 2) constant volume. In a variable volume configuration, the equalization process is designed to provide a constant effluent flow to the downstream treatment unit processes. However, in the case of a constant volume process, the rate of outflow to other treatment unit processes changes with changes in the influent.

For the determination of the equalization process attenuation volume required, mass curve analysis using the Ripple graphical method can be employed (Tchobanoglous, Burton, & Stensel, 2003). The equalization process attenuation basin volume required is determined from a plot of cumulative inflow volume versus time of day as shown in figure 2.6.

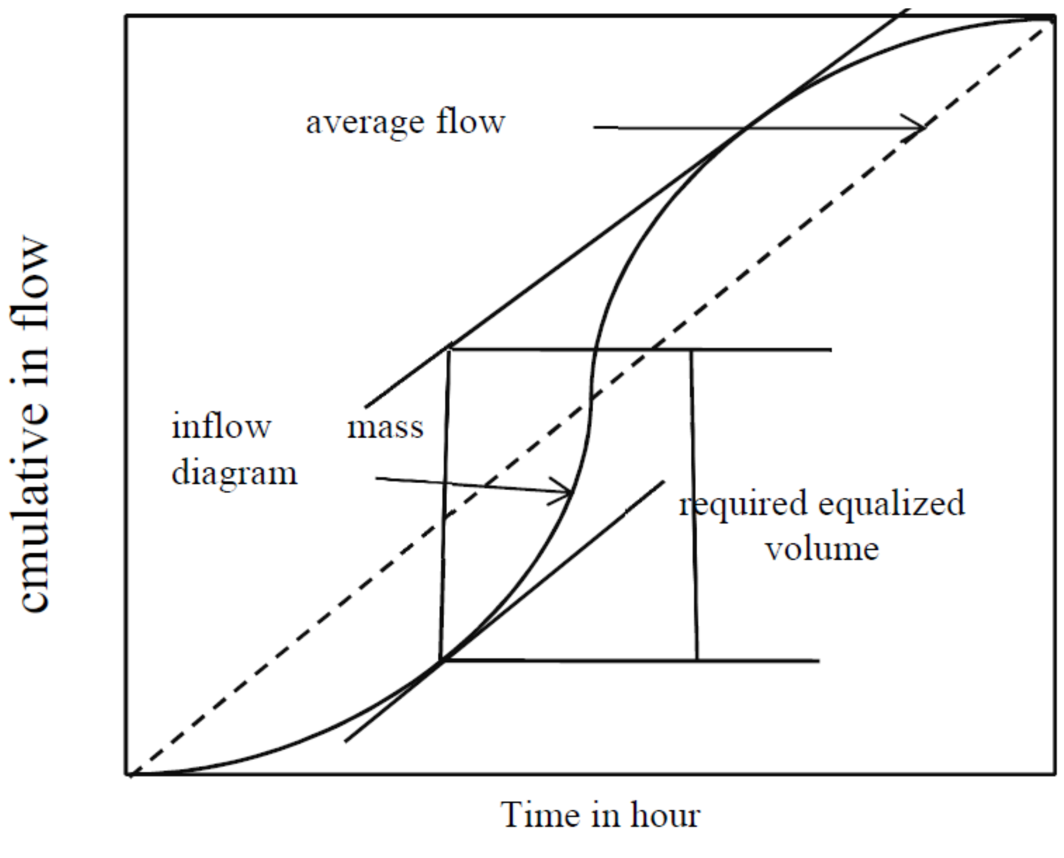


Figure 2. 6: Inflow mass diagram (Ripple diagram method) (Tchobanoglous, Burton, & Stensel, 2003)

The required equalization volume is derived by superimposing tangents (equal in slope to the planned basin discharge) at point/s of maximum curvature of the latter plot and then measuring the cumulative flow between such tangent/s. In cases of a single tangent being possible, the cumulative flow between such tangent and the cumulative basin release rate gives the required basin volume.

The actual final design volume required is determined by applying a factor of increase of 20% of the process volume calculated as outlined. The increase factor allows for unforeseen changes in hydraulic flows (Tchobanoglous, Burton, & Stensel, 2003). However, the graphical methodology for sizing the equalization basin volume can also be determined by doing a time-step numerical analysis of accumulated inflow into and outflow discharged from the basin over the design day. This may serve as a verification of the volumetric determination done according to the graphical method described (Tchobanoglous, Burton, & Stensel, 2003).

2.10.3 Numerical basin volumetric determination

The time-step numerical analysis approach is also used to assess the impact of equalization attenuation provided by the basin on both hydraulic and pollutant constituent load fluctuation during the design day, using an hourly pollutant mass balance, based on changes in the basin volume for the time-step intervals selected for the design day. Various authors (Tchobanoglous, Burton, & Stensel, 2003) and (Manderso, 2018) outlined a clear and detailed analysis of effect of an equalization on pollutant attenuation.

2.10.4 Computer simulation program

The STOAT (Sewage Treatment Operation and Analysis over Time) model is a dynamic model for simulation of wastewater treatment processes. It contains all functionality necessary to design plants for biological nutrient removal (BNR) and allows for inclusion of both conventionally activated sludge processes and sequencing batch reactors. It has a particular application of the activated sludge process that allows provision of the most cost effective route into dynamic modelling analyses (Dudley & Poinel, WRc Stoat: Installation and User Guide., 2013). In this regard, it can be used to design new activated sludge treatment plants, upgrade existing facilities, optimize the performance of assets and as a tool for training plant operators.

2.11 Treatment train and unit processes employed at the Gammams Water Care Works, Windhoek, Namibia

The Gammams Water Care Works employs the following treatment processes:

- Preliminary treatment process:

  Screening and grit removal.
- Primary treatment process:

  Primary sedimentation treatment (PST), flow measurement and a grit/screenings de-watering process.
- Secondary treatment processes:

  Trickling or biological filtration (TF) with associated humus tanks and Activated Sludge Treatment (AST) with associated Secondary settling (or clarifier) tanks.
- Tertiary treatment:

  Maturation ponds, gravity thickening of waste activated sludge and sludge drying beds.
- Sludge treatment:

  Anaerobic digestion of primary and waste activated sludge thickening and sludge drying beds.

In figure 2.7 below the treatment plant process flow diagram outlines the sequence of unit processes employed at the Gammams Water Care Works.

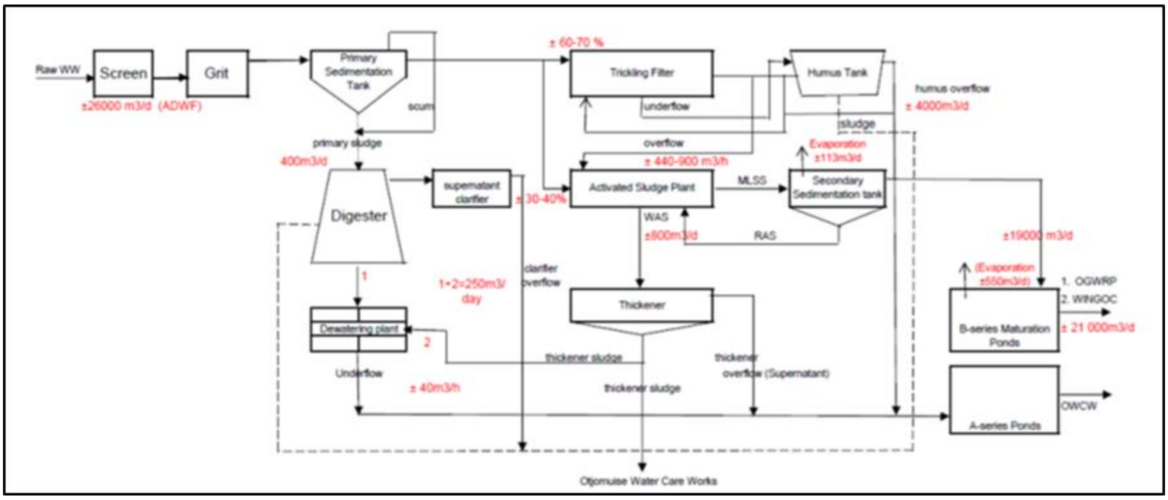


Figure 2. 7: Process flow diagram for the Gammams Water Care Works, Windhoek, Namibia (Haihambo, 2018)

2.11.1. Preliminary Treatment

The preliminary treatment process consists of screening, grit removal and a screenings compactor.

- Screening

Raw wastewater goes through a set of front grating steel bars raked screens of 10mm bar spacing for removal of debris and other coarse materials and large particles such as rags, plastics and other materials. This large size screens are followed by finer screens through parallel of step and back rake screens of 6mm bar spacing.

- Grit Chamber

The Grit chamber retains materials such as sand, gravel, cinder or heavy solids that are usually heavier than the bio-degradable pollutant matter. Within this process, grit is removed from the influent wastewater. The Gammams Water Care Works uses a vortex-type grit remover and the chamber is of a cylindrical shape in which wastewater enters tangentially, creating a vortex flow pattern. Grit then settles by gravity at the bottom of the tank which is then pumped to a compactor for excess water removal before its disposed-off (Haihambo, 2018).

- Compactor

This is an automated system used by Gammams to dewater grit removed from the grit remover and the bar screens before dumping it in a large waste collector bin (6m^3 capacity skip container). This collected waste is then disposed-off at the waste landfill site.

2.11.2. Primary Treatment

Primary sedimentation employed at the Gammams Water Care Works consists of 8 tanks. Seven are of the older Dortmund-type, while the other is a fully automated tank. Table 2.11.2 (see Appendix 2) illustrates the sizes and flow splits of the primary sedimentation tanks (PSTs).

The effluent from the preliminary treatment is conveyed to the primary sedimentation tanks (PSTs) for the settling of suspended pollutant matter. Approximately 60%-70% of settled sewage is conveyed to the biological/trickling filter (TF) plant and 40%-30% to the Activated Sludge Process for biological treatment. The primary settled sludge withdrawn from the PSTs are conveyed to the anaerobic digesters for further treatment.

2.11.3. Secondary Treatment

There are two secondary technologies used in treatment of effluents flowing from the PSTs consisting of Trickling filters (Biofiltration) and Activated sludge treatment.

- Trickling filters (Biofiltration)

Five trickling filters are employed, two being of smaller diameter (31.25m) and the remaining three of larger diameter (35.05m). These trickling filters are conventional rock media filters with a depth of approximately 3.6m. Appendix 2, Table 2.11.3a shows the sizes of the Trickling filters and the respective influent flow splits employed at the Gammams Water Care Works.

The Biological filter receives about 70% of the PST effluent for treatment. At Gammas Water care works, the biological filters are employed as a pre-treatment for the split load received from the PST's before being discharged to the AST process. Operationally approximately 72% (12 000 m^3/day) of biofilter effluent is pumped to the AST process for further biological treatment. The remaining biofilter effluent once settled in the humus tanks is conveyed to the Otjomuise wastewater Treatment Plant. Currently the humus tanks at the Gammams Water Care Works are not de-slugged and high sludge loads are disposed to the Otjiomuise plant.

- Activated Sludge Treatment Process (AST)

The Activated Sludge Process receives approximately 30% of settled sewage from the PSTs. The process employed is according to the modified University of Cape Town (mod UCT) configuration incorporating anaerobic, anoxic and aerobic zones in the bioreactor for biological pollutant, nitrogen and phosphorus nutrient removal. As mentioned earlier in this study, the influent to the AST consists of direct PSTs effluent plus pre-treated biofilter effluent as well as activated return sludge for the secondary settlers (clarifiers sludge or s-recycle). The waste activated sludge (WAS) is withdrawn from the reactor, conveyed to the WAS gravity thickener and in turn delivered to the anaerobic digesters for stabilization. Finally, the AST effluent gravitates to the so-called "B-series" maturation ponds for final effluent polishing.

- Secondary sedimentation tank (Clarifiers)

This comprises of four (4) secondary settling tanks of which three (3) are of the dimension of 4m deep side walls and 5.85m centre depths. The fourth is of 4.5m deep side wall and 6.47m centre depth.

2.11.4. Tertiary treatment

The effluent from the activated sludge process is discharged into the so-called B-series maturation pond system. This system consists of eight (8) sequential artificial ponds with a total capacity of 58 567m^3. At the current flow of treatment effluent takes about three days to pass through the pond series and utilized for the following reclamation purposes or finally disposed into the Gammas River:

- Water resource for the WINGOC Reclamation Plant for potable water reuse.
- Water resource for the Goreangab Reclamation Plant for irrigation water reuse.

2.11.5. Sludge Treatment

Sludge treatment consists of anaerobic digestion of primary sludge.

➢ Anaerobic digestion of primary sludge

Currently, the anaerobic digestion consists of five (5) digesters of which four (4) serve as primary digesters while the 5th as a secondary digester or balancing tank for digested sludge. The digesters are semi-egg shaped with a deep conical bottom, with working volume of approximately 1650m^3 each. The digester's supernatant liquor (SNL) is withdrawn via a series of manually operated draw-off valves from the digester. The SNL is pumped to a small circular "supernatant clarifier" tank where solids are intended to settle out and then returned to the digesters via the primary sludge. Clarifier supernatant is pumped to the "A-Series by-pass channel", from where it is conveyed to the Otjomuise wastewater Treatment Plant via a gravity pipeline for further treatment.

➢ Sludge collection

The final process is the removal of water from the collected sludge. A belt filter press is used to compress the sludge to remove excess moisture before the sludge is dumped in drying beds.

➢ Energy recovery process

The plant produces approximately 270 m^3/day of sludge, of which about 100-200kg is treated and wasted to digesters for anaerobically bio-digestion and utilised for power generation to supplement the power needs at the plant (J. Haihambo, personal interview, July, 2018). According to Haihambo, this process produces about 250 kilowatts per hour of energy.

CHAPTER 3

3. Research and Methodology

Part 1: General approach and procedure of analysis

A desk study considering hydraulic data available for the Gammams Water Care Works as well as sampling and chemical analysis of effluent from the various unit treatment processes were done to assist in developing a understanding of the processes employed and allow considering potential impact of introducing an equalization process on the existing plant process performance.

3.1 Desk study: Treatment plant influent flow/hydraulic load data and influent/effluent pollutant load determination of unit processes

As alluded to, both a hydraulic influent load assessment and chemical analysis of the various unit process of the plant treatment train were carried out. These assessments are described next.

- Hydraulic data

Hydraulic data for the present study was obtained from various sources. First, a desk study was conducted on influent flow data for the 2016 - 2018 period. Second, while the Gammams Water Care Works operational office provided daily historical data, the researcher obtained hydraulic load data from the SCADA data system of the City of Windhoek. Thirdly, rainfall data from the Namibian Meteorological Services including the Namibia Weather Forecast website was obtained. Both Namibian Meteorological Services and the Namibia Weather Forecast website were explored to obtain data on period of rainfall in Namibia.

The identified day of the week with the most representative hydraulic loading, which required consideration of the largest flow variation in relation to the average flow rate under dry weather conditions, was used for the equalization process attenuation capacity design.

Two main factors used to identify the mentioned most representative hydraulic load design day, were identifying: 1) school and university holidays, breaks and public holidays (which result in large population migration away from Windhoek), and (2) business closures, both impacting on extent of hydraulic load generated. In this study it is assumed that holidays and business closures are non-representative design days since during such periods there is a reduced amount of wastewater flow generated daily. Therefore, in the present study April for the period 2016 to 2018 was identified and further analysed as a design month. Specifically, the study established that the maximum influent flow and variation to the average occurred on Thursdays and Fridays of the second week of April between 2016 and 2018.

Since this study is considering current conditions, the current influent flow for Friday, 12 April 2019 was selected as hydraulic design load for sizing the equalization process attenuation capacity.

To assist with identifying the hydraulic design day mentioned above, historical electronic Microsoft Excel format influent data were converted into graphical representations.

Figures 3.1.1 to 3.1.3 below present the month of April historical flow records for 2018, 2017 and 2016.

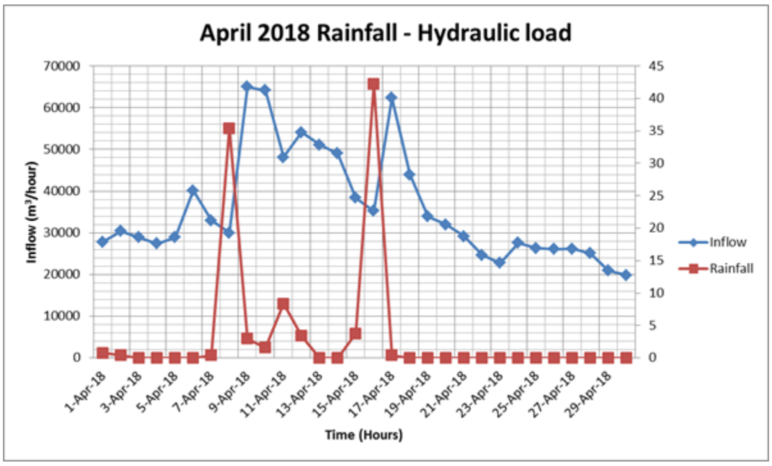


Figure 3.1.1: The month of April historical flow data (2018)

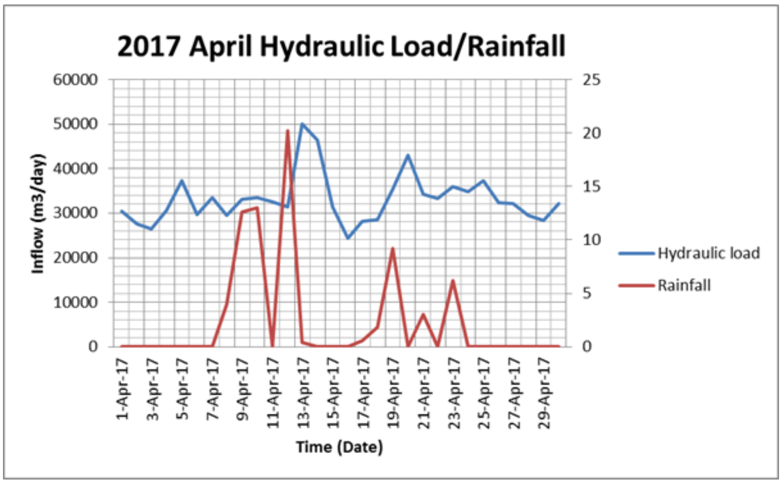


Figure 3.1.2: The month of April historical flow data (2017)

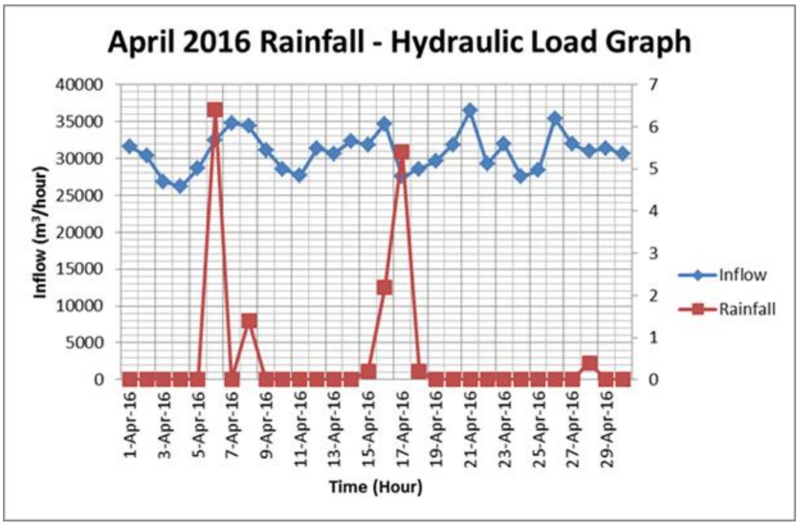


Figure 3.1.3: The month of April historical flow data (2016)

- Unit process influent/effluent pollutant Information

Influent pollutants load data were obtained through wastewater sampling and laboratory testing procedures. Raw wastewater samples were collected using the manual sampling method from the following four (4) locations along the process treatment train:

1) The Inlet works (Preliminary treatment outflow);

2) Primary settler supernatant outflow;

3) The activated sludge clarifier effluent; and

4) Trickling filter humus tank effluent.

A grab sample system was used to collect the wastewater samples at 4-hour intervals during the design day and was subsequently stored in 1 litre glass containers until tested. Samples were submitted to the City of Windhoek scientific laboratory for analysis. Test analyses determined the various pollutant levels at the various unit processes mentioned. The following pollutant parameters were considered in the study:

- COD,
- Total Suspended Solids (TSS),
- Total Dissolved Solids (TDS),
- Total Solids (TS),
- Total Kjeldahl Nitrogen (TKN),
- Ammonia,
- Orthophosphate

The City of Windhoek scientific laboratory personnel conducted all pollutant sample test analyses. The test mentioned aided the comparative pollutant removal trend of the unit processes employed at the plant as well as providing plant influent parameters for the STOAT treatment simulation model employed.

3.1.1 Hydraulic plant influent and unit process pollutant data analysis

As alluded to before, the dry weather influent flow was considered for the days of the dry months (no rain) and excluding school/university holidays, public holidays and long weekends resulting in the identified hydraulic design day being Friday, 12 April 2019.

3.2 Equalization basin volumetric determination

Although the design of an equalization basin requires the evaluation and selection of a number of features mentioned before, the primary focus of this study was on the volumetric process attenuation requirement thereof for adequate influent attenuation to achieve reduction in wastewater pollutant load diurnal variation to subsequent downstream unit processes.

An analysis was done to determine minimum basin attenuation capacity required and accompanying pollutant load variations for alternative basin flow release rates. Basin release flow scenarios considered were: 1) average daily flow, together with, 2) incremental flow release rates. The study used both a graphical analysis and numerical time-step methodology for the determination of the equalization process attenuation volume (being the equalization basin capacity).

The graphical analysis procedure consists of use of an inflow mass diagram where cumulative inflow volume is plotted against time of the day and superimposing various accumulative flow release rates curves thereon (Manderso, 2018).

3.2.1 Graphical analysis

This method is also known as the Ripple graphical method. The procedure consists of a graphical plot of the cumulative inflow and average cumulative flow curves (as well as other alternative increased basin release flow rates) against time of day. Next, two (2) tangents of slope equal to the release flow rate, are superimposed at points of maximum curvature on the cumulative flow curve. The basin volume (in cubic meters) is then derived as the accumulative difference in volume between the mentioned tangent points. For actual practical design purposes, a 20% increase factor is applied to the theoretical attenuation determined to make provision for unforeseen eventualities.

3.2.2 Numerical analysis

A numerical histogram representation was compiled using the average hourly inflow data obtained for the plant. The average inflow with sequential increased discharge rates were then used as release from the equalization basin into the downstream unit processes. As described before, the stepwise analysis done allowed determining the maximum cumulative volume which equals the attenuation volume required for the equalization process for each release flow rate considered.

3.2.3 Research Instruments

The present study used Microsoft Excel for both conducting the numerical time-step analyses as well as the Ripple graphical representation of results for determining the equalization attenuation process basin volume required.

3.3 Analysis and comparison between Equalised and Unequalised overall pollutant load treatment efficiencies

3.3.1 Equalization process impact on pollutant load variation

Both the numerical time-step method and the STOAT simulation software were employed for these analyses.

- Time-step numerical analysis of influent pollution loads vs equalization process flow release rate

The effect of flow equalization on pollutant diurnal loading to subsequent downstream processes were determined. Unequalised and Equalised (without and with equalization) pollutant mass loading results are presented in Part 2 of chapter 3.

- STOAT simulation model software

The Gammams treatment train was configured in the STOAT simulation model software (Dudley & Poinel, STOAT 5.0, 2013) for the purpose of comparative analysis of Unequalised and Equalised scenarios. The STOAT models are semi-calibrated as mentioned before. Detailed information of the processes employed are given in Appendix 2, tables 2.1 to 2.3. Simulation results of the analyses are presented in Part 2 of chapter 3.

The STOAT model unit process inputs for simulation purposes can be described as follows:

As the Gammams Water Care Works receives mostly municipal wastewater, the STOAT modelling option: Municipal wastewater, was selected as the influent input.

The two selected categories of influent dynamic file parameters for the study consisted of flow rate together with seven (7) influent pollutants (namely, COD, TSS, TKN, TSS, TDS, Ammonia and Phosphorus).

Since the STOAT software does not allow input of Total Kjendal Nitrogen (TKN), the model was populated with the following components as recommended by STOAT:

- Pollutant as Nitrogen (N) = TKN – ammonia
- Soluble pollutant N = 40% x pollutant N
- Particulate pollutant N = 60% x pollutant N

The solids compositions were modelled on the assumption that TSS is composed of:

- Volatile solids = 75% x TSS
- Non-volatile solids = 25% x TSS

The COD composition was modelled on the assumption of the following composition:

- Soluble degradable COD = 0.4 x 0.8 x COD
- Soluble non-degradable COD = 0.4 x 0.2 x COD
- Particulate degradable COD = 0.6 x 0.8 x COD
- Particulate non-degradable COD = 0.6 x 0.2 x COD

The Primary settlers were configured according to their physical sizes and as currently operated at the plant. The activated sludge process configuration was done based on its current total reactor volume. The operations of the Activated Sludge process were aligned to that of the modified University of Cape Town (Modified UCT) configuration. The STOAT software assumes that the activated sludge process is configured to control the sludge age by wastage method from the reactor. The model also incorporates the secondary settlers (clarifiers) employed at the plant.

Moreover, for the configuration of the bio filter process the model included the humus tanks together with de-sludging. However, since the humus tanks are not operationally desludged regularly at the plant, the simulated model assumption output is not representative of actual operational conditions. This aspect is referred to in the study result discussion and conclusions.

For the STOAT model simulation runs, an integration algorithm of Explicit stabilized Runge-Kutta (RK) methods and ROCK2 was used. These numerical methods were chosen because they belong to the one-step stabilized methods class, with extended stability domains and do not suffer from the step-size reduction faced by standard explicit methods.

Part 2: Methodology analyses

Section 2.1 Equalization process basin capacity determination

3.4 Existing Unequalised plant: Unit processes: Sample points pollutant concentrations for design day

In Appendix 1 Figure 3.4.1 the locations are given where pollutants were sampled at unit processes along the treatment train. Corresponding laboratory test results of the pollutants sampled are given in Appendix 3 Tables 3.4.1 to 3.4.4.

Figures 4.2.1 to 4.2.4 graphically illustrate the results of a poly regression curve best fit. The regression curves were adjusted in certain instances where negative or incompatible treatment trend values were obtained. Verification was possible based on corresponding historical data for COD and Ammonia only, which indicated a trend variation of less than 10% compared to the regression results obtained.

The results for each sampling location and their graphical illustration were grouped into two separate illustrations, namely: 1) Solids and COD; and 2) organic pollutants. This approach was adopted due to differences in the magnitude of the different pollutant concentrations and to assist to make their graphical illustration clearer.

3.4.1 Preliminary Treatment Outflow

Figures 3.4.1a and b, firstly illustrate the effluent solids together with COD concentration and secondly that of the organic pollutants (TKN, Ammonia and orthophosphate) variations during the design day based on sample tests done with best fit regression curve allowance.

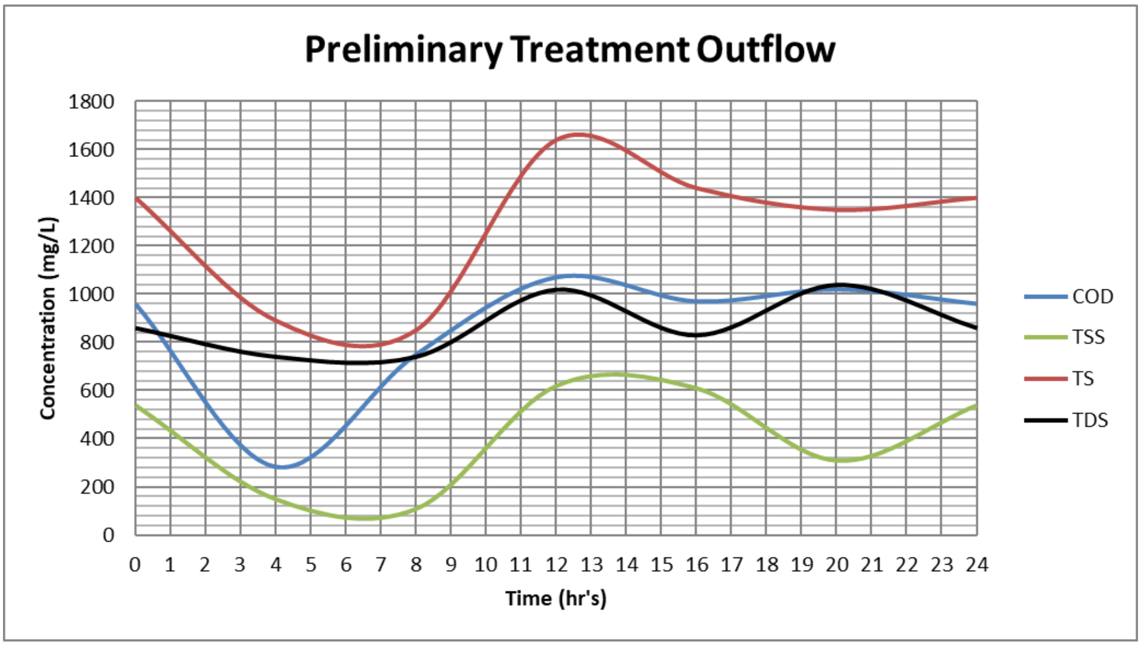


Figure 3.4.1a: Preliminary treatment effluent: Pollutant (Solids and COD) concentrations

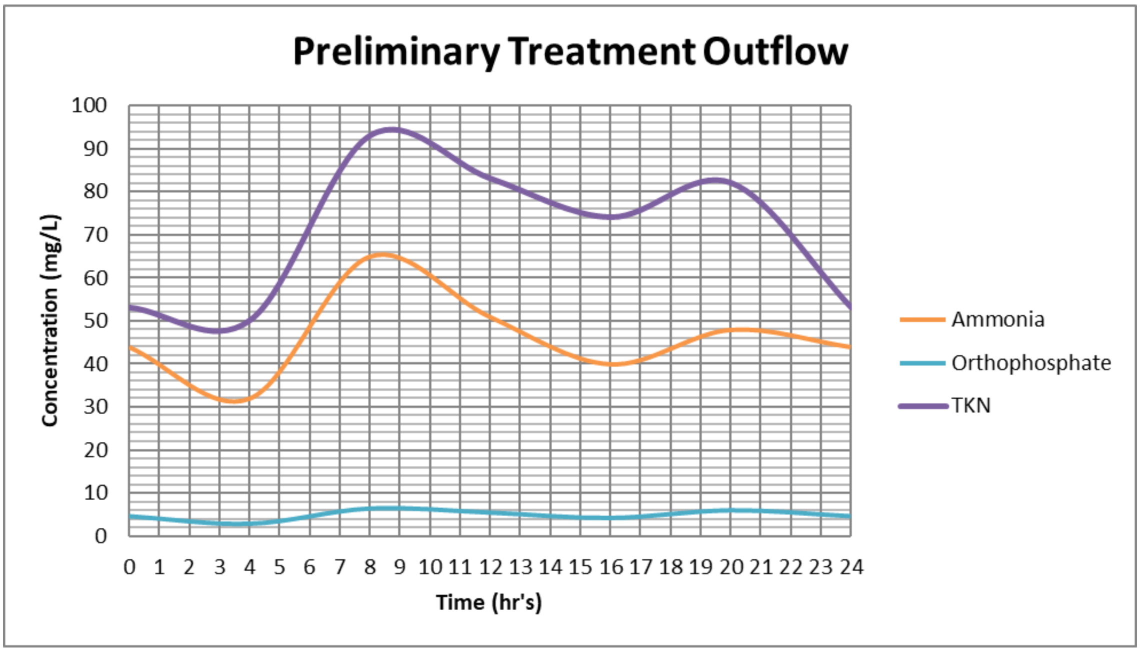


Figure 3.4.1b: Preliminary treatment effluent: Pollutant (TKN, ammonia, orthophosphate) concentrations

The preliminary treatment (or inlet works) effluent pollutant concentrations illustrated in figures 3.4.1a and b, clearly show the extent of concentration variations being minimal (if any) as biochemical treatment does not take place during preliminary treatment as such.

3.4.2 Primary Treatment

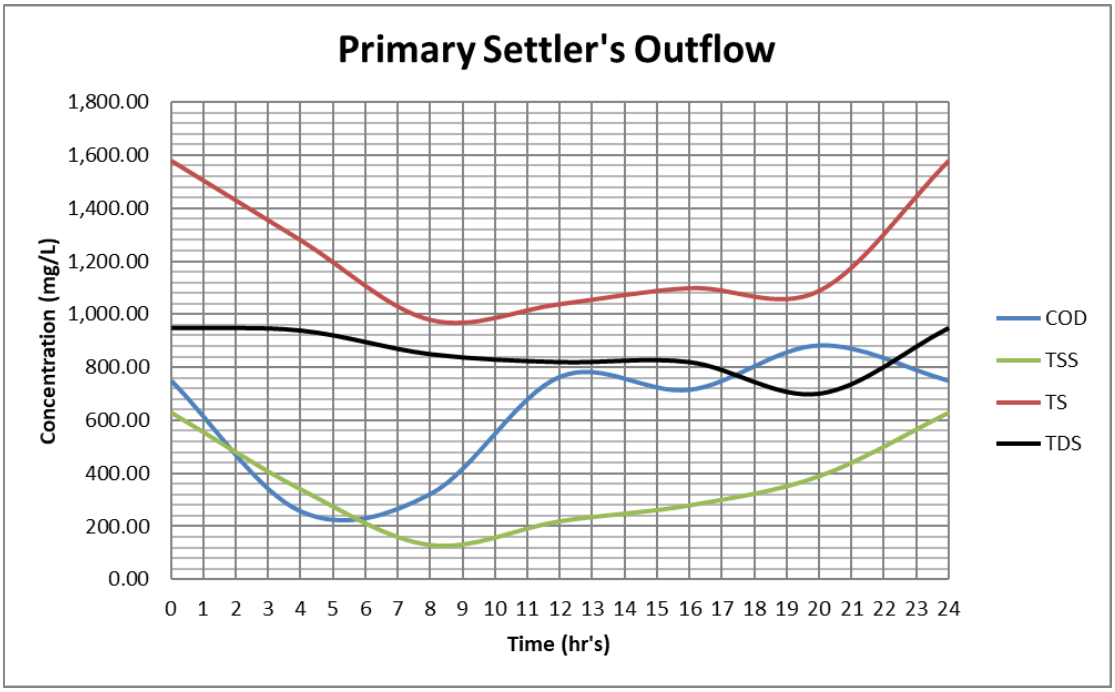


Figure 3.4.2a: Primary settler effluent: Pollutant (Solids and COD) concentrations

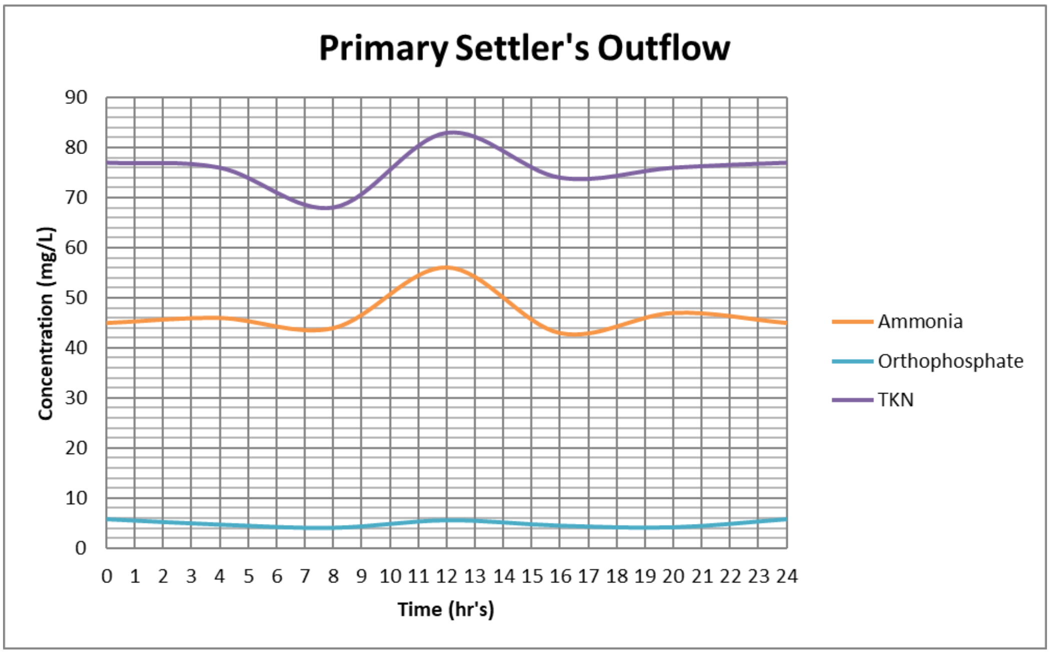


Figure 3.4. 2b: Primary settler effluent: Pollutant (TKN, ammonia, orthophosphate) concentrations

As shown in Figures 3.4.2a and b, the average PSTs effluent concentration of COD, Ammonia, Orthophosphate and TKN decrease in relation to that of the preliminary treatment outflow (Figures 3.4.1a and b). This is due to settler removal of suspended solids in the PSTs resulting in corresponding reduction in the pollutant loading.

3.4.3 Secondary Treatment (Biofilter humus tank) outflow

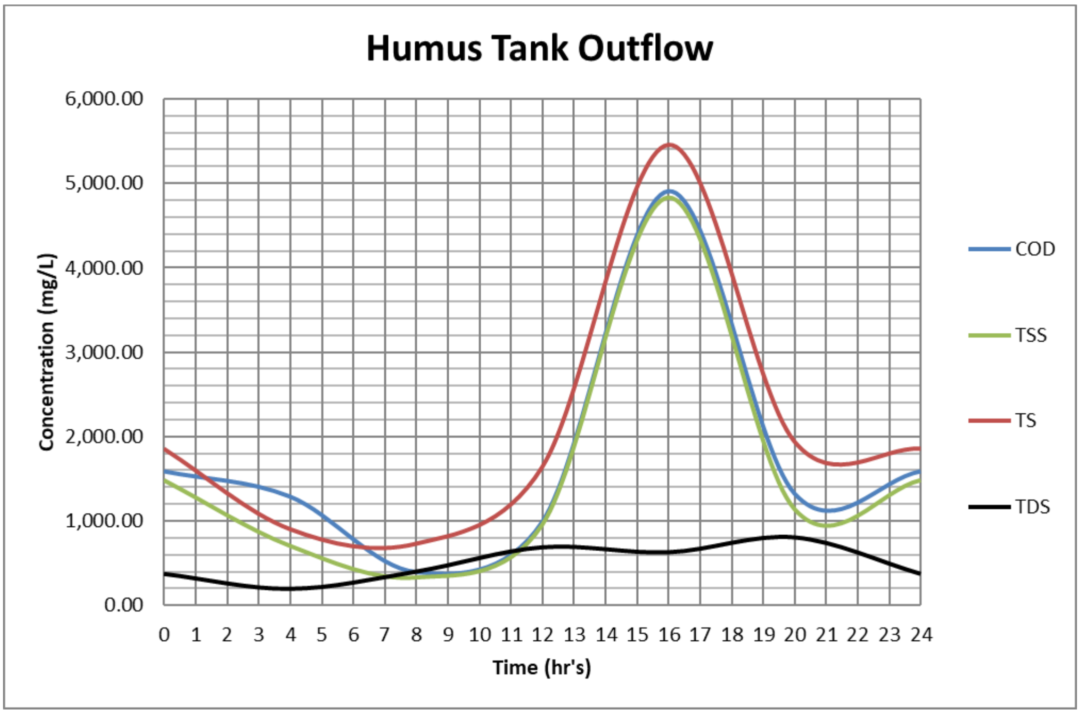


Figure 3.4.3a: Biofilter humus tank effluent: Pollutant (Solids and COD) concentrations

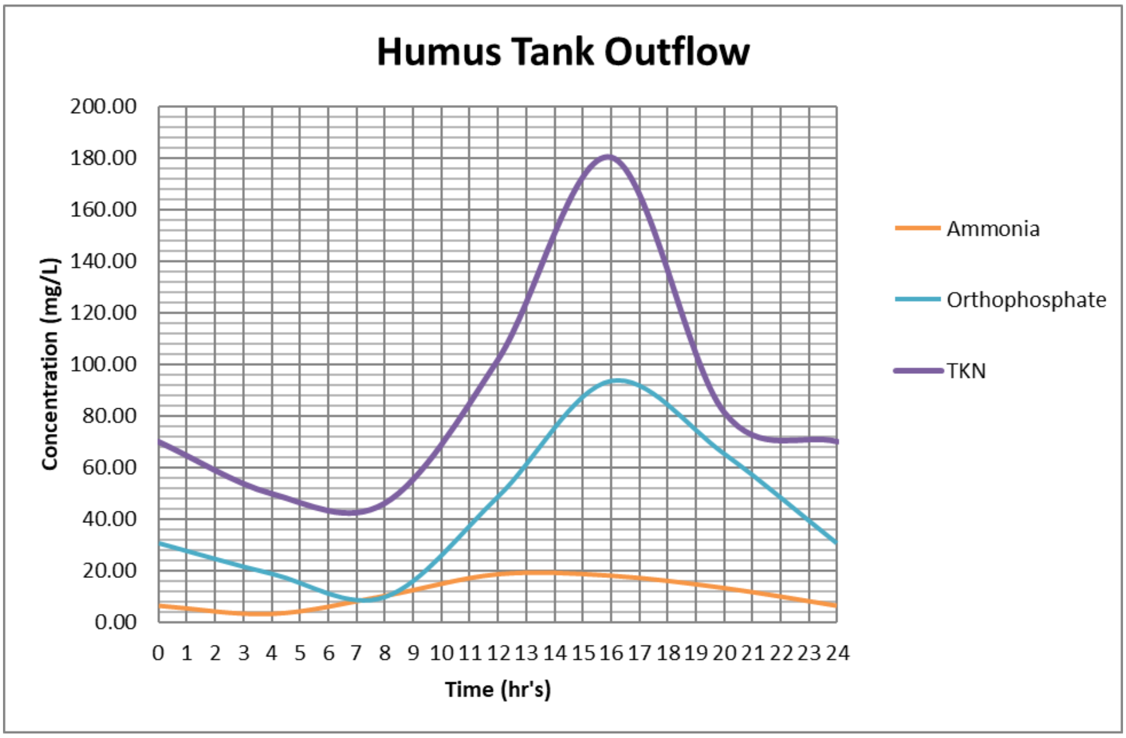


Figure 3.4.3b: Biofilter humus tank effluent: Pollutant (TKN, ammonia, orthophosphate) concentrations

From Figures 3.4.3 a and b it is observed that the effluent from the Biofilter humus tanks indicate an increase in pollutant concentration except for ammonia which was reduced to approximately 19.29 mg/l. This contradictory result to that expected is because currently the humus tanks at the plant are not de-sludged and settling out of any biofilm in filter effluent not taking place.

3.4.4 Secondary Treatment (Activated sludge clarifiers)

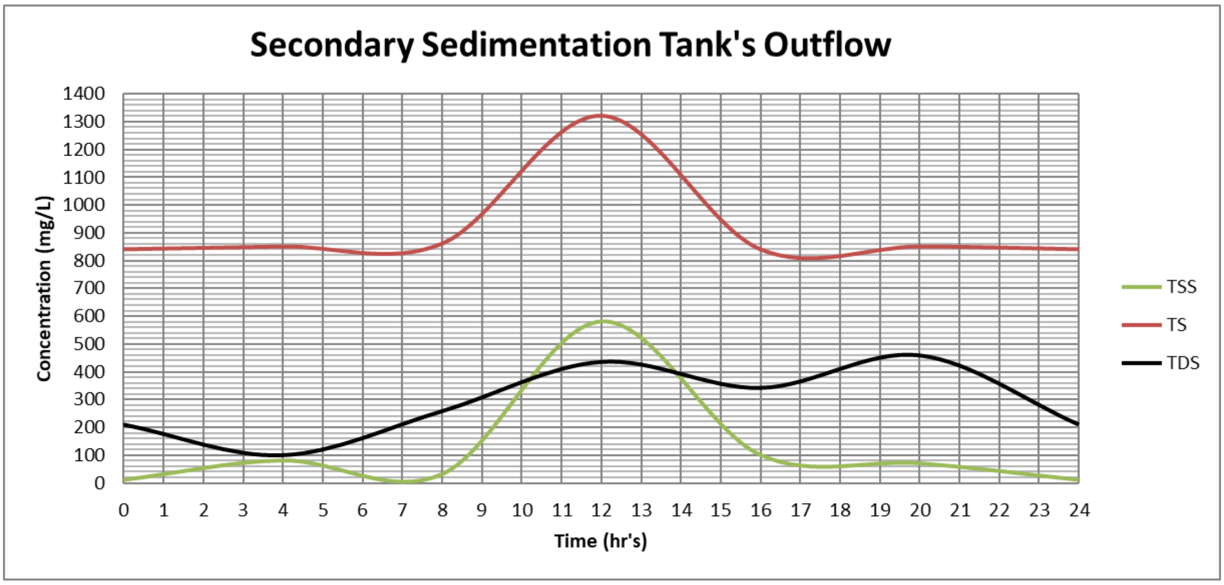


Figure 3.4.4a: Activated sludge clarifiers: Pollutant (Solids) concentrations

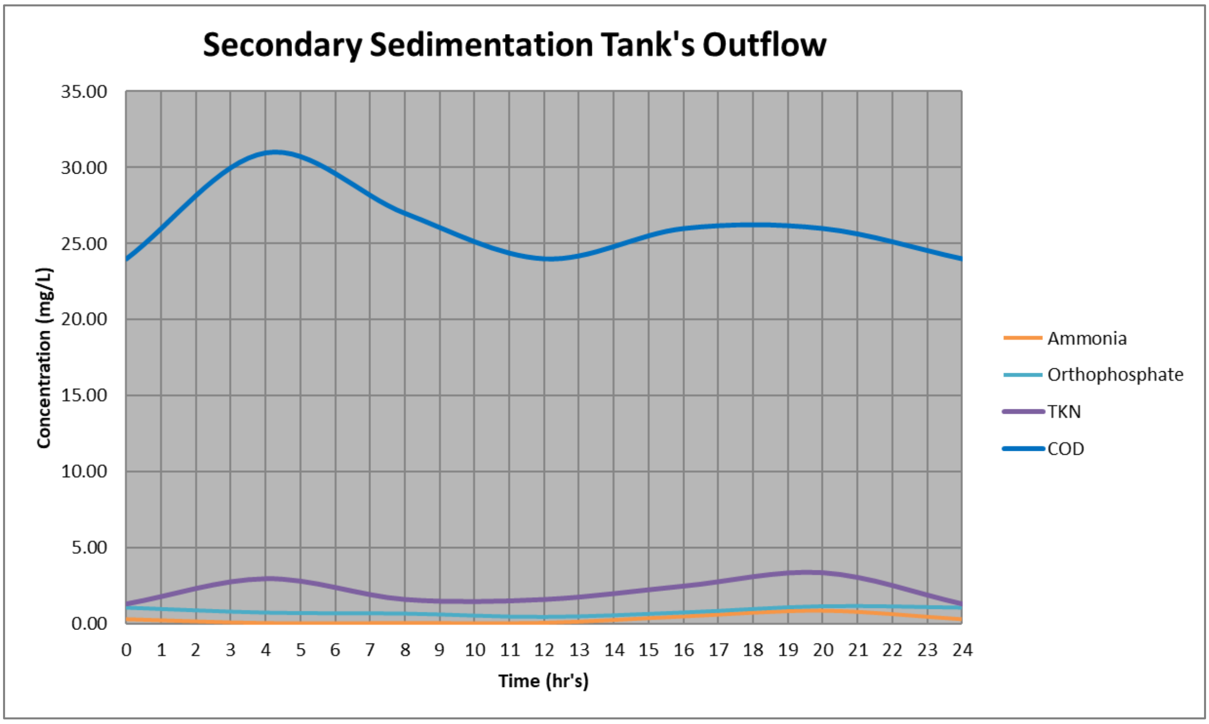


Figure 3.4.4b: Activated sludge clarifiers: Pollutant (TKN, ammonia, orthophosphate and COD) concentrations

The activated sludge pollutant effluent concentrations, as shown in Figures 3.4.4 a and b, indicate good treatment performance of the activated sludge process generally. However, high values are indicated for TS and TSS (580 and 1320 mg/L respectively) compared to that stipulated by the national effluent standard of 25 mg/L (General Standards in the Namibian Government Gazette Regulation R553 of 5 April 1962, in Section 21(1) and 21(2) of the Water Act (Act No 54 of 1956)). As these high values occur over a limited period (see figure 3.4.4a), it would be appropriate to compare average daily effluent values, oppose to the one-day results here, with that stipulated by the standard.

Generally, the reduction trend of pollutant concentrations moving downstream along the treatment train indicates that unit processes are functioning. However, the purpose of this study was to establish the level of performance of unit processes and the extent of removal efficiency achieved based on a STOAT modelling exercise which is discussed later.

3.5 Equalization process: Volumetric attenuation capacity determination

The diurnal daily influent flow pattern of the treatment plant for the equalization process design day was obtained from treatment plant historical SCADA data, and is reflected in figure 3.5.

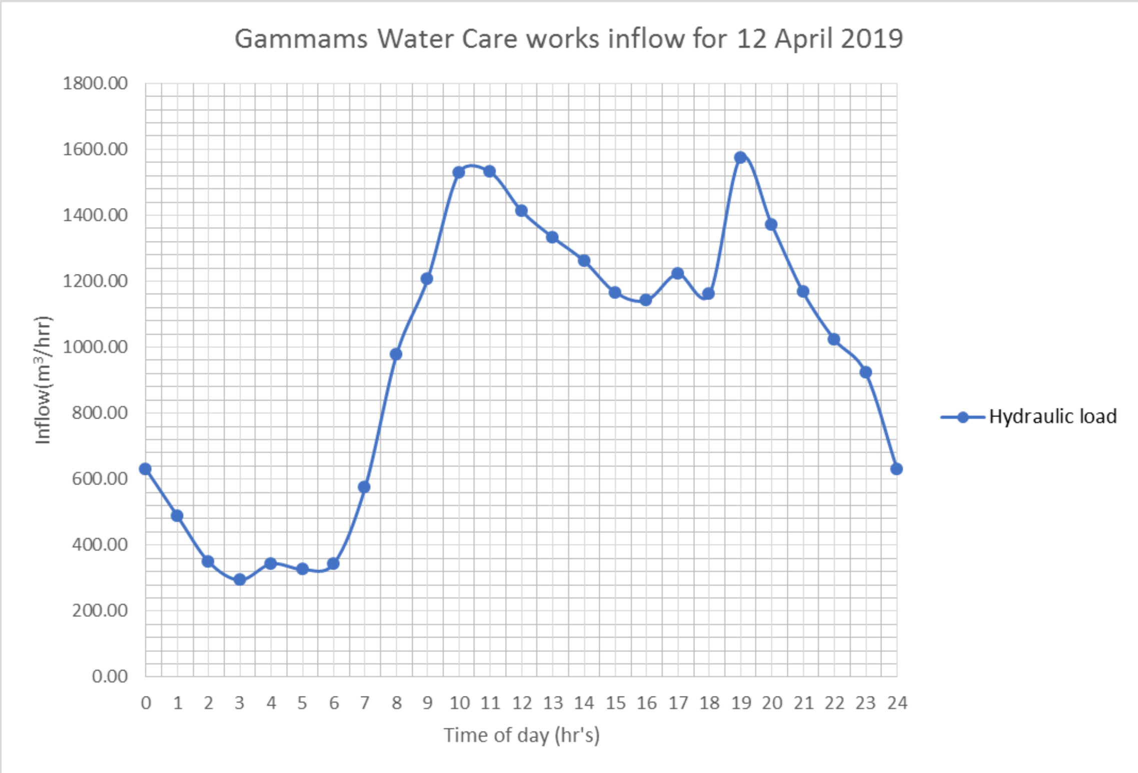


Figure 3.5: Hydraulic load (diurnal flow) pattern for the design day

The graphical Ripple and numerical time-step analyses) methodologies (using Microsoft Excel), were employed for determining the equalization basin attenuation volume required. These methodologies followed are discussed next.

3.5.1 Graphical equalization process attenuation volume determination

The graphical Ripple diagram analysis done for sizing of the equalization process volume (basin volume) for different release flow rates are given ed in Appendix 4, Figures 4.1.1 to 4.1.5. First, a basin release flow rate equal to the average daily flow rate was considered, followed by increased levels thereof with 5% increments from 10% to 25%.

Using the graphical Ripple diagram methodology described before, the required equalization process basin attenuation volumes required were determined as follows:

- Equalization basin release flow rate = average daily flow rate:

From the graph in Appendix 4 Figure 4.1.1, the cumulative flow graph illustrates a point of maximum curvature at 07:00 – 08:00 (when the equalization tank is theoretically empty).

The accumulative mass flow curve at points of maximum curvature (see figure 4.1.1) and the average accumulative flow curve volumes are given in tables 3.5.1.1 and 3.5.1.2, respectively.

Table 3.5.1. 1: Volumes: Cumulative mass flow curve

Maximum Average Cumulative Flow	
Time	Accumulative flow (m^3)
07:00-08:00	7783.02
22:00-23:00	22376.2

Table 3.5.1. 2: Volumes: Cumulative mass flow curve

Maximum cumulative flow	
Time	Ave Accumulative flow (m^3)
07:00-08:00	3525.01
22:00-23:00	22573.2

Therefore,

Equalization basin volume required = (7783.02 – 3525.01) + (22573.2 – 22376.2)

= 4455. 01 m^3

As mentioned previously, making allowance of an additional 20% for unforeseen changes in inflow occurring:

Then

Basin volume required = 4455.01 m^3 x 1.20

= 5346.0 m^3

SAY = 5400 m^3

- Equalization basin release flow rate = average daily flow rate PLUS 10%:

From the graph in Appendix 4 Figure 4.1.3, the accumulative mass flow curve points of maximum curvature are at 07:00 – 08:00 and 21:00 – 22:00.

The volumes for the accumulative mass flow and average accumulative flow curves shown in figure 4.1.2 are presented in tables 3.5.1.3 and 3.5.1.4.

Table 3.5.1. 3: Volumes: Cumulative mass flow curve

Maximum Release Cumulative Flow	
Time	Accumulative flow (m^3)
06:00-07:00	22235
21:00-22:00	21675.716

Table 3.5.1. 4: Volumes: Cumulative mass flow curve

Maximum cumulative flow	
Time	Accumulative flow (m^3)
06:00-07:00	2749.81
21:00-22:00	21600.705

Mass balance = 12 hours

Therefore, Average release x 1.10% = (972.88m^3/hr x 1.10%)

= 1070.17m^3/hr

Therefore, point B on the tangent = 21600.705 m^3 – ((836.67 x 1hr) + (1070.17m^3/hr x 14hr))

= 5781.655 m^3

Equalization basin volume required = (5781.655 m^3 – 2749.81 m^3)

= 3031.845 m^3

As mentioned previously, making a contingency allowance of 20%:

Then

Basin volume required = 3031.845 m^3 x 1.20

= 3638.214 m^3

SAY = 3700 m^3

- Equalization basin release flow rate = average daily flow rate PLUS 15%:

The cumulative mass flow graph in Appendix 4 Figure 4.1.4, has points of maximum curvature at 08:00 – 09:00 and 22:00 – 23:00.

The volumes for the accumulative mass flow and average accumulative flow curves shown in figure 4.1.4 are given in tables 3.5.1.5 and 3.5.1.6.

Table 3.5.1. 5: Volumes: Cumulative mass flow curve

Maximum Cumulative Release Flow	
Time	Cumulative flow (m^3)
07:00-08:00	6615.567
22:00-23:00	22522.12

Table 3.5.1. 6: Volumes: Cumulative mass flow curve

Maximum cumulative flow	
Time	Cumulative flow (m^3)
07:00-08:00	3525.01
22:00-23:00	22573.215

Mass balance = 12 hours

Therefore, Average release x 1.15% = (972.88m^3/hr x 1.15%)

= 1118.812m^3/hr

Therefore, point B on the tangent = 22573.215m^3 – ((3hr x 826.95 m^3/hr) + (1118.812m^3/hr x 12hr))

= 6666.621 m^3

Equalization basin volume required = (6666.621 m^3 – 3525.01 m^3)

= 3141.611 m^3

Making a contingency allowance of 20%:

Then

Basin volume required = 3141.611 m^3 x 1.20

= 3769.933 m^3

SAY = 3800 m^3

- Equalization basin release flow rate = average daily flow rate PLUS 20%:

The cumulative mass flow graph in Appendix 4 Figure 4.1.4, shows points of maximum curvature at 07:00 – 08:00 and 22:00 – 23:00.

The volumes for the accumulative mass flow and average accumulative flow curves shown in figure 4.1.4 are given in tables 3.5.1.7 and 3.5.1.8.

Table 3.5.1. 7: Volumes: Cumulative mass flow release curve

Maximum Cumulative Release Flow	
Time	Cumulative flow (m^3)
07:00-08:00	6226.416
22:00-23:00	22570.763

Table 3.5.1. 8: Volumes: Cumulative mass flow curve

Maximum cumulative flow	
Time	Cumulative flow (m^3)
07:00-08:00	3525.01
22:00-23:00	22573.215

Mass balance = 12 hours

Therefore, Average release x 1.20% = (972.88m^3/hr x 1.20%)

= 1167.812m^3/hr

Therefore,

point B on the tangent = 22573.215 m^3 – ((778.30m^3/hr x 3 hr) + (1167.45m^3/hr x 12hr))

= 6 223.915 m^3

Equalization basin volume required = (6223.915 m^3 – 3525.01 m^3)

= 2 703.905 m^3

Making a contingency allowance of 20%:

Then

Basin volume required = 2 703.905 m^3 x 1.20

= 3 244.686 m^3

SAY = 3300 m^3

- Equalization basin release flow rate = average daily flow rate PLUS 25%:

The cumulative mass flow graph in Appendix 4 Figure 4.1.5, have points of maximum curvature at 08:00 – 09:00 and 22:00 – 23:00.

The volumes for the accumulative mass flow and average accumulative flow curves shown in figure 4.1.5 are presented in tables 3.5.1.9 and 3.5.1.10.

Table 3.5.1. 9: Volumes: Cumulative mass flow curve

Maximum Cumulative release Flow	
Time	Accumulative flow (m^3)
08:00-09:00	6903.72
22:00-23:00	22582.03

Table 3.5.1. 10: *Volumes: Cumulative mass flow curve*

Maximum cumulative flow	
Time	Cumulative flow (m^3)
08:00-09:00	4616.965
22:00-23:00	22573.215

Mass balance = 12 hours

Therefore, Average release x 1.25% = (972.88m^3/hr x 1.25%)

= 1216.10m^3/hr

Therefore,

point B on the tangent = 22573.215 m^3 – ((767.08m^3/hr x 2 hr) + (1216.10m^3/hr x 11hr))

= 7 661.955m^3

Equalization basin volume required = (7 661.955 m^3 – 4616.965 m^3)

= 3 044.99 m^3

Making a contingency allowance of 20%:

Then

Basin volume required = 3 044.99 m^3 x 1.20

= 3 653.988 m^3

SAY = 3 700 m^3

3.5.2 Numerical equalization process attenuation volume determination

As described before, Appendix 5 Tables 5.1a to 5.1e illustrate the numerical time-step analysis that was used for determining the required equalization process attenuation volume for different basin release flow rates (section 3.6.1). Moreover, tables 5.1a to 5.1e highlight data on how the maximum cumulative volume (the attenuation required) was determined.

When release rate is the average daily low rate, the principle of daily flow balance between influent and release flow rate allows the latter to take place over the full 24-hour period. However, when the influent to the basin increases significantly (that is, between two maximum curvature points on a mass flow curve), the increased rates of release of average flow PLUS 10 to 25% can be employed. Thus, to maintain the total daily accumulative influent balance, a lesser flow than the average daily value would have to be released for the remaining hours of the day.

Towards achieving a more accurate basin attenuation capacity determination for the increase flow release rates scenarios greater than average daily flow rate, the numerical time-step methodology was employed with a reduced 1-hour interval. Again, with the numerical methodology described before, the required equalization process basin attenuation volumes required were determined as shown in Appendix 5 Tables 5.1a to 5.1e.

3.5.3 Equalization basin release flow rate variation: Impact on final effluent quality

For the attenuation requirements for different basin release rates (given in Table 3.5.2),

the corresponding final effluent pollutant concentrations were determined using the STOAT model.

Table 3.5.3 below summarizes the impact of release flow rate on final plant effluent quality.

Table 3.5. 3: Equalised average final effluent pollutant concentration for different basin release flow rates: Based on STOAT model analysis

Pollutant	Average Basin release flow rates	Average +10% Basin release flow rates	Average +15% Basin release flow rates	Average +20% Basin release flow rates	Average +25% Basin release flow rates
TSS (mg/L)	8.583	12.133	12.218	12.360	12.502
COD (mg/L)	20.557	30.328	31.918	32.620	32.771
Ammonia (mg/L)	0.574	1.204	1.561	1.256	2.597
Total Nitrogen (mg/L)	6.779	3.045	3.283	3.568	4.062
Total Phosphorus (mg/L)	7.665	5.533	3.164	3.614	4.033

3.6 Equalization attenuation requirements: Allowance for a 5-year growth in wastewater influent to the plant

The study also considered the equalization process attenuation requirements for a 5-year period with growth in wastewater discharged to the plant. The assumption was that the increase in wastewater flow would be equivalent to the projected population growth based on the latest census records. A growth rate of 4.1% per annum was assumed based on the 2011 Namibia Population and Housing Census graphically shown in Appendix 6 Figure 6.1.

Both the Ripple graphical and time-step numerical analysis methodologies were used to analyse the equalization process attenuation basin volume required at the end of the 5-year growth period.

3.6.1 Graphical analysis: 5-year wastewater growth projection

Figure 6.2 in Appendix 6 shows the cumulative flow graph with the tangents at points of maximum curvature. Tables 3.6.1 and 3.6.2 present the points of maximum curvature identified and accumulative flow curve values from Appendix - figure 6.2.

Table 3.6. 1: Graphical methodology: Basin attenuation requirement: 5-year wastewater flow projection: Cumulative Flow values

Maximum Cumulative Flow	
Time	Accumulative flow (m3)
07:00-08:00	4157.299
22:00-23:00	27815.285

Table 3.6. 2: Graphical methodology: Basin attenuation requirement: 5-year wastewater flow projection: Average cumulative Flow values

Maximum average cumulative flow	
Time	Ave Accumulative flow (m^3)
07:00-08:00	9227.655
22:00-23:00	27326.55

Therefore,

Equalization basin attenuation volume required

= (9227.655 – 4157.299) + (27815.285 – 27326.550)

= 5559.10 m^3

3.6.2 Numerical analysis: 5-year wastewater growth projection

From Appendix 6 Table 6.1 for numerical method, the basin attenuation volume required amounts to 5525.73 m^3

3.6.3 5-year wastewater growth projection: Summary of graphical and numerical attenuation capacity

Based on the information given in sections 3.6.1 and 3.6.2, the respectively minimum attenuation required as well as the increased design capacity that makes allowance for unforeseen changes in inflow occurring for both current and a future 5-year growth projection in wastewater flow is reflected in table 3.6.3.

Table 3.6. 3: Summary of 5-year projection Basin required Volume: Graphical and Numerical Analysis

	Basin release flow rate analysis methodologies			
	Graphical		**Numerical**	
Release Flow rates	Graphical analysis: Basin Capacity required (m3)	Graphical analysis: Basin Capacity requiredwith 20% buffer (m3)	Numerical analysis: Basin capacity required (m3)	Numerical analysis: Basin Capacity requiredwith 20% buffer (m3)
Average daily flow	5559.10	6700	5525.73	6700

Comparison of the graphical Ripple and numerical time-step analysis results indicate similar attenuation basin volume sizes (5559.10 m^3 and 5525.73 m^3 respectively), differing by less than 1%. The numerical analysis attenuation capacity is considered more accurate than the graphical one and therefore the resulting design capacity including a 20% contingency allowance amounts to 6700 m^3.

Part 2.2 Unequalised and Equalised treatment plant comparative analyses

3.7 Pollutant loading: Unequalised vs Equalised plant operation

3.7.1 Numerical analysis comparison

The pollutant loading analysis of Unequalised and Equalised scenarios were done using the numerical time-step analysis.

Appendix 7 Figures 7.1 to 7.7 illustrate graphically the potential impact of equalization on the considered pollutant loading for different basin release flow rates.

- Pollutant: COD loading

The impact of equalization on COD loading illustrated in Figure 7.1a, is summarised as follows:

- For average daily flow basin release (Table 3.7.1.1):

Table 3.7.1. 1: Impact on COD loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (Kg/hr)	Max (kg/hr)	Min (Kg/hr)
Average	908.19	905.18	907.85	905.18
Peak	1590.39	90.97	1030.34	625.14
Fluctuation wrt average	75%	90%	13%	31%

Note: Max and Min are the respective diurnal concentration values under daily maximum and minimum flow conditions.

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.1b to e:

➢ Average daily flow rate PLUS 10% increase (Table 3.7.1.2):

Table 3.7.1. 2: Impact on COD loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max (Kg/hr)	Min (Kg/hr)	Max (Kg/hr)	Min (Kg/hr)
Average	908.19	905.18	907.45	905.18
Peak	1590.39	90.97	1137.41	518.35
Fluctuation wrt average	75%	90%	25%	43%

➢ Average daily flow rate PLUS 15% increase (Table 3.7.1.3):

Table 3.7.1. 3: Impact on COD loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average	908.19	905.18	908.45	905.18
Peak	1590.39	90.97	1186.85	492.57
Fluctuation wrt average	75%	90%	31%	46%

➢ Average daily flow rate PLUS 20% increase (Table 3.7.1.4):

Table 3.7.1. 4: Impact on COD loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average	908.19	905.18	908.64	905.18
Peak	1590.39	90.97	1239.16	440.43
Fluctuation wrt average	75%	90%	36%	51%

➢ Average daily flow rate PLUS 25% increase (Table 3.7.1.5):

Table 3.7.1. 5: Impact on COD loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Max (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average	908.19	905.18	908.84	905.18
Peak	1590.39	90.97	1291.55	388.65
Fluctuation wrt average	75%	90%	42%	57%

- Pollutant: Ammonia loading

The impact of equalization on Ammonia loading as illustrated in Appendix 7 Figure 7.2a, is summarised as follows:

- For average daily flow basin release (Table 3.7.2.1):

Table 3.7.2. 1: Impact on Ammonia loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	46.97	46.97	46.51	46.51
Peak (P)	90.22	8.73	63.04	63.04
Variation vs Average (%)	92.09	81.40	26	26
Ratio: P/A	1.92	5.38	1.36	1.36

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.2.2 to 3.7.2.5:

➢ Average daily flow rate PLUS 10% increase (Table 3.7.2.2):

Table 3.7.2. 2: Impact on Ammonia loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max	Min	Max (kg/hr)	Min (kg/hr)
Average (A)	23.68	23.68	23.50	23.50
Peak (P)	48.25	3.99	35.20	35.20
Variation vs Average (%)	103.72	83.14	33	33
Ratio: P/A	2.04	5.93	1.50	1.50

➢ Average daily flow rate PLUS 15% increase (Table 3.7.2.3):

Table 3.7.2. 3: Impact on Ammonia loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	23.68	23.68	23.50	23.50
Peak (P)	48.25	3.99	36.46	36.46
Variation vs Average (%)	103.72	83.14	36	36
Ratio: P/A	2.04	5.93	1.55	1.55

➢ Average daily flow rate PLUS 20% increase (Table 3.7.2.4):

Table 3.7.2. 4: Impact on Ammonia loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	23.68	23.68	23.50	23.50
Peak (P)	48.25	3.99	37.85	37.85
Variation vs Average (%)	103.72	83.14	38	38
Ratio: P/A	2.04	5.93	1.61	1.61

➢ Average daily flow rate PLUS 25% increase (Table 3.7.2.5):

Table 3.7.2. 5: Impact on Ammonia loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	23.68	23.68	23.50	23.50
Peak (P)	48.25	3.99	39.45	39.45
Variation vs Average (%)	103.72	83.14	40	40
Ratio: P/A	2.04	5.93	1.68	1.68

- Pollutant: Orthophosphate loading

The impact of equalization on Orthophosphate loading as illustrated in Appendix 7, Figures 7.3a, is summarised as follows:

- For average daily flow basin release (Table 3.7.3.1):

Table 3.7.3. 1: Impact on Orthophosphate loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	5.0	5.0	5.0	5.0
Peak (P)	8.87	0.72	6.32	4.25
Variation vs Average (%)	76	86	25	16
Ratio: P/A	1.76	0.14	1.25	0.8

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.3.2 to 3.7.3.5:

  - Average daily flow rate PLUS 10% increase (Table 3.7.3.2):

Table 3.7.3. 2: Impact on Orthophosphate loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	2.5	2.5	2.5	2.5
Peak (P)	4.81	0.33	3.48	1.68
Variation vs Average (%)	90	87	38	33
Ratio: P/A	1.90	0.13	1.38	0.7

  - Average daily flow rate PLUS 15% increase (Table 3.7.3.3):

Table 3.7.3. 3: Impact on Orthophosphate loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	2.5	2.5	2.5	2.5
Peak (P)	4.81	0.33	3.65	1.66
Variation vs Average (%)	90	87	44	35
Ratio: P/A	1.90	0.13	1.44	0.7

  - Average daily flow rate PLUS 20% increase (Table 3.7.3.4):

Table 3.7.3. 4: Impact on Orthophosphate loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	2.5	2.5	2.5	2.5
Peak (P)	4.81	0.33	3.81	1.51
Variation vs Average (%)	90	87	50	40
Ratio: P/A	1.90	0.13	1.50	0.6

  - Average daily flow rate PLUS 15% increase (Table 3.7.3.5):

Table 3.7.3. 5: Impact on Orthophosphate loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	2.5	2.5	2.5	2.5
Peak (P)	4.81	0.33	5.47	-2.77
Variation vs Average (%)	90	87	115	209
Ratio: P/A	1.90	0.13	2.15	-1.1

- Pollutant: Total Kjeldahl Nitrogen (TKN) loading

The impact of equalization on TKN loading illustrated in Appendix 7 Figure 7.4a, is summarised as follows:

- For average daily flow basin release (Table 3.7.4.1):

Table 3.7.4. 1: Impact on TKN loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	74.71	74.71	74.70	74.70
Peak (P)	134.57	13.22	91.45	60.18
Variation vs Average (%)	80	82	22	19
Ratio: P/A	1.80	0.18	1.22	0.81

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.4.2 to 3.7.4.5:

- Average daily flow rate PLUS 10% increase (Table 3.7.4.2):

Table 3.7.4. 2: Impact on TKN loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	37.71	37.71	38.05	38.05
Peak (P)	70.29	6.04	50.33	25.50
Variation vs Average (%)	86	84	32	33
Ratio: P/A	1.86	0.16	1.32	0.67

- Average daily flow rate PLUS 15% increase (Table 3.7.4.3):

Table 3.7.4. 3: Impact on TKN loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	37.71	37.71	38.05	38.05
Peak (P)	70.29	6.04	52.72	24.67
Variation vs Average (%)	86	84	39	35
Ratio: P/A	1.86	0.16	1.39	0.65

- Average daily flow rate PLUS 20% increase (Table 3.7.4.4):

Table 3.7.4. 4: Impact on TKN loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	37.71	37.71	38.05	38.05
Peak (P)	70.29	6.04	55.02	22.63
Variation vs Average (%)	86	84	45	41
Ratio: P/A	1.86	0.16	1.45	0.59

- Average daily flow rate PLUS 25% increase (Table 3.7.4.5):

Table 3.7.4. 5: Impact on TKN loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	37.71	37.71	38.05	38.05
Peak (P)	70.29	6.04	79.00	-38.12
Variation vs Average (%)	86	84	108	200
Ratio: P/A	1.86	0.16	2.08	-1.00

- Pollutant: Total Suspended Solids (TSS) loading

The impact of equalization on TSS loading as illustrated in Appendix 7, Figure 7.5a, is summarised as follows:

- For average daily flow basin release (Table 3.7.5.1):

Table 3.7.5. 1: Impact on TSS loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	430.88	430.88	431.85	431.85
Peak (P)	940.38	38.58	620.46	133.89
Variation vs Average (%)	118	91	44	69
Ratio: P/A	2.18	0.09	1.44	0.31

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.5.2 to 3.7.5.5:

➢ Average daily flow rate PLUS 10% increase (Table 3.7.5.2):

Table 3.7.5. 2: Impact on TSS loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	214.85	214.85	211.30	211.30
Peak (P)	456.47	19.78	343.47	53.94
Variation vs Average (%)	112	91	63	74
Ratio: P/A	2.12	0.09	1.63	0.26

➢ Average daily flow rate PLUS 15% increase (Table 3.7.5.3):

Table 3.7.5. 3: Impact on TSS loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	214.85	214.85	211.30	211.30
Peak (P)	456.47	19.78	361.70	42.81
Variation vs Average (%)	112	91	71	80
Ratio: P/A	2.12	0.09	1.71	0.20

➢ Average daily flow rate PLUS 20% increase (Table 3.7.5.4):

Table 3.7.5. 4: Impact on TSS loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	214.85	214.85	211.30	211.30
Peak (P)	456.47	19.78	379.24	31.81
Variation vs Average (%)	112	91	79	85
Ratio: P/A	2.12	0.09	1.79	0.15

➢ Average daily flow rate PLUS 25% increase (Table 3.7.5.5):

Table 3.7.5. 5: Impact on TSS loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	430.88	430.88	433.83	433.83
Peak (P)	940.38	38.58	790.19	77.42
Variation vs Average (%)	118	91	82	82
Ratio: P/A	2.18	0.09	1.82	0.18

- Pollutant: Total Solids (TS) loading

The impact of equalization on TS loading as illustrated in Appendix 7, Figure 7.6a to e, is summarised as follows:

- For average daily flow basin release (Table 3.7.6.1):

Table 3.7.6. 1: Impact on TS loading: Average daily flow basin release

Equalization	Unequalised		Equalised	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	1359.79	1359.79	1360.93	1360.93
Peak (P)	2413.46	210.66	1597.09	852.38
Variation vs Average (%)	77	85	15	37
Ratio: P/A	1.77	0.15	1.17	0.63

- For increased flow rate release (above average flow rate) the data is given in Tables 3.7.6.2 to 3.7.6.5:

- Average daily flow rate PLUS 10% increase (Table 3.7.6.2):

Table 3.7.6. 2: Impact on TS loading: Average daily flow PLUS 10%

Equalization	Unequalised		Equalised	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	671.27	671.27	671.269	671.2686902
Peak (P)	1158.55	108.02	884.082	308.78
Variation vs Average (%)	73	84	24	54
Ratio: P/A	1.73	0.16	1.317	0.46

- Average daily flow rate PLUS 15% increase (Table 3.7.6.3):

Table 3.7.6. 3: Impact on TS loading: Average daily flow PLUS 15%

Equalization	Unequalised		Equalised	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	1359.79	1359.79	1396.58	1396.58
Peak (P)	2413.46	210.66	1881.81	623.20
Variation vs Average (%)	77.49	84.51	25.79	55.38
Ratio: P/A	1.77	0.15	1.35	0.45

➢ Average daily flow rate PLUS 20% increase (Table 3.7.6.4):

Table 3.7.6. 4: Impact on TS loading: Average daily flow PLUS 20%

Equalization	Unequalised		Equalised	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	1359.79	1359.79	1403.34	1403.34
Peak (P)	2413.46	210.66	1978.22	497.52
Variation vs Average (%)	77.49	84.51	29.06	64.55
Ratio: P/A	1.77	0.15	1.41	0.35

➢ Average daily flow rate PLUS 25% increase (Table 3.7.6.5e):

Table 3.7.6. 5: Impact on TS loading: Average daily flow PLUS 25%

Equalization	Unequalised		Equalised	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	1359.79	1359.79	1403.344	1403.344209
Peak (P)	2413.46	210.66	1978.223	497.52
Variation vs Average (%)	77	85	29	65
Ratio: P/A	1.77	0.15	1.410	0.35

- Pollutant: Total Dissolved Solids (TDS) loading

The impact of equalization on TDS loading as illustrated in Figures 3.7.1.7a to e, is summarised as follows:

o For average daily flow basin release (Table 3.7.7.1):

Table 3.7.7. 1: Impact on TS loading: Average daily flow basin release

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	874.56	874.56	874.98	874.98
Peak (P)	1531.46	229.78	978.48	726.49
Variation vs Average (%)	75	74	12	17
Ratio: P/A	1.75	0.26	1.12	0.83

o For increased flow rate release (above average flow rate) the data is given in Tables 3.7.7.2 to 3.7.7.5:

➢ Average daily flow rate PLUS 10% increase (Table 3.7.7.2):

Table 3.7.7. 2: Impact on TDS loading: Average daily flow PLUS 10%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	435.19	435.19	435.95	435.95
Peak (P)	761.36	114.40	538.69	339.51
Variation vs Average (%)	75	74	24	22
Ratio: P/A	1.75	0.26	1.24	0.78

➢ Average daily flow rate PLUS 15% increase (Table 3.7.7.3):

Table 3.7.7. 3: Impact on TDS loading: Average daily flow PLUS 15%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	435.19	435.19	435.89	435.89
Peak (P)	761.36	114.40	564.07	306.02
Variation vs Average (%)	75	74	29	30
Ratio: P/A	1.75	0.26	1.29	0.70

➢ Average daily flow rate PLUS 20% increase (Table 3.7.7.4):

Table 3.7.7. 4: Impact on TDS loading: Average daily flow PLUS 20%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	435.19	435.19	435.82	435.82
Peak (P)	761.36	114.40	589.40	288.71
Variation vs Average (%)	75	74	35	34
Ratio: P/A	1.75	0.26	1.35	0.66

➢ Average daily flow rate PLUS 25% increase (Table 3.7.7.5):

Table 3.7.7. 5: Impact on TDS loading: Average daily flow PLUS 25%

Equalization	Unequalized		Equalized	
	Max (kg/hr)	Min (kg/hr)	Max (kg/hr)	Min (kg/hr)
Average (A)	435.19	435.19	435.77	435.77
Peak (P)	761.36	114.40	615.02	271.17
Variation vs Average (%)	75	74	41	38
Ratio: P/A	1.75	0.26	1.41	0.62

3.8 STOAT simulation model comparative analysis: Unequalised vs equalised operation

3.8.1 Scenario 1: Unequalised operation (current)

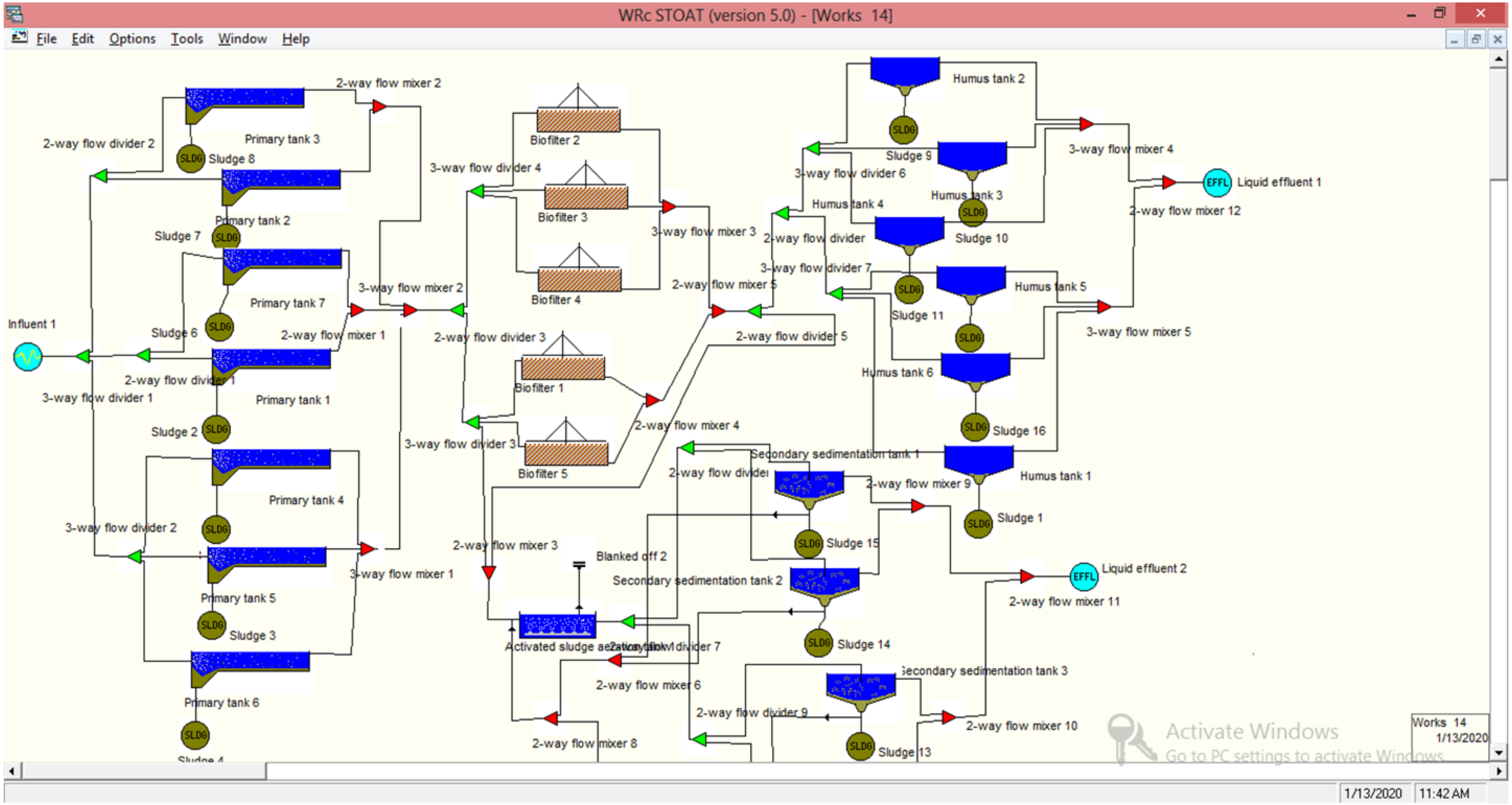


Figure 3.8. 1: STOAT model: Unequalised treatment process flow diagram

Figure 3.8.1 gives a typical graphical illustration of the sequence of unit processes in the current unequalised treatment train as compiled in the STOAT model software. Preliminary treatment was not modelled in STOAT since it is only of a mechanical nature (screening and grit removal operations alone) and biological treatment does not occur during this process step.

The current unequalised treatment plant operation was modelled using the STOAT simulation software, and the pollution parameter loading graphical results are reflected in Figures 3.8.2.1a to Figure 3.8.2d for the various unit processes along the treatment train.

Corresponding pollutant average, maximum and minimum peak load as well as the corresponding variations of the latter in relation to the average load are given in Tables 3.8.1 to 3.8.4.

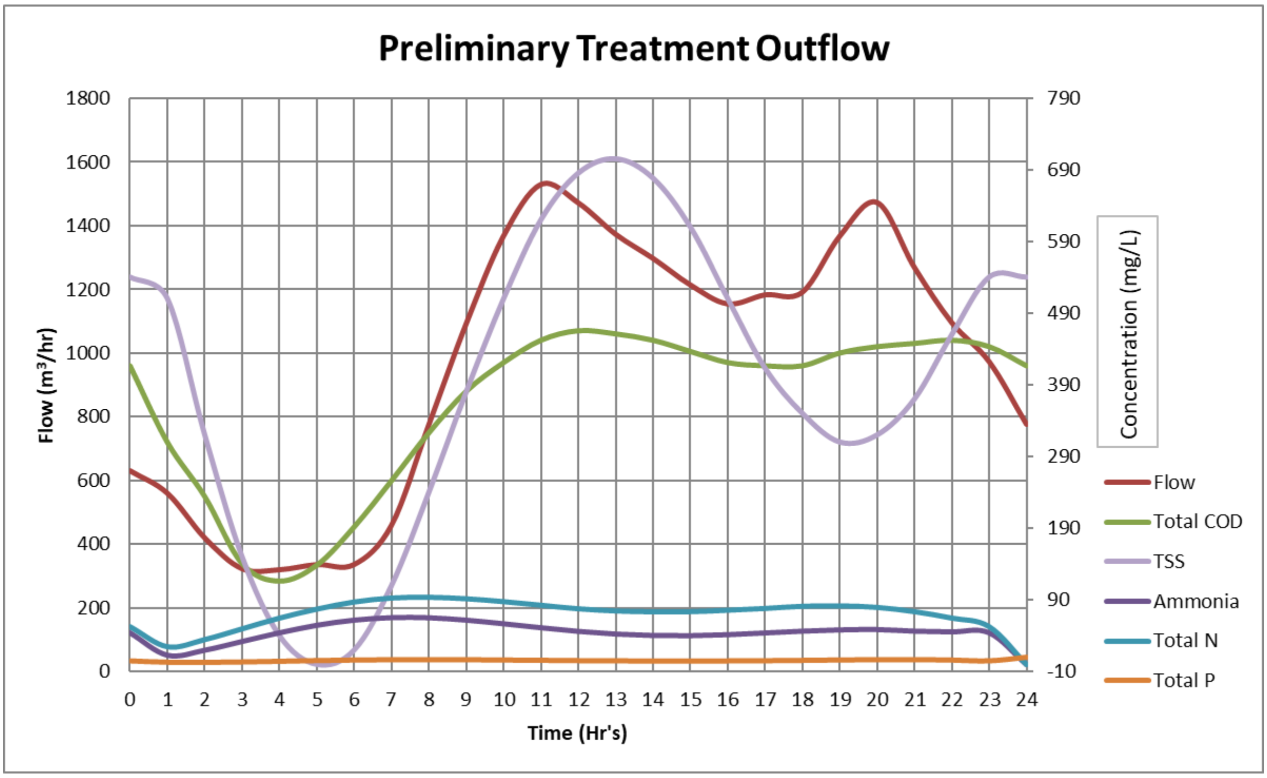


Figure 3.8. 2: STOAT simulation results: Preliminary Treatment Outflow – Unequalised (Flow; COD; TSS; NH4; N & P)

Table 3.8. 1: STOAT simulation results: Preliminary Treatment Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	397.2	845.8	45.8	5.0	72.2
Max Peak (P)	705.0	1070.0	65.0	6.5	94.0
Min Peak (P)	0.0	285.0	13.0	2.2	25.0
(Max) Variation vs Average (%)	77.5	26.5	41.8	31.1	30.1
(Min) Variation vs Average (%)	100.0	66.3	71.6	55.2	65.4

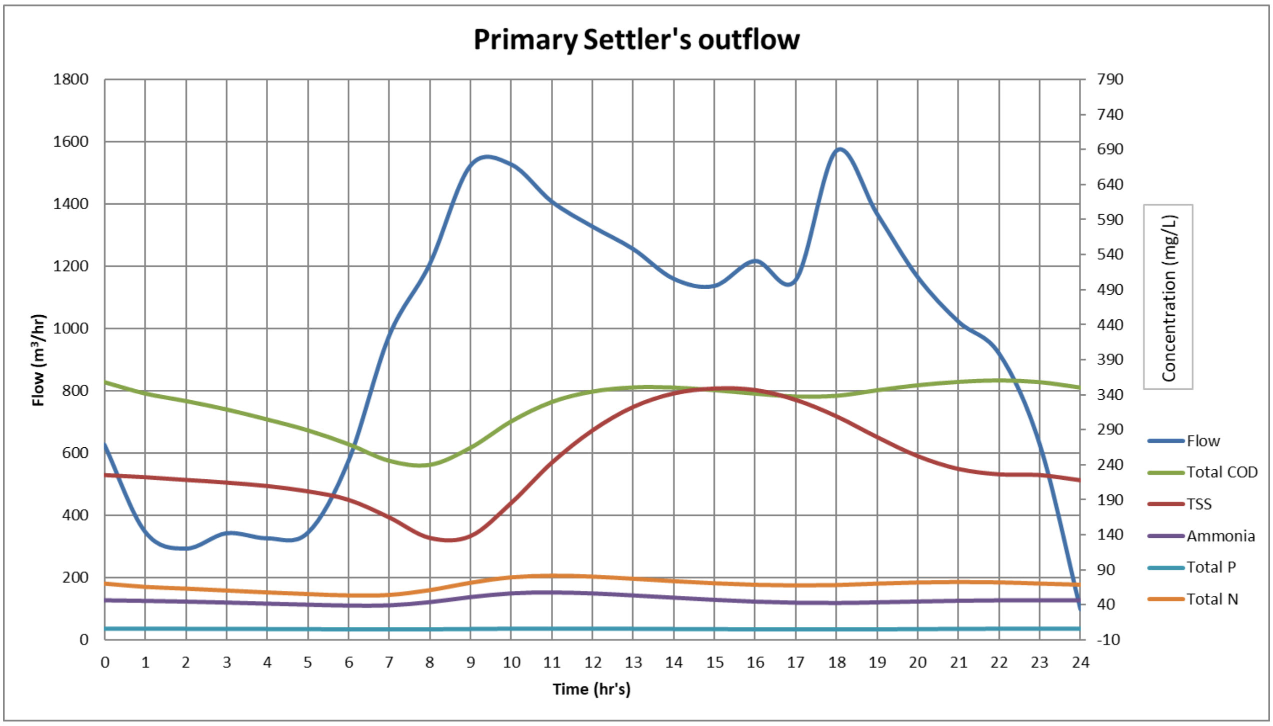


Figure 3.8. 3: STOAT simulation results: Primary Setter Outflow – Flow; COD; TSS; NH4; N & P

Table 3.8. 2: STOAT simulation results: Primary Settler Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	243	753	46	5	68
Max Peak (P)	349	835	58	6	81
Min Peak (P)	135	562	39	5	53
(Max) Variation vs Average (%)	44	11	24	12	20
(Min) Variation vs Average (%)	44	25	16	13	22

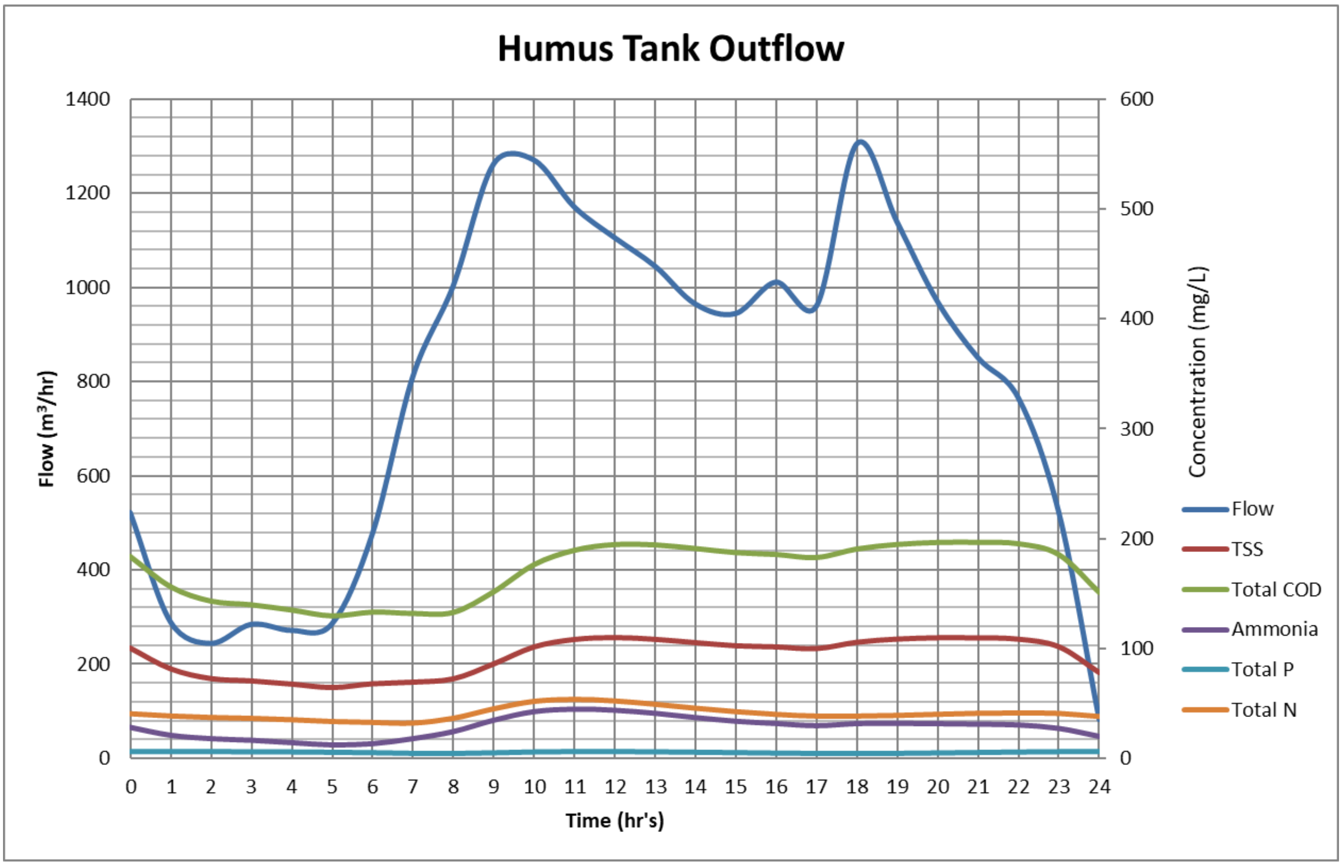


Figure 3.8. 4: STOAT simulation results: Humus Tank Outflow – Unequalised (Flow; COD; TSS; NH4; N & P)

Table 3.8. 3: STOAT simulation results: Humus Tank Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	215.3	398.5	28.6	5.3	40.7
Max Peak (P)	255.4	458.9	44.4	5.9	53.2
Min Peak (P)	151.0	302.8	12.4	4.6	32.0
(Max) Variation vs Average (%)	18.6	15.2	55.5	10.3	30.7
(Min) Variation vs Average (%)	29.9	24.0	56.7	13.8	21.3

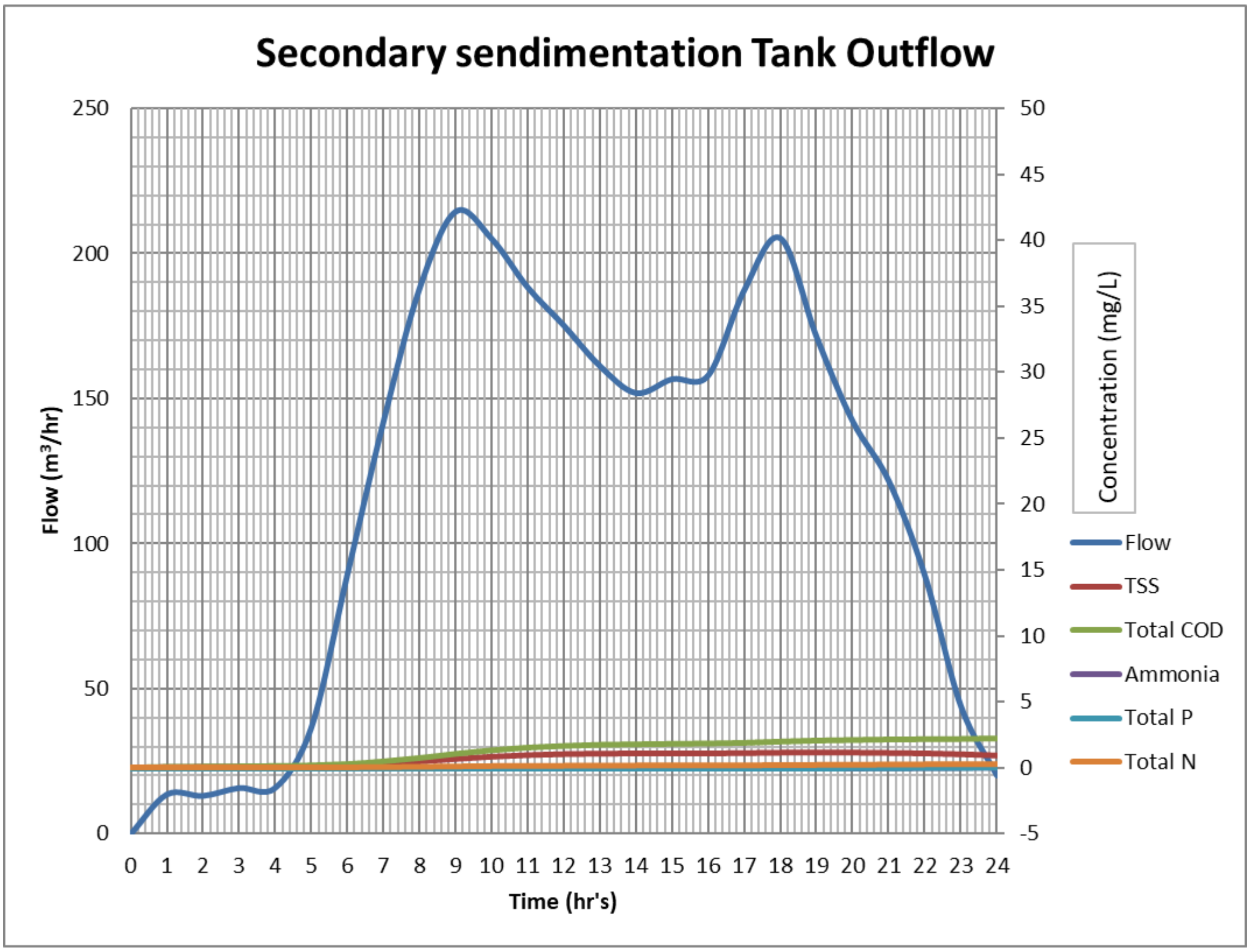


Figure 3.8. 5: STOAT simulation results: Secondary Settler Outflow – Unequalised (Flow; COD; TSS; NH4; N & P)

Table 3.8. 4: STOAT simulation results: Secondary Settler Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	18.99	360.06	22.75	4.73	27.52
Max Peak (P)	26	419.3	28.3	5.37	34.52
Min Peak (P)	10	142.3	19.4	4.07	22.4
(Max) Variation vs Average (%)	36.9	16.5	24.4	13.5	25.4
(Min) Variation vs Average (%)	47.3	60.5	14.7	14.0	18.6

3.8.2 Scenario 2: Equalised operation (new process intervention considered)

Figure 3.9.1 gives a typical graphical illustration of the sequence of unit processes of the equalised treatment train as compiled in the STOAT software model incorporating an equalization process.

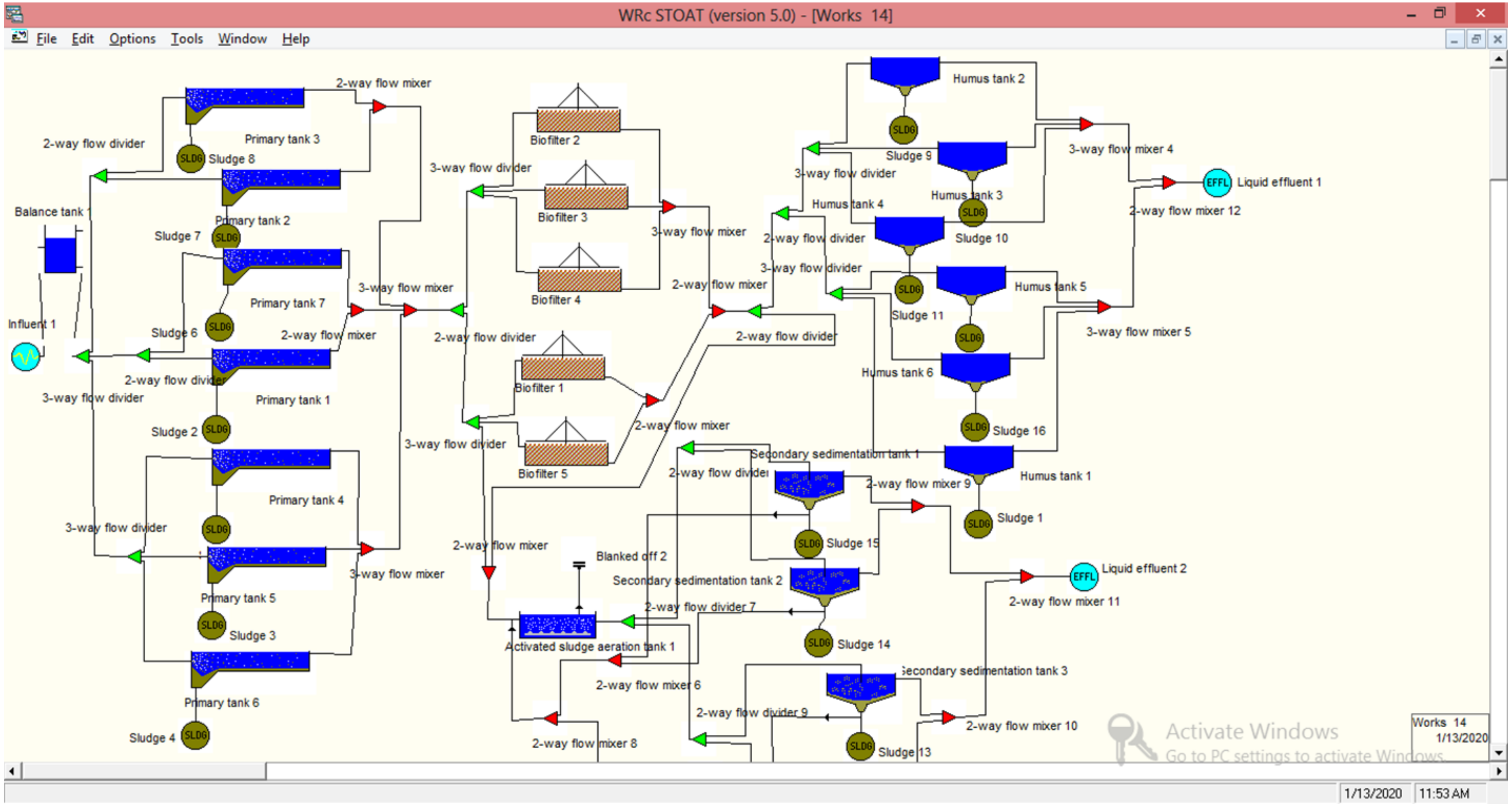


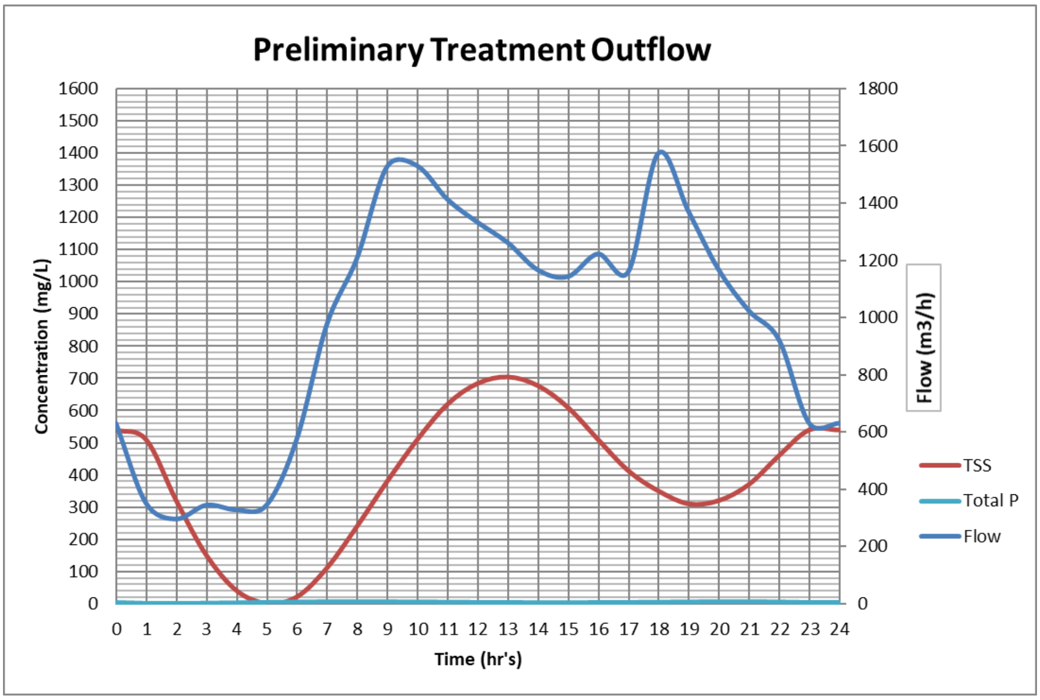
Figure 3.9. 1: STOAT model: Equalised treatment process flow diagram

The attenuation required for the equalization process was modelled based on a variable volume configuration, whereby the basin is configured to release a designated effluent flow to the downstream unit treatment processes. The release in flows considered were average daily flow rate as well as increased rates of 10% to 25% above the average daily flow, at 5% increments of flow.

As the equalization basin is positioned directly downstream of the Preliminary Treatment outflow, unequalised and equalised scenarios for the latter are identical. The equalised simulation therefore commences with the equalization basin followed by the unit processes downstream from that in the treatment train, the first being the Primary settlers.

Figures 3.8.3.1a to 3.8.3.1d illustrate the various unit processes pollutant effluent concentrations. For the graphical representation of pollution parameter results two separate graphs illustrations with respect to flow are used, being: 1) Total P and TSS; and 2) COD, Total N and Ammonia.

Tables 3.8.5 to 3.8.



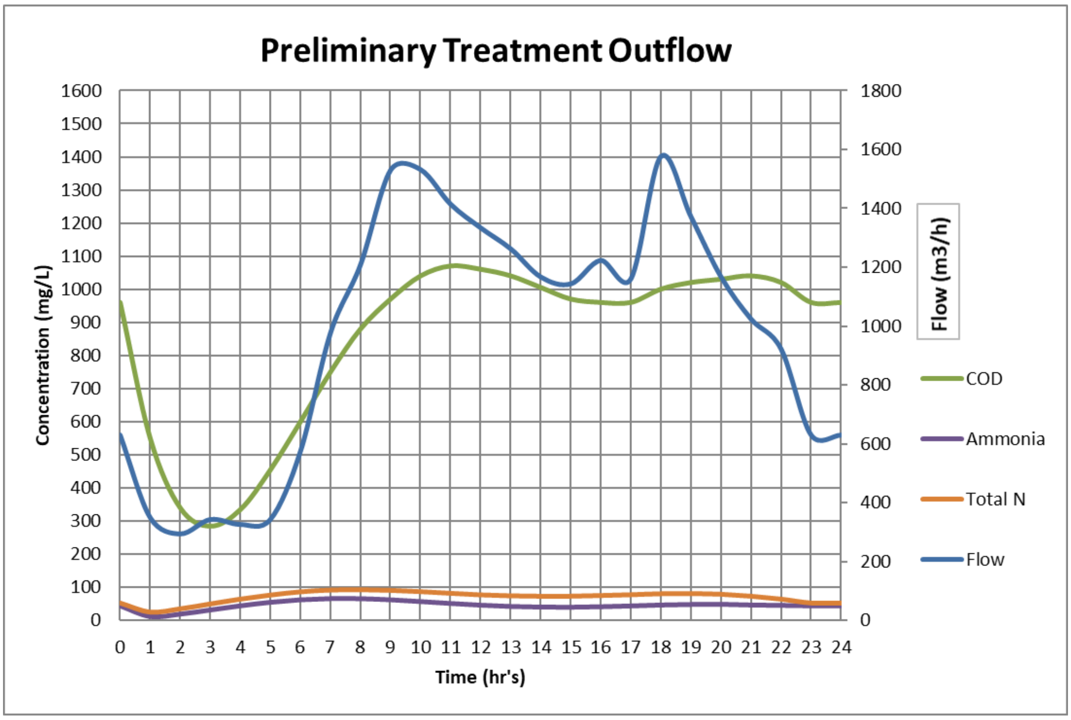


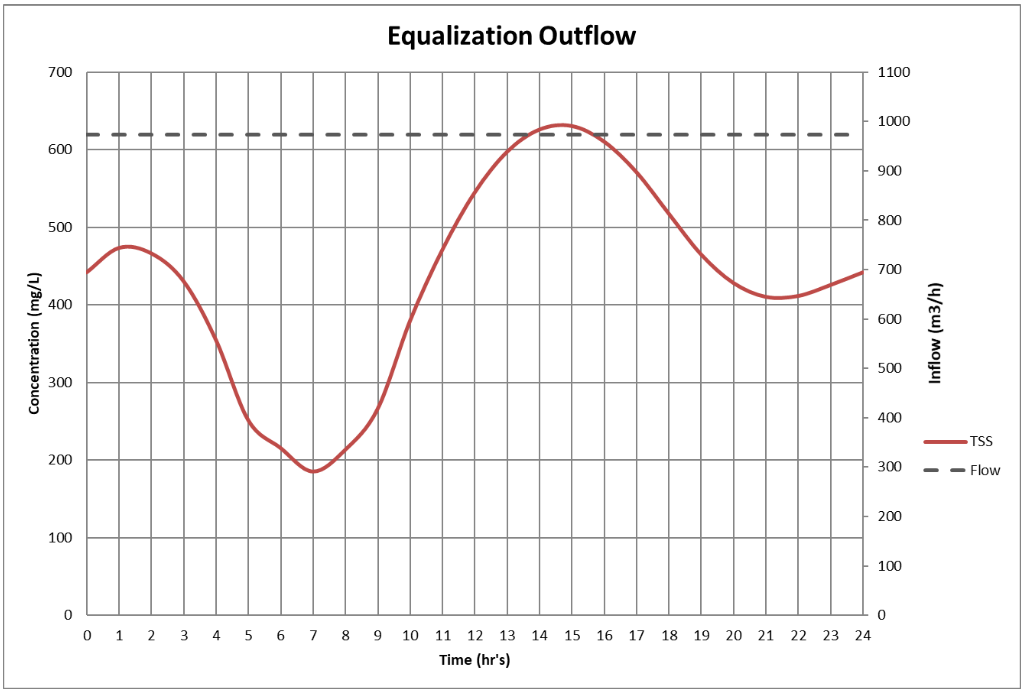
Figure 3.9. 2: STOAT simulation results: Preliminary Treatment Outflow:

1. Flow; TSS & Total P
2. Flow; COD; NH4; Total N

Table 3.8. 5: STOAT simulation results: Preliminary Treatment Outflow:

Flow and Pollutant concentration for Equalised Treatment

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	397.2	845.8	45.8	5.0	72.2
Max Peak (P)	705.0	1070.0	65.0	6.5	94.0
Min Peak (P)	0.0	285.0	13.0	2.2	25.0
(Max) Variation vs Average (%)	77.5	26.5	41.8	31.1	30.1
(Min) Variation vs Average (%)	100.0	66.3	71.6	55.2	65.4



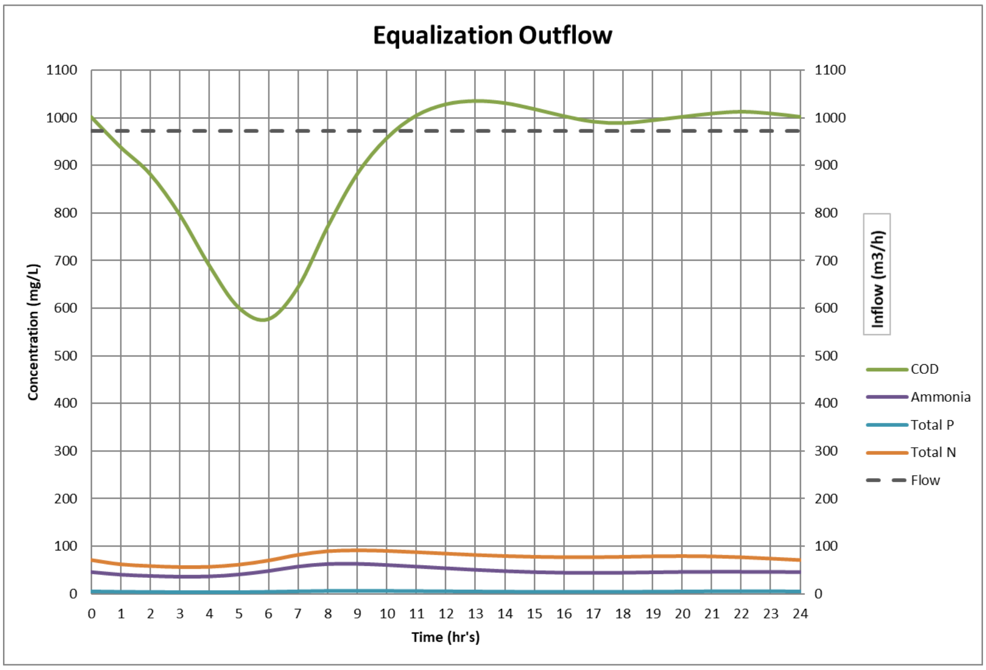
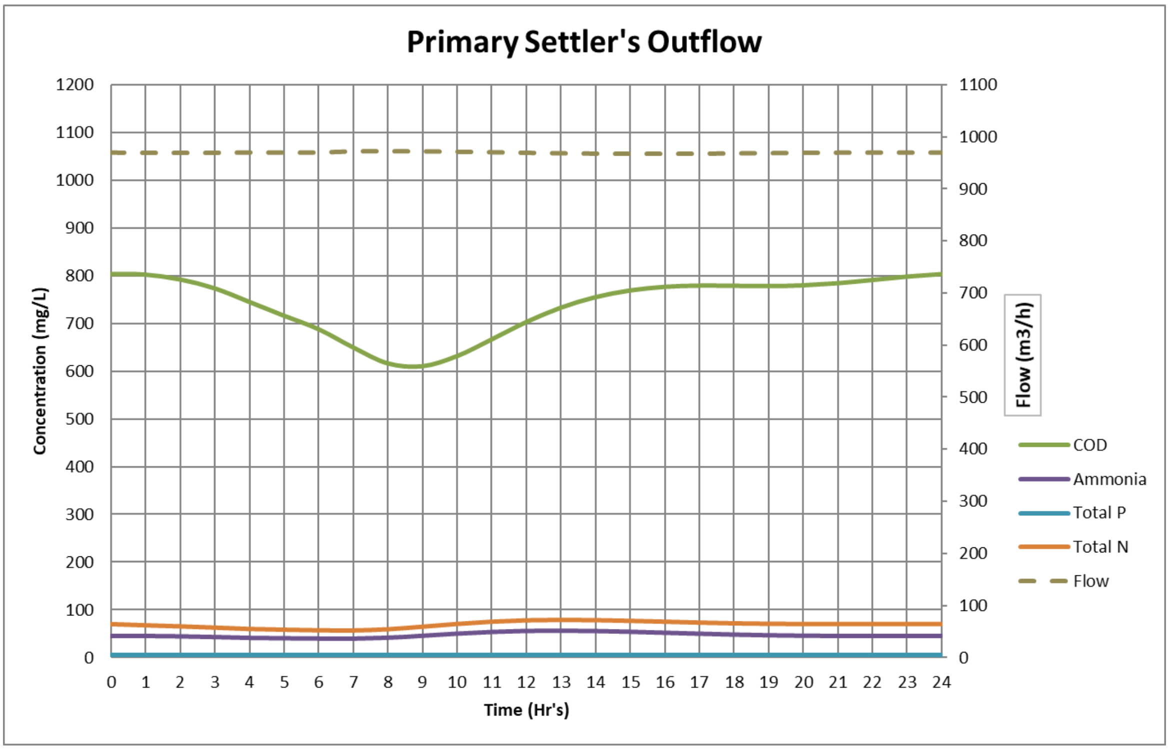


Figure 3.9. 3: STOAT simulation results: Equalization Outflow:
1. Flow & TSS
2. Flow; COD; NH4; Total N & Total P

Table 3.8. 6: STOAT simulation results: Equalization Outflow:

Flow and Pollutant concentrations for Equalised Treatment

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	433	912	48	5	76
Max Peak (P)	631	1036	63	6	92
Min Peak (P)	185	576	36	4	56
(Max) Variation vs Average (%)	46	14	32	22	21
(Min) Variation vs Average (%)	57	37	25	21	26



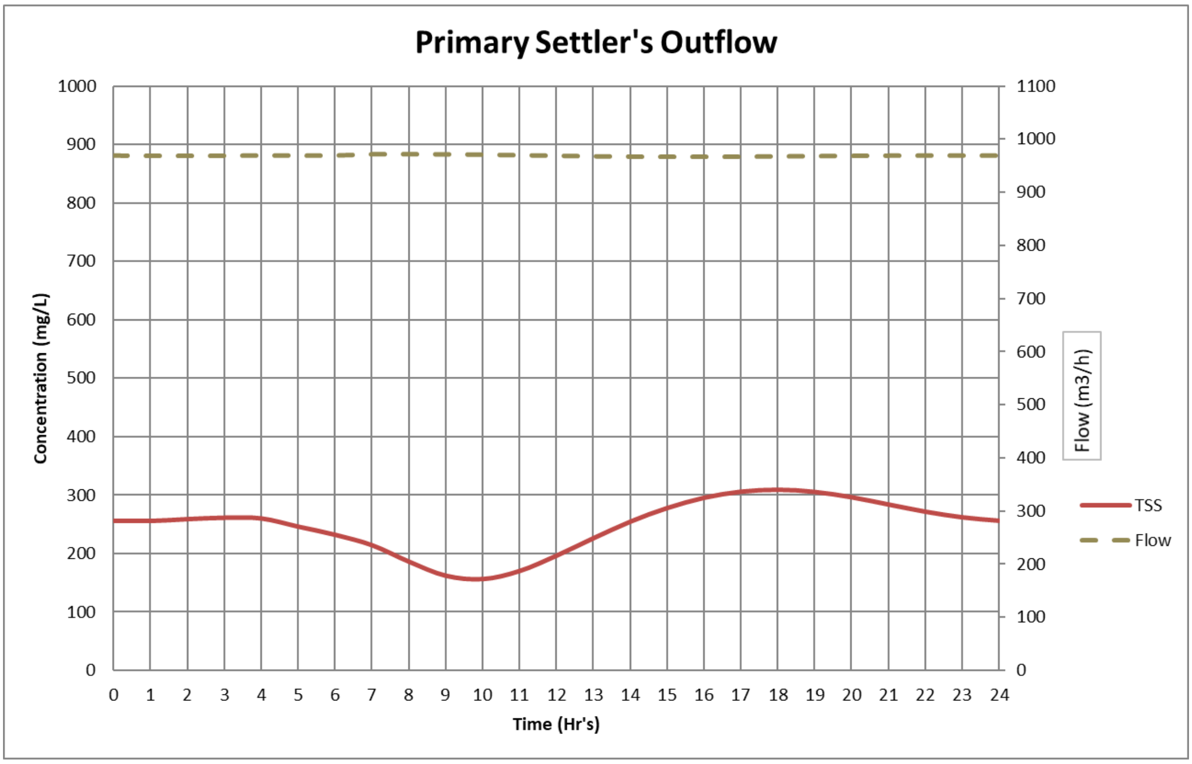
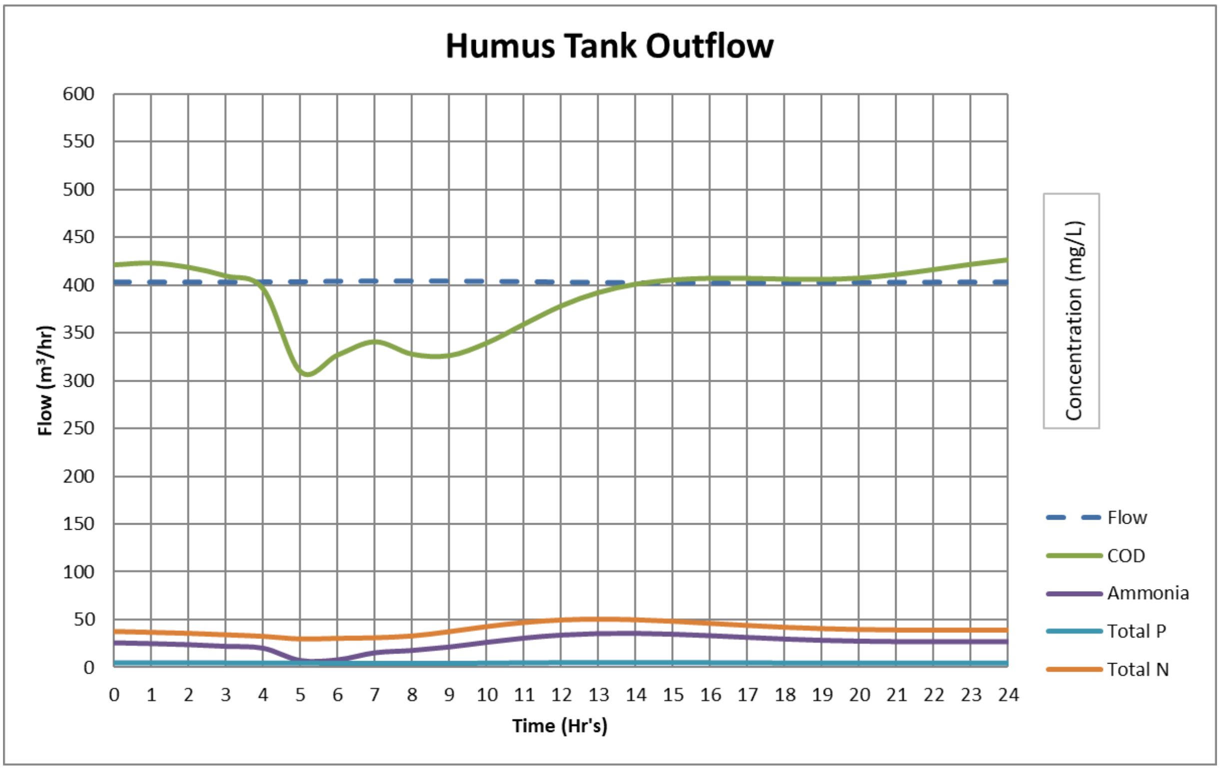


Figure 3.9. 4: STOAT simulation results: Primary Settler Outflow:
1. Flow; COD; NH4; Total P & Total N
2. Flow; TSS

Table 3.8. 7: STOAT simulation results: Primary Settler Outflow:
Flow and Pollutant concentrations for Primary Treatment

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	218	728	43	5	65
Max Peak (P)	310	803	56	6	79
Min Peak (P)	157	610	40	5	56
(Max) Variation vs Average (%)	42	10	31	11	21
(Min) Variation vs Average (%)	28	16	7	10	14



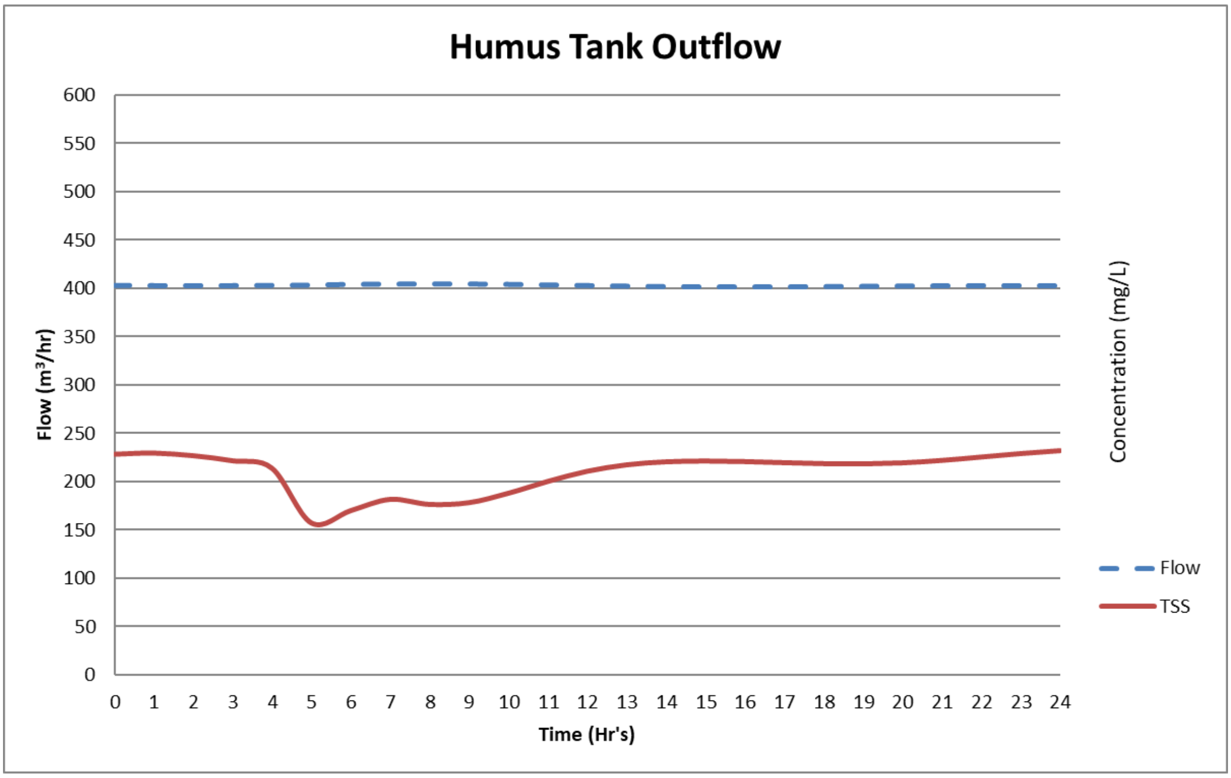


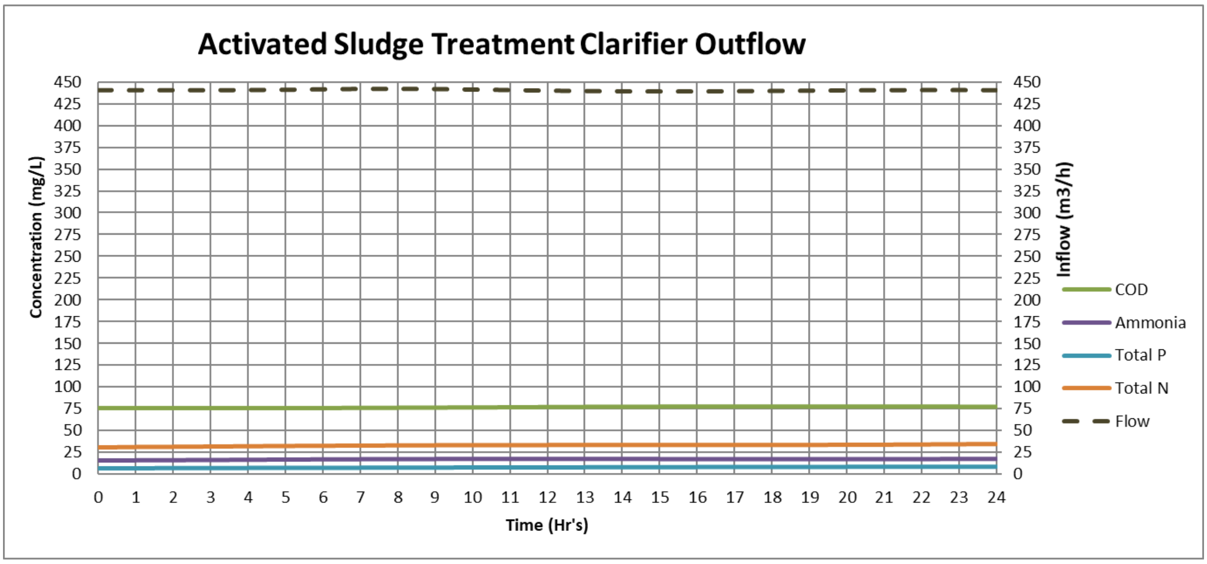
Figure 3.9. 5: STOAT simulation results: Humus Tank Outflow:

1. Flow; COD; NH4; Total P & Total N
2. Flow; TSS

Table 3.8. 8: STOAT simulation results: Humus Tank Outflow:

Flow and Pollutant concentration for Equalised Treatment

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	198.2	316.0	15.4	5.2	38.0
Max Peak (P)	232.2	426.7	35.4	5.8	50.5
Min Peak (P)	157.0	309.9	7.1	4.6	30.0
(Max) Variation vs Average (%)	17.1	35.0	129.9	10.8	32.8
(Min) Variation vs Average (%)	20.8	1.9	54.2	10.7	21.1



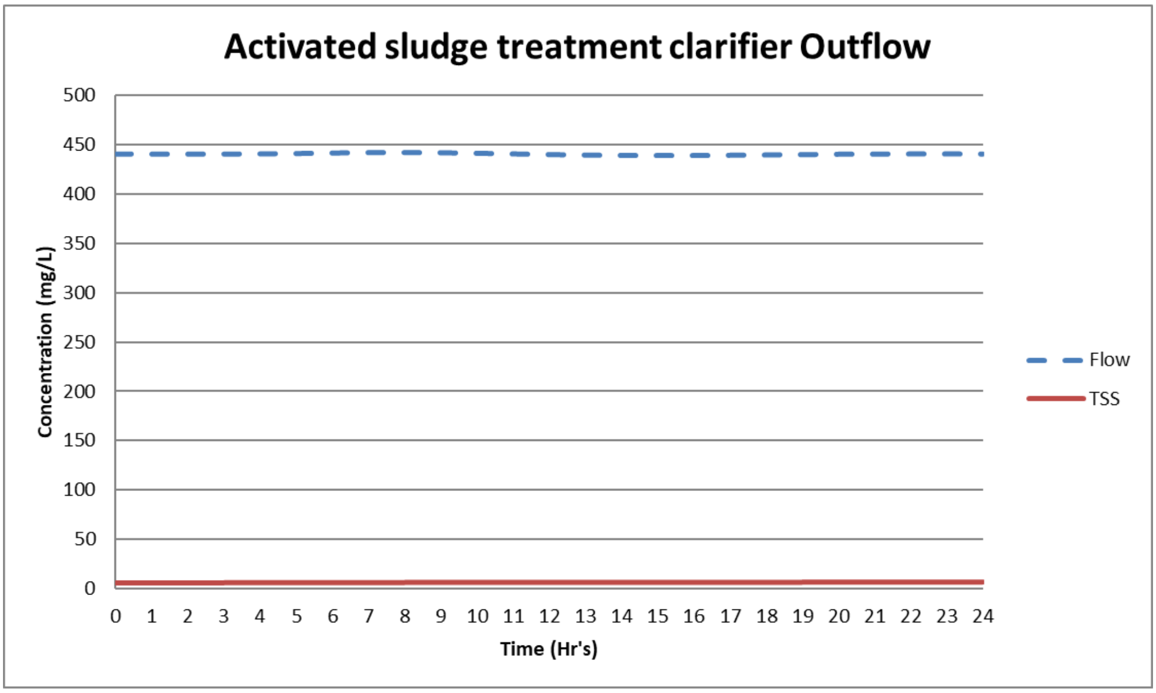


Figure 3.9. 6: STOAT simulation results: Secondary Settler Outflow:

1. Flow; COD; NH4; Total P & Total N
2. Flow; TSS

Table 3.8. 9: STOAT simulation results: Secondary Settler Outflow: Flow and Pollutant concentration for Secondary Settler Treatment

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	6.4	69.6	1.0	0.9	2.3
Max Peak (P)	6.6	71.1	1.9	1.0	2.5
Min Peak (P)	5.7	67.7	0.2	0.8	2.1
(Max) Variation vs Average	3.2	2.1	87.3	11.6	10.1
(Min) Variation vs Average	10.4	2.7	83.3	13.4	5.4

3.8.3 Treatment plant efficiency: Comparison of Unequalised vs Equalized scenarios

For pollutant removal efficiency purposes, the study has considered two stages, viz 1) an intermediate stage after the Primary settlers; and 2) for the plant overall level (after the Secondary settlers).

The treatment plant pollutant concentrations for the outflows of the Primary and Secondary settlers obtained (based on the STOAT model simulations), are given in Tables 3.9.1.1 and 3.9.1.2 for unequalised and equalised operational scenarios respectively.

Table 3.9.1. 1: Average pollutant concentration: Primary Settler's outflow

	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Othophosphate (mg/l)	TKN (mg/l)
Unequalized Operational scenario	243	753	46	5	68
Equalized Operational scenario	218	728	43	5	65

Table 3.9.1. 2: Average pollutant concentration: Secondary Settler's outflow

Operational scenario	**Total SS (mg/l)**	**Total COD (mg/l)**	**Ammonia (mg/l)**	**Othophosphate (mg/l)**	**TKN (mg/l)**
Unequalized	18.99	360.06	22.75	4.73	27.52
Equalized	8.62	20.70	0.57	7.56	6.93

CHAPTER 4

4. RESULTS AND DISCUSSION

4.1 Current operation compared to regulated effluent standards

The laboratory pollutant test result data obtained for the various unit process effluents are given in Appendix 3. The daily average concentrations derived from a best fit poly curve regression analysis are summarized in Table 4.1.

It must be borne in mind that these results are for a single day and plant operation dynamics varies daily. However, the results can be considered as indicative and not a true representative quantification and therefore conclusions made are focussed on the general pollution removal trend over consecutive unit processes employed only.

Table 4.1. 1: The current treatment operation: Average pollutant concentrations outflow

	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Inlet Works	397.240	840.80	43.996	5.161	69.352
PST outflow	374.29	762.14	46.57	4.91	75.86
Humus Tank	3081.43	3668.57	19.29	77.86	170.86
SST outflow	125.71	26.00	1.00	0.86	2.10

Table 4.1. 2: The current treatment operation: Variations for hydraulic and pollutants concentrations

Unit Process	TSS (mg/L)		COD (mg/L)		Ammonia (mg/L)		Total P (mg/L)		Total N (mg/L)		Flow (m^3/hr)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Inlet works	110	620	285	1070	32	65	2.9	6.3	50	93	295	1574.68
PST	130	630	415	1190	44	56	4.1	5.7	68	83		
Humus Tank	710	6970	850	7080	15	26	26	135	92	260		
SST	10	580	24	31	<0.15	1.5	1.5	1.2	1.3	3.4		

Commencing with Preliminary treatment (inlet works) and the PST's effluent, solids removal in the latter result in reduced levels of both TSS (from 397.24 mg/L to 374.29 mg/L) and COD (from 840.80 mg/L to 762.14 mg/L) as expected. However, the humus tank effluent reflects significant increases of TSS (from 374.29 mg/L to 3081.43 mg/L) and COD (from 762.14 mg/L to 3668.57 mg/L).

Over the PST's the reduction in Ammonia, Phosphorus and Nitrogen are negligible as no significant biochemical processes occur. As mentioned before, being daily average values considered, the slightly higher or lower values compared to PST influent can be attributed to this.

Considering the Secondary settler (SST) effluent (being also the plant final effluent), pollutant levels have reduced compared to that of the plant influent. Although COD, Ammonia, Phosphorus and Nitrogen have clearly reduced to regulated effluent standard levels (comparative values of 75mg/L, 10mg/L, 2mg/L and 10mg/L respectively), TSS levels exceed required effluent standards of 25 mg/L. Again, effluent values mentioned are for a single 24-hour period and the dynamics of treatment is not being fully reflected in a steady state result considered here.

- The TSS concentration varies between 620.0 mg/L and 110 mg/L at the inlets and between 630.0 mg/L and 130 mg/L at the primary settler's outflow, amounting to approximately an 83% variation at inlets and 78% variation for primary settlers' outflow. The humus tank has a TSS variation of 90%.
- The COD concentration at the Inlets varies between 285 mg/L and 1070 mg/L, which equates to a 73% variation and 65% variations at the primary settler's outflow.
- The ammonia concentration varies between 32 mg/L and 65 mg/L at the Inlet works, being approximately 51% variations.
- Total phosphorus concentration at the inlets varies between 2.9 mg/L and 6.3 mg/L, amounting to 54% variation and the primary settler's outflow at 4.1 mg/L and 5.7 mg/L being 28% variation respectively.
- Total nitrogen concentration varies between 50 mg/L and 93 mg/L at the inlets and between 68 mg/L and 83 mg/L at the primary settler's outflow amounting to approximately 46% and 18% respectively.

4.2 Equalization process attenuation capacity: Summary of graphical and numerical methodologies used

The equalization process attenuation capacity determination was done using both graphical and numerical methods of analysis with corresponding results presented in table 4.2.1.

Table 4.2. 1: Summary of Basin attenuation volumes required (excluding contingency allowance: Numerical Analysis

Release Flow rates	Graphical analysis: Basin capacity required (m^3)	Numerical analysis: Basin capacity required (m^3)
Average daily flow	4455.01	4455.41
Average daily flow + 10%	3031.85	3462.71
Average daily flow + 15%	3141.61	3141.66
Average daily flow + 20%	2703.91	2703.86
Average daily flow + 26%	3044.99	2611.6

The attenuation basin capacity volumes for the graphical and numerical methods are slightly different. This can be attributed to the numerical time-step analysis being more accurate compared to the graphical analysis.

When comparing the two sets of results it is evident that the attenuation capacities required for the two different methodologies are of similar magnitude varying at most by 12%. The difference in values is attributed by the numerical time-step analysis that assume a linear progression in flow.

4.3 Comparison of STOAT model simulation: Unequalised vs Equalised process configurations

The STOAT model simulation results for Preliminary treatment, Primary settlers, Humus tanks and Secondary clarifiers obtained for unequalised and equalised process configurations are discussed next.

Table 3.7.1.1 to table 3.7.7.5 gives a summary of unit process effluent pollutant results under the different basin release flow rates.

Tables 3.8.2 to table 3.8.9 illustrate a summary for unit process effluent pollutant concentration results based on the use of the STOAT simulation model, for equalised and unequalised scenarios.

- Preliminary treatment outflow

The level of pollutant load removal in the preliminary treatment for both scenarios are similar. This is obviously due to the equalization process basin being located downstream of preliminary treatment it does not impact any upstream process thereof.

- Primary settler outflow

There is a change in the level of pollutant load removal in the primary treatment for the equalised compared to the unequalised scenario. This is obviously due to the equalization process basin being located upstream of the primary treatment. By comparing the primary treatment process for unequalised (in table 3.8.2) and equalised (in table 3.8.7) configurations, the process average effluent pollutant concentration has reduced from 233mg/L to 210mg/L. This represents a reduction of approximately 10% if an equalization process is introduced.

- Humus tank outflow

The current humus tank operation based on effluent samples tested, illustrates poor treatment performance as compared to that of the STOAT simulation software. It indicates high values in both the pollutants except for ammonia which indicates a reduction in concentration load. By comparing humus tank operation in STOAT simulation software for unequalised (in table 3.8.3) and equalised (in table 3.8.8) configurations, the process average effluent pollutant concentration has reduced from 138mg/L to 115mg/L. This represents a reduction of approximately 17% if an equalization process is introduced.

- Activated sludge secondary settler outflow

Activated sludge secondary settler operation in STOAT simulation software for unequalised (in table 3.8.4) and equalised (in table 3.8.9) configurations shows that the process average effluent pollutant concentration has reduced from 87mg/L to 16mg/L. This represents a reduction of approximately 82% if an equalization process is introduced.

4.3.1 Equalization impact on diurnal effluent pollutant concentration variation with the daily average value

By comparison of Tables 3.8.2 and 3.8.9 it is evident that the equalization (attenuated) process pollutant concentration variation with respect to the average daily value, are impacted by equalization for the different pollutants, as follows:

- The TSS concentration reduced from 36.9% and 47.3% to 34% and 37% for the diurnal maximum and minimum values respectively. The average daily concentration of TSS reduced from 18.99mg/L to 6.4 mg/L, amounting to a 66% reduction.

- The COD concentration reduced from 16.5% and 60.5% to about 2.1% and 2.7% for the diurnal maximum and minimum values respectively. The average COD concentration reduced from 360.06 mg/L to 69.6 mg/L which equates to an 80% reduction.

- The ammonia concentration reduced from 28.3mg/L maximum and 19.4 mg/L minimum to about 7.8mg/L maximum and 2.6mg/L minimum. The average ammonia concentration achieved with equalization was 3.0 mg/L compared to 22.71 mg/L without equalization.

- Total phosphorus concentration variation was reduced from 52% maximum and 36.4% minimum to 11.6% maximum and 10.0% minimum.

- The Total nitrogen concentration variation was reduced to 10.1 mg/L maximum and 5.4 mg/L minimum from 25.4mg/L maximum and 16.8mg/L minimum, while the average value reduced to 2.3 mg/L from 27.52 mg/L.

4.3.2 Comparison of Unequalised vs Equalised treatment over PST's and Secondary clarifiers: STOAT model simulations.

➢ **Primary settler's**

By comparison of tables 4.3.2.1 and 4.3.2.2 it is evident that the equalization (attenuated) process impact on primary settler's outflow pollutant parameter loading achieved, is as follows:

(It is assumed that system internal flow losses are minimal and for the PST's that influent and effluent flows over the unit process remains constant).

Table 4.3.2. 1: STOAT simulation results: Primary Settler Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	243	753	46	5	68
Max Peak (P)	349	835	58	6	81
Min Peak (P)	135	562	39	5	53
(Max) Variation vs Average (%)	44	11	24	12	20
(Min) Variation vs Average (%)	44	25	16	13	22

Table 4.3.2. 2: STOAT simulation results: Primary Settler Outflow - Equalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	218	728	43	5	65
Max Peak (P)	310	803	56	6	79
Min Peak (P)	157	610	40	5	56
(Max) Variation vs Average (%)	42	10	31	11	21
(Min) Variation vs Average (%)	28	16	7	10	14

- The TSS average daily concentration was reduced from 243mg/L to 218 mg/L, amounting to approximately a 10% increase in removal level.
- The average COD concentration reduced from 753 mg/L to 728 mg/L, which equates to a 3% increase in removal.
- The average ammonia concentration reduced from 46 mg/L to 43 mg/L, being approximately a 7% increase in removal level.
- Total average phosphorus concentration remained the same at 5 mg/L and therefore no improvement in removal level was achieved.
- Total average nitrogen concentration reduced to 65 mg/L from 68 mg/L and the improved removal level is approximately 4%.

- **Secondary clarifiers**

By comparison of Tables 4.3.2.3 and 4.3.2.4 it is evident that the equalization (attenuated) process impact on primary settler outflow pollutant loading is as follows:

(It is assumed that system internal flow losses are minimal and for the secondary clarifiers that influent and effluent flows over the unit process remains constant).

Table 4.3.2. 3: STOAT simulation results: Secondary Settler Outflow - Unequalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	18.99	360.06	22.75	4.73	27.52
Max Peak (P)	26	419.3	28.3	5.37	34.52
Min Peak (P)	10	142.3	19.4	4.07	22.4
(Max) Variation vs Average (%)	36.9	16.5	24.4	13.5	25.4
(Min) Variation vs Average (%)	47.3	60.5	14.7	14.0	18.6

Table 4.3.2. 4: STOAT simulation results: Secondary Settler Outflow - Equalised

Parameter	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Average (A)	8.6	20.7	0.6	7.6	6.9
Max Peak (P)	9.8	67.7	1.7	11.8	10.6
Min Peak (P)	5.7	15.5	0.1	0.8	2.1
(Max) Variation vs Average (%)	13.6	227.0	197.6	55.7	52.7
(Min) Variation vs Average (%)	33.9	25.2	80.7	88.8	69.1

- The TSS average daily concentration was reduced from approximately 19 mg/L to 8.6 mg/L, amounting to a 55% increase in removal level.
- The average COD concentration reduced from 360 mg/L to approximately 21 mg/L, which equates to nearly a 94% increase in removal.
- The average ammonia concentration reduced from nearly 23 mg/L to 0.6 mg/L, being approximately a 97% increase in removal level.
- Total average phosphorus concentration increased from just under 5 mg/L to approximately 7.6 mg/L and therefore the improved removal level is -52 %.
- Total average nitrogen concentration reduced to just above 6.9 mg/L from approximately 28 mg/L. This gives an improved removal level of approximately 75%.

4.3.3 Comparison of pollutant removal efficiency of Primary settlers and overall treatment plant: STOAT model simulation results

Comparison of the treatment plant influent pollutant load and that of the Primary settler allowed for a first stage over this process, while compared to Secondary clarifier effluents an overall plant removal efficiency can be ascertained for the various pollutants. The comparison was done on an average daily flow and pollutant concentration basis for the representative design day considered in the study based on the STOAT model simulation results.

As mentioned before, the Biofilter process act as a pre-treatment of a portion of the PST effluent as the former process effluent is conveyed to the Activated sludge process for further advanced treatment. The total flow therefore end-up as final effluent of the activated sludge secondary clarifiers of the plant. The influent flow because of the process configuration for both Primary settlers and Secondary clarifiers are considered as remaining the same as that of the plant influent, as treatment system internal flow losses are considered as being minimal.

The removal efficiencies of the Primary settlers and Secondary settler are determined next.

Individual pollutant removal efficiencies

- **Primary settler's**
  - Unequalised scenario

Considering the average pollutant results of Preliminary treatment and Primary settler outflows for unequalised conditions as contained in tables 3.8.1 and 3.8.2, the removal efficiency can be determined as follows:

TSS	= (397 - 243)/397 x 100 = 38.7 %
Total COD	= (845.8 - 753)/845.8 x 100 = 10.9 %
Ammonia	= (45.8 - 46)/45.8 x 100 ≈ zero %
Total P	= (5 – 5)/5 x 100 ≈ zero %
Total N	= (72.2 - 68)/72.2 x 100 = 5.8 %

➢ Equalised scenario

Considering the average pollutant results of Preliminary treatment and Primary settler outflows for equalised conditions as contained in tables 3.8.6 and 3.8.7, the removal efficiency can be determined as follows:

(The summarized result is given in Table 4.3.3.1)

TSS = (433 - 218)/433 x 100 = 49.6 %

Total COD = (912 - 728)/912 x 100 = 20.2 %

Ammonia = (48 - 43)/48 x 100 = 10.4 %

Total P = (5 – 5)/5 x 100 ≈ zero %

Total N = (76 - 65)/76 x 100 = 14.5 %

Table 4.3.3. 1: STOAT model: Primary settler pollutant removal efficiencies

Process	Pollutant removal efficiency (%)				
	TSS	COD	NH3	P	N
Unequalized	38.8	10.9	0	0	5.8
Equalised	49.6	20.2	10.4	0	14.5

- **Secondary clarifiers**

➢ Unequalised scenario

Considering the average pollutant results of Preliminary treatment and Secondary clarifier outflows for unequalised conditions as contained in tables 3.8.1 and 3.8.4, the removal efficiency can be determined as follows:

TSS = (397.2 – 18.99)/397.2 x 100 = 95.2 %

Total COD = (845.8 – 360.06)/845.8 x 100 = 57.4 %

Ammonia = (45.8 – 22.75)/45.8 x 100 = 50.3 %

Total P = (5 – 4.73/5 x 100 = 5.4 %

Total N = (72.2 – 27.52)/72.2 x 100 = 61.9 %

- Equalised scenario

Considering the average pollutant results of the Equalisation basin and Secondary clarifier outflows for equalised conditions as contained in Tables 3.8.6 and 3.8.9, the removal efficiency can be determined as follows:

TSS = (433 – 8.6)/433 x 100 = 98 %

Total COD = (912 – 20.5)/912 x 100 = 98 %

Ammonia = (48 – 0.57)/48 x 100 = 98.8 %

Total P = (5 – 7.6)/5 x 100 = -52 %

Total N = (76 – 6.8)/76 x 100 = 91 %

Table 4.3.3. 2: STOAT model: Secondary settler pollutant removal efficiencies

Process	**Pollutant removal efficiency (%)**				
	TSS	**COD**	**Ammonia**	**Phosphorus**	**Nitrogen**
Unequalised	95.2	57.4	50.3	5.4	61.9
Equalised	98	98	98.8	-52	91

Combined removal efficiencies based on weighted average ratio of individual pollutant efficiencies to the concentration ratio.

- **Primary settler's**

The combined weighted average removal efficiencies established for unequalised and equalised scenarios are given in Table 4.2.3.

Table 4.3.3. 3: STOAT model: Primary settler combined pollutant removal efficiencies

Operation scenario	Effluent pollutant concentrations (mg/L)					
	TSS	COD	Ammonia	Phosphorus	Nitrogen	**SUM**
Unequalised	243	753	46	5	68	1115
Equalised	218	728	43	5	65	1059
	Pollutant individual removal efficiencies (%)					
Unequalised	38.7	10.9	0	0	5.8	
Equalised	45.18	13.9	6.1	0	9.9	
	Pollutant contribution to combined removal efficiency (%)					**Combined efficiency**
Unequalised	8.43	7.36	0.00	0.00	0.35	16.15
Equalised	9.30	9.56	0.25	0.00	0.61	19.71

- **Secondary clarifiers**

The combined weighted average removal efficiencies established for unequalised and equalised scenarios are given in Table 4.2.4.

Table 4.3.3. 4: STOAT model: Secondary settler combined pollutant removal efficiencies

Operation scenario	**Effluent pollutant concentrations (mg/L)**					
	TSS	COD	Ammonia	Phosphorus	Nitrogen	**SUM**
Unequalised	18.99	360.06	22.75	4.73	27.52	434.05
Equalised	6.4	69.6	1	0.9	2.3	80.2
	Pollutant individual removal efficiencies (%)					
Unequalised	95.2	57.4	50.3	5.4	61.9	
Equalised	98	98	98.8	-52	91	
	Pollutant contribution to combined removal efficiency (%)					**Combined efficiency (%)**
Unequalised	4.17	47.62	2.64	0.06	3.92	58.40
Equalised	7.82	85.05	1.23	-0.58	2.61	96.13

Using the Gammams treatment train configuration and comparing unequalised and equalised scenarios based on STOAT model simulations as illustrated in Tables 4.3.3.3 to 4.3.3.4, it can clearly be concluded that the treatment train with an equalization process has the potential to improve the treatment performance of the plant.

The results in Tables 4.3.3.5 and 4.3.3.6 reflect STOAT model simulation results for unequalised and equalised scenarios respectively.

Table 4.3.3. 5: STOAT unequalised simulation: Pollutant concentrations: Primary settler's outflow

	Total SS (mg/l)	**Total COD (mg/l)**	**Ammonia (mg/l)**	**Total P (mg/l)**	**Total N (mg/l)**
Inlet Works	397.240	840.80	43.996	5.161	69.352
PST outflow	243.06	752.85	46.47	5.26	68.05
Humus Tank	216.66	396.99	28.61	5.28	40.69
SST outflow	0.68	1.11	0.00	0.00	0.11

Table 4.3.3. 6: Stoat unequalised simulation: Pollutant concentrations: Primary settler's outflow

	Total SS (mg/l)	Total COD (mg/l)	Ammonia (mg/l)	Total P (mg/l)	Total N (mg/l)
Inlet Works	0.000	0.00	0.000	0.000	0.000
PST outflow	131.23	9.30	0.10	-0.35	7.80
Humus Tank	2864.77	3271.58	-9.33	72.57	130.17
SST outflow	125.04	24.89	1.00	0.86	1.99

4.4 Equalization attenuation capacity requirements vs. basin release flow rate

Table 4.4.1 gives attenuation capacity requirements for increased basin release flow rates and the resulting pollutant diurnal pattern variation.

Although the increase in the attenuation basin release flow rate result in a reduction in required attenuation capacity, this result in higher levels of pollutant diurnal pattern variations.

Table 4.4. 1: Equalization basin release flow rate vs attenuation capacity and diurnal pollutant pattern variation

Basin release flow rate (units)	Attenuation capacity required (m^3)	Pollutant diurnal pattern variation (%)				
		TSS	COD	NH3	P	N
Ave daily flow (ADF)	4455.41	57	22	36	21	21
ADF Plus 10%	3462.71	69	34	33	36	33
ADF plus 15%	3141.66	76	39	36	40	37
ADF plus 20%	2703.86	82	44	38	45	43
ADF plus 25%	2611.60	82	50	40	62	54

The increase basin release flow rates do result in a reduction in required attenuation with lower associated construction cost benefit. However, the higher diurnal variation pattern variation can negatively impact on unit process pollutant removal efficiency as well as final plant effluent quality.

Careful consideration would therefore be necessary to determine a desired balance between basin release flow rate, associated attenuation capacity required and resulting plant treatment efficiency and final plant effluent quality standard compliance.

4.5 STOAT model simulation results: Comparison of pollutant removal efficiency of Primary settlers and the treatment plant as a whole.

From comparison of results in tables 4.3.2.1 and 4.3.2.2, the following conclusions are made:

1. Primary settler pollutant removal efficiency

Equalised treatment has increased removal efficiencies compared to the unequalised scenario. The increase for individual pollutants varies from 3% to approximately 7%, except for soluble Phosphorus being zero as this is not removed over Primary settlers. In case of the combined efficiency for all pollutants considered in the study, the increase amounts to approximately 4%.

2. Secondary settler pollutant removal efficiency

As secondary settling is preceded by biochemical processes in the treatment train, the removal efficiencies of pollutants are considerably larger than that observed at Primary settlers. Equalised treatment has resulted in increased removal efficiencies which is attributed to generally reduced diurnal pattern variations with near uniform diurnal pattern conditions in the activated sludge process and clarifiers.

For individual pollutant removal efficiency, the increase amounts to approximately 3% for TSS, while for COD and Total N it is 35%. For Ammonia and Phosphorus, it is 48% and 77% respectively. The combined removal efficiency inclusive for all pollutant studied. the efficiency increased from 58% to as high as 92% equating to a potential increase of approximately 34% should an inline equalization process be introduced.

3. Phased attenuation capacity employment through staged basin release flow rates.

Figure 4.5.1 illustrates relationship between required basin release flow rates for any attenuation capacity provided. It also allows for doing so in relation to future increased influent flow into the treatment plant (i.e. a 5-year growth in wastewater inflow).

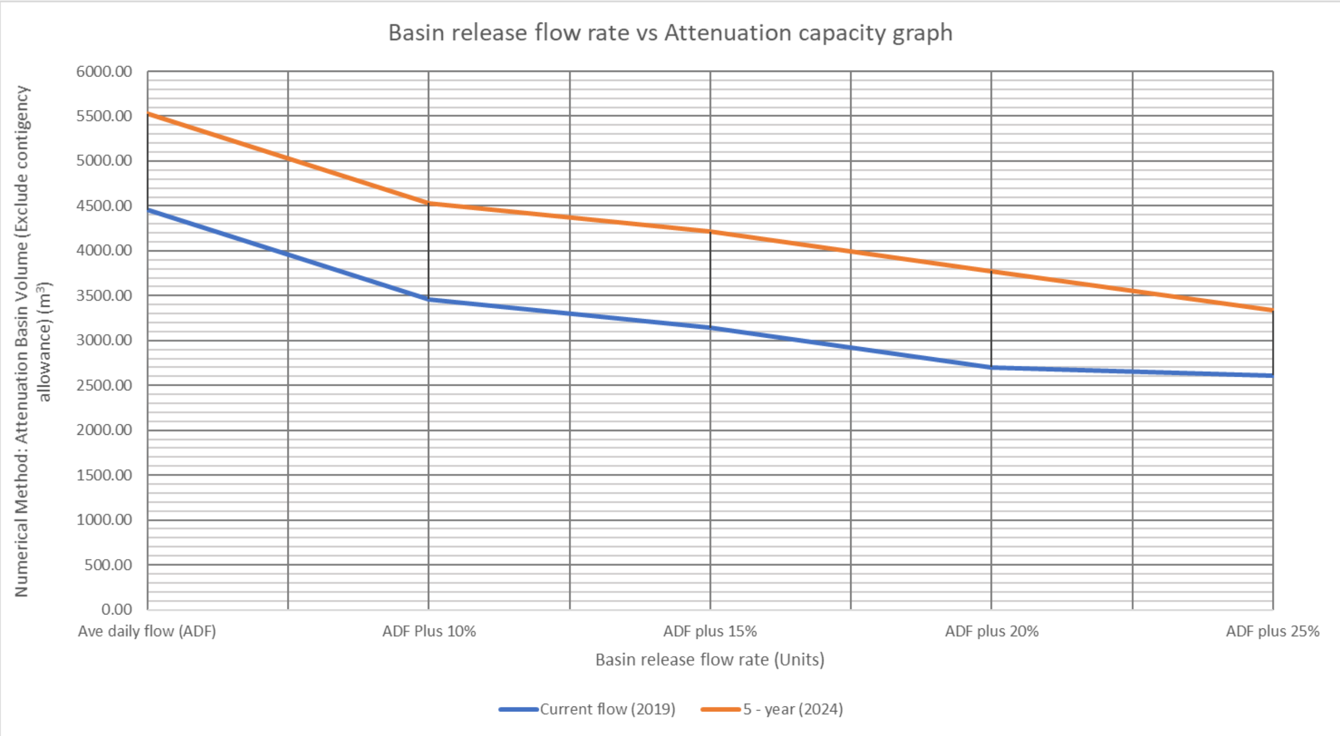


Figure 4.5. 1: Basin release flow rate vs attenuation capacity graph

From Figure 4.5.1, for a release flow rate equal to the plant current average daily influent flow (ADF), the required attenuation amounts to 4455.41m3 (excluding a contingency allowance). Based on the attenuation capacity mentioned, a release flow rate of approximately ADF plus 11% would be required to attenuate the future 5-year increase in wastewater flow.

Should a higher release flow rate be employed currently than ADF, the required attenuation capacity can be reduced to a value obtained from the graph and the corresponding release rate for the 5-year growth period from the curve labelled "5-year".

Figure 4.5.1 can be used to assist with decision making for finding a balance between attenuation capacity size and cost, and release flow rate required and compliance with regulated plant final effluent quality standards.

Due to the initial plant being designed for unequalised diurnal flow and pollutant patterns, some spare/buffer treatment capacity in unit process may be available as equalised diurnal patterns are less than unequalised as evident from the findings of this study.

However, as the influent flow to the plant increases over time, the hydraulic and biochemical treatment loading of existing unit process will increase accordingly, and any available buffer capacity will diminish in time.

CHAPTER 5

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Comparing the **current operation** with the regulated standards, it can be concluded that Preliminary treatment (inlet works) and commencing to the PST's effluent, solids removal in the latter results in reduced levels of both TSS and COD as expected, though still caused by huge daily pollutant diurnal load variations. However, the humus tank effluent reflects significant increases of the mentioned pollutants due to these tanks not being desludged regularly which require to be addressed for ensuring improved effluent quality. Over the PST's the reduction in Ammonia, Phosphorus and Nitrogen are negligible as no significant biochemical processes occur.

Considering the Secondary settler (SST) effluent (being the plant final effluent), pollutant levels have reduced compared to that of the plant influent. Although COD, Ammonia, Phosphorus and Nitrogen have clearly reduced to regulated effluent standard levels (comparative values of 75mg/L, 10mg/L, 2mg/L and 10mg/L respectively), TSS levels exceed required effluent standards of 25 mg/L.

The current treatment plant's diurnal hydraulic variation between 295 m^3/hr and 1575 m^3/hr, can be firmed by a release flow rate equal to the plant current average daily influent flow (ADF). This average daily influent flow (ADF) will then require an attenuation capacity amounts to 4455.41m^3 (excluding a contingency allowance) using both numerical and graphical analysis methodologies. Based on the attenuation capacity mentioned, a release flow rate of approximately ADF plus 11% would be required to attenuate the future 5-year increase in wastewater flow.

It can be concluded that, should a higher release flow rate be employed currently than ADF, the required attenuation capacity can be reduced. Although the increase in the attenuation basin release flow rate result in a reduction of the required attenuation capacity, this result in higher levels of pollutant diurnal pattern variations.

Secondary treatment at Gammams consists of a two-stream approach of employing low technology biological (trickling) filters and high technology activated sludge treatment. The former process was part of the initial plant, while the latter was introduced with the subsequent upgrading of the plant. Maintaining the lower technology biofilters stream and directing effluent therefrom to the activated sludge process, serves as pre-treatment and ensures a reduced load on the latter. This has overall treatment benefits of extended utilization of the low technology and associated minimal energy inputs required together with an increased bioreactor and associated high energy requirements if the higher activated sludge technology had to treat the full secondary treatment load alone being avoided.

Through the STOAT model simulation, the equalised primary treatment increased pollutant removal efficiencies to approximately 4% compared to the unequalised scenario, while the secondary treatment increased to approximately 34% should an inline equalization process be introduced. However, phosphorus demonstrated a negative pollutant removal efficiency with secondary treatment, due to the effluent concentration being higher than the influent to the activated sludge process.

From an operational point of view such a situation can be attributed to release of phosphate by aerobe bacteria back into solution due to a low sludge return cycle and its accumulation causing anaerobic conditions in the clarifier. The buffer capacity in the clarifiers receiving equalized reduced hydraulic load compared to the designed capacity for unequalised conditions, result in a longer retention period with associated anaerobic conditions and release of phosphate and increased levels thereof.

It must however be borne in mind that these efficiencies are based on simulations for a single day selected as representative of typical plant conditions. More extended-time simulations and specific model calibration of plant kinetics would be required for a more accurate and representative efficiency removal outcome. These mentioned results should therefore be indicative of improvement based on a first stage study only and not exact estimates whatsoever. It would be appropriate to assess general plant pollutant removal efficiency after doing extended-period simulations over a period of 1 month or more.

5.2 Recommendations

1. It is recommended that the City of Windhoek considers the introduction of an inline equalization process into the Gammams Water Care Works treatment train. The attenuation capacity can be sized based on a phased approach and facilitating increase in wastewater influent by increased flow rates over for example a 5-year period.

2. Also, regarding the provision of equalization attenuation capacity, this could be done in a phased modular fashion. As indicated in the study, the attenuation recommended if provided now (5400m^3), could continue to provide a reasonable level of attenuation by introducing an increase in flow release rate above average by approximately 15% and still achieve acceptable levels of plant performance.

3. It is further recommended that decision makers should consider an In-line equalization process, because this configuration allows all the influent to pass through the equalization basin, which will dampen the pollutant concentration more effectively. To avoid settling of suspended solids in the tank, it is recommended that tank agitation in terms of mechanical stirrer should be implemented.

4. It is further recommended that the existing Humus tanks of the plant be fixed to improve the treatment performance of the plant.
5. It is recommended that further studies be done in the following areas:

- Conceptualisation and formulation of a graphical analysis approach where variation of equalization basin release flow rates are considered. Particular focus on accuracy of the graphically determined attenuation capacity compared to that achieved with numerical analyses methodologies. Consideration could be given to determining a virtual uniform equivalent basin release rate such that attenuation capacity estimation equal to results obtained using a numerical analysis.

- Extended period simulations and calibration of the model incorporating both further steady state and dynamic analyses to replicate current plant effluent quality through improvements and better approximation of kinetics dynamics of biological processes.

- Consideration of recycle of equalization process effluent back to the basin inlet for determining the potential of achieving improved attenuation together with associated energy cost and operational implications.

- Development of a water management decision making tool for optimal water resource use particularly relevant to semi-arid areas such as Namibia. This includes compiling an integrated management model utilising appropriate simulation software to allow for holistic consideration of the full water cycle of surface water and groundwater resource use and losses, wastewater collection and treatment and return flow factors, wastewater reuse and reclamation as well as final disposal options.

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APPENDIX 1: Location of sampling points along the Gammams treatment train.

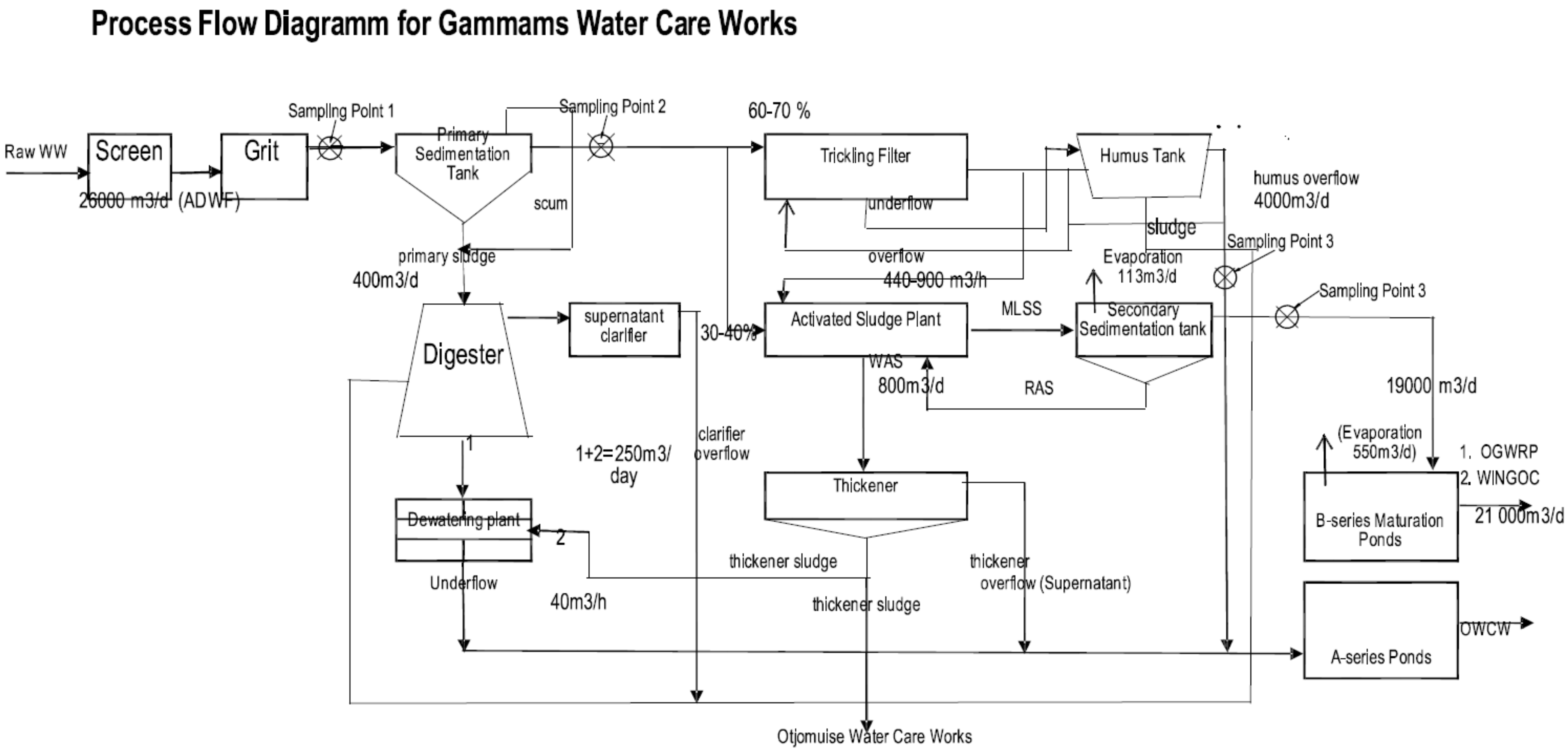


Figure 3. 1: Unit process flow diagram for Gammams Water Care Works

Determinant	Unit	Typical domestic wastewater characteristics
COD	$mgO_2/ℓ$	500-700
Suspended solids	mg/ℓ	200-350
Settleable solids	mℓ /ℓ	8-10
TKN	mg/ℓ	35-85
Phosphate	mg/ℓ	10-13
Ammonia	mg/ℓ	40-60

Figure 2.1: Typical domestic wastewater characteristics for South Africa, adopted by Namibia
(Niekerk, Seetal, Dama-Fakir,, Boyd, & Gaydon, December 2009)

APPENDIX 2: Gammams Water Care Works Unit processes sizes.

Table 2.8.2: Gammams Water Care Works Primary sedimentation tanks sizes and flow splits

Tank No.	Diameter (m)	Surface Area (m^2)	Volume (m^2)	Flow Split (%)
1 to 7	11	95	440	70 (10 each)
8	32	805	3700	30

No.	Stones media volume (m^3)	Stones Media surface area (m^2)	Flow Split (%)
1	2760	767	17
2	2760	767	17
3	3470	965	22
4	3470	965	22
5	3470	965	22

Table 2.8.3a: Gammams Water Care Works Trickling Filters sizes and flow splits

No.	Stones media volume (m^3)	Stones Media surface area (m^2)	Flow Split (%)
1	2760	767	17
2	2760	767	17
3	3470	965	22
4	3470	965	22
5	3470	965	22

Table 2.8.3b: Gammams Water Care Works Activated Sludge plant

Process component	No.	Capacity and Units
Anaerobic zone	2	1st: 759 m^3; 2nd: 1641 m^3 Total: 2400m^3
Anoxic Zone	2	1st: 583 m^3; 2nd: 1311 m^3 Total:1894 m^3
Aerobic Zone	4	1st: 583 m^3; 2nd: 1311 m^3; 3rd: 1311 m3; 4th: 1311 m3; Total:1894 m^3
Total Reactor volume		12057 m^3
Secondary clarifiers	4	Dia: 32m; Side depth; 3.5m
a-recycle	1	~60ML/d (one of the two pumps operating at 2500 m^3/hr)
r-cycle	1	~12.8ML/d (operating at 533 m^3/hr)
s-cycle	1	~17ML/d (operating at 708.33 m^3/hr)
Mixers	5	Anaerobic: 3 x 2.2kW Anoxic: 2 x 7.5kW
Aerators	10	1st aerobic: 2 x 90kW installed 2nd Aerobic: 2 x 75kW installed 3rd aerobic: 3 x 45kW installed 4th Aerobic: Diffused air 3 x 22kW blowers

APPENDIX 3: The analysis results for various pollutants considered in the thesis and their various sampling locations along the treatment train. Unit process operational outputs based on sampled tests done vs STOAT model.

Table 3.4. 1: Pollutant concentrations at the Inlet Works

Inlet Works							
Time (hrs)	COD (mg/l)	Ammonia (mg/l as N)	Othorphosphate (mg/l)	Total Kjeldahl nitrogen (TKN-N) (mg/l)	Total Suspended Solids (mg/l)	Total Solids: TS 105 (mg/l)	Total Dissolved Solids: TDS 180 (mg/l)
0:00	960	44	4.6	53	540	1400	860
4:00	285	32	2.9	50	150	890	740
8:00	750	65	6.3	93	110	850	740
12:00	1070	51	5.4	83	620	1640	1020
16:00	970	40	4.2	74	610	1440	830
20:00	1020	48	5.9	82	310	1350	1040
0:00	960	44	4.6	53	540	1400	860
Average	**859.29**	**46.29**	**4.84**	**69.71**	**411.43**	**1281.43**	**870**

Table 3.4. 2: Pollutant concentrations at the Primary Settling Tanks outflow

Primary Settling Tank Outflow							
Time (hrs)	COD (mg/l)	Ammonia (mg/l as N)	Othorphosphate (mg/l)	Total Kjeldahl nitrogen (TKN-N) (mg/l)	Total Suspended Solids (mg/l)	Total Solids: TS 105 (mg/l)	Total Dissolved Solids: TDS 180 (mg/l)
0:00	1190	45	5.7	77	630	1580	950
4:00	800	46	4.7	76	340	1280	940
8:00	415	44	4.1	68	130	980	850
12:00	520	56	5.5	83	220	1040	820
16:00	620	43	4.5	74	280	1100	820
20:00	600	47	4.2	76	390	1090	700
0:00	1190	45	5.7	77	630	1580	950
Average	**762.14**	**46.57**	**4.91**	**75.86**	**374.29**	**1235.71**	**861.43**

Table 3.4. 3: Pollutant concentrations at the Humus Tank outflow

Humus Tank							
Time (hrs)	COD (mg/l)	Ammonia (mg/l as N)	Othorphosphate (mg/l)	Total Kjeldahl nitrogen (TKN-N) (mg/l)	Total Suspended Solids (mg/l)	Total Solids: TS 105 (mg/l)	Total Dissolved Solids: TDS 180 (mg/l)
0:00	4200	17	81	185	3920	4920	1000
4:00	6730	17	98	260	3680	4740	1060
8:00	850	22	21	99	710	1580	870
12:00	1120	21	55	115	1080	1860	780
16:00	7080	26	135	260	6970	7880	910
20:00	1500	15	74	92	1290	2200	910
0:00	4200	17	81	185	3920	4920	1000
Average	**3668.57**	**19.29**	**77.86**	**170.86**	**3081.43**	**4014.29**	**932.86**

Table 3.4. 4: Pollutant concentrations at the Activated Sludge Process Clarifiers

Secondary Clarifiers							
Time (hrs)	COD (mg/l)	Ammonia (mg/l as N)	Othorphosphate (mg/l)	Total Kjeldahl nitrogen (TKN-N) (mg/l)	Total Suspended Solids (mg/l)	Total Solids: TS 105 (mg/l)	Total Dissolved Solids: TDS 180 (mg/l)
0:00	24	1.3	1.1	1.3	10	840	830
4:00	31	0.54	0.75	3	80	850	770
8:00	27	0.25	0.68	1.6	30	860	830
12:00	24	<0.15	0.45	1.6	580	1320	740
16:00	26	1.1	0.76	2.5	100	840	740
20:00	26	1.5	1.2	3.4	70	850	780
0:00	24	1.3	1.1	1.3	10	840	830
Average	**26**	**1.00**	**0.86**	**2.1**	**125.71**	**914.29**	**788.57**

APPENDIX 4: Graphical Ripple diagram methodology for determining the equalization process basin volumes required at different flows.

- Equalization basin release flow rate = average daily flow rate

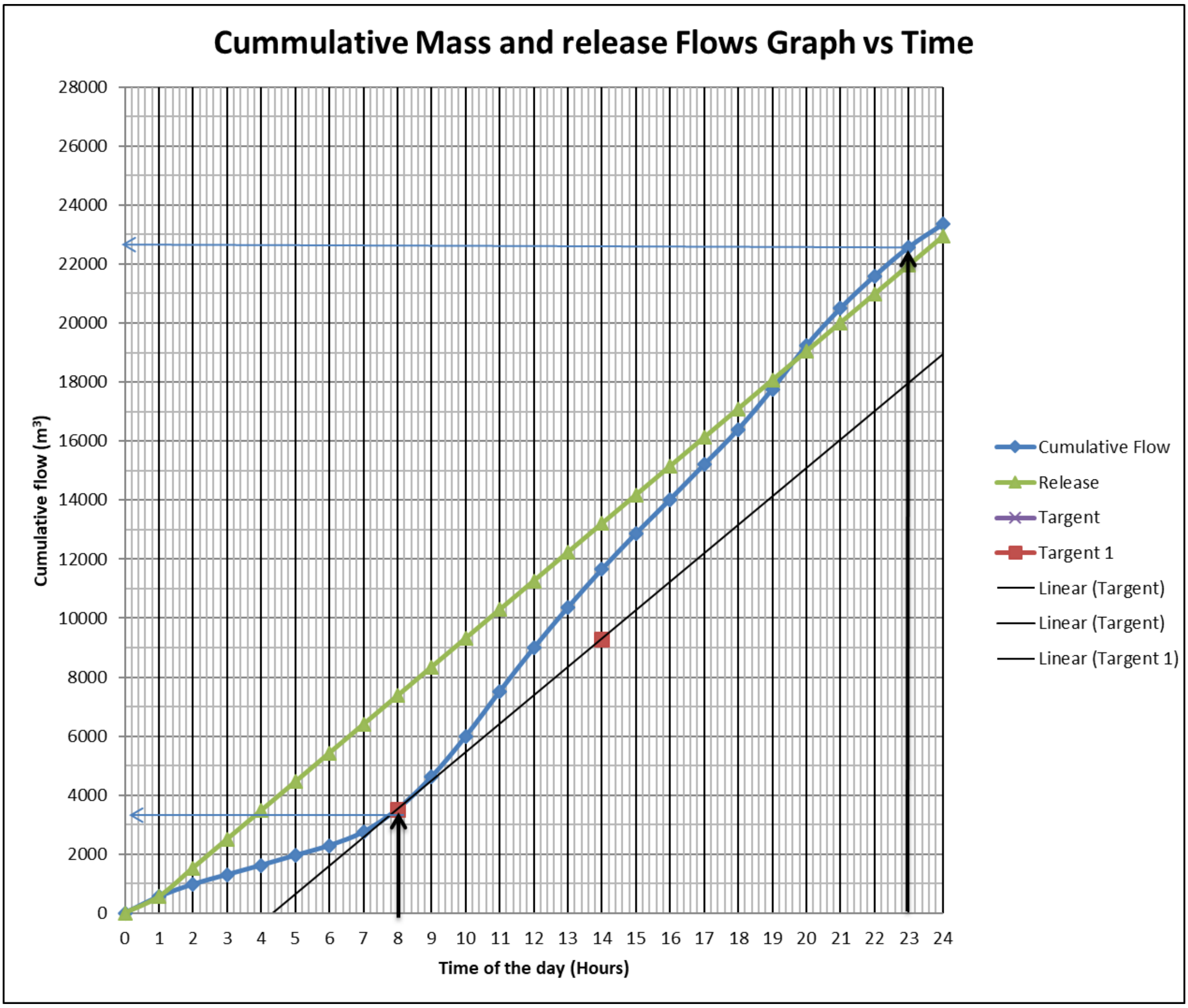


Figure 4.1. 1: Equalization process volume: Graphical analysis: Release flow rate = average daily flow rate

- Equalization basin release flow rate = average daily flow rate PLUS 10%:

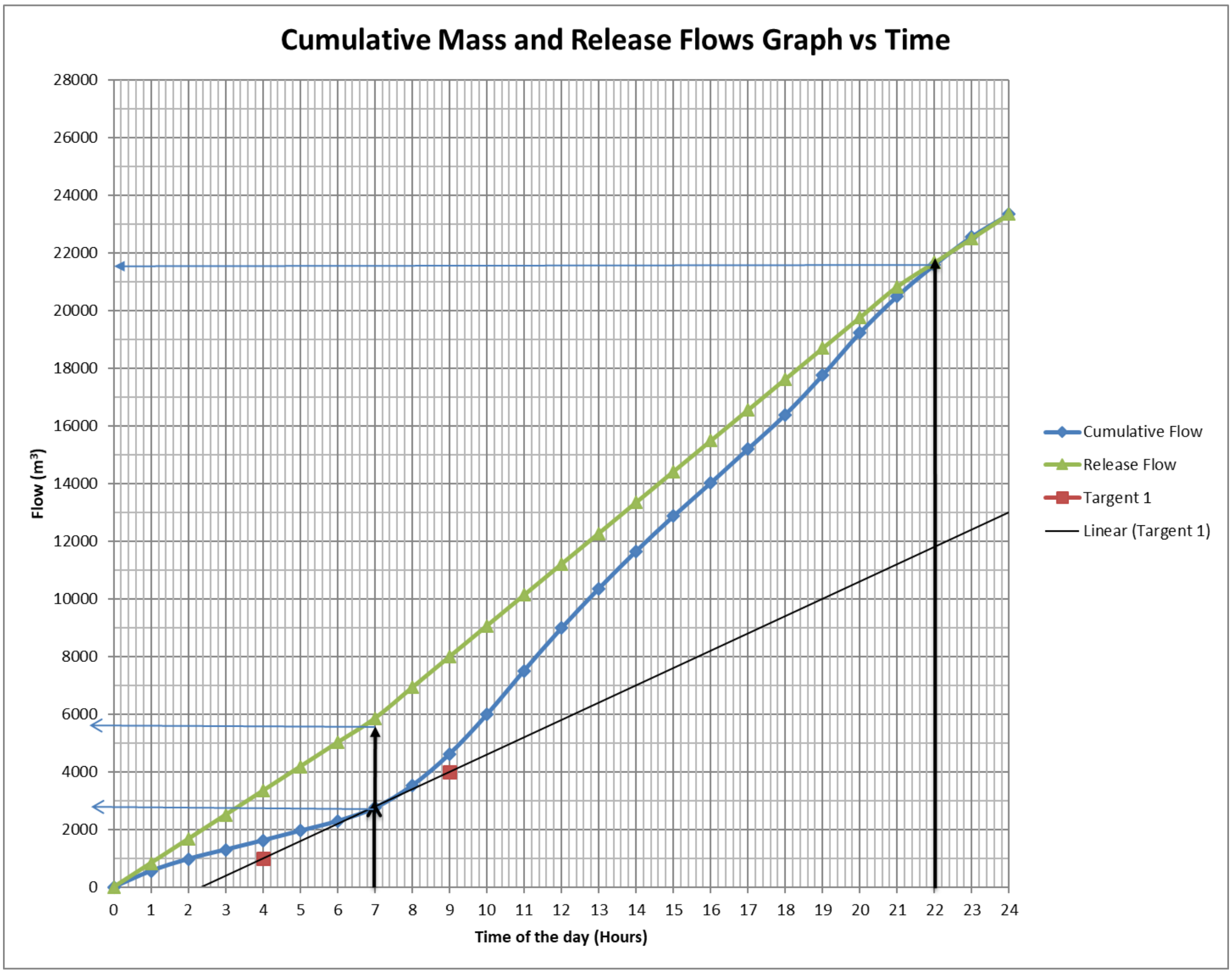


Figure 4.1. 2: Equalization process volume: Graphical analysis: Release flow rate = average daily flow rate PLUS 10% increase

- Equalization basin release flow rate = average daily flow rate PLUS 15%:

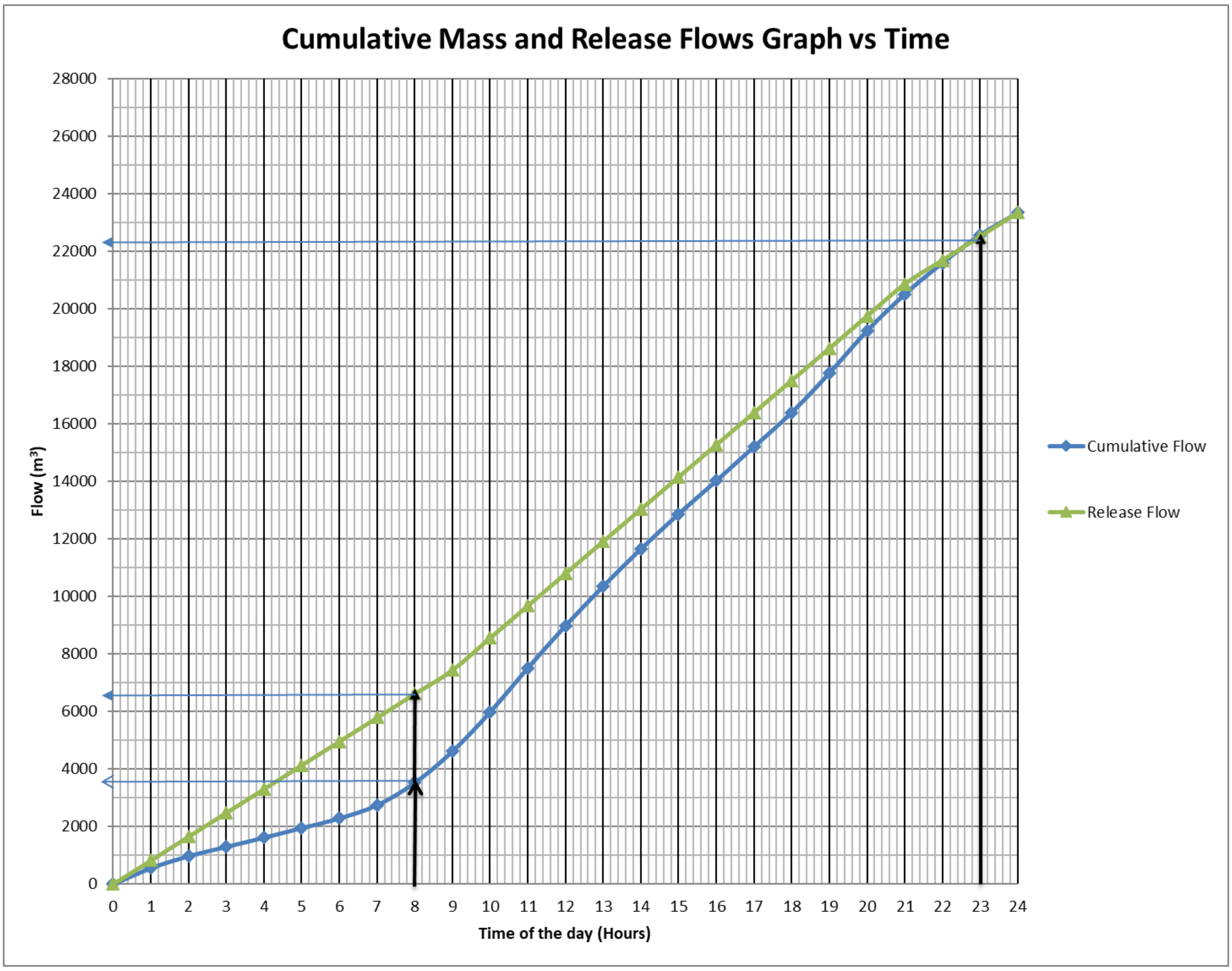


Figure 4.1. 3: Equalization process volume: Graphical analysis: Release flow rate = average daily flow rate PLUS 15% increase

- Equalization basin release flow rate = average daily flow rate PLUS 20%:

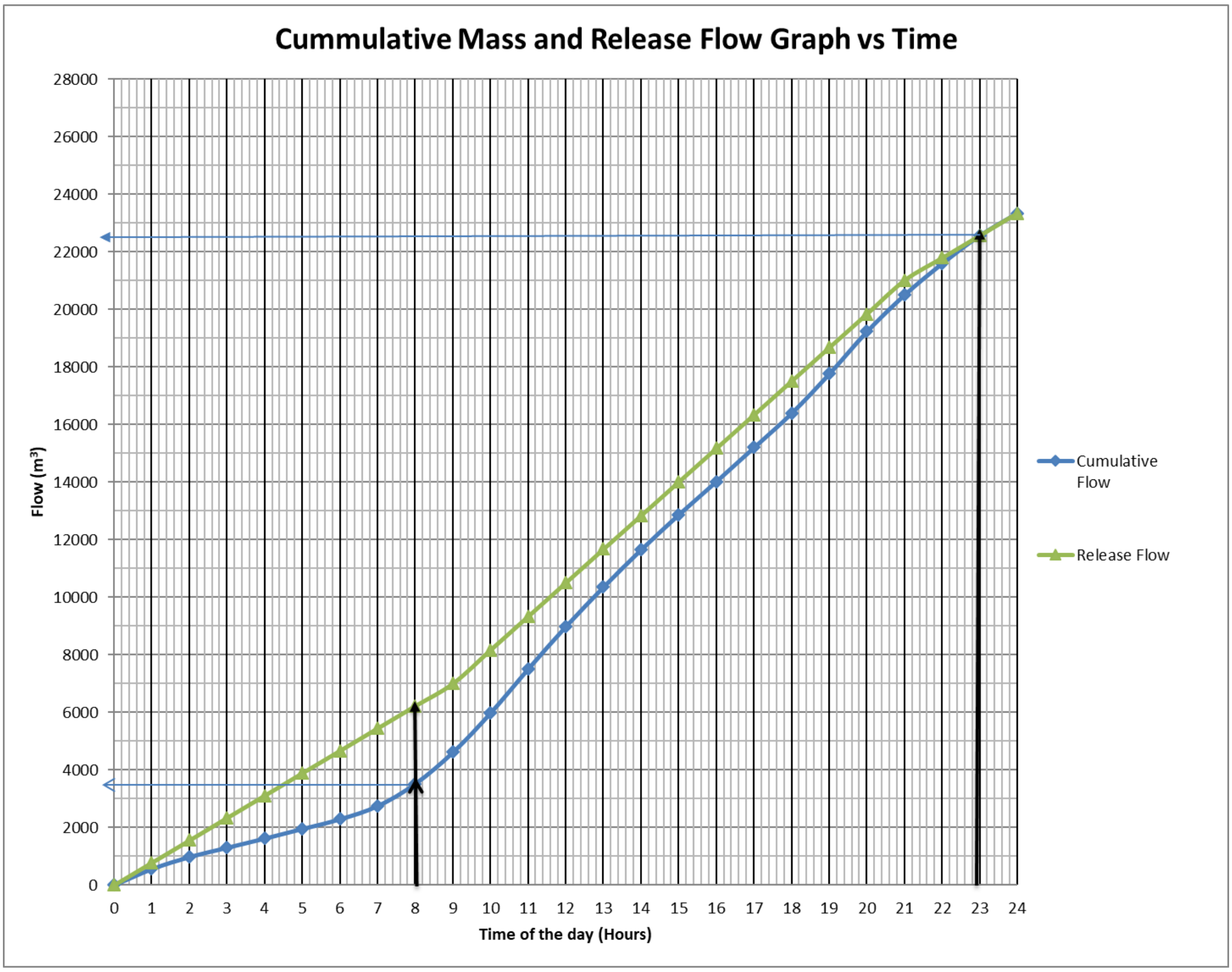


Figure 4.1. 4: Equalization process volume: Graphical analysis: Release flow rate = average daily flow rate PLUS 20% increase

- Equalization basin release flow rate = average daily flow rate PLUS 25%:

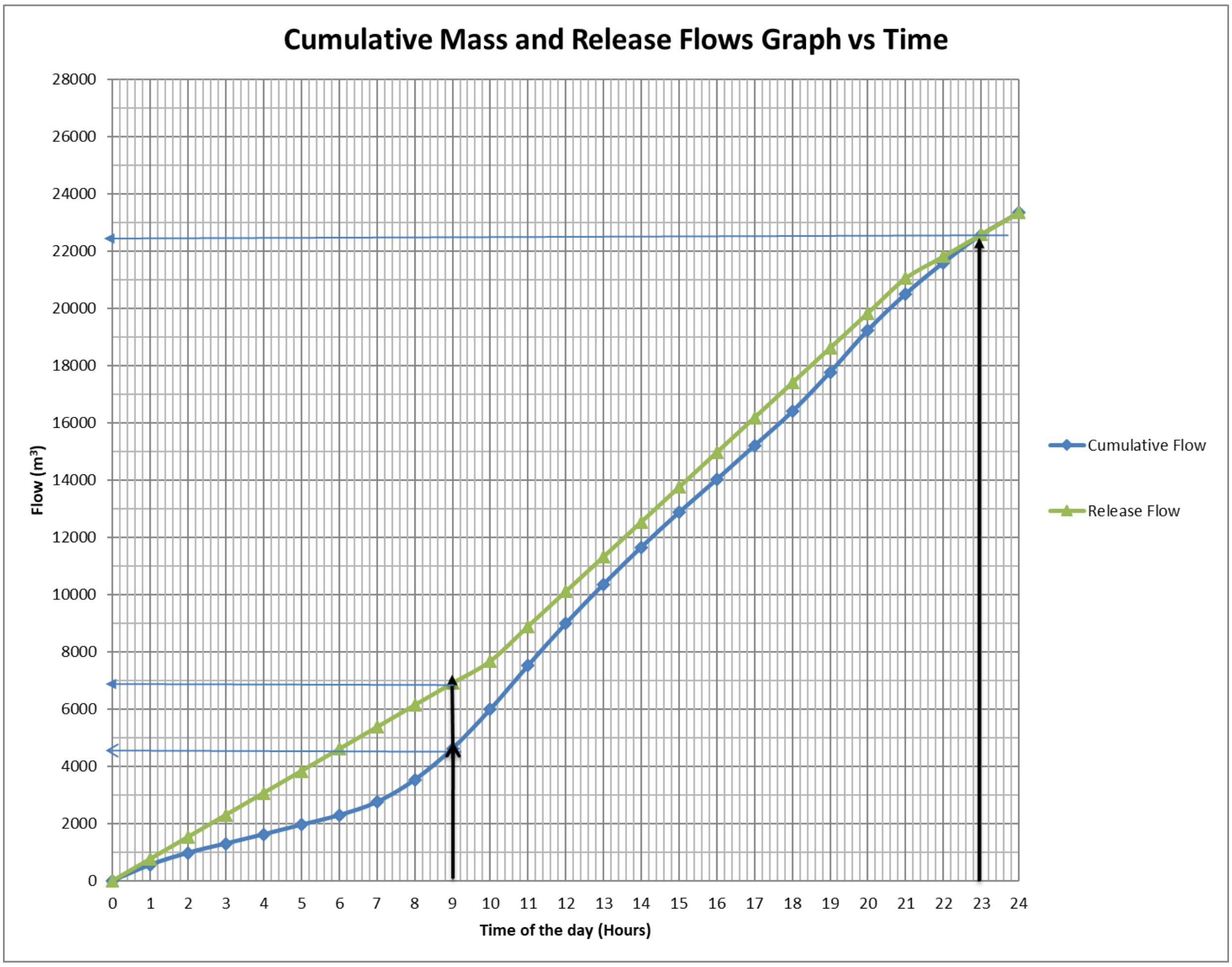


Figure 4.1. 5: Equalization process volume: Graphical analysis: Release flow rate = average daily flow rate PLUS 25% increase

APPENDIX 5: Numerical analysis comparison

Table 5.1a: Numerical analysis: Equalization basin volume: Release flow rate = average daily flow

		Ave flow	972.88		
Time interval (hr's)	**Flow in** m3/hr	**Vol $_{IN}$ ave** (m^3/hr)	**Vol $_{OUT}$** (m^3/hr)	**Δ Vol = Vol $_{IN}$ - Vol $_{OUT}$** (m^3/hr)	**Accum volume** (m^3)
23-24	630.13				4258.01
24-01	488.41	559.27	972.88	-413.61	3844.40
01-02	349.90	419.16	972.88	-553.72	3290.68
02-03	294.74	322.32	972.88	-650.56	2640.12
03-04	343.66	319.20	972.88	-653.68	1986.45
04-05	326.86	335.26	972.88	-637.62	1348.83
05-06	344.02	335.44	972.88	-637.44	711.39
06-07	574.31	459.17	972.88	-513.71	197.68
07-08	976.09	775.20	972.88	-197.68	0.00
08-09	1207.82	1091.96	972.88	119.08	119.08
09-10	1528.07	1367.95	972.88	395.07	514.15
10-11	1530.38	1529.23	972.88	556.35	1070.49
11-12	1412.86	1471.62	972.88	498.74	1569.24
12-13	1332.77	1372.82	972.88	399.94	1969.17
13-14	1261.58	1297.18	972.88	324.30	2293.47
14-15	1165.95	1213.77	972.88	240.89	2534.36
15-16	1142.71	1154.33	972.88	181.45	2715.81
16-17	1222.36	1182.54	972.88	209.66	2925.47
17-18	1159.81	1191.09	972.88	218.21	3143.68
18-19	1574.68	1367.25	972.88	394.37	3538.04
19-20	1370.44	1472.56	972.88	499.68	4037.73
20-21	1166.49	1268.47	972.88	295.59	4333.31
21-22	1023.46	1094.98	972.88	122.10	**4455.41**
22-23	921.56	972.51	972.88	-0.37	4455.04
23-24	630.13	775.85	972.88	-197.03	4258.01
	Ave				
				Storage req =	**4455.41**

Start of capacity analysis

Table 5.1b: Numerical analysis: Equalization basin volume: Release flow rate = average daily flow PLUS 10%

					Hours
Max release (Ave+10%)			1070.17		14
	Minimum release		836.67		10
		Ave flow	972.88		
Time	**Flow in**	**Vol $_{IN}$**	**Vol $_{OUT}$**	**Δ Vol =**	**Accum**
interval		**ave**		**Vol $_{IN}$ - Vol $_{OUT}$**	**volume**
(hr's)	m3/hr	(m^3/hr)	(m^3/hr)	(m^3/hr)	(m^3)
23-24	630.13				3401.88
24-01	488.41	559.27	836.67	-277.40	3124.47
01-02	349.90	419.16	836.67	-417.52	2706.95
02-03	294.74	322.32	836.67	-514.35	2192.60
03-04	343.66	319.20	836.67	-517.47	1675.12
04-05	326.86	335.26	836.67	-501.41	1173.71
05-06	344.02	335.44	836.67	-501.23	672.47
06-07	574.31	459.17	836.67	-377.51	294.97
07-08	976.09	775.20	1070.17	-294.97	0.00
08-09	1207.82	1091.96	1070.17	21.79	21.79
09-10	1528.07	1367.95	1070.17	297.78	319.57
10-11	1530.38	1529.23	1070.17	459.06	778.63
11-12	1412.86	1471.62	1070.17	Start of capacity analysis 401.45	1180.08
12-13	1332.77	1372.82	1070.17	302.65	1482.73
13-14	1261.58	1297.18	1070.17	227.01	1709.74
14-15	1165.95	1213.77	1070.17	143.60	1853.34
15-16	1142.71	1154.33	1070.17	84.16	1937.51
16-17	1222.36	1182.54	1070.17	112.37	2049.88
17-18	1159.81	1191.09	1070.17	120.92	2170.80
18-19	1574.68	1367.25	1070.17	297.08	2467.88
19-20	1370.44	1472.56	1070.17	402.39	2870.27
20-21	1166.49	1268.47	1070.17	198.30	3068.57
21-22	1023.46	1094.98	836.67	258.30	3326.87
22-23	921.56	972.51	836.67	135.84	3462.71
23-24	630.13	775.85	836.67	-60.83	3401.88
	Ave				
				Storage req =	3462.71

Table 5.1c: Numerical analysis: Equalization basin volume: Release flow rate = average daily flow PLUS 15%

					Hours
Max release (Ave+15%)			1118.81		12
	Minimum release		826.95		12
		Ave flow	972.88		
Time interval	**Flow in**	**Vol $_{IN}$ ave**	**Vol $_{OUT}$**	**Δ Vol = Vol $_{IN}$ - Vol $_{OUT}$**	**Accum volume**
(hr's)	m3/hr	(m^3/hr)	(m^3/hr)	(m^3/hr)	(m^3)
23-24	630.13				3090.56
24-01	488.41	559.27	826.95	-267.68	2822.88
01-02	349.90	419.16	826.95	-407.79	2415.09
02-03	294.74	322.32	826.95	-504.63	1910.46
03-04	343.66	319.20	826.95	-507.75	1402.72
04-05	326.86	335.26	826.95	-491.69	911.03
05-06	344.02	335.44	826.95	-491.51	419.53
06-07	574.31	459.17	826.95	-367.78	51.75
07-08	976.09	775.20	826.95	-51.75	0.00
08-09	1207.82	1091.96	826.95	265.01	265.01
09-10	1528.07	1367.95	1118.81	249.14	514.15
10-11	1530.38	1529.23	1118.81	410.42	924.56
11-12	1412.86	1471.62	1118.81	Start of capacity analysis 352.81	1277.37
12-13	1332.77	1372.82	1118.81	254.01	1531.38
13-14	1261.58	1297.18	1118.81	178.37	1709.74
14-15	1165.95	1213.77	1118.81	94.96	1804.70
15-16	1142.71	1154.33	1118.81	35.52	1840.22
16-17	1222.36	1182.54	1118.81	63.73	1903.95
17-18	1159.81	1191.09	1118.81	72.28	1976.22
18-19	1574.68	1367.25	1118.81	248.44	2224.66
19-20	1370.44	1472.56	1118.81	353.75	2578.41
20-21	1166.49	1268.47	1118.81	149.66	2728.06
21-22	1023.46	1094.98	826.95	268.03	2996.09
22-23	921.56	972.51	826.95	145.56	**3141.66**
23-24	630.13	775.85	826.95	-51.10	3090.56
	Ave				
				Storage req =	**3141.66**

Table 5.1d: Numerical analysis: Equalization basin volume: Release flow rate = average daily flow PLUS 20%

					Hours
Max release (Ave+20%)			1167.45		12
	Minimum release		778.30		12
		Ave flow	972.88		
Time interval	**Flow in**	**Vol $_{IN}$ ave**	**Vol $_{OUT}$**	**Δ Vol = Vol $_{IN}$ - Vol $_{OUT}$**	**Accum volume**
(hr's)	m3/hr	(m^3/hr)	(m^3/hr)	(m^3/hr)	(m^3)
23-24	630.13				2701.41
24-01	488.41	559.27	778.30	-219.03	2482.37
01-02	349.90	419.16	778.30	-359.15	2123.23
02-03	294.74	322.32	778.30	-455.98	1667.25
03-04	343.66	319.20	778.30	-459.10	1208.14
04-05	326.86	335.26	778.30	-443.04	765.10
05-06	344.02	335.44	778.30	-442.86	322.24
06-07	574.31	459.17	778.30	-319.14	3.10
07-08	976.09	775.20	778.30	-3.10	0.00
08-09	1207.82	1091.96	778.30	313.65	313.65
09-10	1528.07	1367.95	1167.45	200.49	514.15
10-11	1530.38	1529.23	1167.45	361.77	875.92
11-12	1412.86	1471.62	1167.45	Start of capacity analysis 304.17	1180.08
12-13	1332.77	1372.82	1167.45	205.36	1385.45
13-14	1261.58	1297.18	1167.45	129.72	1515.17
14-15	1165.95	1213.77	1167.45	46.31	1561.48
15-16	1142.71	1154.33	1167.45	-13.12	1548.36
16-17	1222.36	1182.54	1167.45	15.08	1563.44
17-18	1159.81	1191.09	1167.45	23.63	1587.07
18-19	1574.68	1367.25	1167.45	199.79	1786.86
19-20	1370.44	1472.56	1167.45	305.11	2091.97
20-21	1166.49	1268.47	1167.45	101.01	2192.98
21-22	1023.46	1094.98	778.30	316.67	2509.66
22-23	921.56	972.51	778.30	194.21	**2703.86**
23-24	630.13	775.85	778.30	-2.46	2701.41
	Ave				
				Storage req =	**2703.86**

Table 5.1e: Numerical analysis: Equalization basin volume: Release flow rate = average daily flow PLUS 25%

					Hours
Max release (Ave+25%)			1216.10		11
	Minimum release		767.08		13
		Ave flow	972.88		
Time interval (hr's)	**Flow in** m3/hr	**Vol $_{IN}$ ave** (m^3/hr)	**Vol $_{OUT}$** (m^3/hr)	**Δ Vol = Vol $_{IN}$ - Vol $_{OUT}$** (m^3/hr)	**Accum volume** (m^3)
23-24	630.13				2611.60
24-01	488.41	559.27	767.08	-207.81	2403.80
01-02	349.90	419.16	767.08	-347.92	2055.87
02-03	294.74	322.32	767.08	-444.76	1611.12
03-04	343.66	319.20	767.08	-447.88	1163.24
04-05	326.86	335.26	767.08	-431.82	731.42
05-06	344.02	335.44	767.08	-431.64	299.79
06-07	574.31	459.17	767.08	-307.91	-8.12
07-08	976.09	775.20	767.08	8.12	0.00
08-09	1207.82	1091.96	767.08	324.88	324.88
09-10	1528.07	1367.95	767.08	600.87	925.75
10-11	1530.38	1529.23	1216.10	313.13	1238.88
11-12	1412.86	1471.62	1216.10	Start of capacity analysis 255.52	1494.40
12-13	1332.77	1372.82	1216.10	156.72	1651.12
13-14	1261.58	1297.18	1216.10	81.08	1732.19
14-15	1165.95	1213.77	1216.10	-2.33	1729.86
15-16	1142.71	1154.33	1216.10	-61.77	1668.10
16-17	1222.36	1182.54	1216.10	-33.56	1634.53
17-18	1159.81	1191.09	1216.10	-25.01	1609.52
18-19	1574.68	1367.25	1216.10	151.15	1760.67
19-20	1370.44	1472.56	1216.10	256.46	2017.13
20-21	1166.49	1268.47	1216.10	52.37	2069.50
21-22	1023.46	1094.98	767.08	327.90	2397.40
22-23	921.56	972.51	767.08	205.43	2602.83
23-24	630.13	775.85	767.08	8.77	**2611.60**
	Ave				
				Storage req =	**2611.60**

APPENDIX 6: Equalization attenuation requirements: Allowance for a 5- year growth in wastewater influent to the plant.

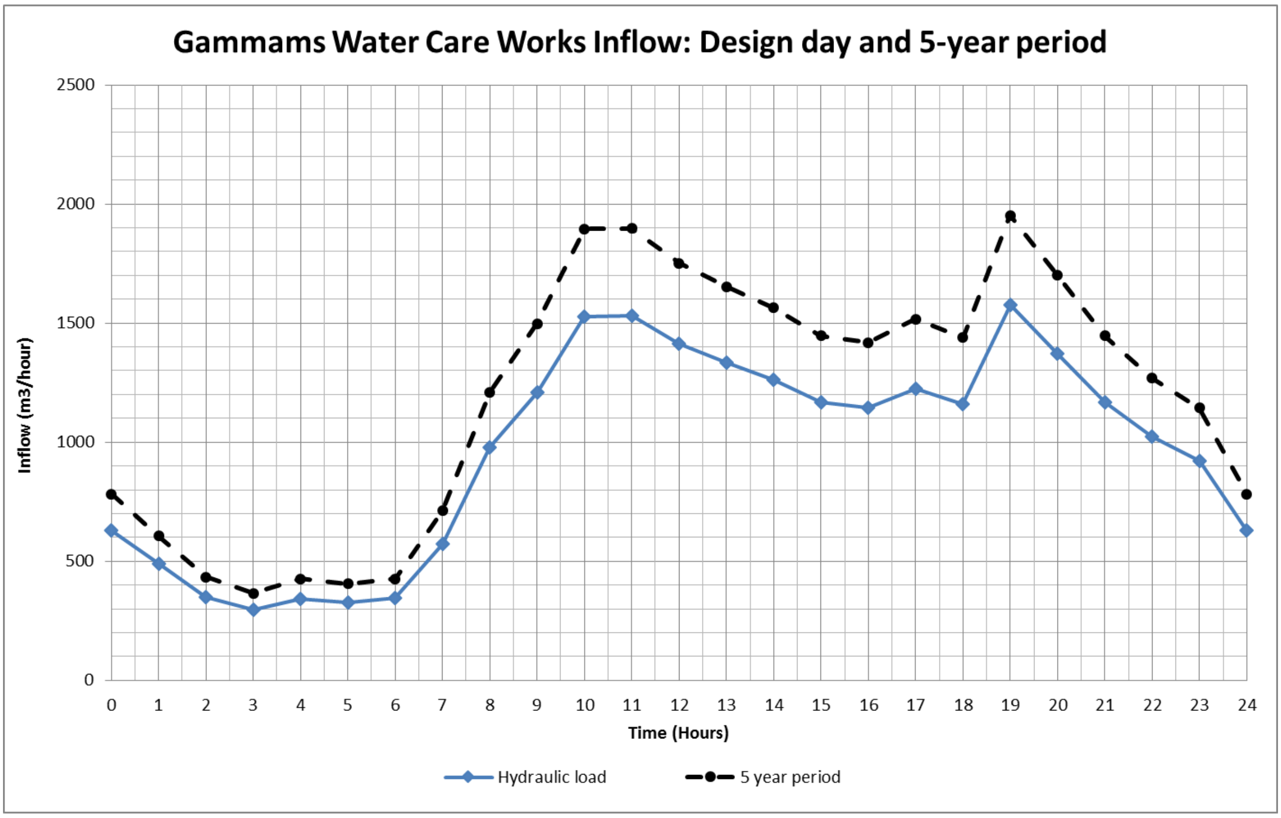


Figure 6. 1: Hydraulic load (Diurnal flow) pattern for the design day and projected 5-year growth in wastewater flow

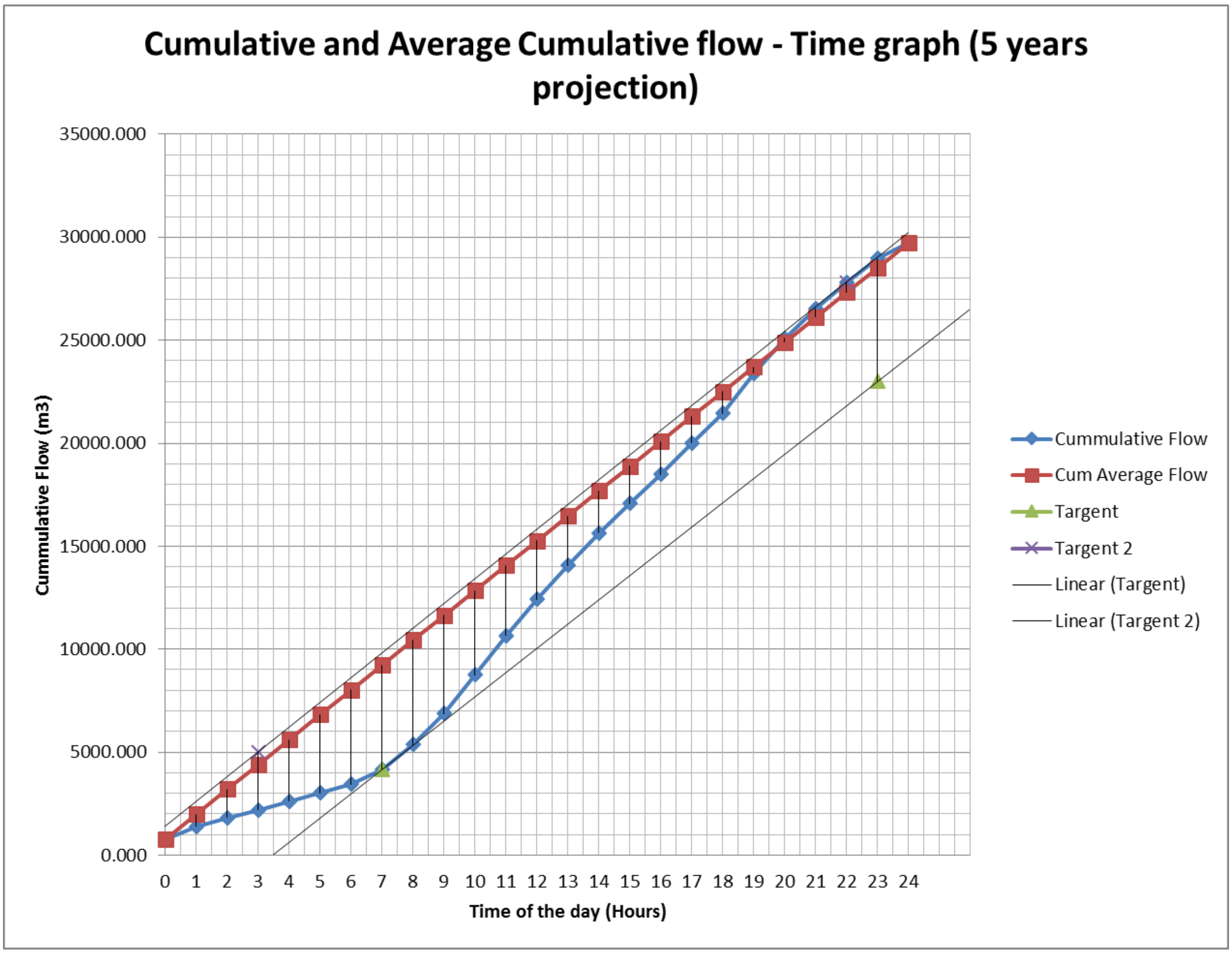


Figure 6. 2: Equalization process volume: Graphical analysis of 5-year growth: Release flow rate = average daily flow rate

Table 6.1: Equalization process volume: Numerical analysis of 5-year growth: Release flow rate = average daily flow rate

Time interval (hr's)	Flow in m3/hr	Vol_{IN} ave (m^3/hr)	Vol_{OUT} (m^3/hr)	Δ Vol = Vol_{IN} - Vol_{OUT} (m^3/hr)	Accum volume (m^3)
23-24	781.50				5280.91
24-01	605.74	693.62	1206.59	-512.97	4767.94
01-02	433.96	519.85	1206.59	-686.74	4081.20
02-03	365.55	399.75	1206.59	-806.84	3274.36
03-04	426.22	395.89	1206.59	-810.71	2463.65
04-05	405.38	415.80	1206.59	-790.79	1672.86
05-06	426.66	416.02	1206.59	-790.57	882.29
06-07	712.28	569.47	1206.59	-637.12	245.16
07-08	1210.58	961.43	1206.59	-245.16	0.00
08-09	1497.98	1354.28	1206.59	147.69	147.69
09-10	1895.16	1696.57	1206.59	489.97	637.66
10-11	1898.02	1896.59	1206.59	690.00	1327.66
11-12	1752.27	1825.15	1206.59	Start of capacity analysis 618.55	1946.21
12-13	1652.94	1702.61	1206.59	496.01	2442.22
13-14	1564.64	1608.79	1206.59	402.20	2844.42
14-15	1446.04	1505.34	1206.59	298.75	3143.17
15-16	1417.23	1431.64	1206.59	225.04	3368.22
16-17	1516.01	1466.62	1206.59	260.03	3628.25
17-18	1438.43	1477.22	1206.59	270.63	3898.88
18-19	1952.96	1695.70	1206.59	489.10	4387.98
19-20	1699.66	1826.31	1206.59	619.72	5007.70
20-21	1446.72	1573.19	1206.59	366.60	5374.30
21-22	1269.33	1358.02	1206.59	151.43	**5525.73**
22-23	1142.95	1206.14	1206.59	-0.46	5525.27
23-24	781.50	962.23	1206.59	-244.37	5280.91
			Retained	5280.91	

APPENDIX 7: Numerical analysis results: Comparison of daily diurnal pollutant load variation of Unequalised and Equalised scenarios

- Pollutant: COD loading

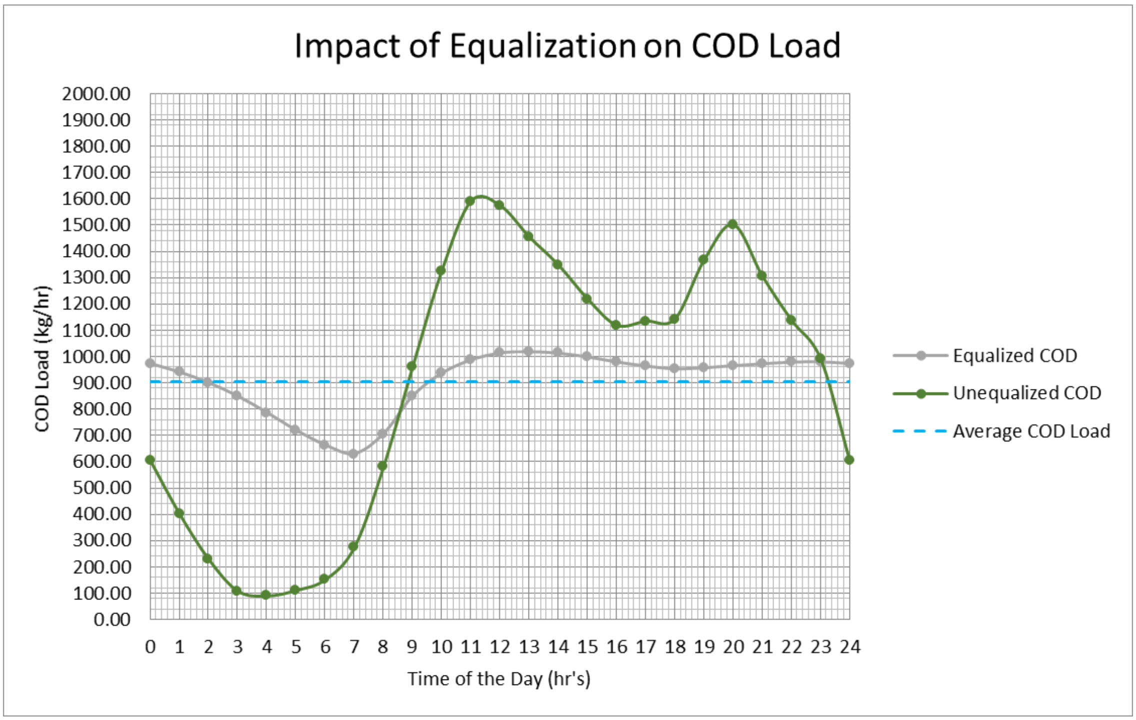


Figure 7.1. 1: Numerical analysis: Average daily flow (ADF) release: Unequalised vs Equalised COD loading comparison

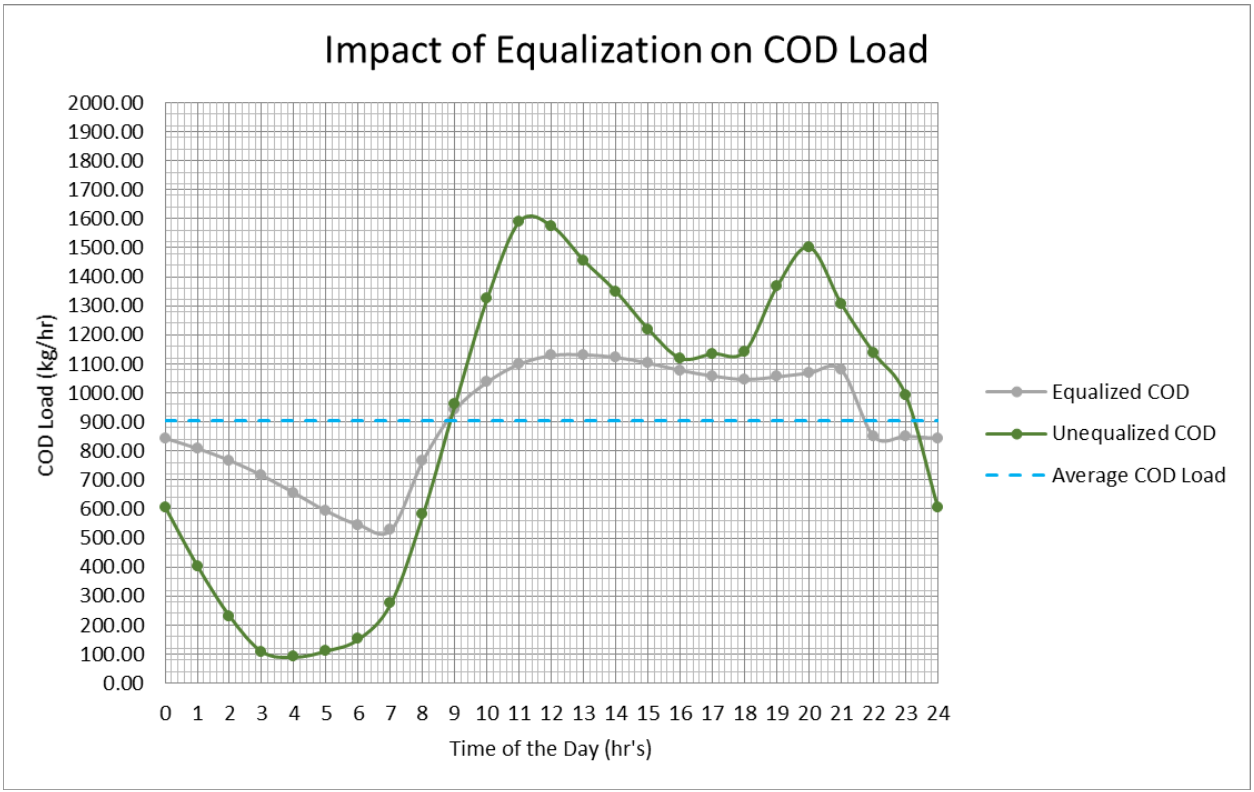


Figure 7.1. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised COD loading comparison

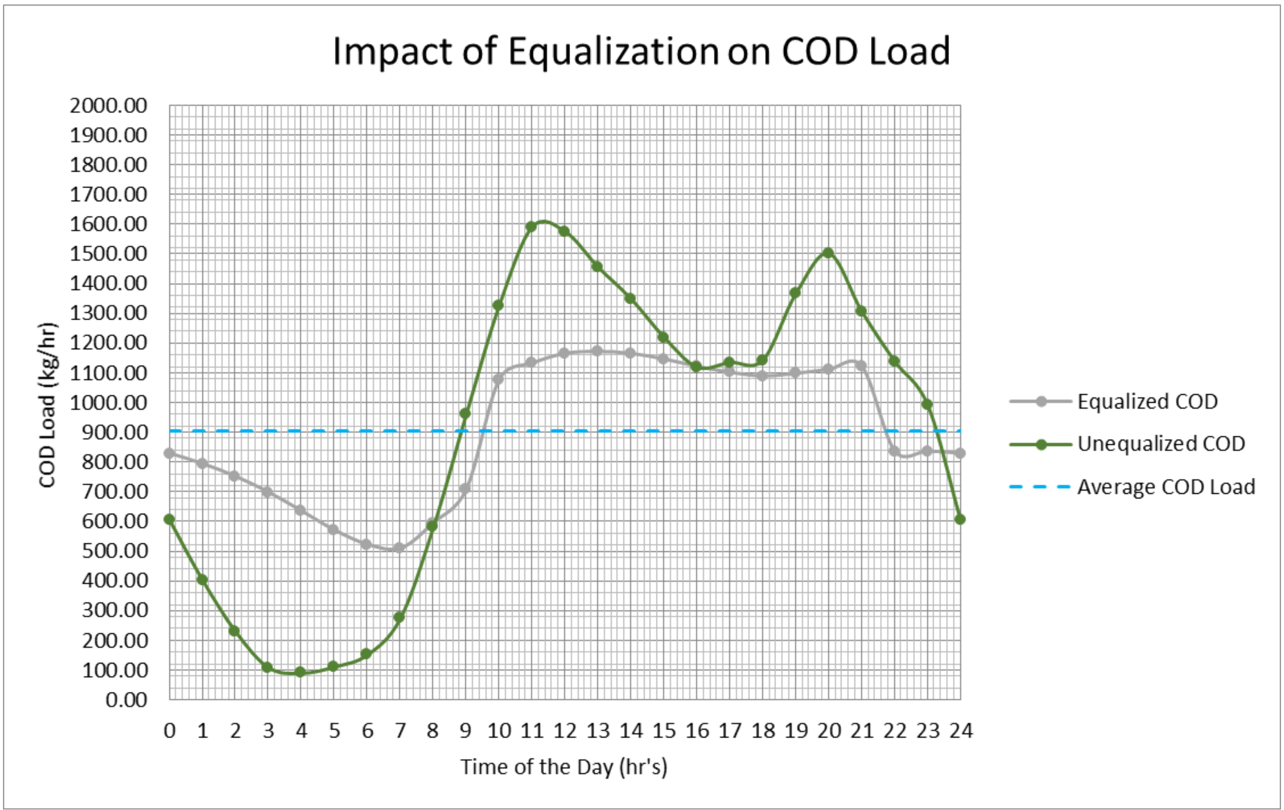


Figure 7.1. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised COD loading comparison

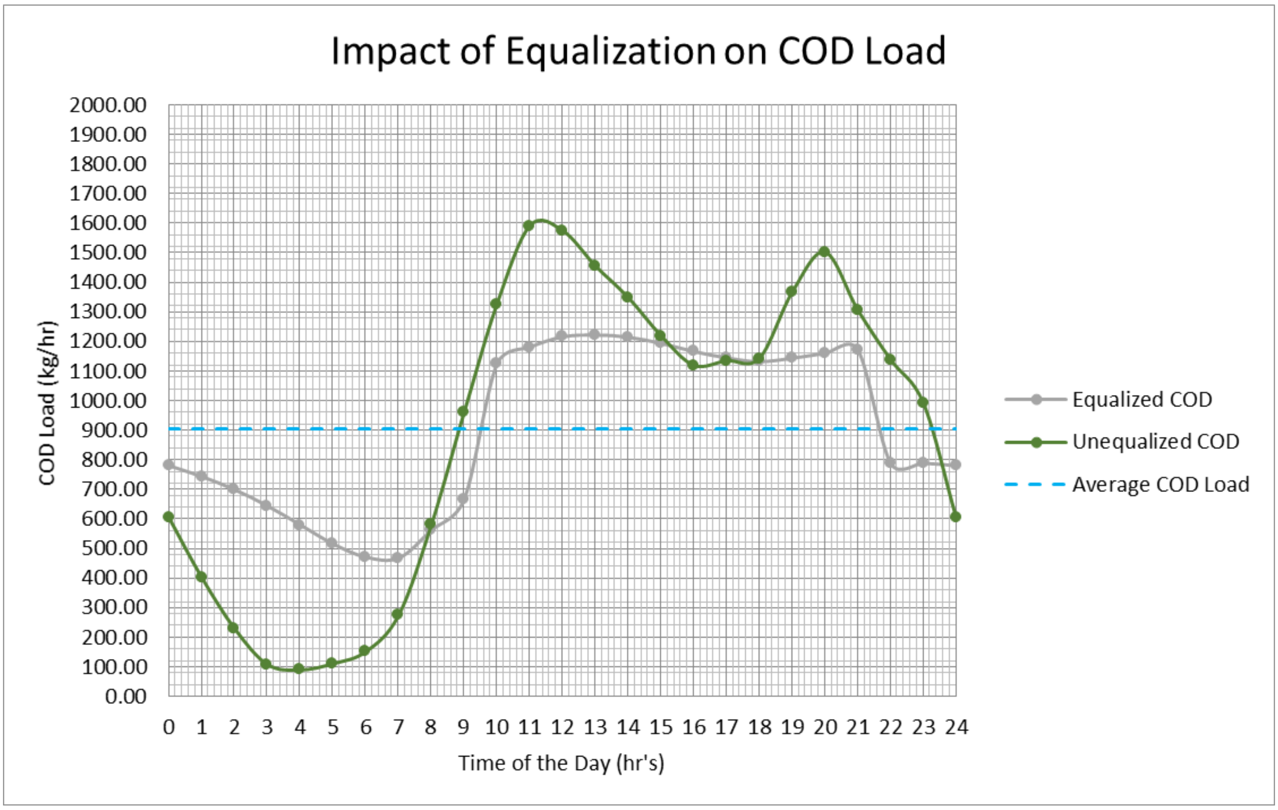


Figure 7.1. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised COD loading comparison

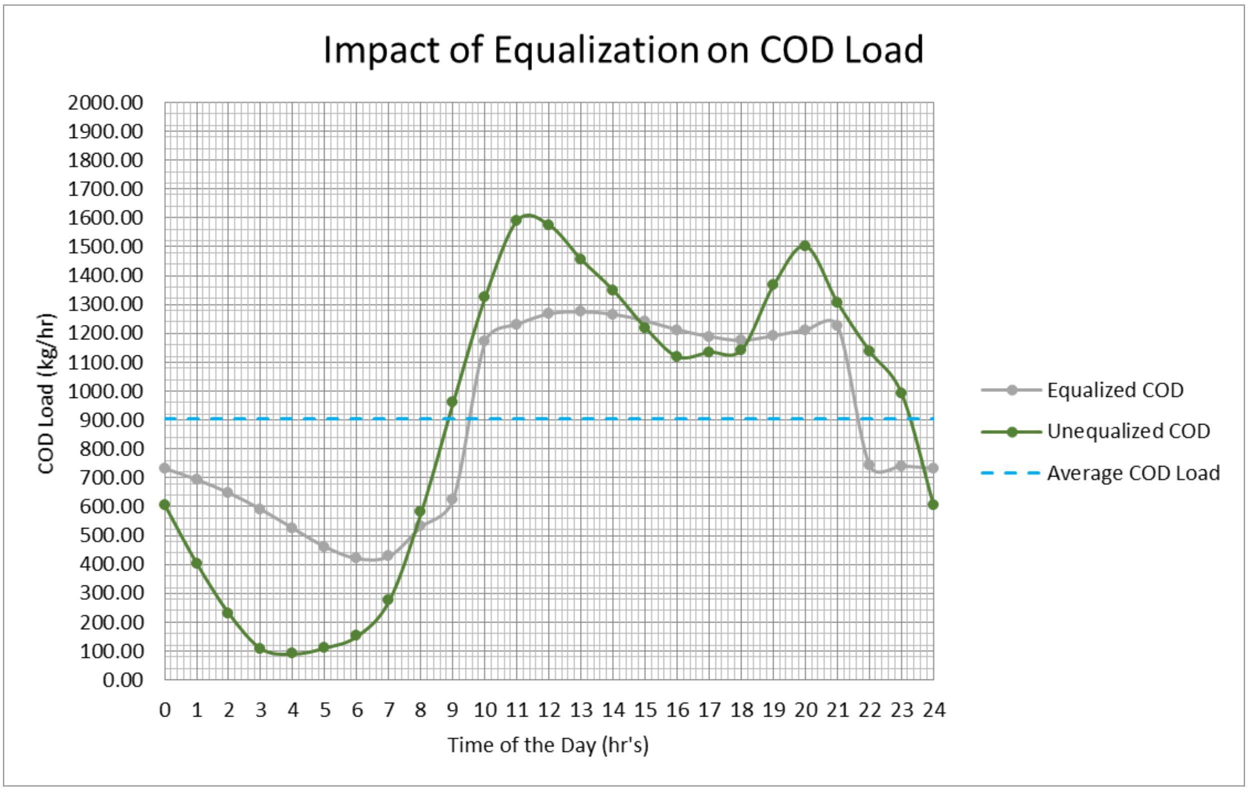


Figure 7.1. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised COD loading comparison

- Pollutant: Ammonia loading

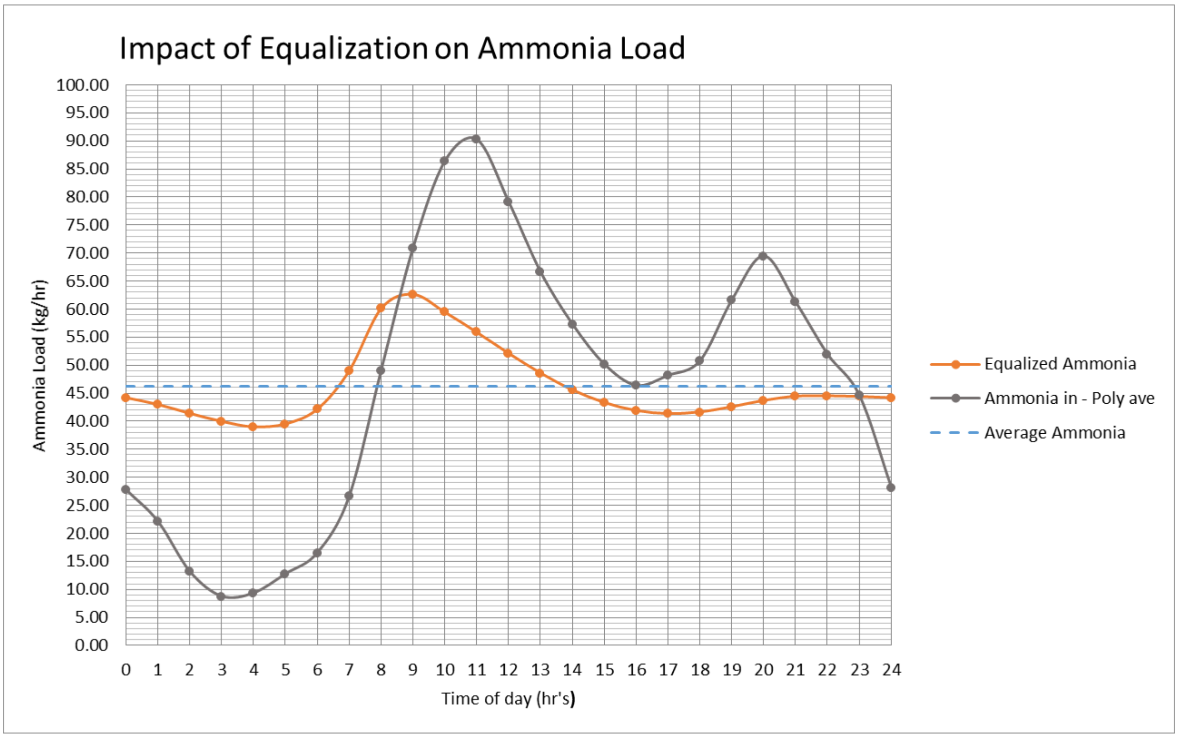


Figure 7.2. 1: Numerical analysis: ADF release: Unequalised vs Equalised Ammonia loading comparison

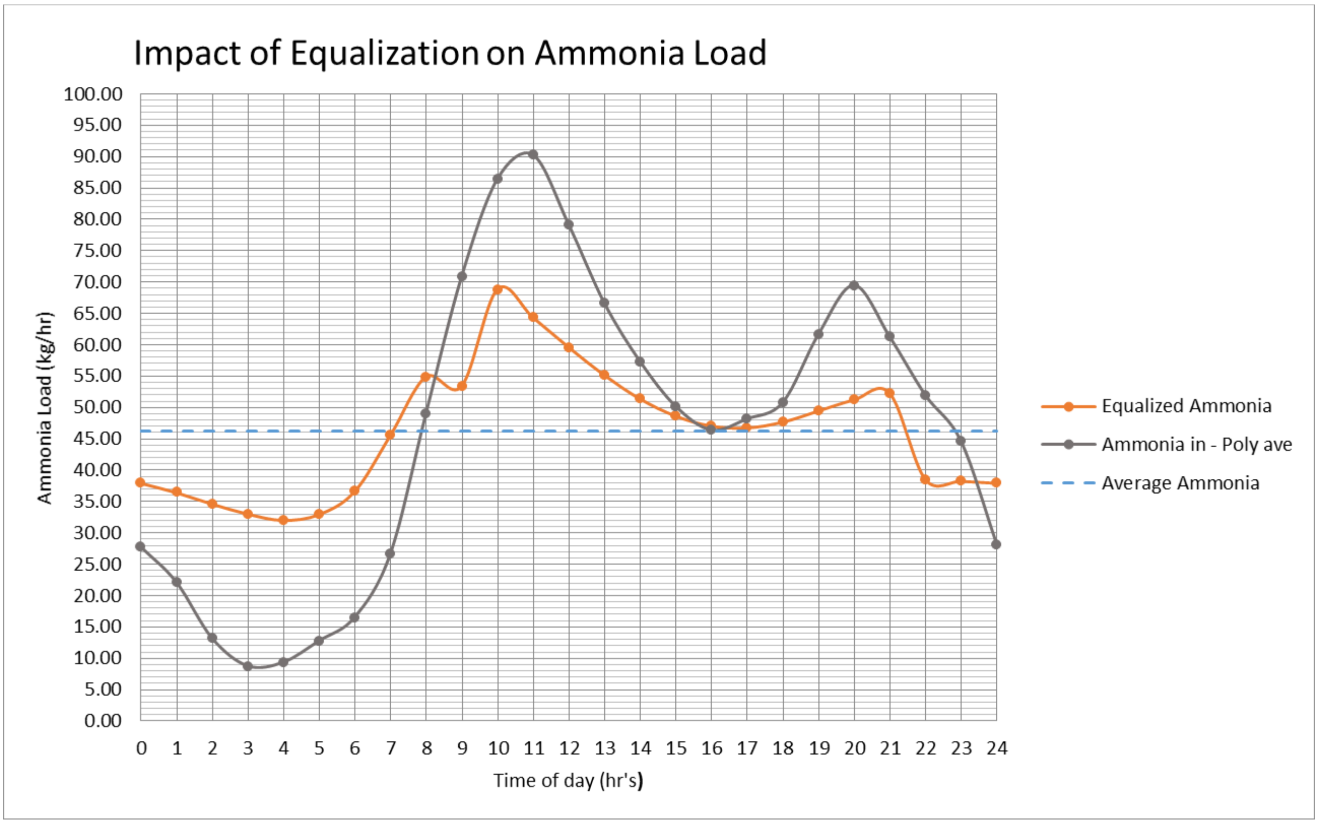


Figure 7.2. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised COD loading comparison

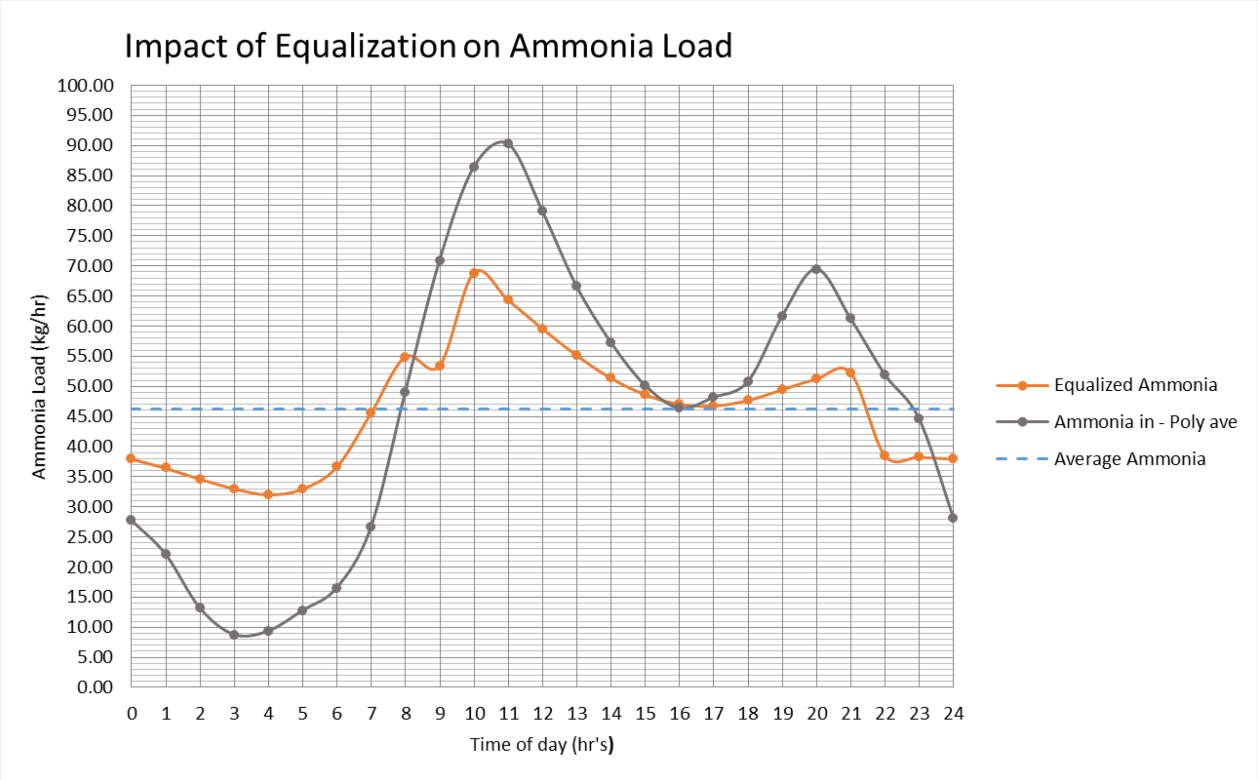


Figure 7.2. 3 Numerical analysis: ADF + 15% release: Unequalised vs Equalised Ammonia loading comparison

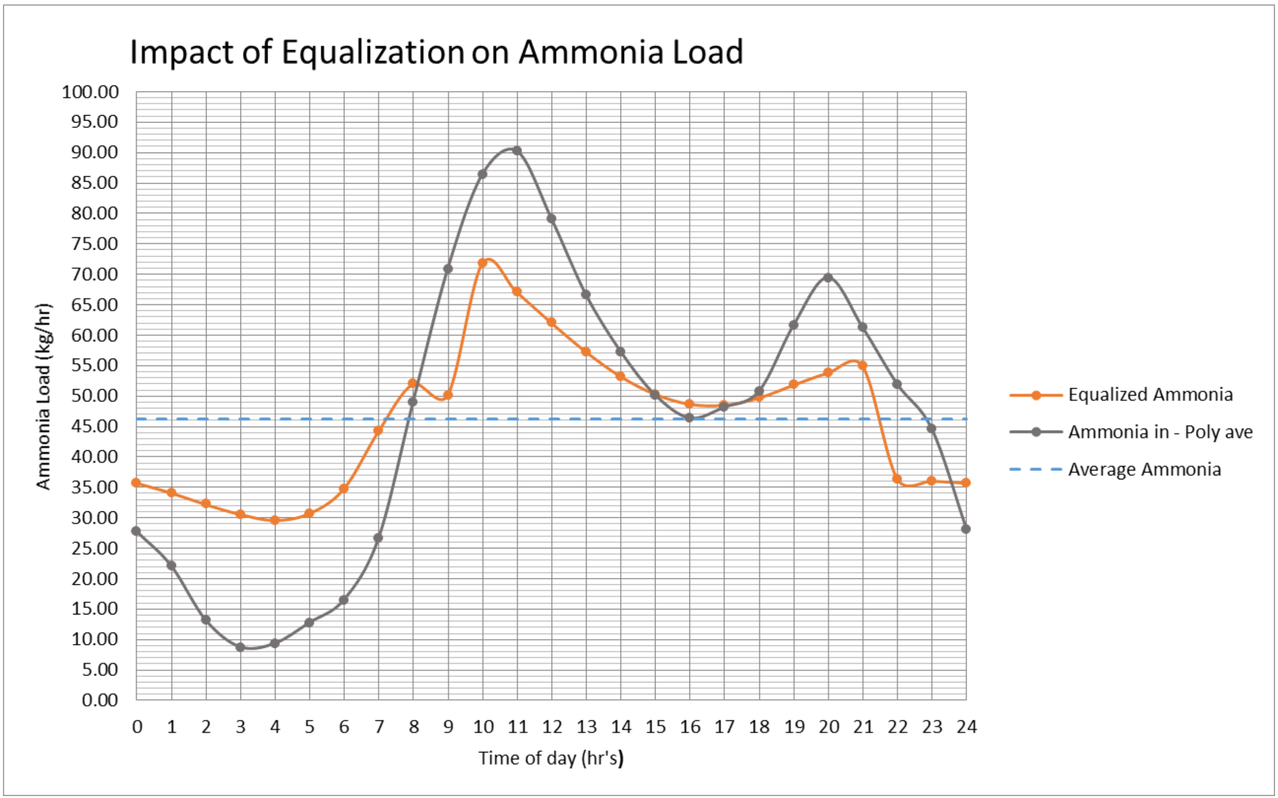


Figure 7.2. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised Ammonia loading comparison

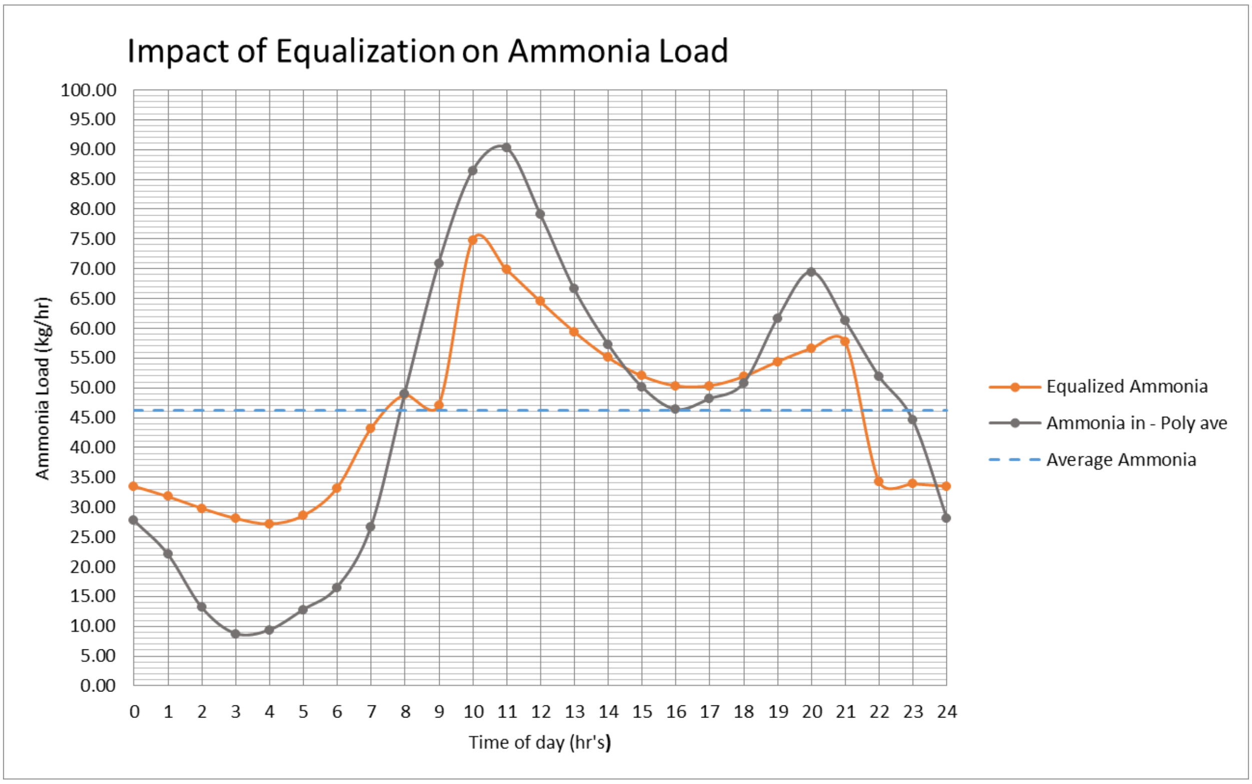


Figure 7.2. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised Ammonia loading comparison

- Pollutant: Orthophosphate loading

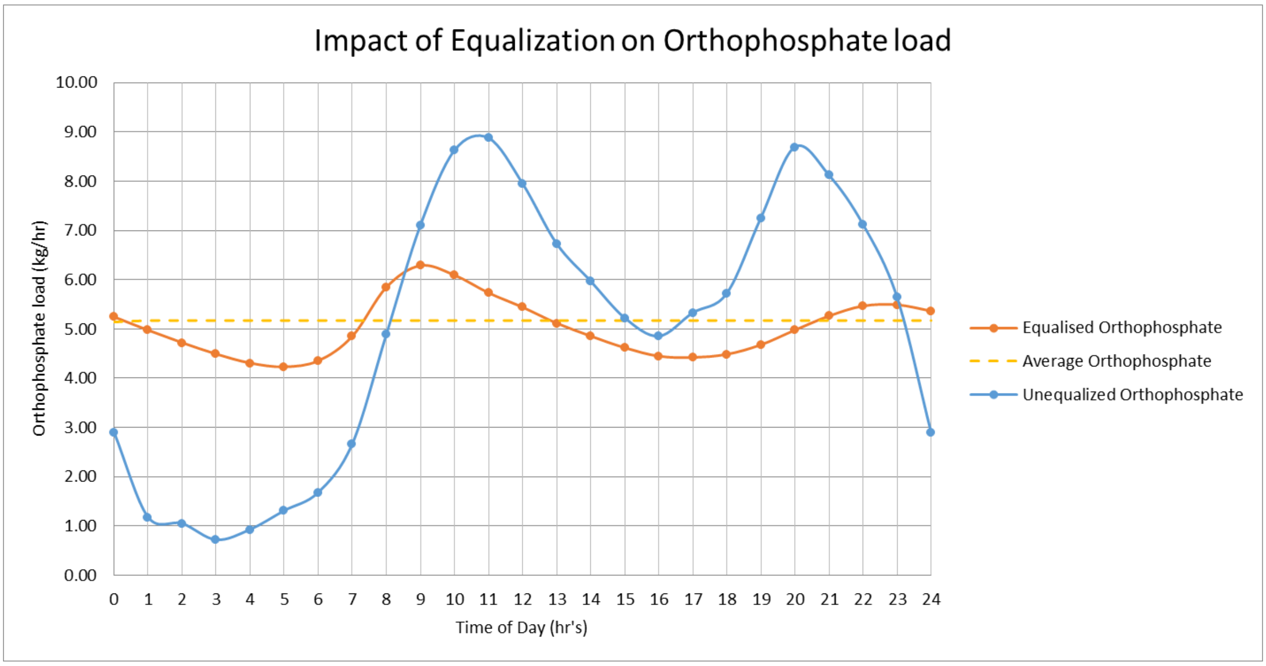


Figure 7.3. 1: Numerical analysis: ADF release: Unequalised vs Equalised Orthophosphate loading comparison

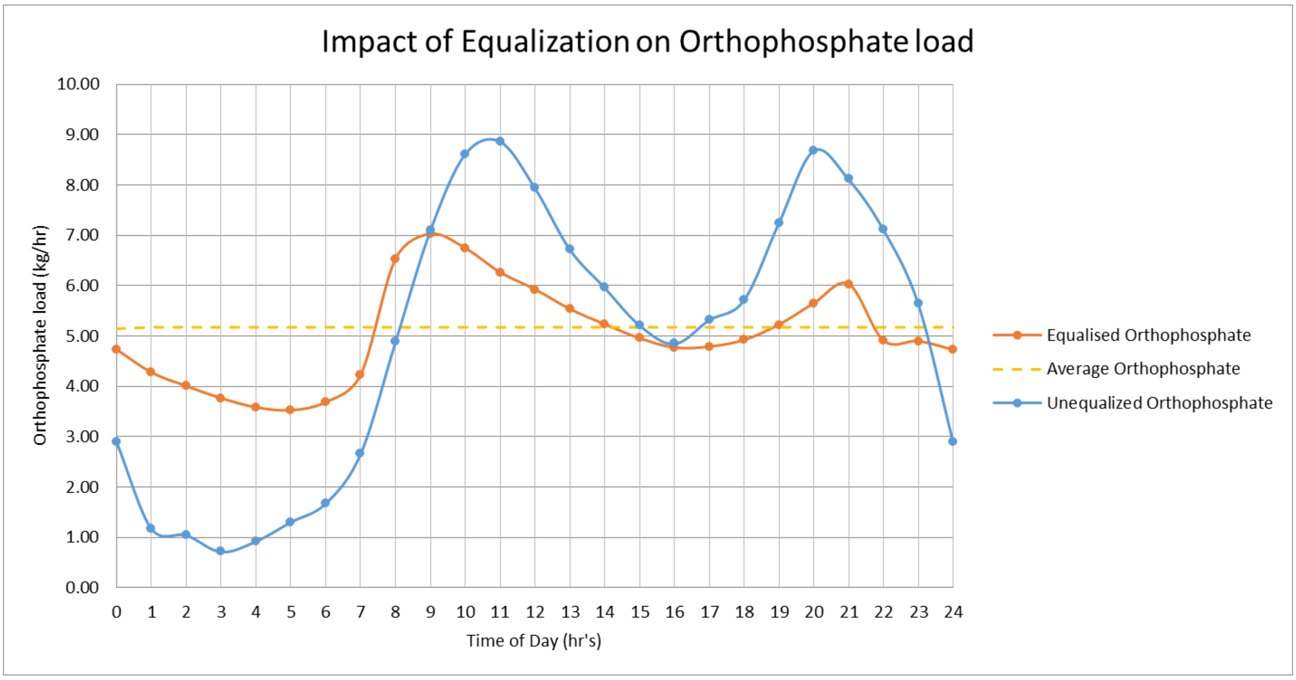


Figure 7.3. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised Orthophosphate loading comparison

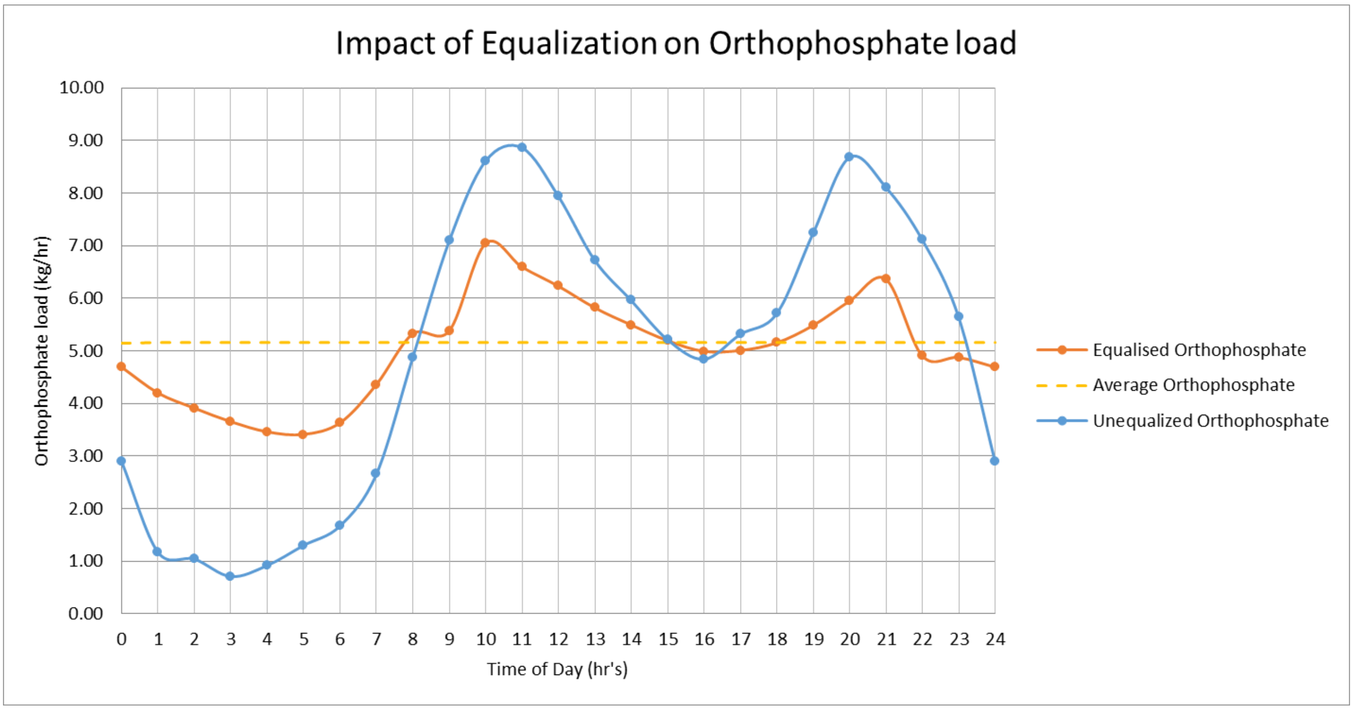


Figure 7.3. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised Orthophosphate loading comparison

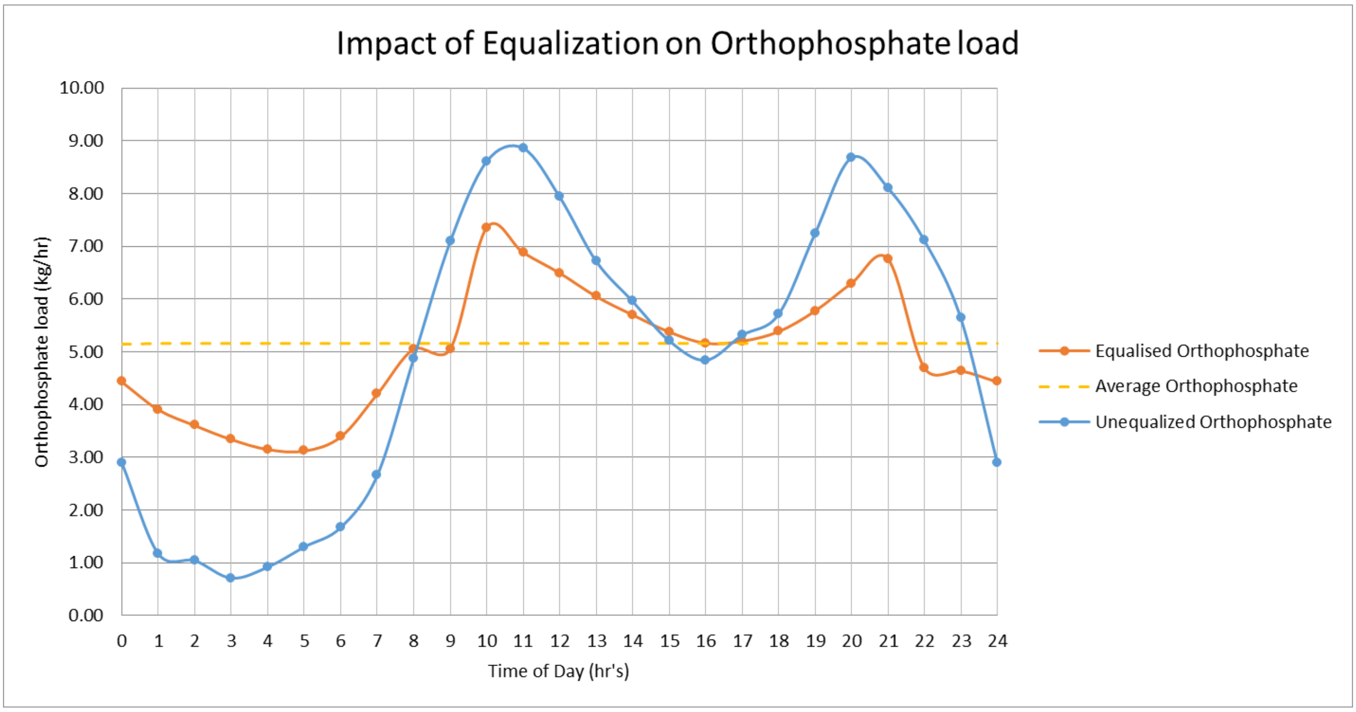


Figure 7.3. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised Orthophosphate loading comparison

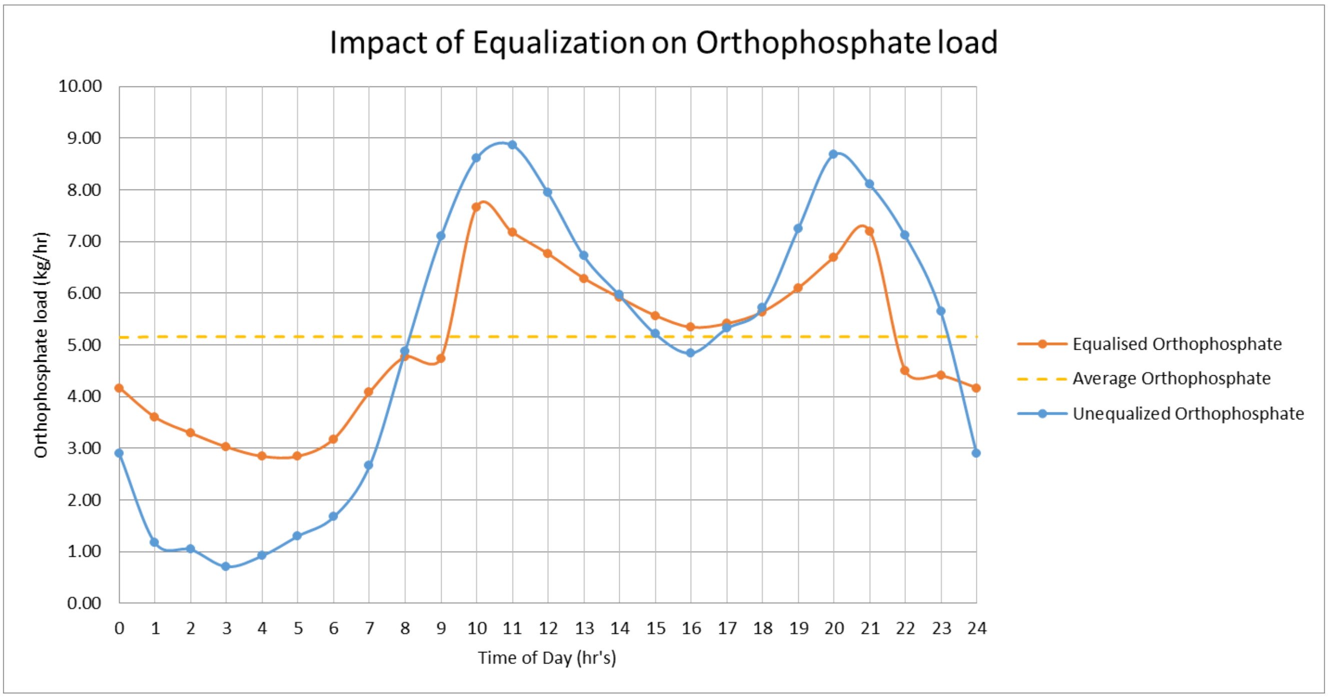


Figure 7.3. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised Orthophosphate loading comparison

- Pollutant: Total Kjeldahl Nitrogen (TKN) loading

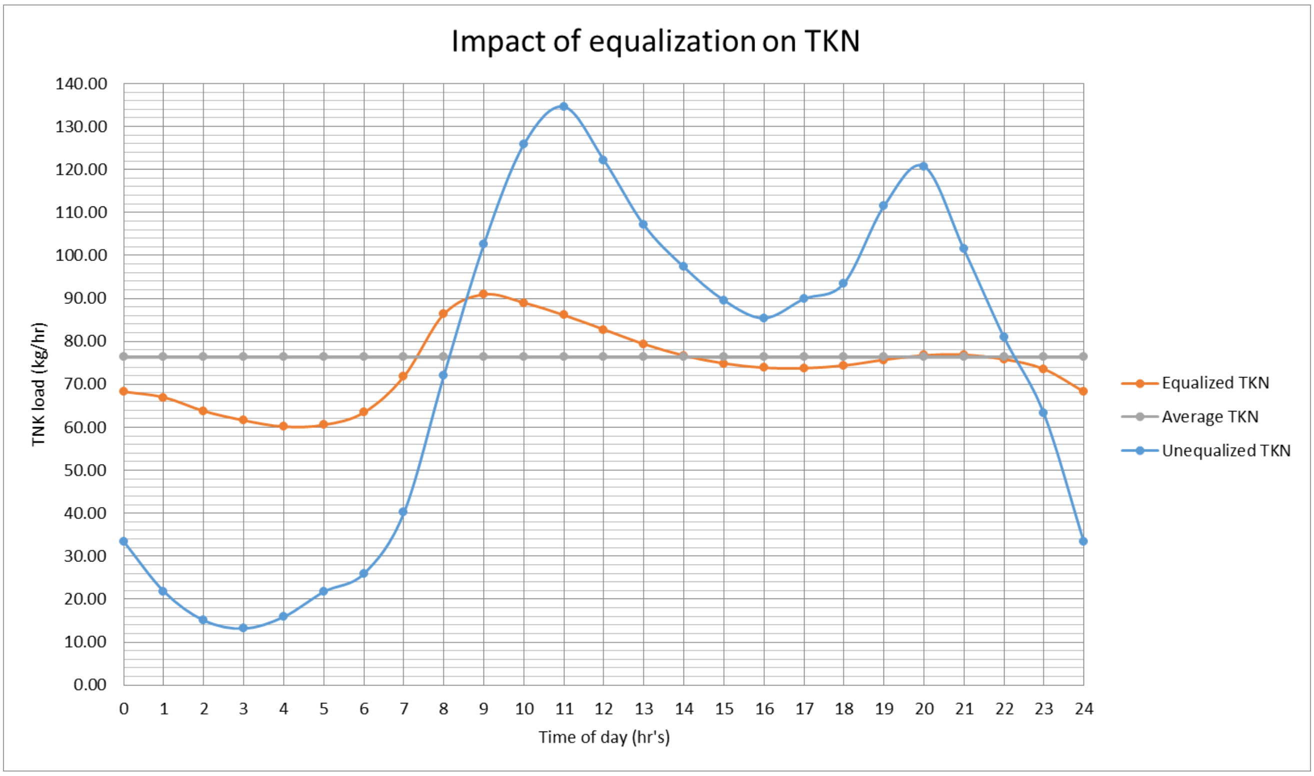


Figure 7.4. 1: Numerical analysis: ADF: Comparative Unequalised vs Equalised Total Kjeldahl Nitrogen (TKN) loading

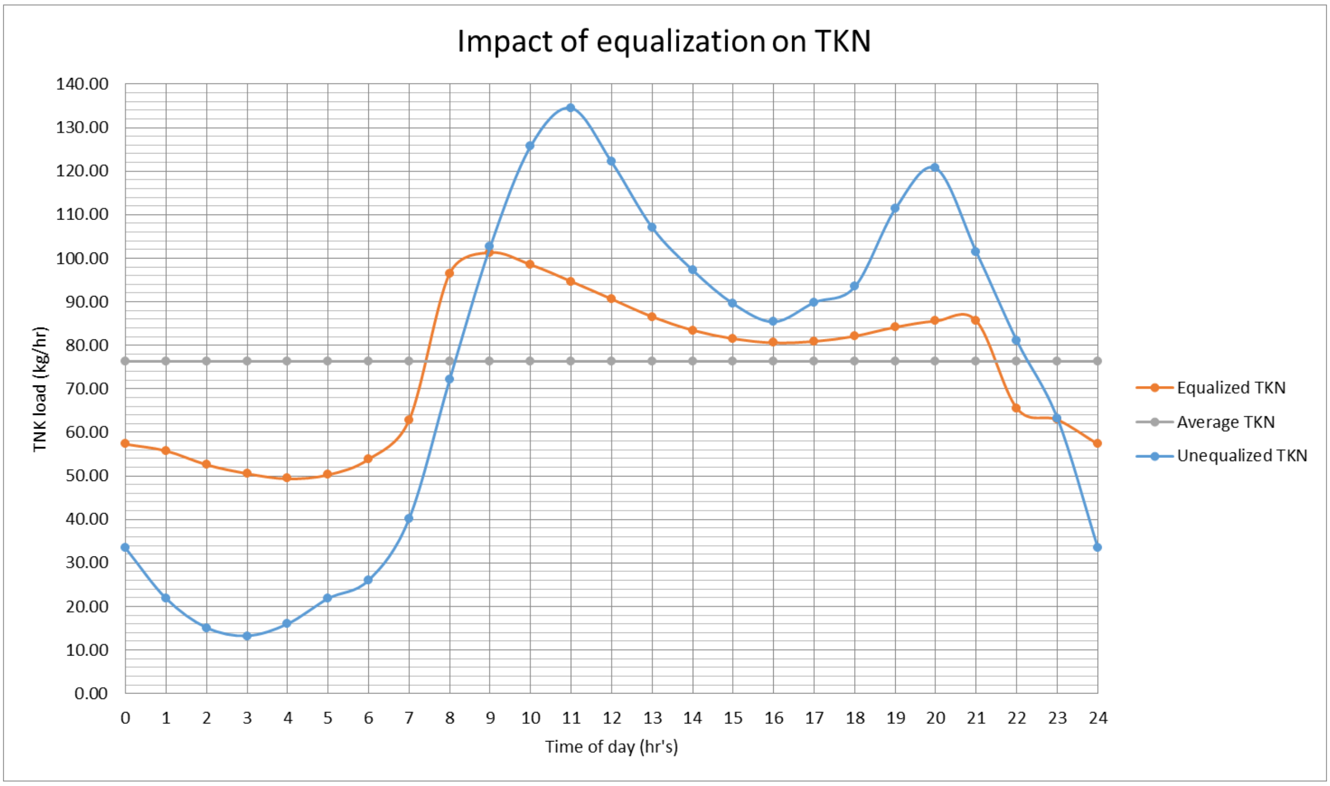


Figure 7.4. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised TKN loading comparison

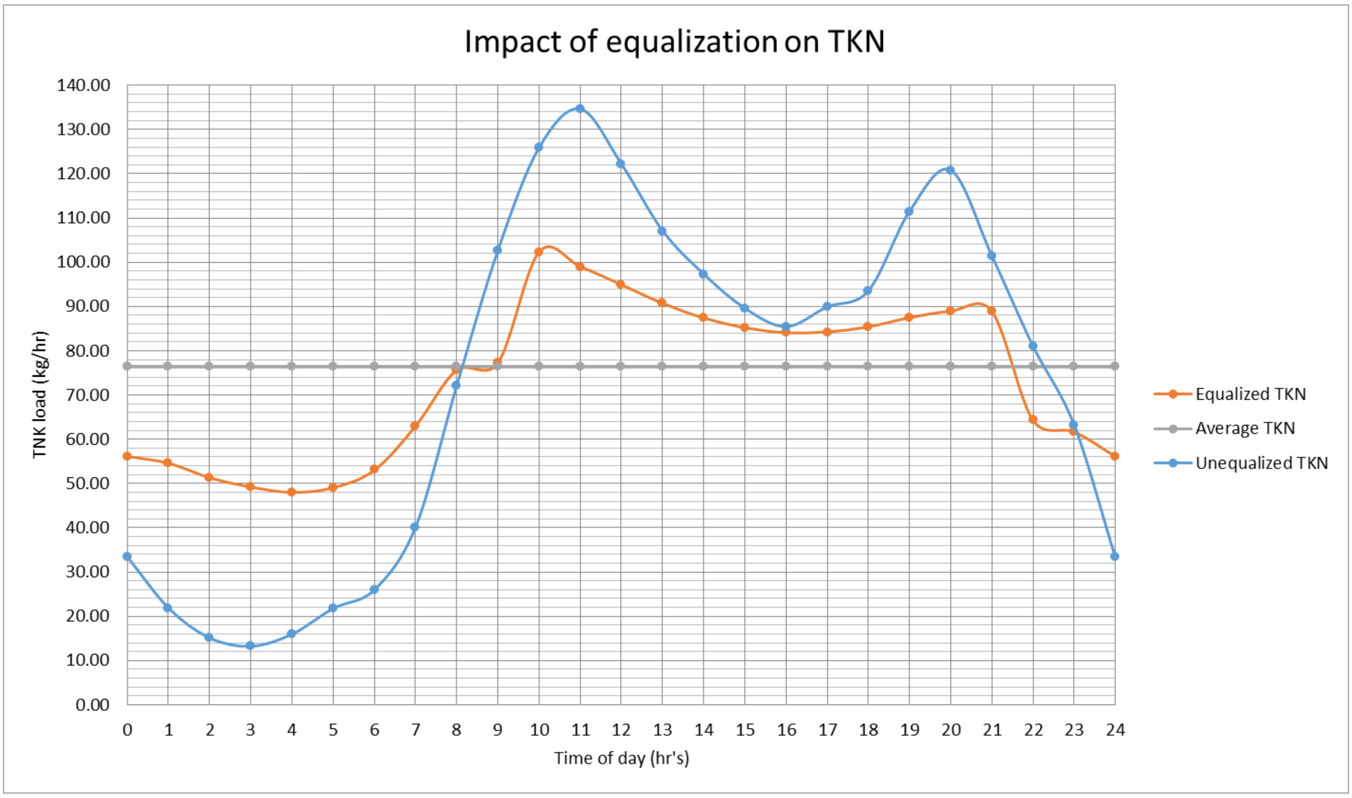


Figure 7.4. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised TKN loading comparison

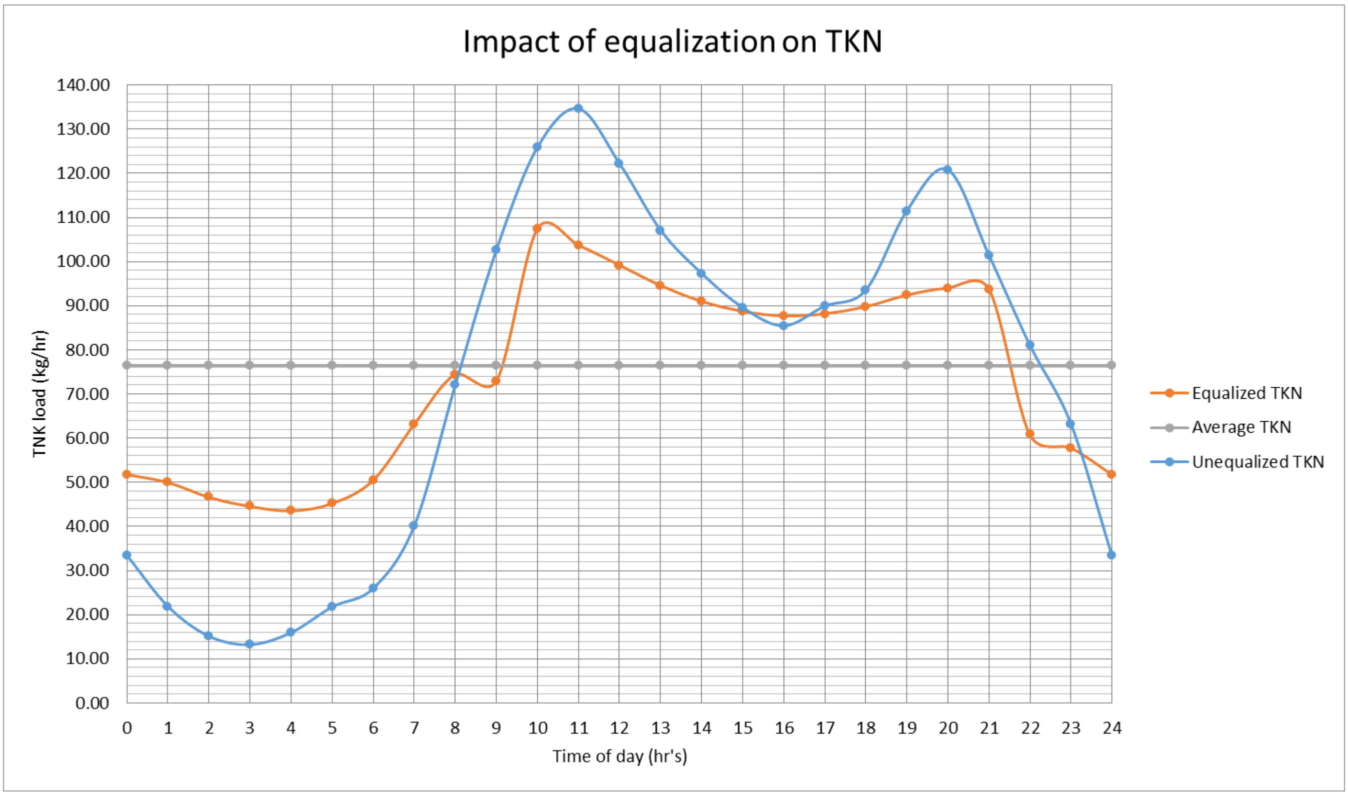


Figure 7.4. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised TKN loading comparison

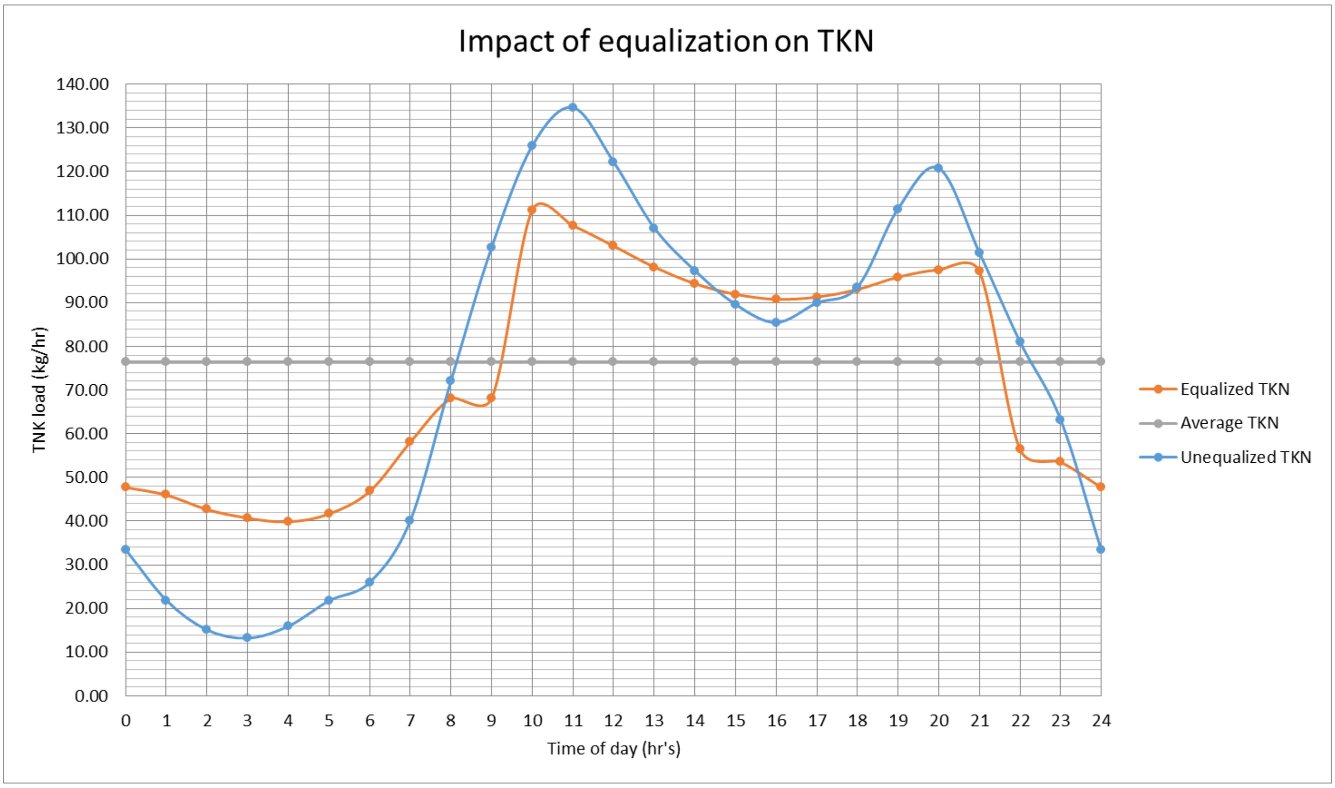


Figure 7.4. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised TKN loading comparison

- Pollutant: Total Suspended Solids (TSS) loading

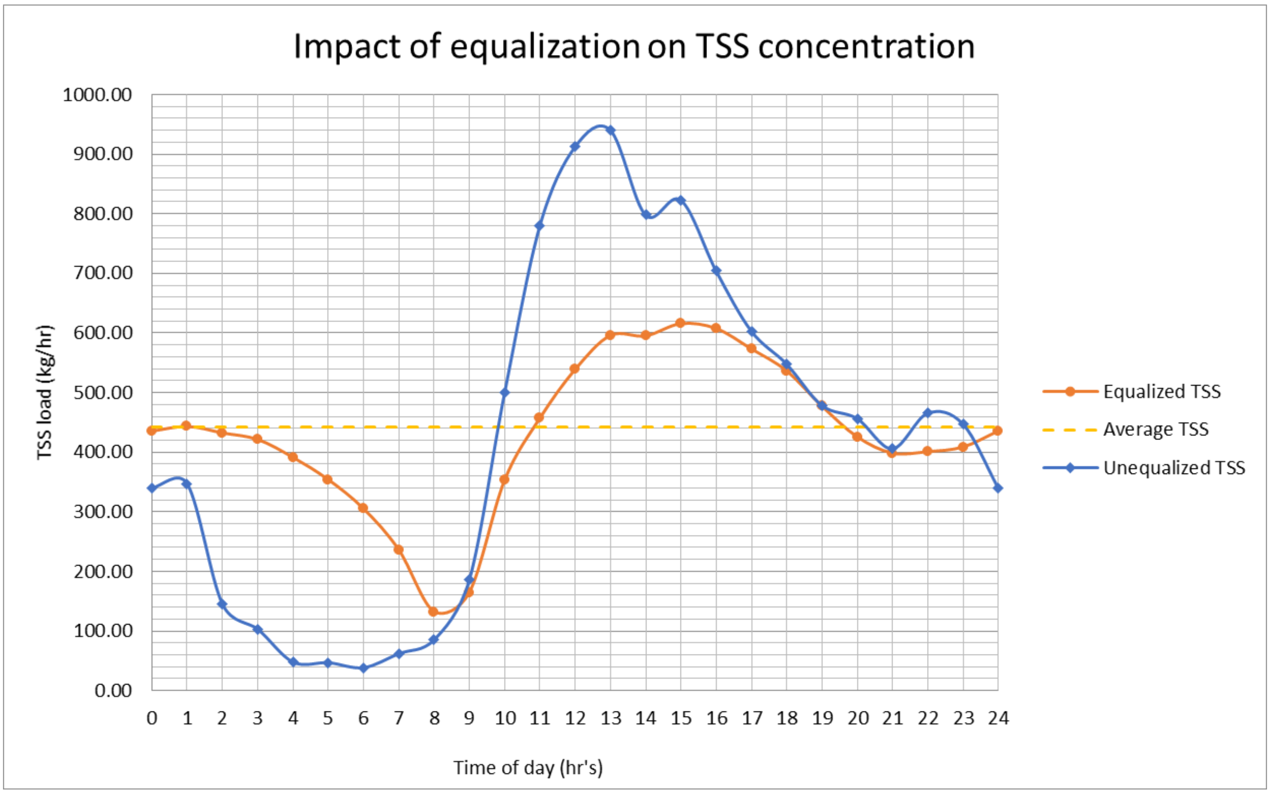


Figure 7.5. 1: Numerical analysis: ADF release: Comparative Unequalised vs Equalised Total Suspended Solids (TSS) loading

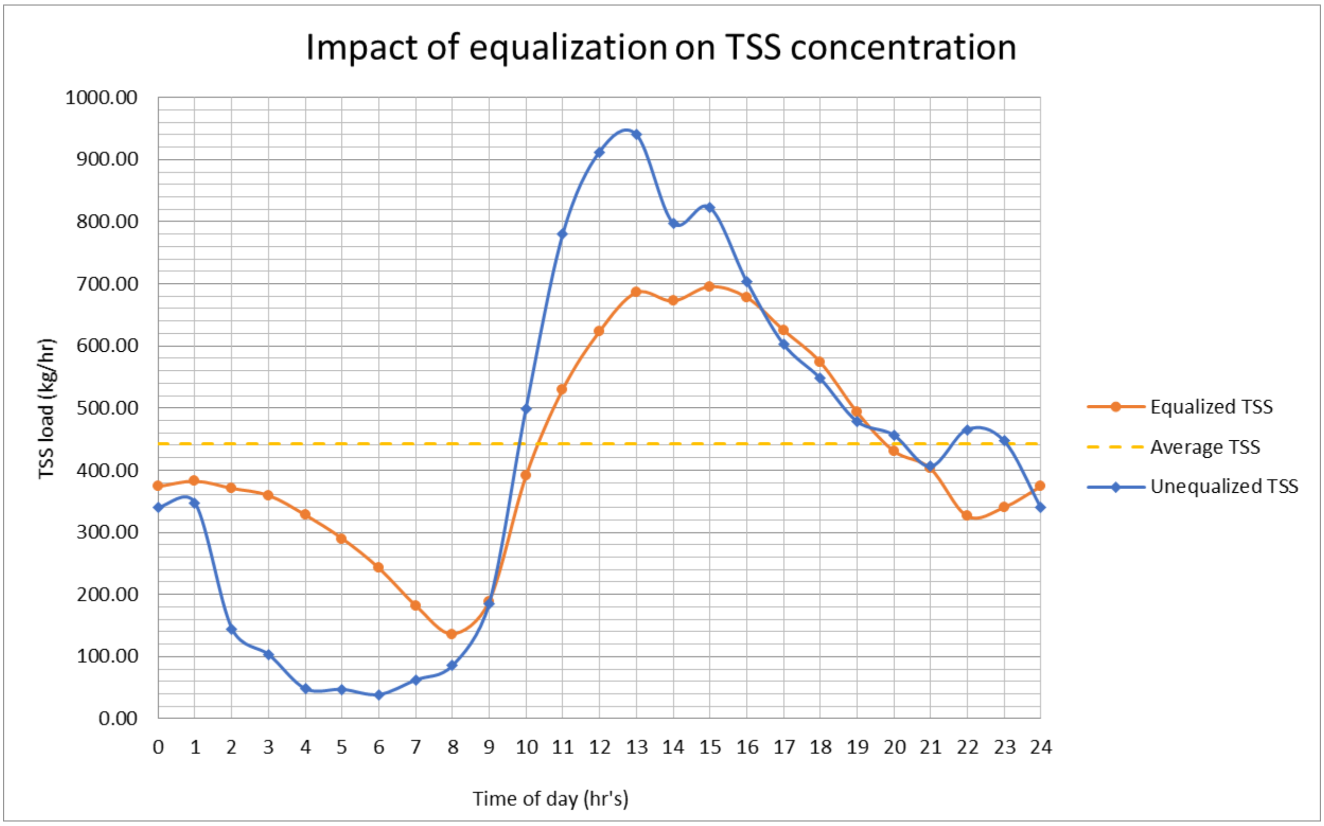


Figure 7.5. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised TSS loading comparison

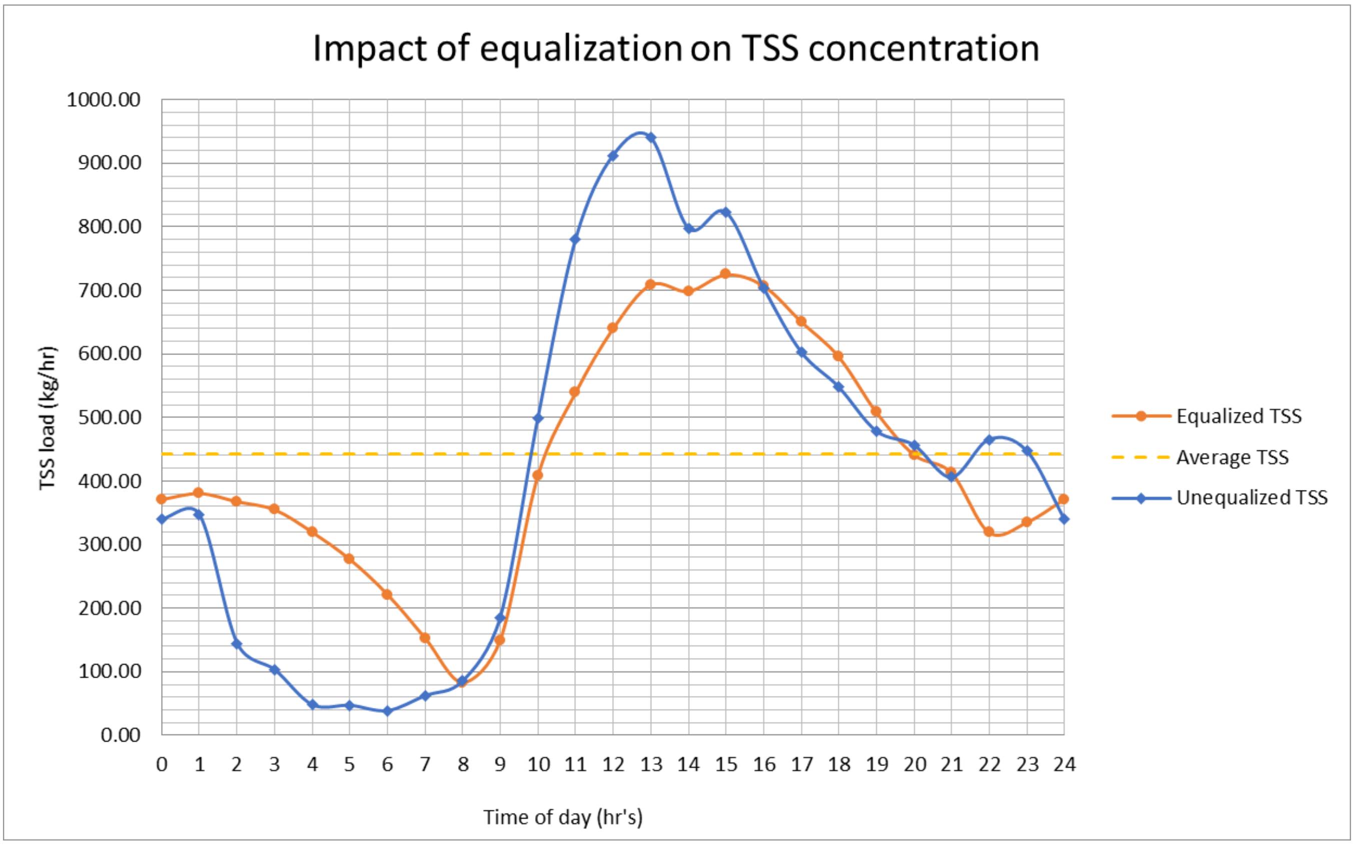


Figure 7.5. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised TSS loading comparison

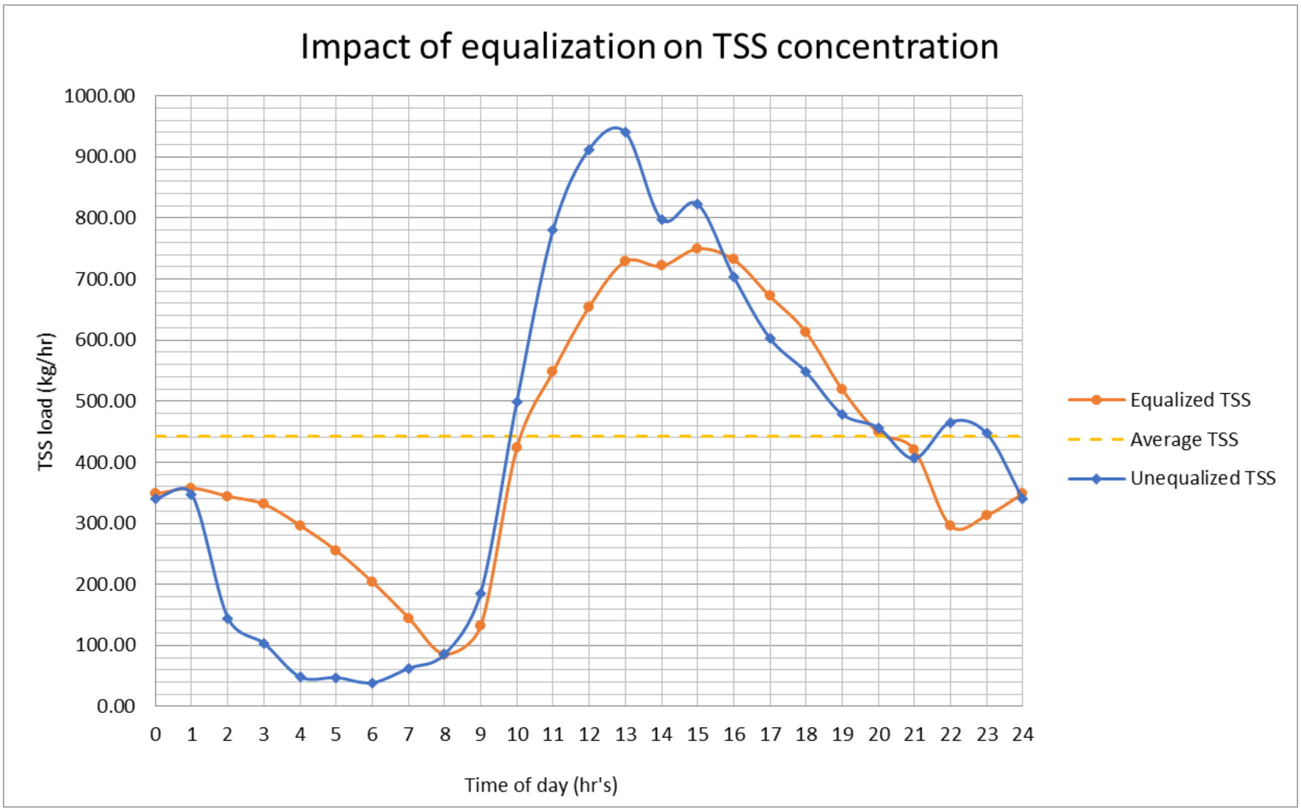


Figure 7.5. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised TSS loading comparison

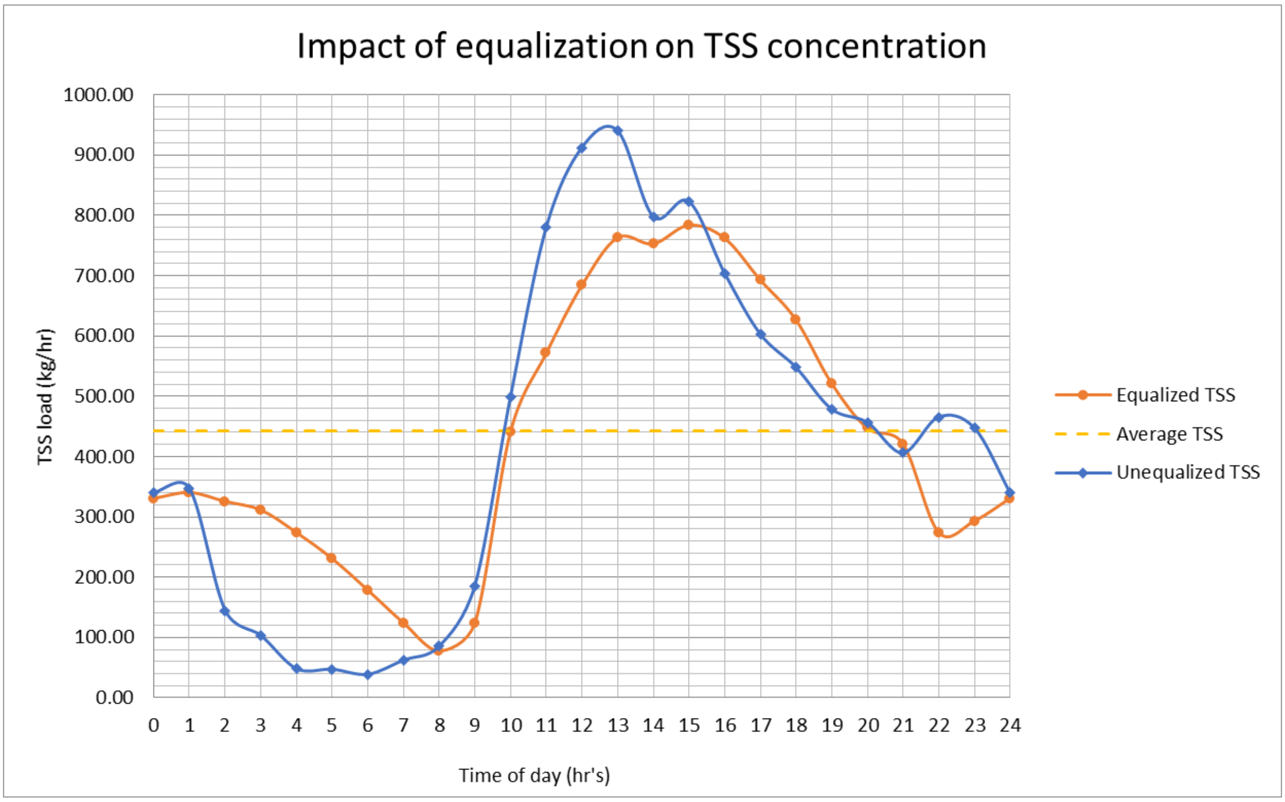


Figure 7.5. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised TSS loading comparison

- Pollutant: Total Solids (TS) loading

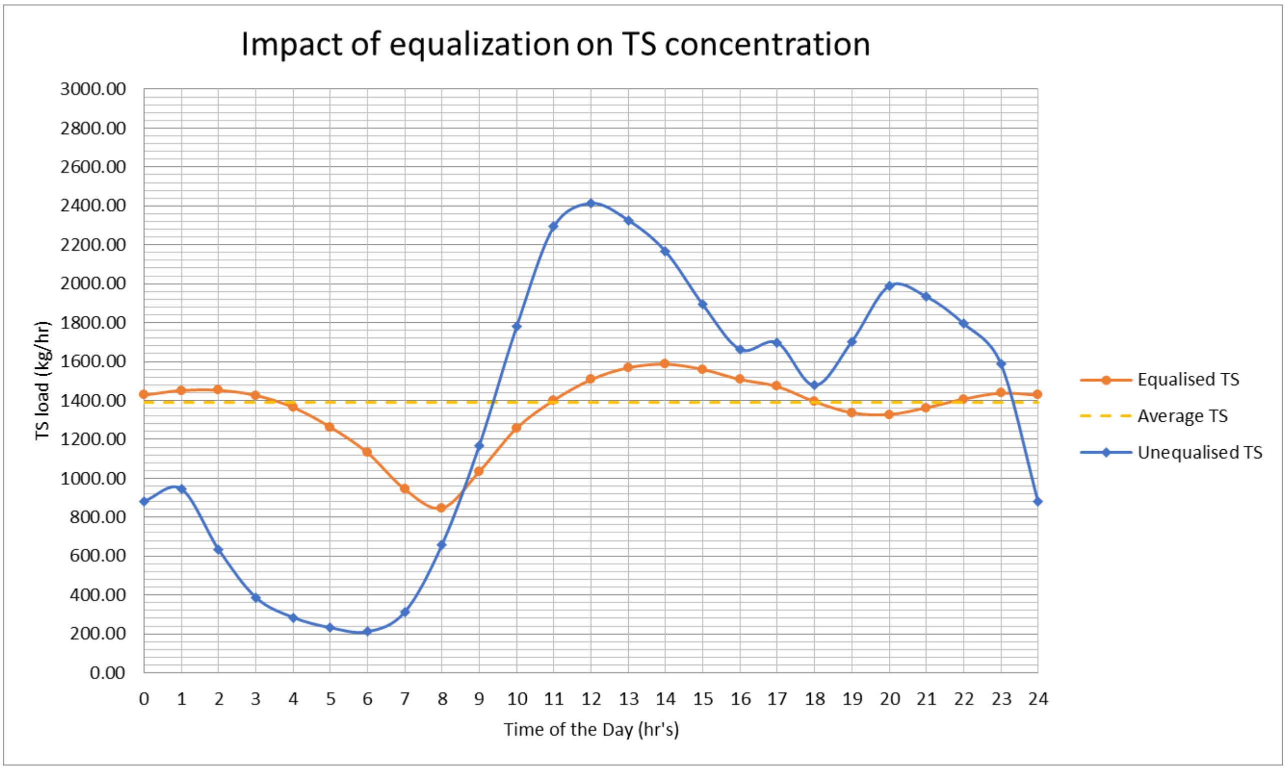


Figure 7.6. 1: Numerical analysis: ADF release: Comparative Unequalised vs Equalised Total Solids (TS) loading

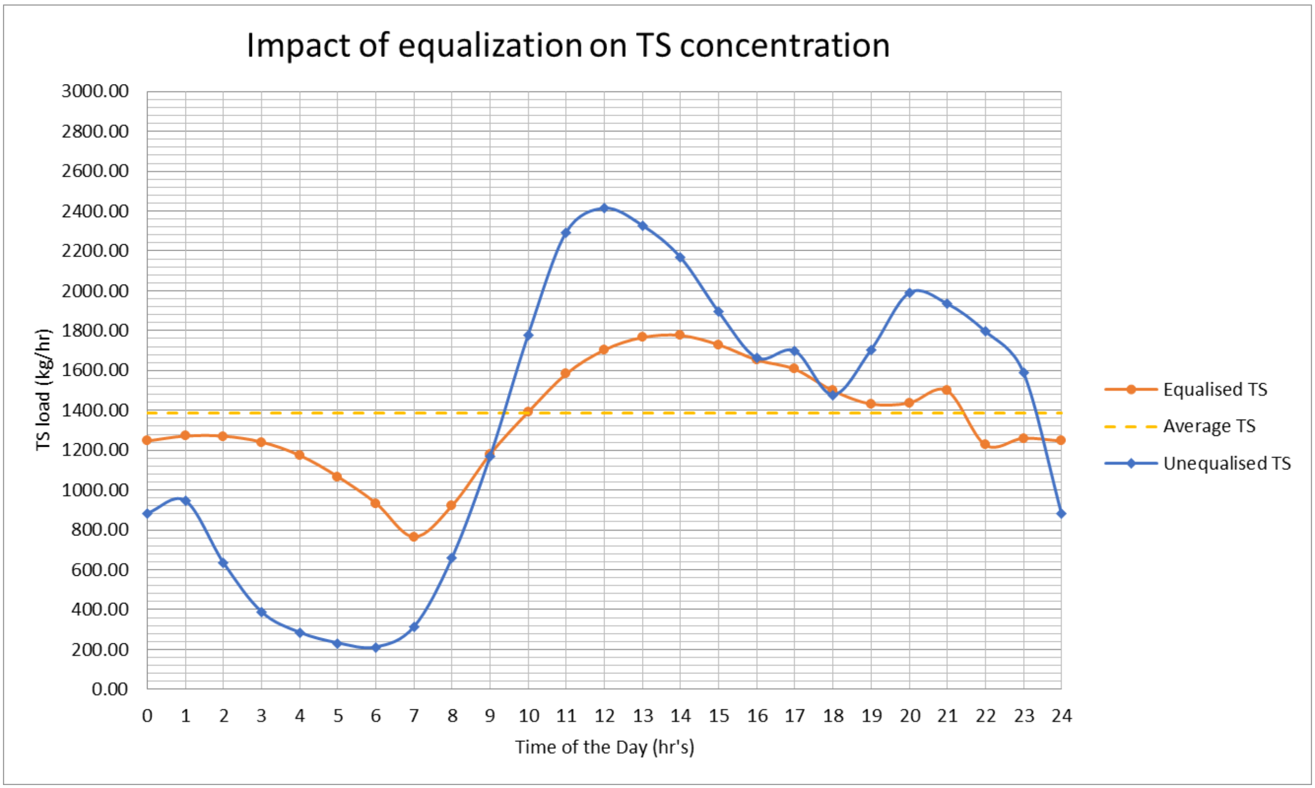


Figure 7.6. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised Total Solids (TS) loading comparison

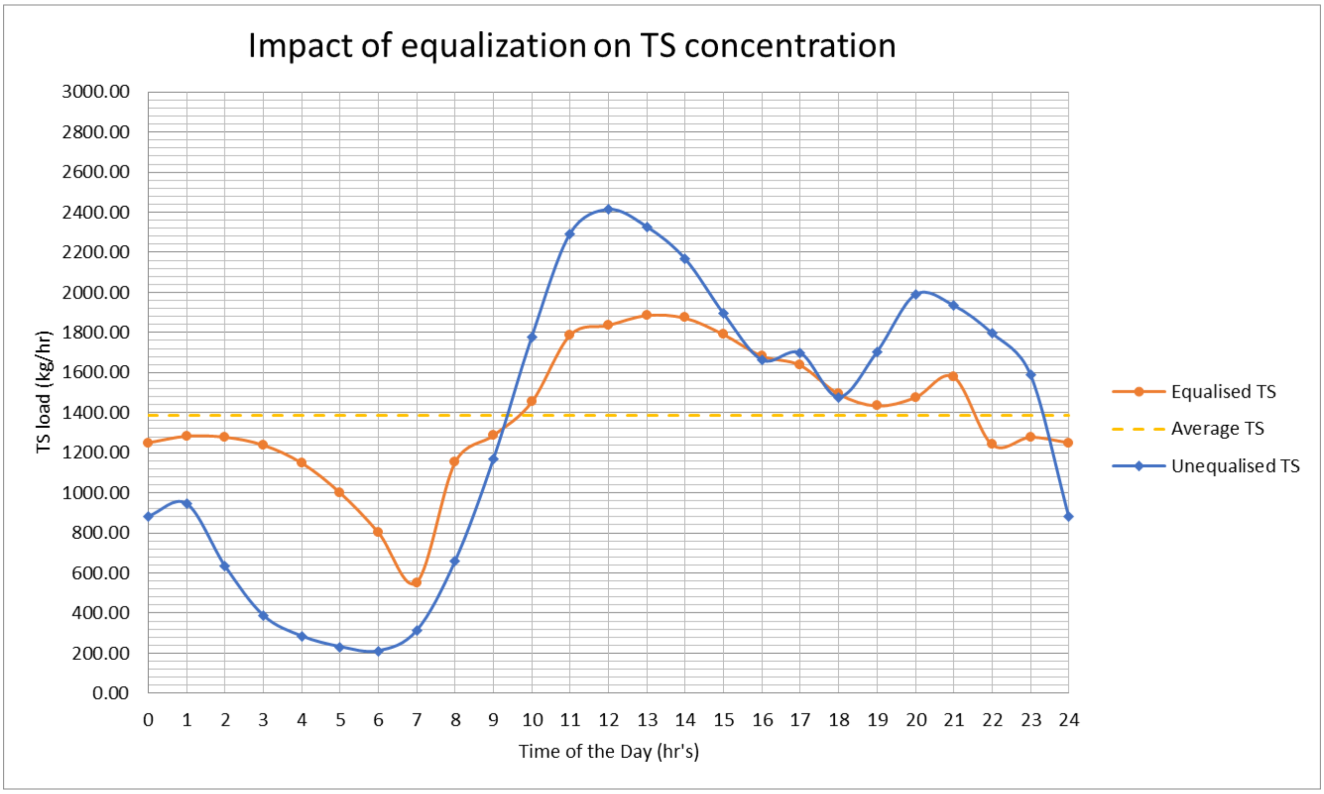


Figure 7.6. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised Total Solids (TS) loading comparison

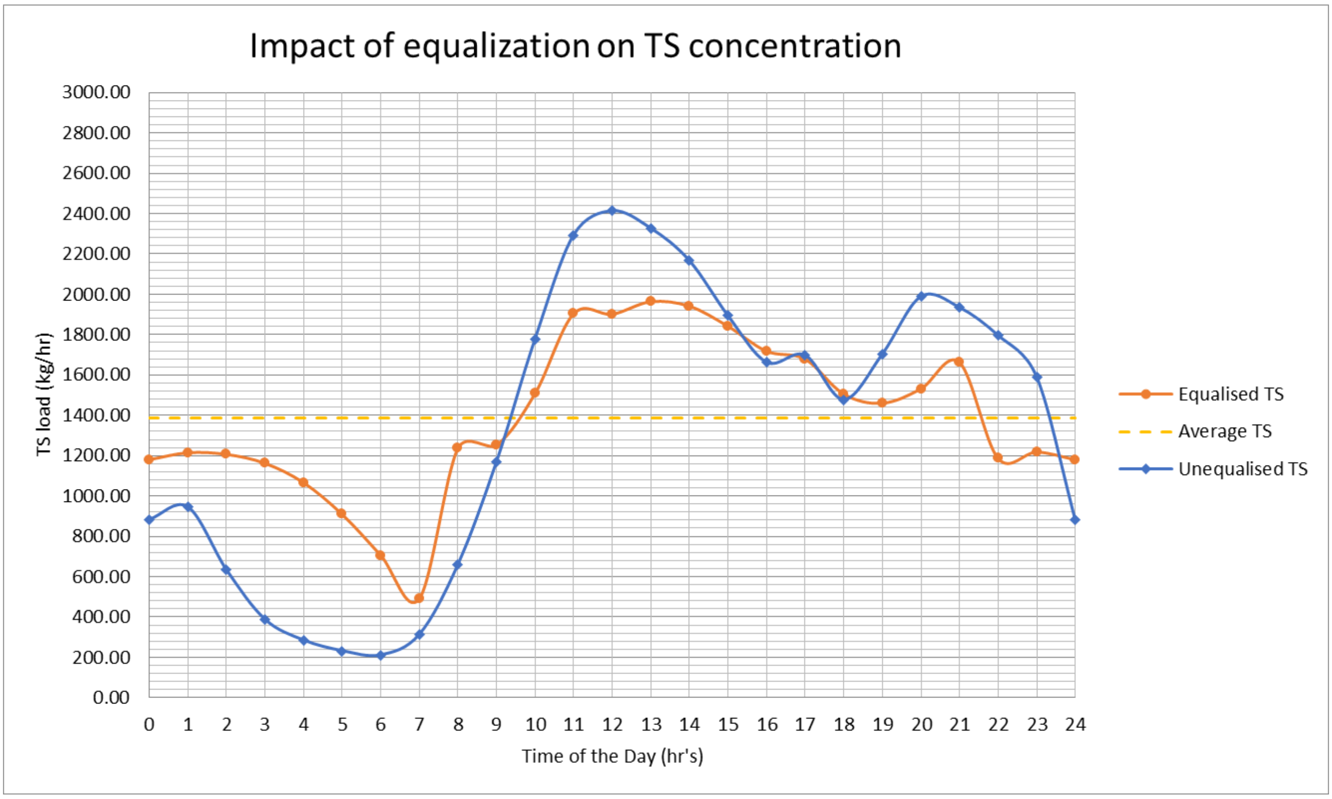


Figure 7.6. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised Total Solids (TS) loading comparison

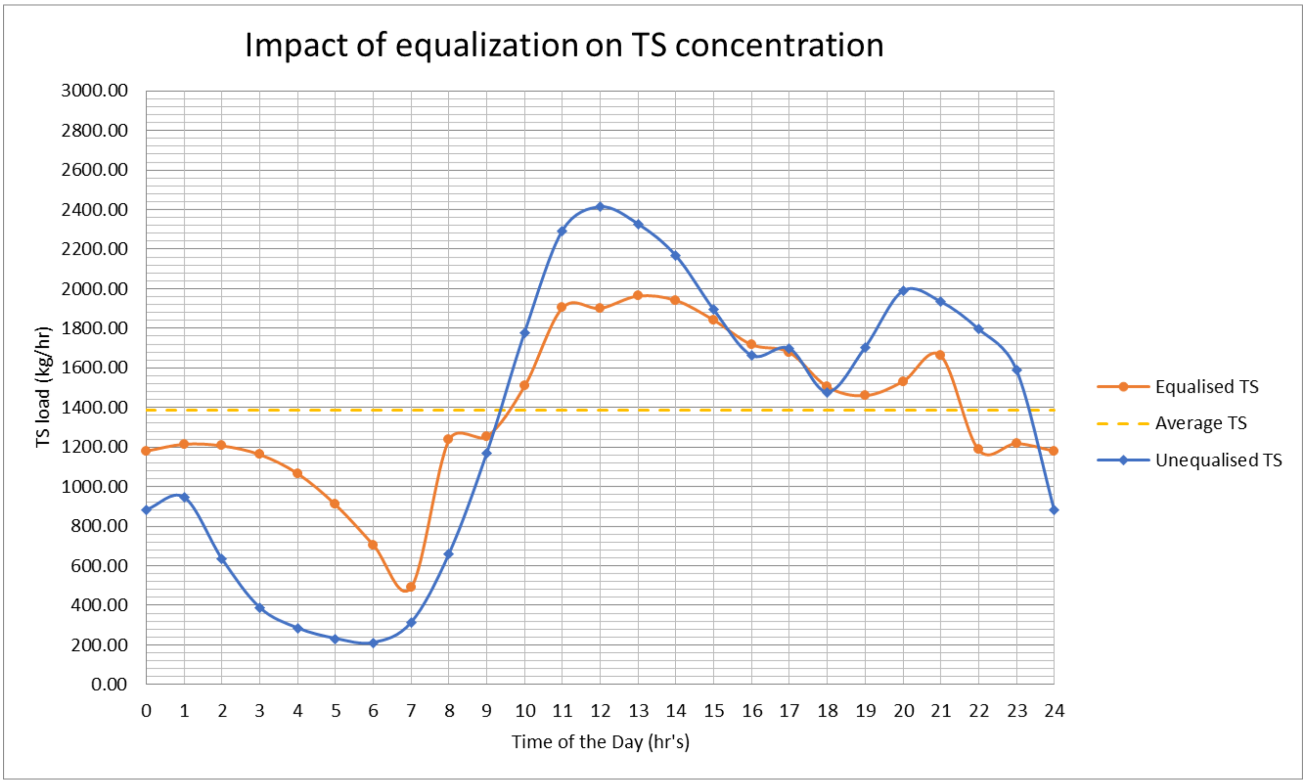


Figure 7.6. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised Total Solids (TS) loading comparison

- Pollutant: Total Dissolved Solids (TDS) loading

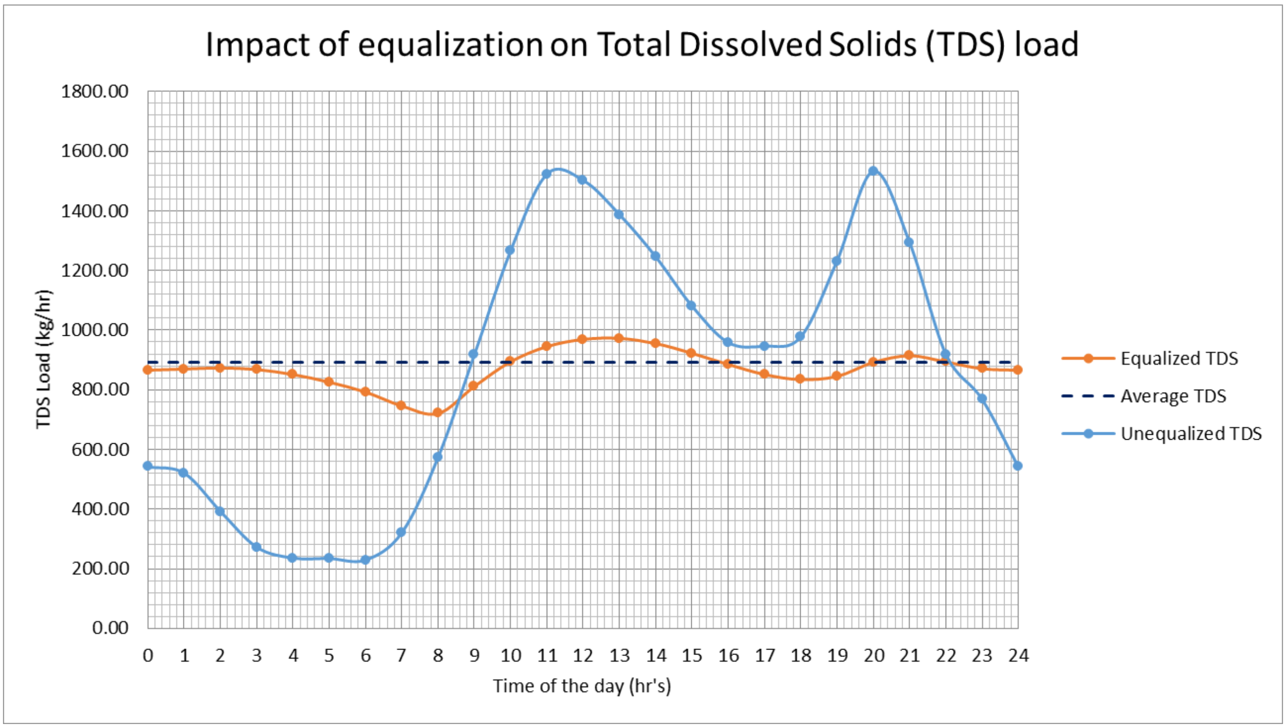


Figure 7.7. 1: Numerical analysis: ADF release: Comparative Unequalised vs Equalised Total Dissolved Solids (TDS) loading

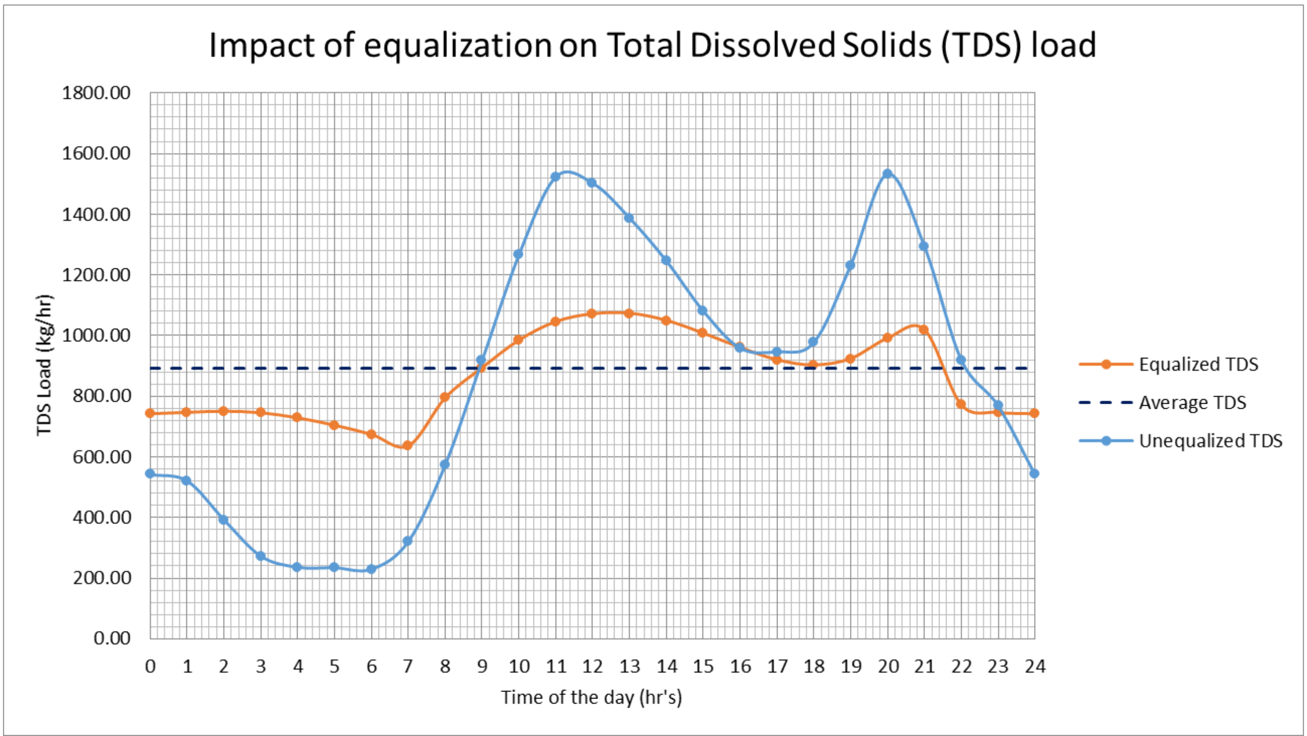


Figure 7.7. 2: Numerical analysis: ADF + 10% release: Unequalised vs Equalised Total Dissolved Solids (TDS) loading comparison

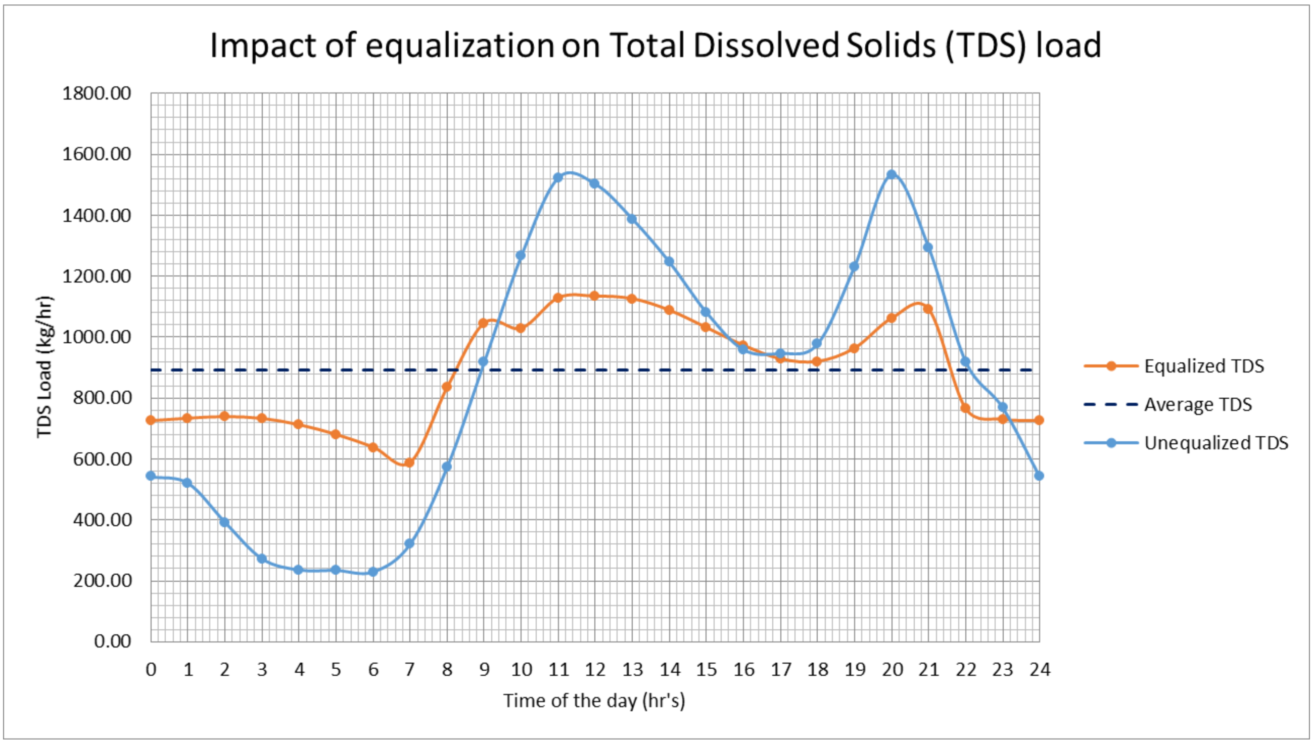


Figure 7.7. 3: Numerical analysis: ADF + 15% release: Unequalised vs Equalised Total Dissolved Solids (TDS) loading comparison

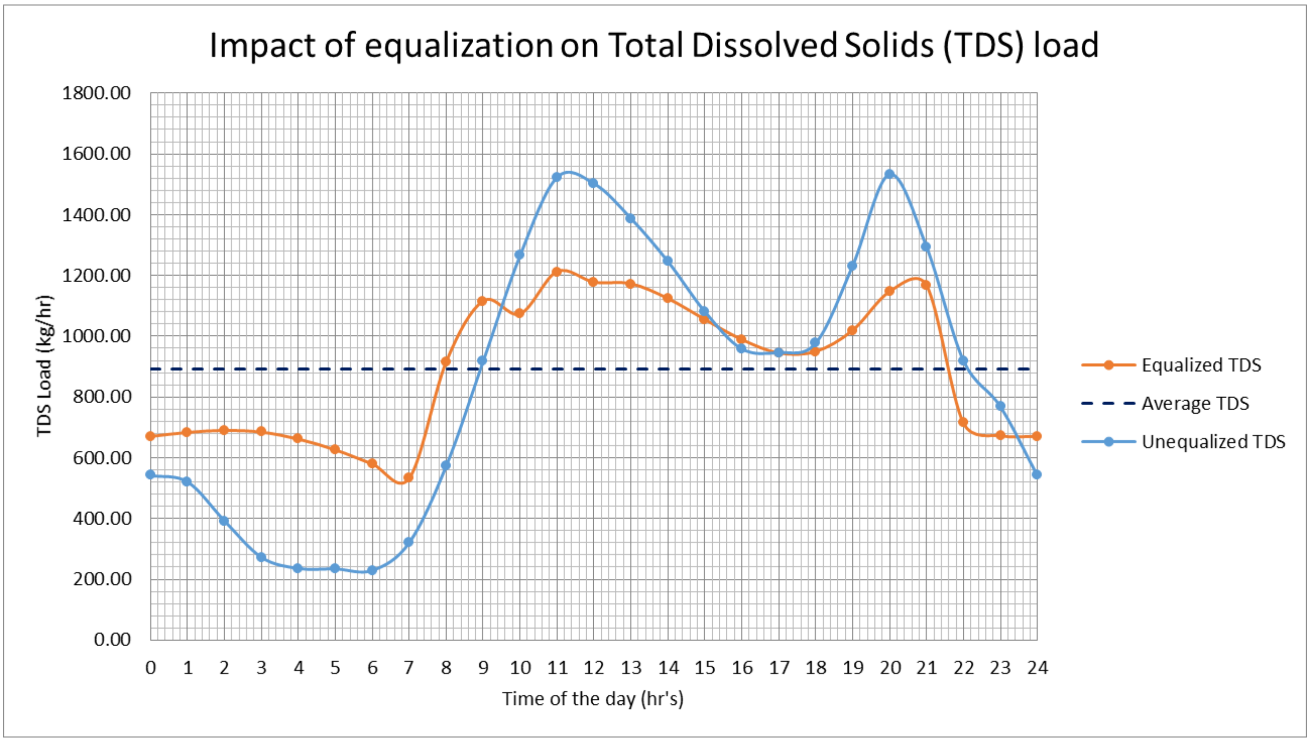


Figure 7.7. 4: Numerical analysis: ADF + 20% release: Unequalised vs Equalised Total Dissolved Solids (TDS) loading comparison

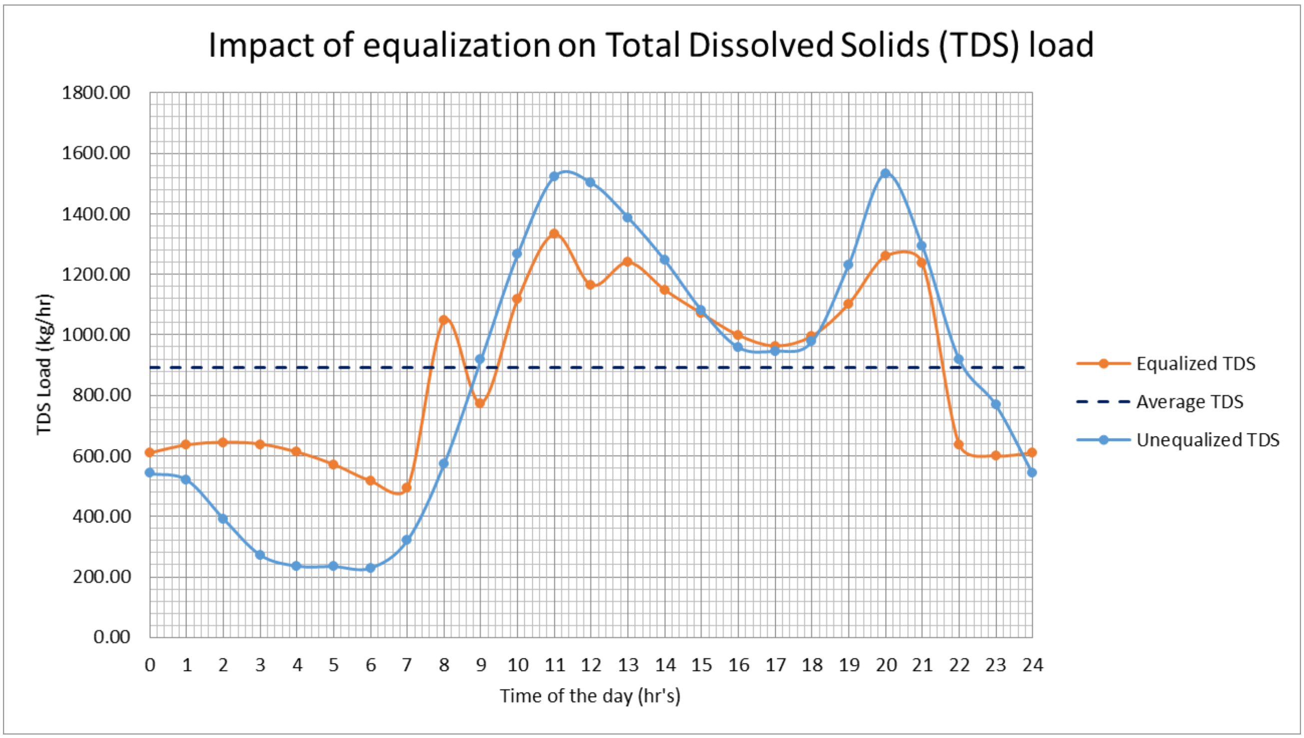


Figure 7.7. 5: Numerical analysis: ADF + 25% release: Unequalised vs Equalised Total Dissolved Solids (TDS) loading comparison

APPENDIX 8: Current treatment plant operation: Pollutants load daily diurnal variation (based on sampled test analysis of various unit process outflows)

The current Unequalised treatment plant operation pollutant daily diurnal load variation results based on sampled tests done for the unit processes along the treatment train are illustrated in Figures 8.1 to 8.8.

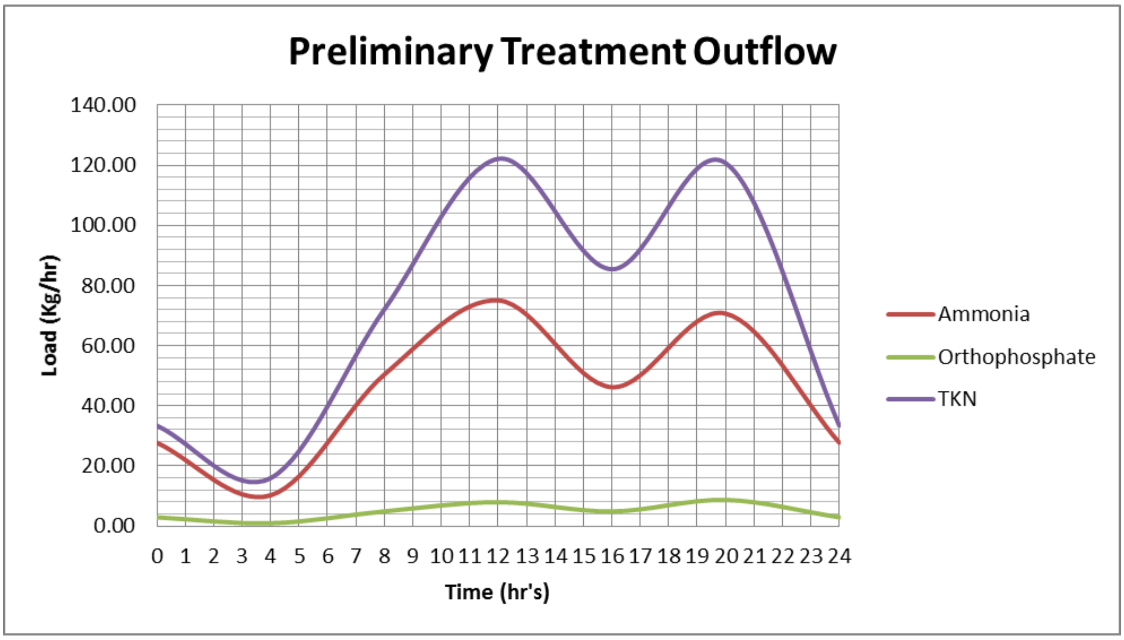


Figure 8. 1: Preliminary Treatment Outflow: Current treatment operation - NH_4, PO_4 & TKN

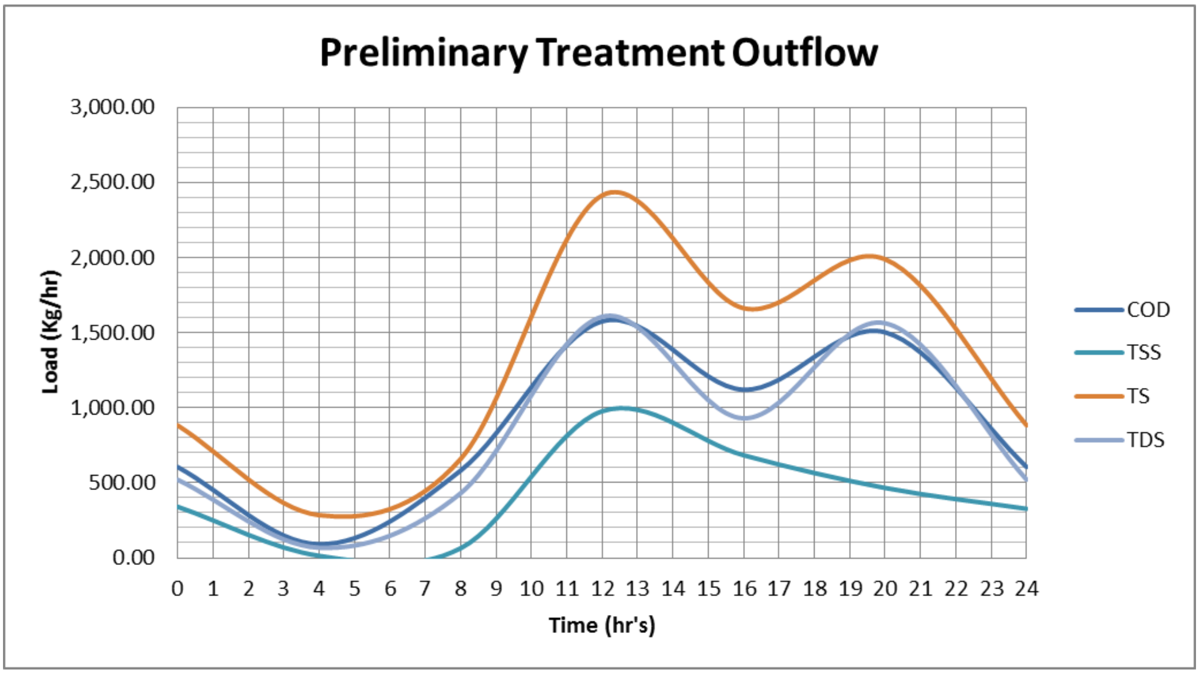


Figure 8. 2: Preliminary Treatment Outflow: Current treatment operation - COD; TSS; TS & TDS

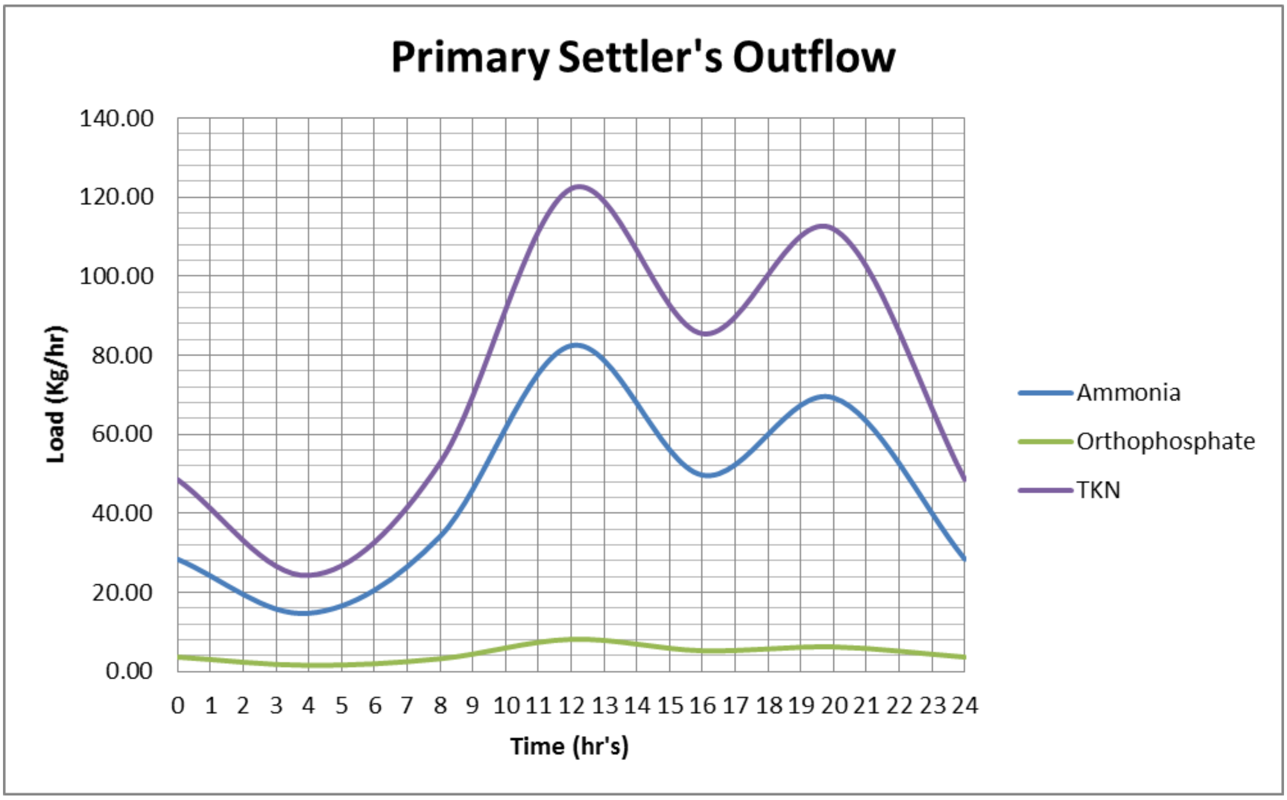


Figure 8. 3: Primary Treatment Outflow: Current treatment operation - NH_4, PO_4 & TKN

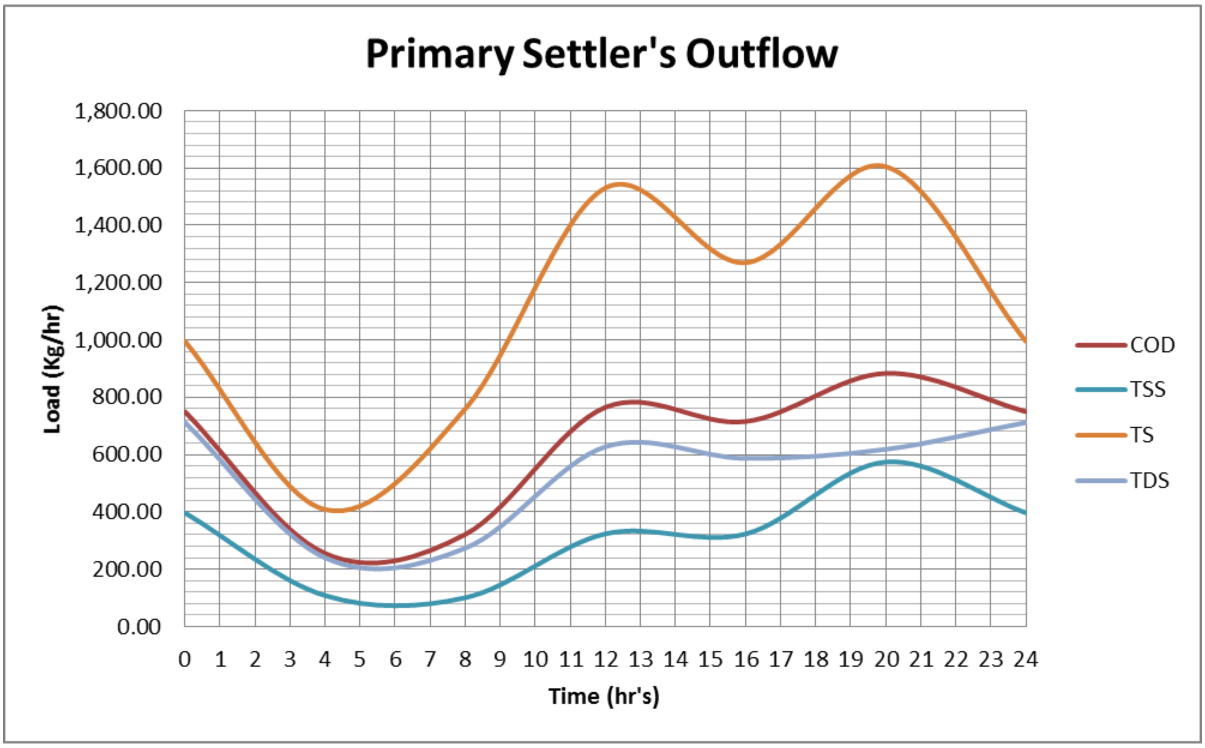


Figure 8. 4: Primary Treatment Outflow: Current treatment operation - COD; TSS; TS & TDS

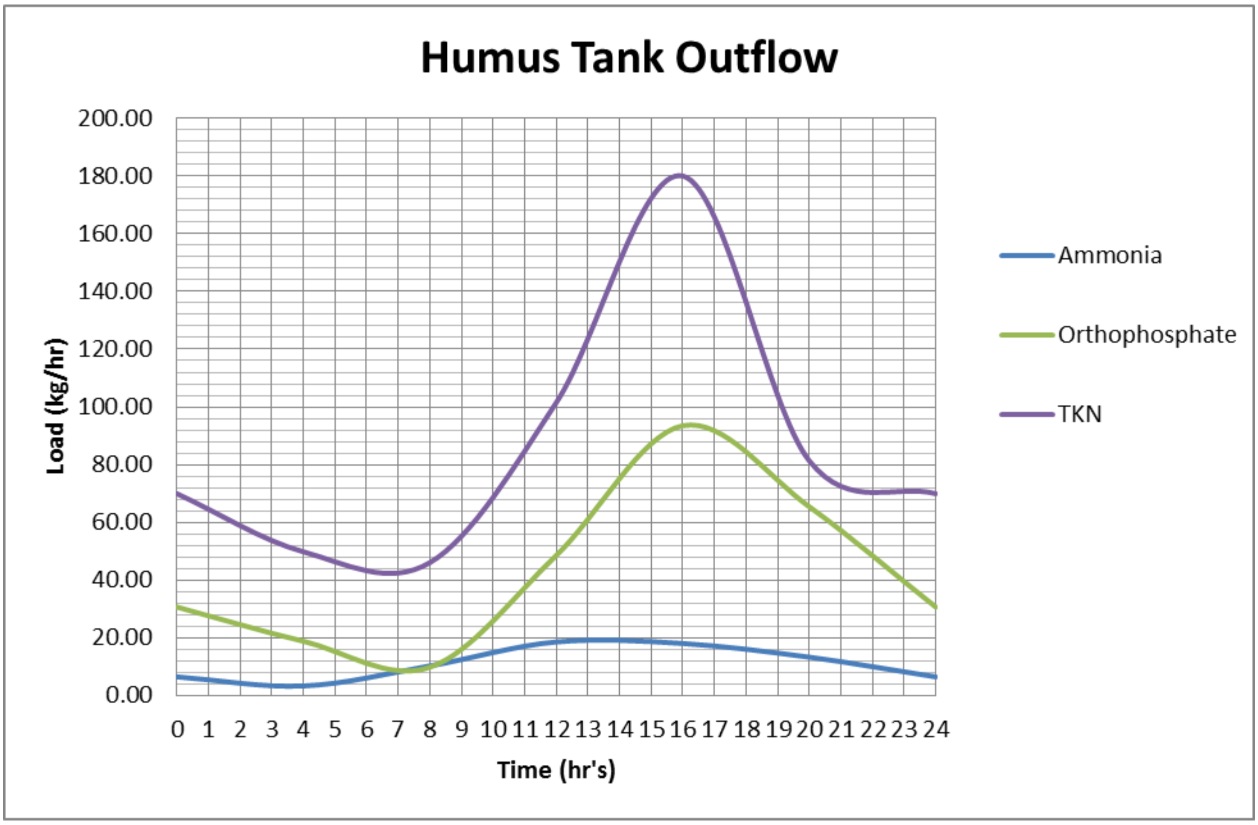


Figure 8. 5: Humus Tank Outflow: Current treatment operation - NH_4, PO_4 & TKN

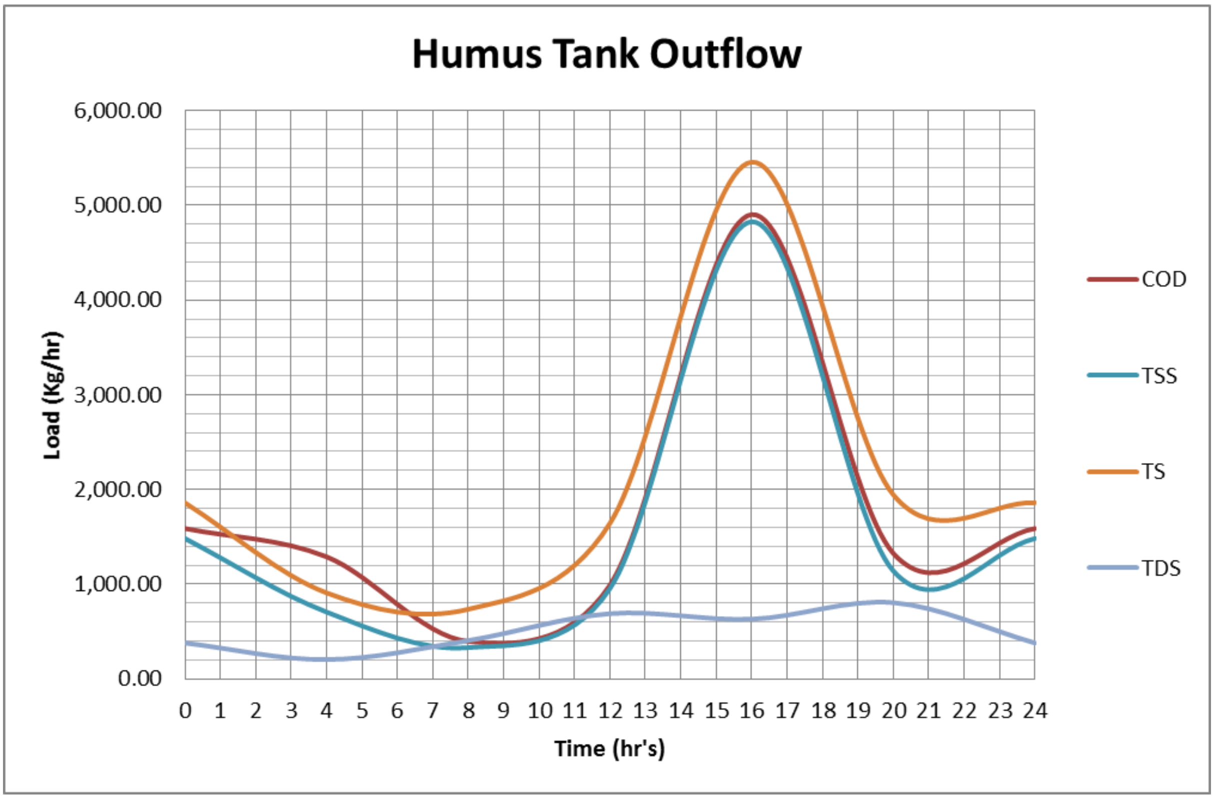


Figure 8. 6: Humus Tank Outflow: Current treatment operation - COD; TSS; TS & TDS

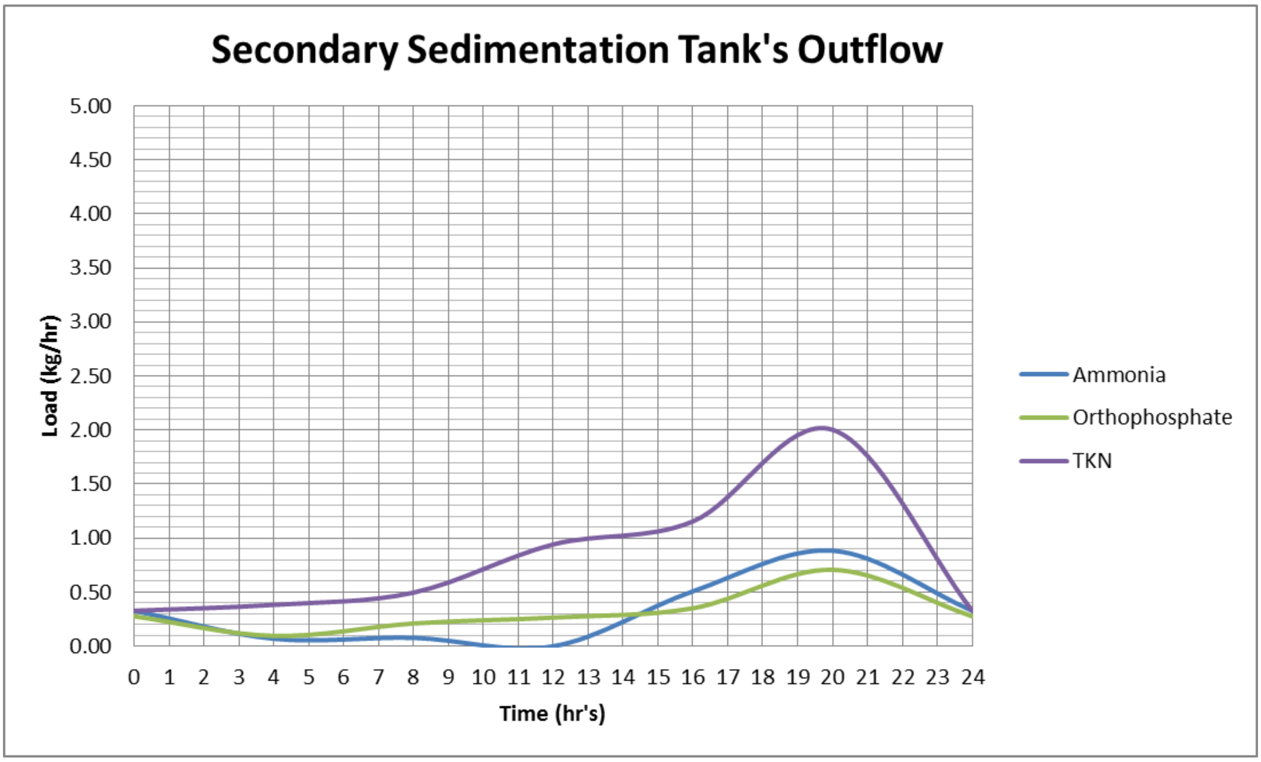


Figure 8. 7: Secondary Sedimentation Tank Outflow: Current treatment operation - NH_4, PO_4 & TKN

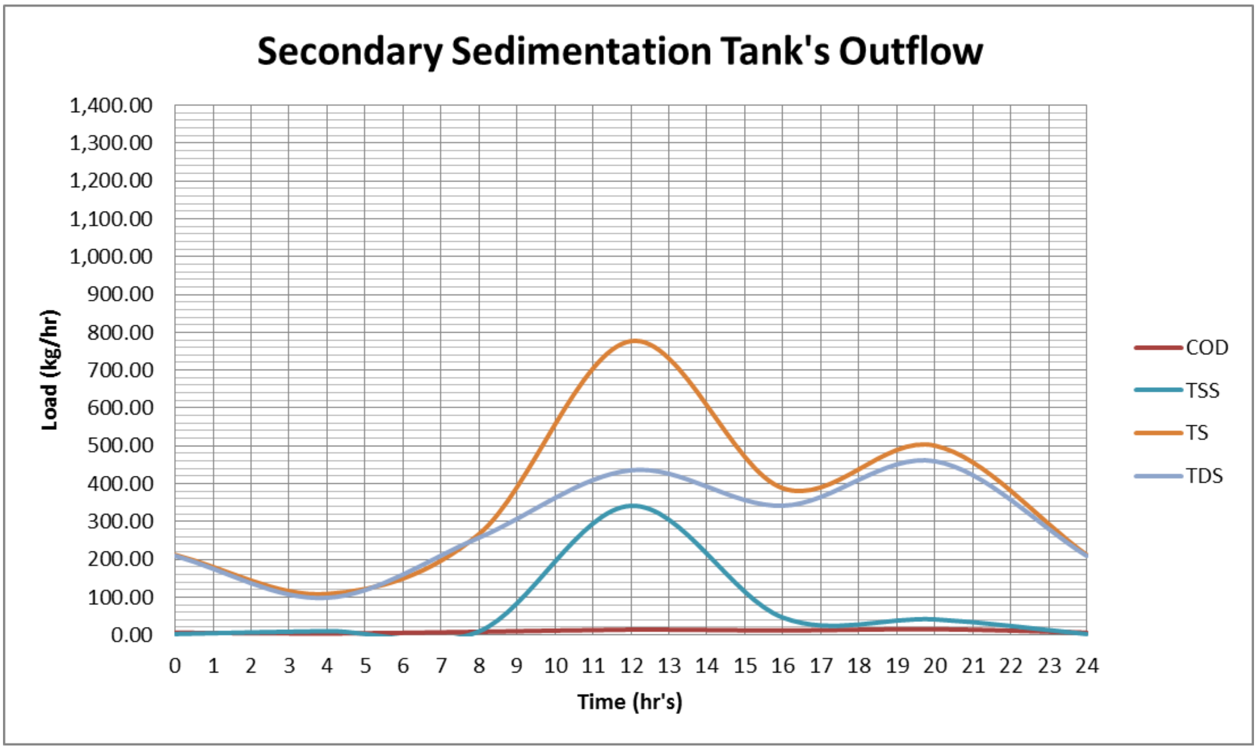
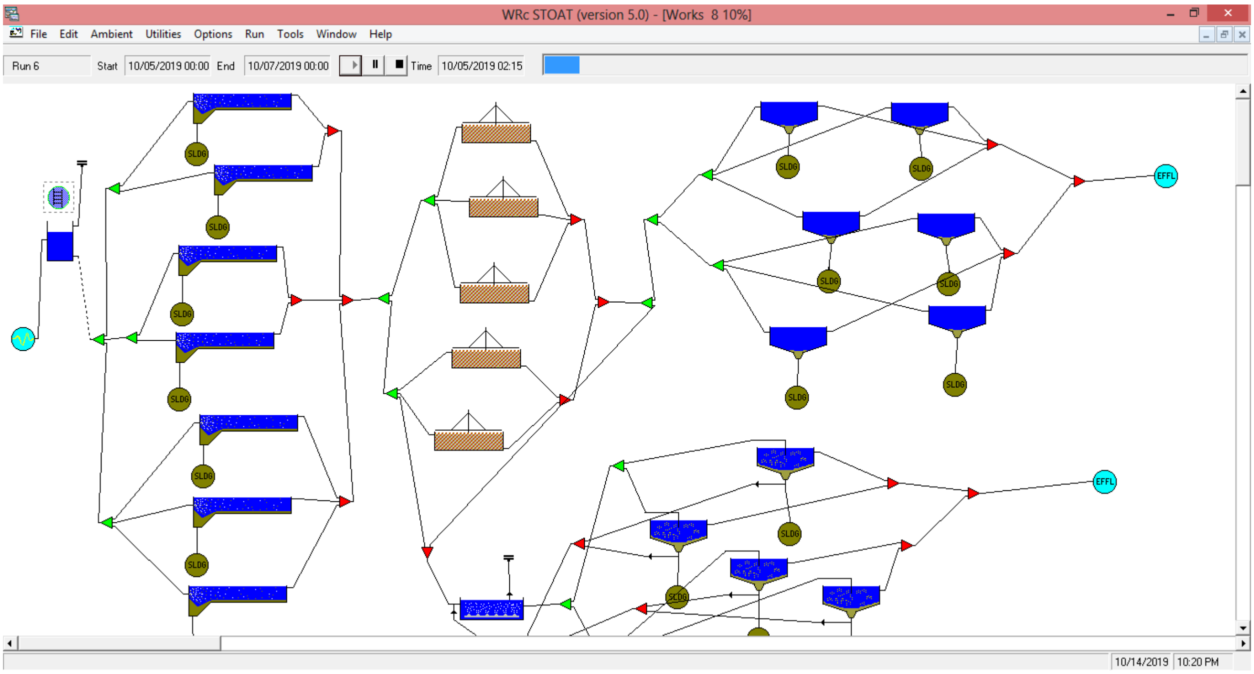


Figure 8. 8: Secondary Sedimentation Tank Outflow: Current treatment operation – COD; TSS; TS & TDS

APPENDIX 9: STOAT unit processes descriptions



1. Influent file

Influent profile [F:\Nust\Data\Stoat\Query\Database\Correct Latested Improved Influent edited.cod]

	Elapsed time (h)	Flow (m^3/h)	Temperature (deg. C)	Volatile fatty acids (mg COD/l)	Soluble biodegrable COD (mg/l)	Soluble nondegradable COD (mg/l)	Particulate biodegradable COD (mg/l)	Particulate nondegradable COD (mg/l)	Volatile solids (mg/l)	Non-volatile solids (mg/l)	Ammonia (mg/l)
1	0.000000	630.130000	15.000000	5.000000	307.200000	76.800000	460.800000	115.200000	405.000000	135.000000	40.000000
2	1.000000	488.410000	15.000000	5.000000	230.400000	57.600000	345.600000	86.400000	465.000000	155.000000	40.000000
3	2.000000	349.900000	15.000000	5.000000	176.000000	44.000000	264.000000	66.000000	382.500000	127.500000	40.000000
4	3.000000	294.740000	15.000000	5.000000	108.800000	27.200000	163.200000	40.800000	240.000000	80.000000	40.000000
5	4.000000	343.660000	15.000000	5.000000	91.200000	22.800000	136.800000	34.200000	112.500000	37.500000	40.000000
6	5.000000	326.860000	15.000000	5.000000	107.200000	26.800000	160.800000	40.200000	30.000000	10.000000	40.000000
7	6.000000	344.020000	15.000000	5.000000	145.600000	36.400000	218.400000	54.600000	0.000000	0.000000	40.000000
8	7.000000	574.310000	15.000000	5.000000	192.000000	48.000000	288.000000	72.000000	15.000000	5.000000	40.000000
9	8.000000	976.090000	15.000000	5.000000	240.000000	60.000000	360.000000	90.000000	82.500000	27.500000	40.000000
10	9.000000	1207.820000	15.000000	5.000000	281.600000	70.400000	422.400000	105.600000	180.000000	60.000000	40.000000
11	10.000000	1528.070000	15.000000	5.000000	310.400000	77.600000	465.600000	116.400000	285.000000	95.000000	40.000000
12	11.000000	1530.380000	15.000000	5.000000	332.800000	83.200000	499.200000	124.800000	382.500000	127.500000	40.000000
13	12.000000	1412.860000	15.000000	5.000000	342.400000	85.600000	513.600000	128.400000	465.000000	155.000000	40.000000
14	13.000000	1332.770000	15.000000	5.000000	339.200000	84.800000	508.800000	127.200000	513.750000	171.250000	40.000000
15	14.000000	1261.580000	15.000000	5.000000	332.800000	83.200000	499.200000	124.800000	528.750000	176.250000	40.000000
16	15.000000	1165.950000	15.000000	5.000000	321.600000	80.400000	482.400000	120.600000	508.500000	169.500000	40.000000
17	16.000000	1142.710000	15.000000	5.000000	310.400000	77.600000	465.600000	116.400000	457.500000	152.500000	40.000000
18	17.000000	1222.360000	15.000000	5.000000	307.200000	76.800000	460.800000	115.200000	382.500000	127.500000	40.000000
19	18.000000	1159.810000	15.000000	5.000000	307.200000	76.800000	460.800000	115.200000	309.750000	103.250000	40.000000
20	19.000000	1574.680000	15.000000	5.000000	320.000000	80.000000	480.000000	120.000000	262.500000	87.500000	40.000000
21	20.000000	1370.440000	15.000000	5.000000	326.400000	81.600000	489.600000	122.400000	232.500000	77.500000	40.000000
22	21.000000	1166.490000	15.000000	5.000000	329.600000	82.400000	494.400000	123.600000	240.000000	80.000000	40.000000
23	22.000000	1023.460000	15.000000	5.000000	332.800000	83.200000	499.200000	124.800000	277.500000	92.500000	40.000000
24	23.000000	921.560000	15.000000	5.000000	326.400000	81.600000	489.600000	122.400000	345.000000	115.000000	40.000000
25	24.000000	630.130000	15.000000	5.000000	307.200000	76.800000	460.800000	115.200000	405.000000	135.000000	40.000000

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	Non-volatile solids (mg/l)	Ammonia (mg/l)	Nitrates (mg/l)	Soluble org. nitrogen (mg/l)	Particulate org. nitrogen (mg/l)	Soluble phosphate (mg/l)	Dissolved oxygen (mg/l)	Sol. non-deg. org. N (mg N/l)	Part. non-deg. org. N (mg N/l)	Particulate degradable org. P (mg P/l)	Particulate nondegradable org. P (mg P/l)
1	135.000000	40.000000	0.000000	3.600000	5.400000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
2	155.000000	40.000000	0.000000	4.120000	6.180000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
3	127.500000	40.000000	0.000000	4.800000	7.200000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
4	80.000000	40.000000	0.000000	6.000000	9.000000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
5	37.500000	40.000000	0.000000	7.200000	10.800000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
6	10.000000	40.000000	0.000000	8.400000	12.600000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
7	0.000000	40.000000	0.000000	9.240000	13.860000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
8	5.000000	40.000000	0.000000	10.400000	15.600000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
9	27.500000	40.000000	0.000000	11.200000	16.800000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
10	60.000000	40.000000	0.000000	11.680000	17.520000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
11	95.000000	40.000000	0.000000	12.200000	18.300000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
12	127.500000	40.000000	0.000000	12.600000	18.900000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
13	155.000000	40.000000	0.000000	12.800000	19.200000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
14	171.250000	40.000000	0.000000	12.800000	19.200000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
15	176.250000	40.000000	0.000000	13.080000	19.620000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
16	169.500000	40.000000	0.000000	13.400000	20.100000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
17	152.500000	40.000000	0.000000	13.600000	20.400000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
18	127.500000	40.000000	0.000000	13.840000	20.760000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
19	103.250000	40.000000	0.000000	13.880000	20.820000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
20	87.500000	40.000000	0.000000	14.080000	21.120000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
21	77.500000	40.000000	0.000000	13.600000	20.400000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
22	80.000000	40.000000	0.000000	12.600000	18.900000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
23	92.500000	40.000000	0.000000	11.080000	16.620000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
24	115.000000	40.000000	0.000000	7.880000	11.820000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000
25	135.000000	40.000000	0.000000	3.600000	5.400000	10.000000	0.000000	0.000000	0.000000	0.000000	0.000000

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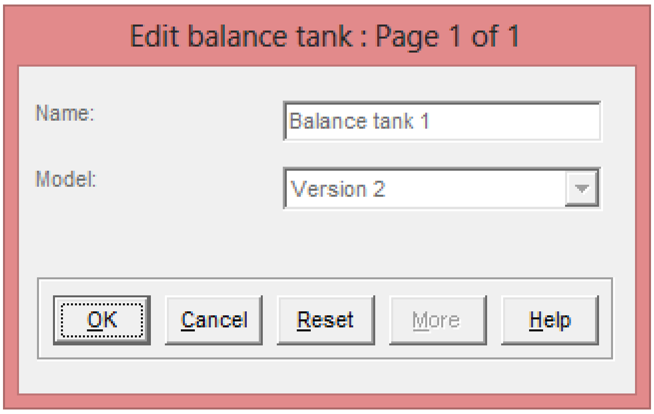
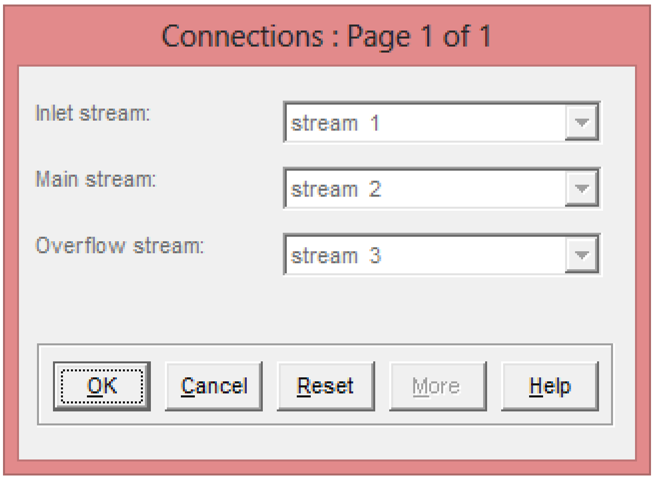
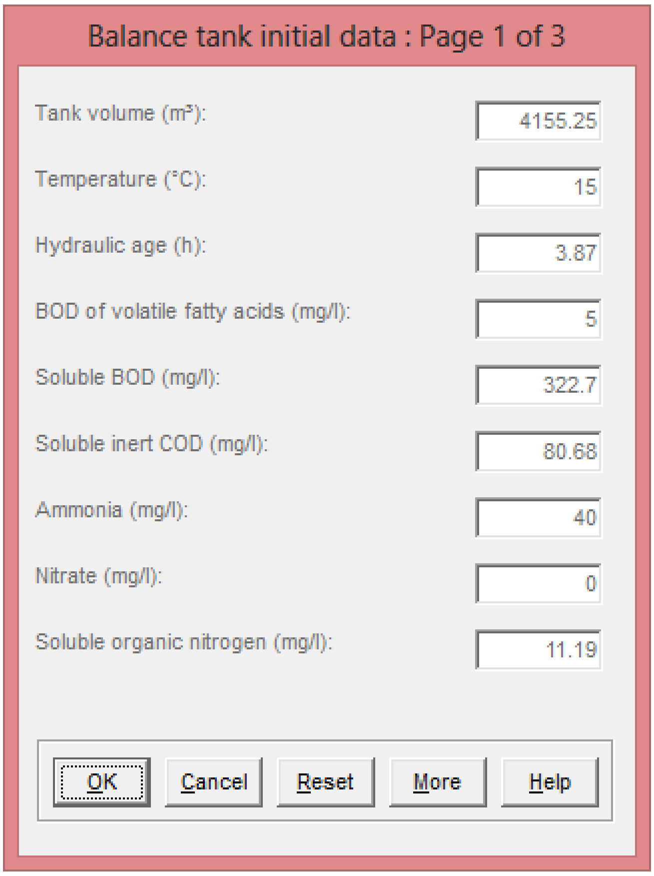
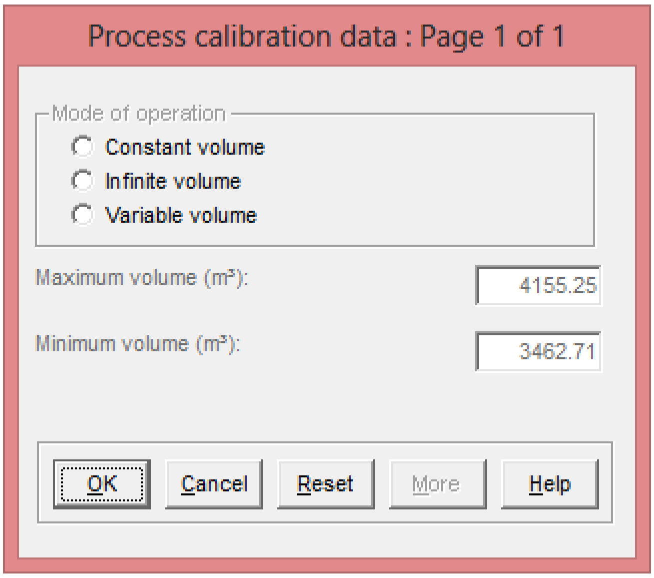
2. Equalization tank



3. Streams

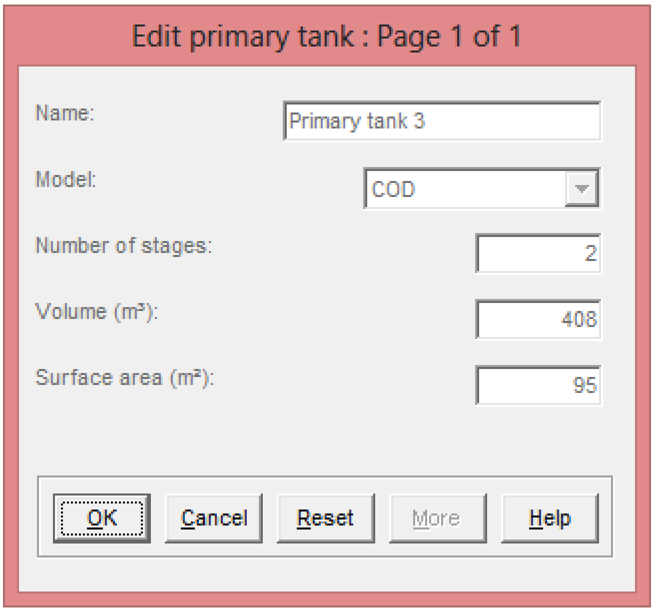

Connections : Page 1 of 1
Inlet stream:
stream 1
Main stream:
stream 2
Overflow stream:
stream 3
OK
Cancel
Reset
More
Help

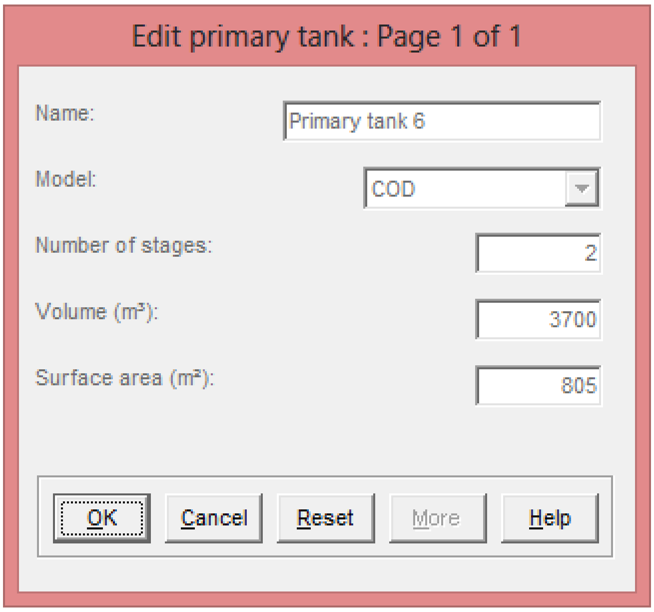

Balance tank initial data : Page 1 of 3
Tank volume (m³):
4155.25
Temperature (°C):
15
Hydraulic age (h):
3.87
BOD of volatile fatty acids (mg/l):
5
Soluble BOD (mg/l):
322.7
Soluble inert COD (mg/l):
80.68
Ammonia (mg/l):
40
Nitrate (mg/l):
0
Soluble organic nitrogen (mg/l):
11.19
OK
Cancel
Reset
More
Help



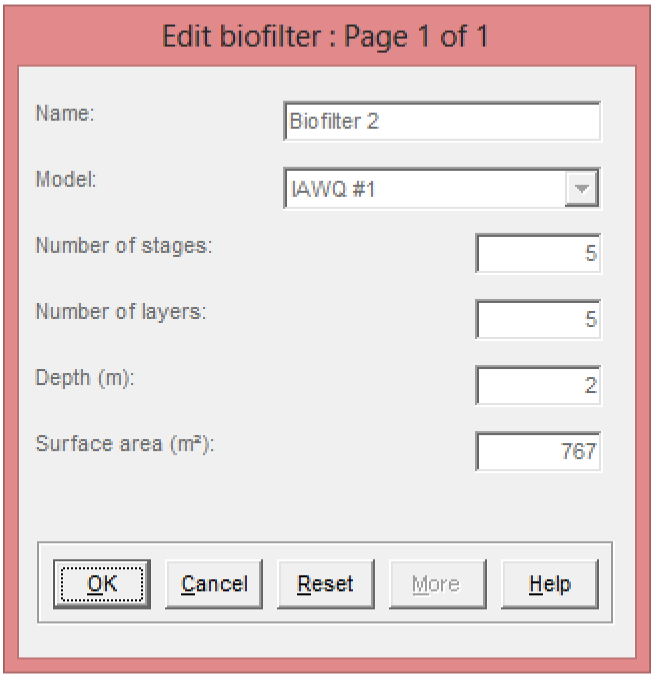
The results after the simulation.

4. Primary settler

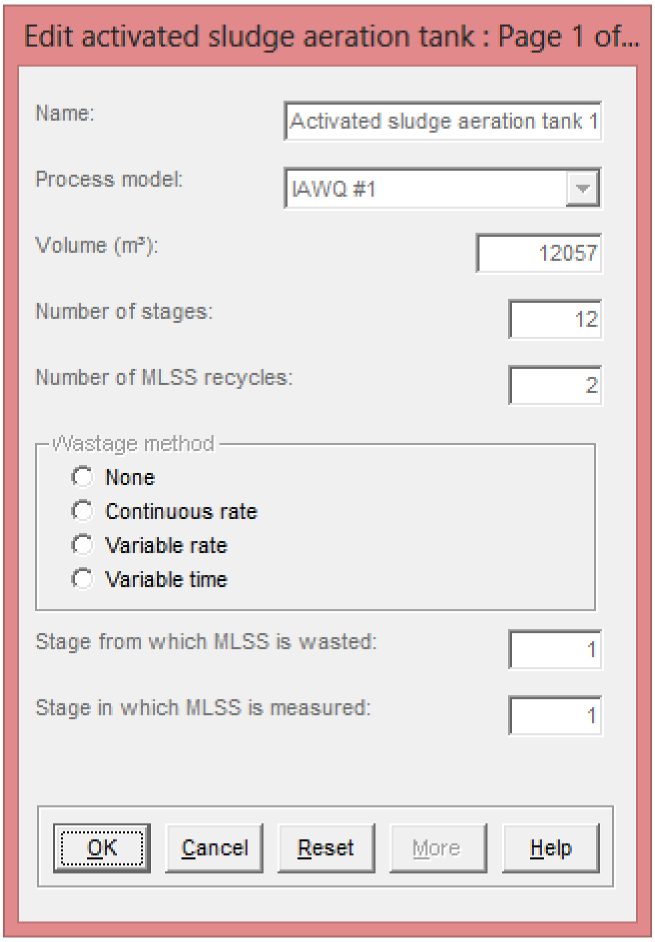


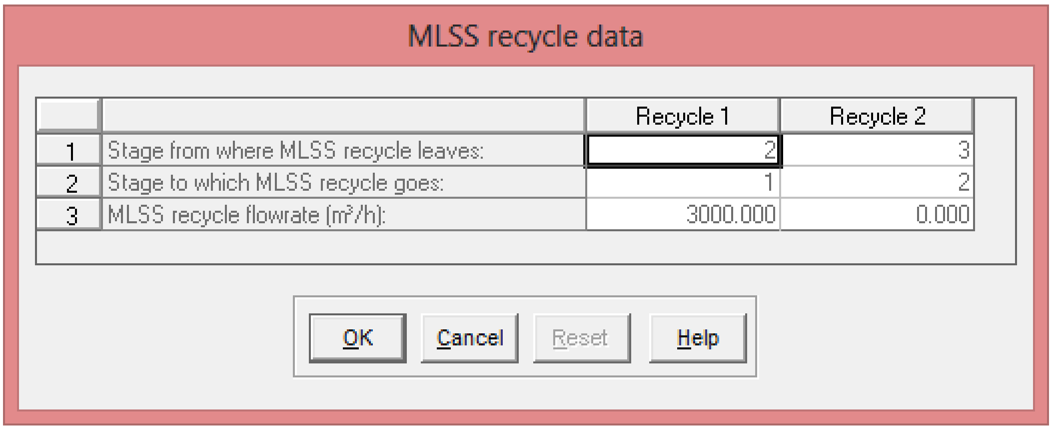


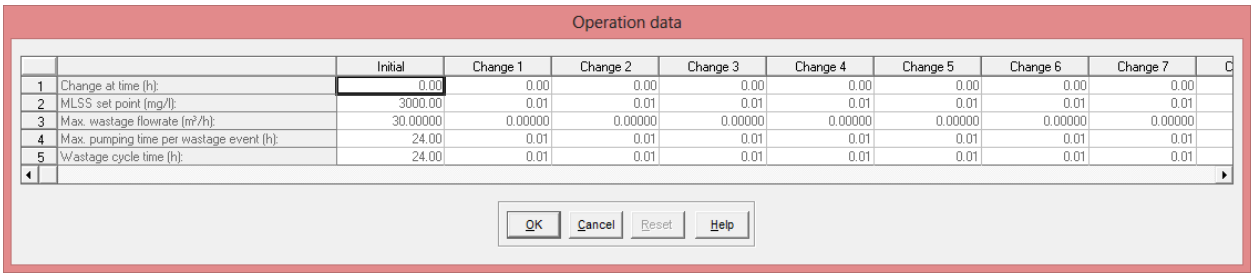
5. Bio filters



6. Activated Sludge process



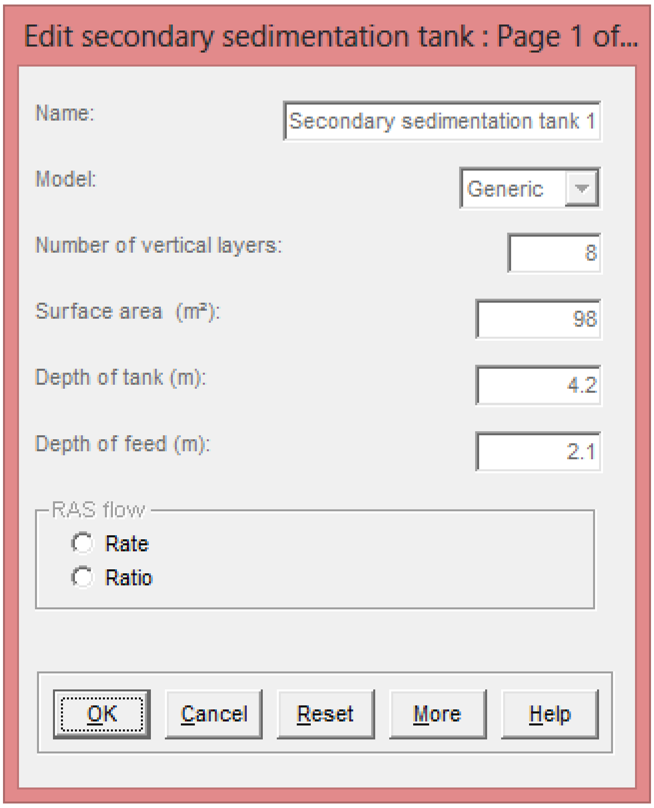


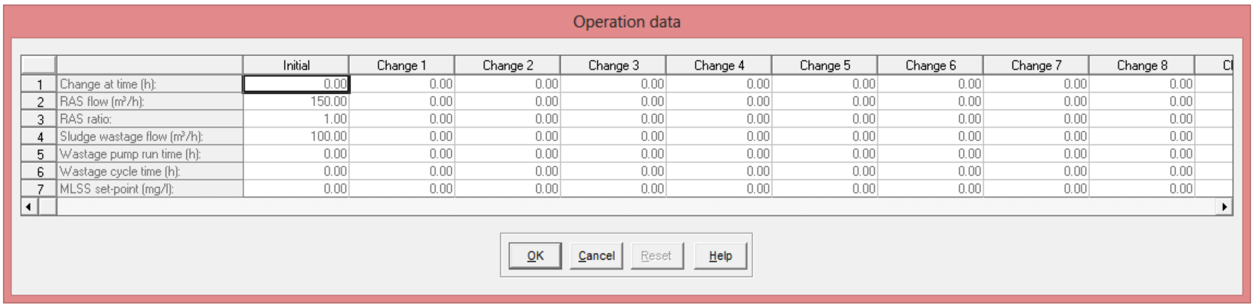


Operation data

		Initial	Change 1	Change 2	Change 3	Change 4	Change 5	Change 6	Change 7	C
1	Change at time (h):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
2	MLSS set point (mg/l):	3000.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	
3	Max. wastage flowrate (m³/h):	30.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	
4	Max. pumping time per wastage event (h):	24.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	
5	Wastage cycle time (h):	24.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	

7. Secondary Settlers

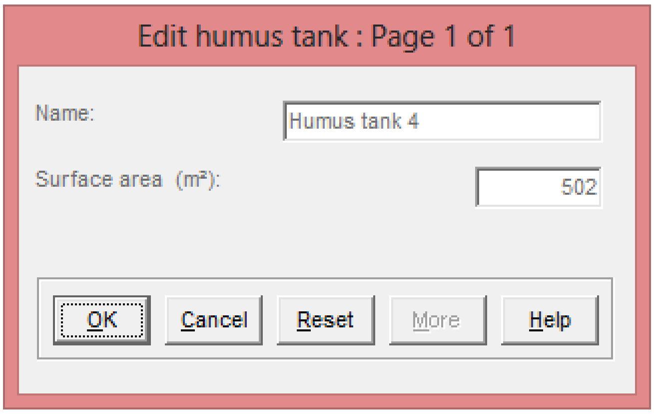




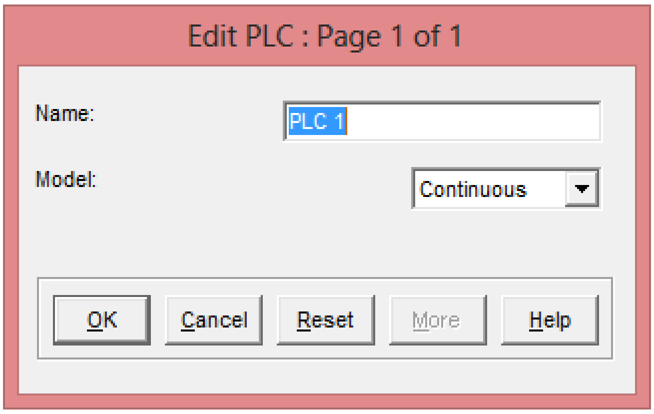
Operation data

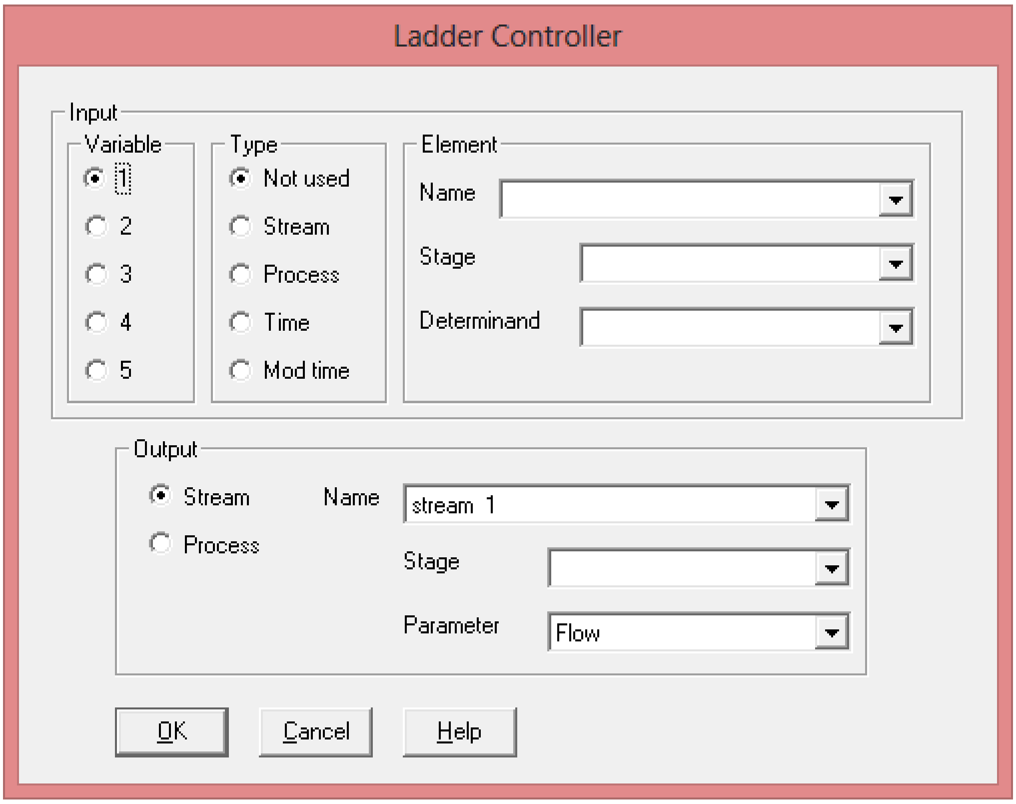
		Initial	Change 1	Change 2	Change 3	Change 4	Change 5	Change 6	Change 7	Change 8	Cl
1	Change at time (h):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
2	RAS flow (m³/h):	150.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
3	RAS ratio:	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
4	Sludge wastage flow (m³/h):	100.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
5	Wastage pump run time (h):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
6	Wastage cycle time (h):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
7	MLSS set-point (mg/l):	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

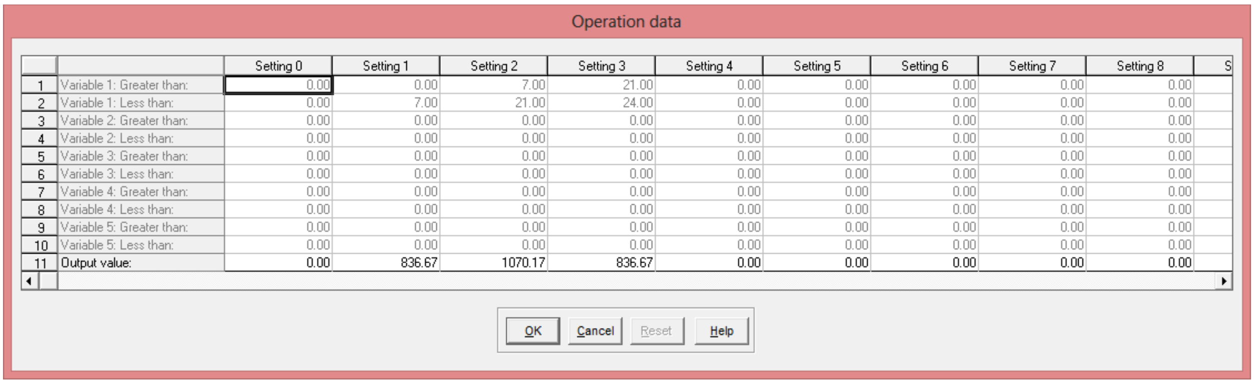
8. Humus Tank



9. Controller






Operation data

		Setting 0	Setting 1	Setting 2	Setting 3	Setting 4	Setting 5	Setting 6	Setting 7	Setting 8	S
1	Variable 1: Greater than:	0.00	0.00	7.00	21.00	0.00	0.00	0.00	0.00	0.00	
2	Variable 1: Less than:	0.00	7.00	21.00	24.00	0.00	0.00	0.00	0.00	0.00	
3	Variable 2: Greater than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
4	Variable 2: Less than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
5	Variable 3: Greater than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
6	Variable 3: Less than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
7	Variable 4: Greater than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
8	Variable 4: Less than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
9	Variable 5: Greater than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
10	Variable 5: Less than:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
11	Output value:	0.00	836.67	1070.17	836.67	0.00	0.00	0.00	0.00	0.00	

APPENDIX 10: STOAT modelling pollutant concentration results: Different equalization basin release flow rates simulations

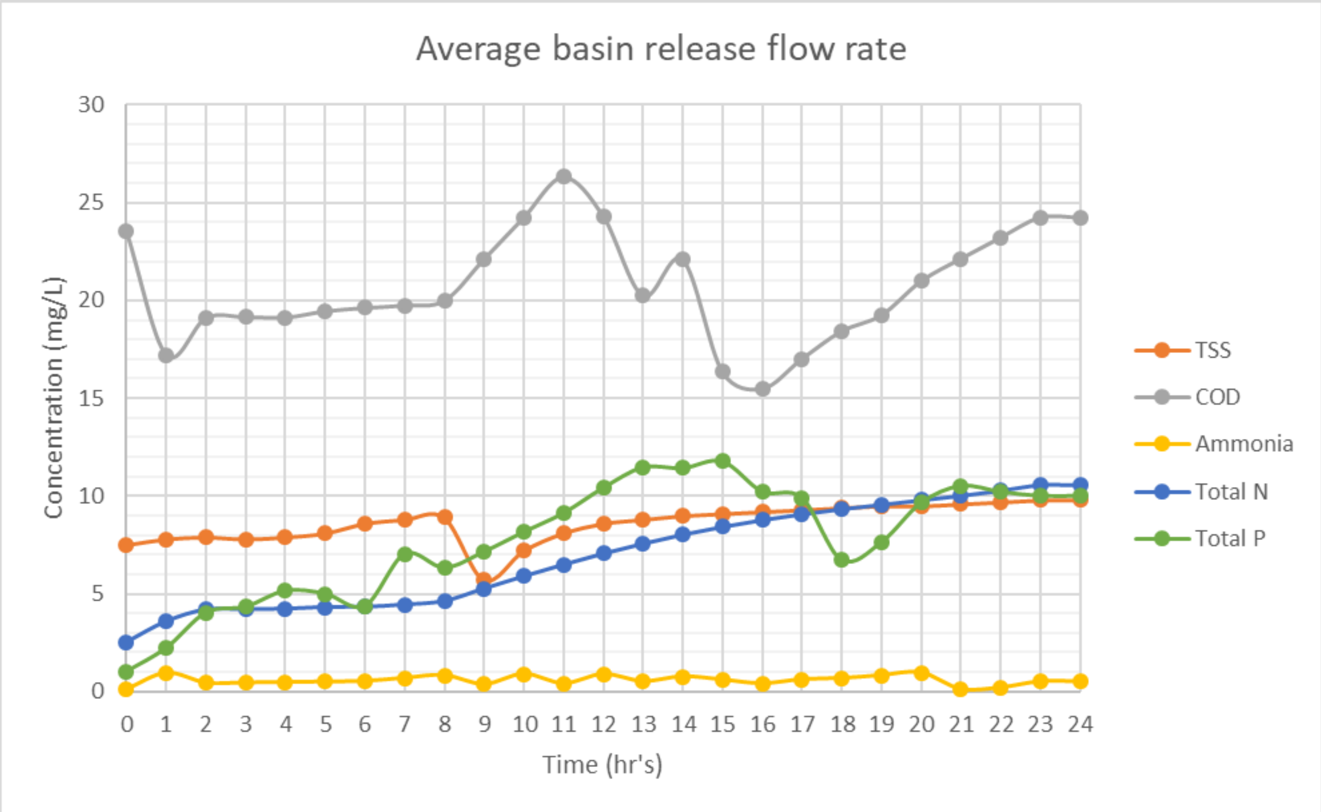


Figure 10. 1: STOAT simulation analysis at average release

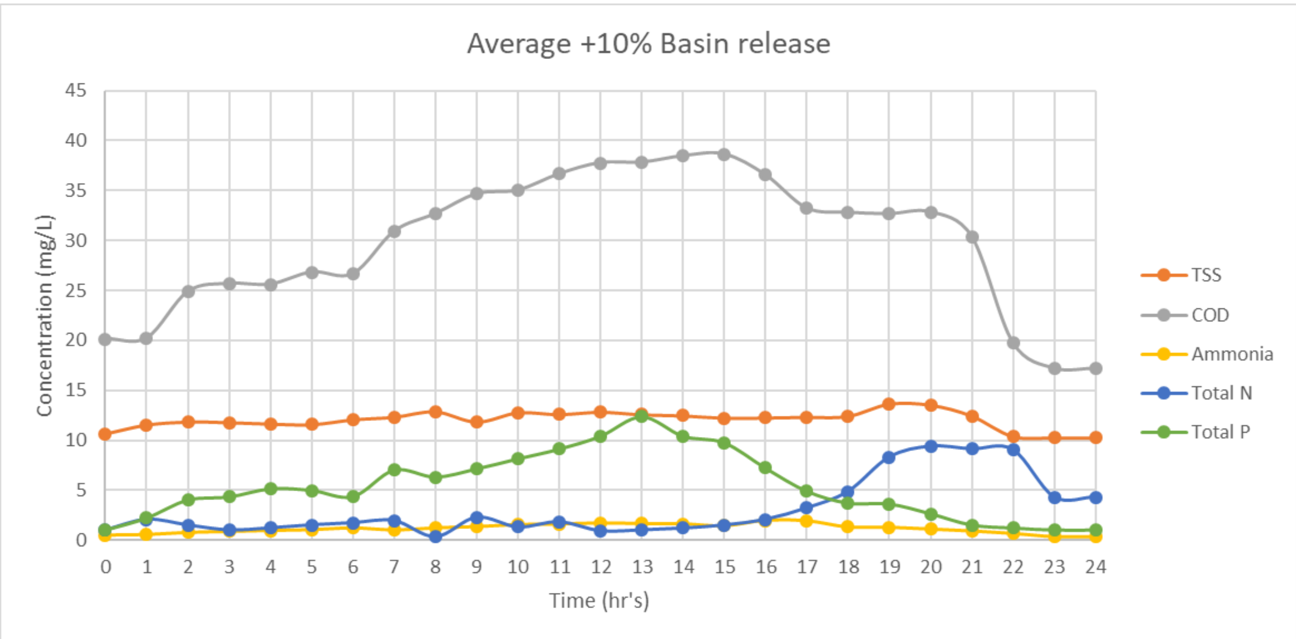


Figure 10. 2: STOAT simulation analysis at average + 10% release

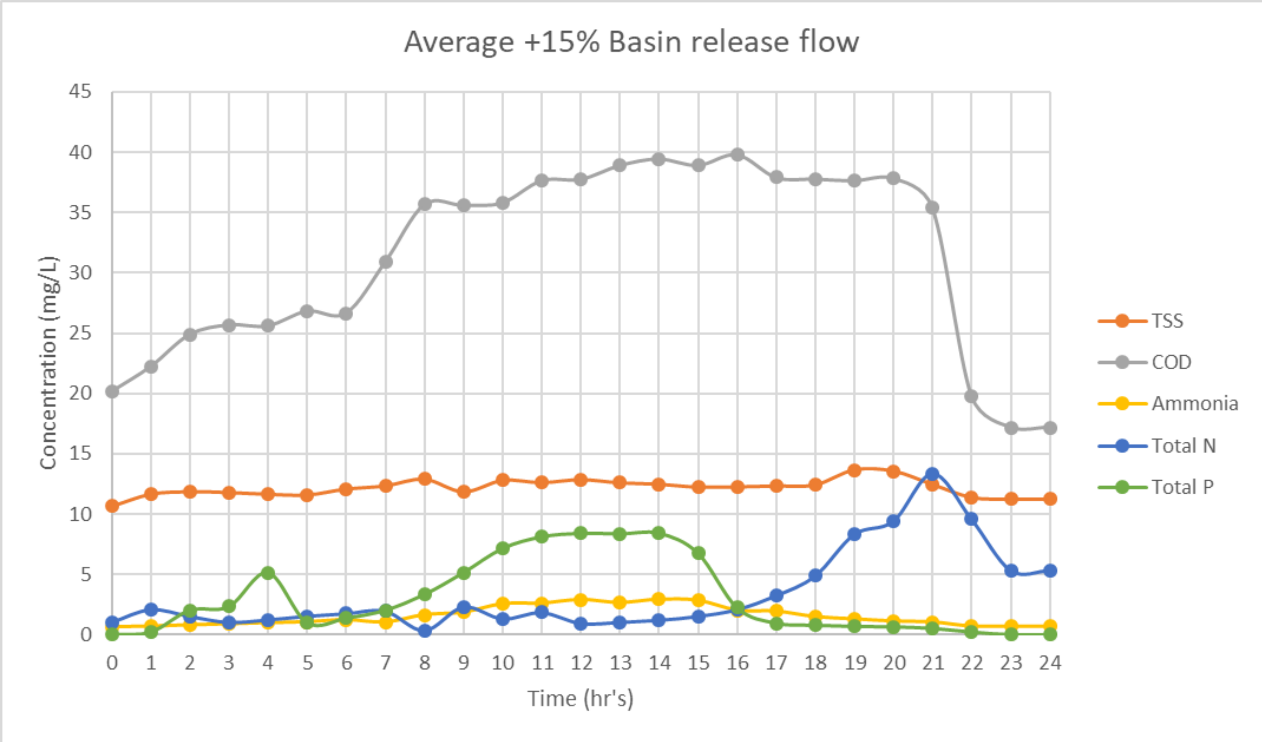


Figure 10. 3: STOAT simulation analysis at average + 15% release

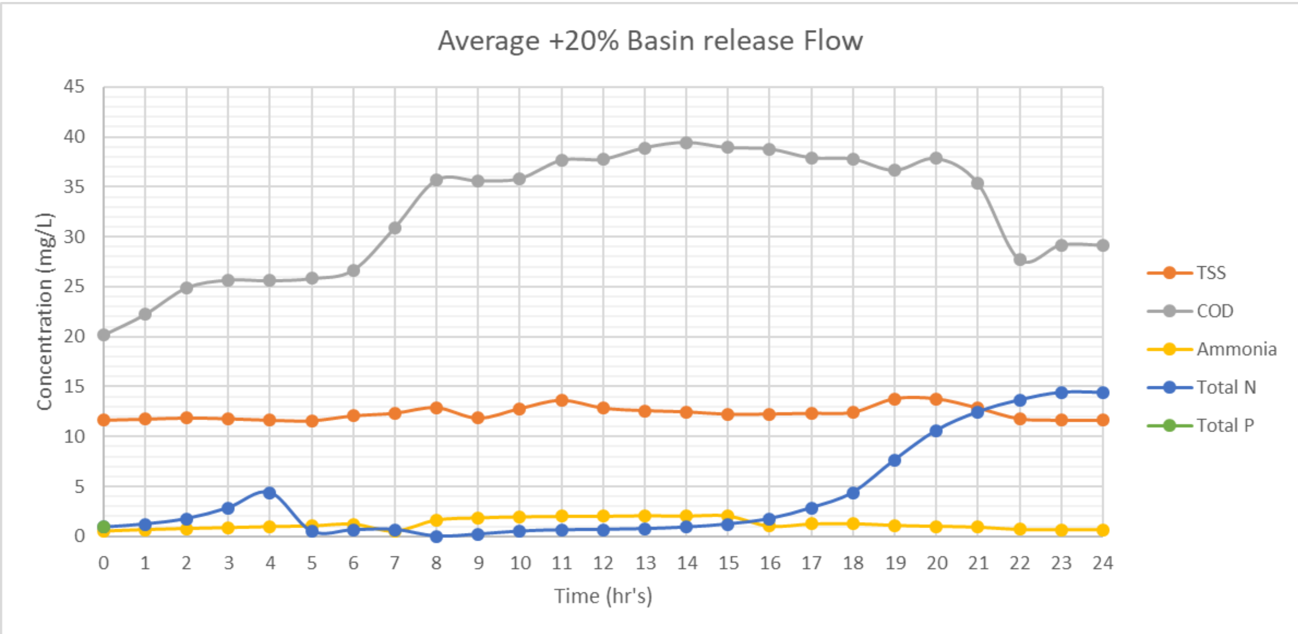


Figure 10. 4: STOAT simulation analysis at average + 20% release

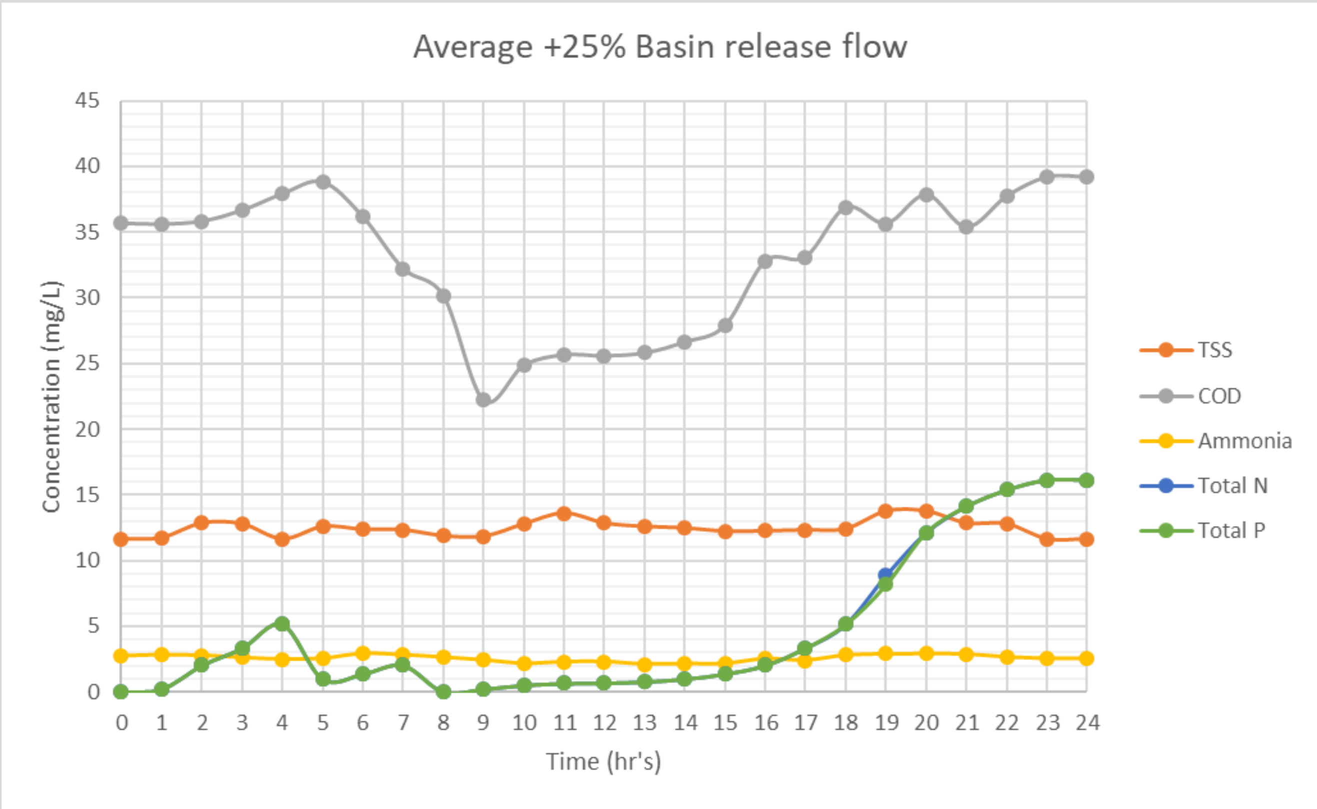


Figure 10. 5: STOAT simulation analysis at average + 25% release