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Integral Water Treatment Plant (WTP) Design Guide for Resource Circularity in the Caribbean: Part 1

Kompleksowy przewodnik projektowania stacji uzdatniania wody (WTP) na potrzeby obiegu zamkniętego zasobów na Karaibach: Część 1

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Abstract
Small Island Developing States (SIDS) have unique challenges in water resource management due to their small geographical area, reliance on fragile ecosystems, and vulnerability to climate change. This paper presents an Integral Urban Water Treatment Plant (WTP) Design Guide to enhance water circularity as a long-term solution for SIDS. The guide emphasises the integration of water treatment, water circularity, and nature-based solutions that can be easily adapted to Caribbean SIDS's unique geographical, social, and economic conditions. By fostering circular water techniques with conventional water treatment processes, water harvesting and sustainability. This design guide focuses on water resources provisions from rivers, groundwater reservoirs or dams in the West Indies and critical infrastructure requirements which provides long-term viability of potable water supply. Benefits of the suggested WTP design includes step-by-step processes for improved water security, increased resistance to climatic changes, and less dependency on external water supplies. The paper presents a replicable paradigm that can be easily applied across Caribbean SIDS to promote sustainable development and self-sufficiency in urban, sub-urban integrated water resources management (IWRM).

Słowa kluczowe: *małe rozwijające się państwa wyspiarskie (SIDS), stacja uzdatniania wody, projekt, zrównoważony rozwój, odporność na zmiany klimatu, zintegrowane zarządzanie zasobami wodnymi (IWRM) na Karaibach.*

Streszczenie
Małe rozwijające się państwa wyspiarskie (SIDS) stoją przed wyjątkowymi wyzwaniami w zakresie gospodarki wodnej ze względu na niewielki obszar geograficzny, zależność od wrażliwych ekosystemów i podatność na zmiany klimatu. Niniejszy dokument przedstawia kompleksowy przewodnik projektowania miejskich stacji uzdatniania wody (WTP) w celu poprawy obiegu wody jako długoterminowego rozwiązania dla państw wyspiarskich. W przewodniku położono nacisk na integrację uzdatniania wody, obiegu wody i rozwiązań opartych na naturze, które można łatwo dostosować do wyjątkowych warunków geograficznych, społecznych i ekonomicznych karaibskich państw. Poprzez promowanie technik obiegu wody w połączeniu z konwencjonalnymi procesami uzdatniania wody, zbieraniem wody i zrównoważonym rozwojem. Niniejszy przewodnik koncentruje się na zasobach wodnych pochodzących z rzek, zbiorników wód gruntowych lub zapór wodnych na Karaibach oraz na krytycznych wymaganiach infrastrukturalnych, które zapewniają długoterminową rentowność dostaw wody pitnej. Korzyści wynikające z proponowanego projektu WTP obejmują stopniowe procesy poprawy bezpieczeństwa wodnego, zwiększenia odporności na zmiany klimatyczne i zmniejszenia zależności od zewnętrznych dostaw wody. Dokument przedstawia powtarzalny paradygmat, który można łatwo zastosować w małych rozwijających się państwach wyspiarskich regionu Karaibów w celu promowania zrównoważonego rozwoju i samowystarczalności w zakresie zintegrowanego zarządzania zasobami wodnymi (IWRM) w miastach i na przedmieściach.

INTRODUCTION

Available water resources, wastewater management, water reuse, and circularity have become crucial for sustainable development and human health. According to a UN-Habitat report from 2021, 50% of water supply worldwide is being treated to acceptable international standards, indicating advancements in water management and water supply infrastructure. Even with these advancements, difficulties still exist. Approximately half of the world's untreated wastewater still finds its way into rivers, lakes, and oceans, posing serious hazards to the environment and public health, according to research by the United Nations Environment Programme (UNEP). This alarming statistic highlights the urgent need for effective wastewater treatment and reuse solutions. Untreated wastewater poses significant risks to public health and the environment. It harbours harmful pathogens and contaminants, leading to waterborne diseases and ecological degradation. Addressing this global challenge is imperative, as it could significantly reduce global mortality rates and improve environmental quality. In the developing world, approximately 8-10 % of municipal and industrial wastewater undergoes treatment of any kind, warning of the increasing damage to public health and the environment across

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a) b)

Fig. 1: (a) The Navet Reservoir, Trinidad, West Indies (W.I), with a volumetric capacity of 1.9 million m3 and area covering 324 hectares of land (b) Secondary Treatment stages of the Caroni Water Treatment Plant, Trinidad, W.I which has a production capacity of 75 million gallons per day. [48]

Rys. 1: (a) Zbiornik Navet na Trynidadzie na Karaibach (W.I) o pojemności 1,9 mln m3 i powierzchni 324 hektarów (b) Etapy wtórnego oczyszczania w zakładzie uzdatniania wody Caroni na Trynidadzie na Karaibach (W.I), którego zdolność produkcyjna wynosi 75 mln galonów dziennie. [48]

Latin America and the Caribbean. Naturally formed toxins are riddled within untreated wastewater across the globe, spreading diseases, and with it, needless deaths. It is a common concern that has rallied numerous nations across the world into providing efficient methods that supply their public with it. The biggest concern with little access to sustainable water resources is what resides, hidden and within it. To date, more than 840,000 people die from water borne illnesses, of which around 60% fatalities are down to avoidable cases. It is predicted that, solving this worldwide issue would result in a 10% drop in unnatural mortalities that highlighted by the Water and Sewerage Authority (WASA) [48]. The importance of wastewater treatment, remediation and reuse for a water circularity system, and the detrimental effect if it is not delivered, is obvious to all. Thus, water quantity, variance in water quality continues to influence political decisions with it comes to water infrastructure and related potable water supply treatment facilities (**Fig 1**).

In this paper, we have created a typical water treatment plant (WTP) also referred to as water treatment works (WTW) specific to the West Indies (W.I). It has been designed to remove/break down common particles typically found in water streams and natural Hydrosystems which are required for public health and safety for potable consumption. To meet stringent environmental regulations and public health standards, the WTP must be capable of treating and distributing a significant volume of municipal or domestic water supply. This involves removing specific contaminants to a predefined level, ensuring the treated water is suitable for its intended use. The final design of a WTP or WTW must adhere to several environmental regulations as well as reach a significant number of expectations. For this case study, the WTP / WTW must be capable of treating and supplying into a distribution network, approximately 140,000 m^3/d of potable water supply.

The first requirement is this case-study example of a water treatment plant (WTP) / water treatment works (WTW) design must be

Table 1 Drinking Water Quality Standards[50,51]

Tabela 1 Normy jakości wody pitnej [50,51]

Water Quality Parameters	World Health Organisation (WHO)	United States of America Environment Protections Agency (USEPA)	Water quality Parameters	Water quality values	Water quality Parameters	Water quality values
pH	6.5-8.5	6.0-8.5	Enterococci	0	Mercury	1.0
Total Dissolved Solids (TDS (mg/l)	< 500	< 500	*Escherichia coli (E. coli)*	0	Nickel (ii)	20
Suspended Solids (mg/l)	< 5.5	< 5.0	Coliform bacteria	0	Nitrate (iii)	50
Electrical conductivity (μS/cm)	-	< 300	Acrylamide	0.1	Aldrin	0.030
Dissolved Oxygen (DO (mg/l)	At least 4-6	At least 4-6	Antimony	5.0	Dieldrin	0.03
Biochemical Oxygen Demand (BOD) (mg/l)	< 6	< 5	Arsenic	10	Heptachlor	0.03
Chemical Oxygen Demand (COD) (mg/l)	<10	<4	Benzene	1	Heptachlor epoxide	0.03
Cl- (mg/l)	500	240	Benzo(a)pyrene	0.01	other pesticides	0.1
Turbidity (NTU)	< 1	< 1	Boron	1	Polycyclic aromatic hydrocarbons (vii)	0.1
Na^+ (mg/l)	6.5	10	Bromate	10	Selenium	10
K^+(mg/l)	1.2	1.5	Cadmium	5	Tetrachloroethene and Trichloroethene (viii)	10
Ca^{2+} (mg/l)	100	100	Chromium	5	Trihalomethanes: Total (ix)	100
Mg^{2+} (mg/l)	150	30	Copper(ii)	2	Vinyl chloride	0.5
NH_4^+ (mg/l)	0.5	0.5	Cyanide	50	Aluminium	200
NO_3^- (mg/l)	10	10	1, 2 dichloroethane	3.0	Colour	20
SO_4^{2-} (mg/l)	100	100	Epichlorohydrin	0.1	Iron	200
Total Coliforms	Absent in 100 mL	Absent in 100 mL	Fluoride	1.5	Sodium	200
E. coli	Absent in 100 mL	Absent in 100 mL	Lead (ii)	10	Manganese	50

Notes: Specific Caribbean countries may have more stringent standards. WHO: World Health Organisation; US EPA: United States Environmental Protection Agency; UK: United Kingdom Drinking Water Inspectorate; mBOD: Biochemical Oxygen Demand; COD: Chemical Oxygen Demand; NTU: Nephelometric Turbidity Units capable of bringing in, treating, and distributing 140,000 m^3/d of fresh surface water. It must also be able to completely remove or at the very least reduce specific, pre-given, contaminant levels from the influent water resources to internationally acceptable water quality standards. This paper outlines the design of a typical water treatment plant (WTP) or water treatment works (WTW) for non-potable water reuse. The WWTP is designed to remove or reduce common contaminants found in wastewater, ensuring the safety and quality of the treated water for various applications (drinking water resources, agricultural applications, commercial as well as industrial consumption).

Typical potable water contaminants of concern

Monitoring and adhering to drinking water standards are crucial for public health and environmental protection. These standards ensure that the water we consume is free from harmful microorganisms, chemicals, and other contaminants [27]. By consistently monitoring water quality, we can identify potential contamination sources, implement necessary treatment measures, and prevent the spread of waterborne diseases that highlighted by Global Vision International [15]. Adherence to these standards also helps protect aquatic ecosystems, as untreated wastewater can harm aquatic life and disrupt the delicate balance of the environment. Furthermore, learning from these potable water supply standards allows us to continuously improve water treatment technologies, develop more effective pollution control measures, and ensure the long-term sustainability of our water resources.

Coliforms: Coliforms are a bacterium, commonly found in the water ways and streams of wastewater being treated or pumped from. Coliforms cover two groups of bacteria, those that live in vegetation and their soils and those that live in the gut of warm--blooded animals, such as humans that highlighted by Kavanaugh [20]. Kavanaugh [20] found that the reason they effectively inhabit the human intestines is down to the environment which is at a perfect 37°C and a high acidity. Such a condition encourages rapid breeding of the bacteria. Generally, coliforms are not harmful when consumed by humans; however, there are the few individual types which are just that (*E. coli*) [23,45]. The United States Environment Protection Agency (US EPA) set the Maximum Contaminant Level Goal (MCLG) for total coliforms at zero (0) because there have been cases of waterborne disease outbreaks which occurred at very low levels of coliforms when periodic monitoring occurred that highlighted by Tota-Maharaj et al. [36]. Though more importantly, the mere presence of coliforms is concrete evidence of far more toxic pathogens in the water. The testing of pathogens (e.g. Viruses) is a lengthy and therefore expensive task due to the numerous forms one could test for. Therefore, one single test for the presence of coliforms is a good indicator of whether or not pathogens exist in the water sample [7,42,43]. There are four methods of testing coliform methods in a single water sample, membrane filter, multiple tube fermentation, Most Probable Number (MPN) and the Minimal Medium- 4-methylumbelliferyl-β-D-glucuronide MMO-MUG method. For this water treatment plant to be designed effectively, the MPN (Most Probable Number) technique was adopted. It works by placing the water samples in small, inclined test tubes. The tubes are placed in an environment at around 35°C which allows for rapid growth of coliforms, which produces Carbon Dioxide (CO_2). The CO_2 is trapped as it rises and the quantity is examined, the value of CO_2 collected determines the quantity of Coliforms in the water which has done by the US EPA [41]. According to Dublin [3] due to the fact that coliforms highlight possible pathogens, it is for this reason a value of 0CFU/100mL must be found within any given sample of water before the treated water is distributed. CFU is the Colony Forming Unit of bacteria, measures the number of 'active live organisms.

Escherichia coli (E. coli): *E. coli*, like coliforms, are not inherently harmful to humans. In fact, there are approximately one million *E. coli* cells per gram of faces. However, certain strains, such as Enterotoxigenic *E. coli,* are primary causes of diarrhoea, particularly problematic in developing countries. Even rarer, but more lethal, strains like Enterohemorrhagic E. coli can lead to bloody stool and severe kidney damage. The potential severity of *E.coli* infections is why a strict 0 CFU/100mL standard is enforced in final water samples[5,14]. Testing for E. coli often begins with a coliform test. If no coliforms are detected, *E. coli* testing is unnecessary, as one cannot exist without the other. When coliforms are present, a further test, known as the membrane filter method, is used to confirm *E. coli*. This involves filtering a water sample through a membrane to capture bacteria. The captured bacteria are then transferred to a Thermotolerant membrane and incubated for two hours at 35°C to revive damaged cells. A second incubation period of 22 hours at 44.5°C follows. Finally, the cells are placed on a filter pad soaked in urea, and the number of bacterial colonies is counted under a fluorescent microscope after 15 minutes [47].

Turbidity: Turbidity can be simply defined as the cloudiness in the water, for the greater the cloudiness, the greater the number of suspended particles and therefore the higher the turbidity. High levels of turbidity are clearly visible to the naked eye (>5NTU) [49]. There are two main concerns with high level of turbidity in the water, which 110NTU is. The floating compounds themselves generally have no negative implications on human health; but they can act as a host to far more deadly pathogens. In a sense, remove the host, the parasite will go with it. The second concern is that for disinfection by chlorine to be effective, the turbidity level must be less than 1NTU as a maximum. However, 95% of all samples taken of presumed clean water must have a turbidity value less than 0.4NTU [28]. Testing turbidity is done by the use of light. A sample is placed in a clear glass beaker where a white light is passed through it. The amount of light rays that are dedicated on the other side is measured. The less light measured the higher turbidity, as more rays are being absorbed or deflected from their original path. The values collected are defined as a NTU or Nephelometric Turbidity Units [40].

Biochemical Oxygen Demand (BOD): According to the WHO [52] the BOD level is described as the quantity of oxygen which bacteria use to break down organic compounds found in the water to provide it with the nutrition to thrive, it does so by the process of oxidation. Therefore, similar to coliforms, the lethality of BOD is not down to it directly but the fact that it is an obvious sigh of hidden dangers. There are two main tests that can be used to measure BOD levels. The first is the dilution method, the second is known as a manometric method. It is the latter that will be used for this water plant because of its simplicity of not needing chemical analysis, therefore also saving time. However, both do not supply accurate BOD values, more of an approximation. It works by placing a sample of water and a sealed container, the air pressure in the flask is measured. If bacterium does reside in the water, the air pressure will drop as the microorganism use what is in the air to break-down organics. It must be made clear that unlike with coliforms, the BOD level should not be 0. This is because there need to be some oxygen level in the water to support life, such as the common bacteria we find in the guts of humans to support our digestive system [44].

Chemical Oxygen Demand (COD): As stated before, the BOD (Biochemical Oxygen Demand) measures the amount of oxygen consumed by microorganisms while decomposing organic matter within water samples and the COD (Chemical Oxygen Demand), on the other hand, quantifies the total amount of oxygen required to oxidise all organic and inorganic substances present in water. A higher COD value often indicates a higher level of organic pollution, suggesting the presence of more harmful compounds. A common method for determining COD is redox titration. In this process, a water sample is oxidised using a strong oxidising agent like potassium dichromate. The amount of oxygen consumed during this process is directly proportional to the organic matter content. This method offers a significant advantage over traditional BOD tests as it provides a rapid estimation of organic pollution levels, reducing the need for lengthy incubation periods [22].

Potential Hydrogen (pH): The pH is a scale which represents the acidity (<7) or alkaline (>7) level of the water across the treatment plant. High or low levels can make the perfect environment for pathogens, such as coliforms which is known to prefer high acidity. However, the level of 8 recorded is within the US EPA's required range. The treatment plant will look to reduce this number though, to the neutral 7. This is more for plant operation advantages, such as reducing corrosion in the pipes and therefore limiting maintenance costs. The pH level can be record very easily by the use of a pH meter, which works by measuring the Hydrogen ions in any given solution simply by placing a rod in the water sample [13,37].

Calcium: The primary cause of water hardness is calcium, which typically ranges from 5 to 500 mg/l as $CaCO_3$. Although almost all rocks contain calcium, gypsum and limestone have the highest concentrations. In the majority of natural waters, calcium ions are the main cations. Reduction of calcium is necessary for cooling tower composition treatment. In boiler feed applications, textile processes, and metal finishing, total removal is necessary.

Magnesium: Using a softener or purification exchanger in hydrogen form can lower magnesium to less than 1 mg/l. See "Hardness" as well.

Iron: If the pH of the water is higher than 6.8 and the ferrous iron (also known as clear water iron) is less than 0.5 ppm per grain of hardness, it can be eliminated with a softener. Before being eliminated by mechanical filtration, ferrous iron that exceeds 5.0 ppm must be changed into ferric iron by coming into contact with an oxidising agent, such as chlorine. Mechanical filtration is an easy way to get rid of ferric iron, also known as red water iron. An organic scavenger anion resin or chlorine oxidation followed by mechanical filtration are two methods for removing heme iron. If iron bacteria are present, oxidising agents like chlorine will also destroy them.

Sodium: The hydrogen form cation exchanger part of a deioniser can be used to remove sodium. Sodium will be reduced by 94–98% by reverse osmosis. Sodium will also be eliminated by distillation.

Aluminium: A cation exchanger can extract aluminium from water, but regeneration is required to extract the aluminium from the resin using sulphuric or hydrochloric acid. Although this can be used in an industrial setting, it is not advised for home use unless it is a cation exchange tank. Reverse osmosis will cut drinking water's aluminium content by more than 98%. Water's aluminium content will be reduced by more than 99% through distillation. The reduction of aluminium is another area in which electro-dialysis excels.

Methodology / design procedure

Design process for typical water treatment plant (WTP)

The following sections of this paper focuses on the identification and creation of an appropriate treatment facility, which can be incorporated as a typical water treatment plant suitable for the Caribbean, which works to cleanse surface water from a natural source (lake/reservoir/dam/river). The design processes involved in this case study, has been carried out to remove/break down common particles found in water which are detrimental to humans after consumption. The first requirement is the plant must be capable of bringing in, treating and distributing 140,000m³/d of surface water . It must also be able to completely remove or at the very least reduce specific, pre-given, contaminant levels from the inflowing water. These levels are given in tabular form in **Table 2**. All the levels signified in Table 2 are far above the level expected by the major global water agencies, from the EU to the World Health Organisation (WHO). However, the organisations whose standards will be met, is the USA's Environmental Protection Agency (EPA). The reason for meeting these specific requirements is simply down to the design being based around a typical North American plant. These standards are shown in Table 1.

Table 2 Average Pollutant levels measured at entry point to a Small Island Developing State (SIDS)-Water Treatment Facility.

Tabela 2. Średnie poziomy zanieczyszczeń zmierzone w punkcie wejścia do stacji uzdatniania wody w małym rozwijającym się państwie wyspiarskim (SIDS)

Contaminant	Case-Study Quantity & Units (_)	Maximum allowable contaminant levels & Units (_) by the US EPA
Total Coliforms	8000 (CFU/100mL)	≈0* (CFU/100mL)
Escherichia Coli	2400 (CFU/100mL)	≈0* (CFU/100mL)
Turbidity	25 (NTU)	≈ 0.4 (NTU)
Biochemical Oxygen Demand (BOD)	1840 (mg/L)	< 5 (mg/L)
Chemical Oxygen Demand (COD)	3512 (mg/L)	< 5 (mg/L)
Potential Hydrogen (pH)	7.0-8.0 (pH units)	6.5-8.5 (pH units)

IV. Water treatment and remediation stages

In conventional water treatment facilities, there are typically three (3) main stages, primary, secondary, and tertiary phases of water treatment processes. These steps under these three banners vary from plant to plant. Generally, the plants begin with the removal of the largest foreign substances in water and slowly working down to the destruction of the smallest. The quantity of foreign substances removed per stage is judged by a percentage. The maximum potential of the three stages is assumed in terms of how much of the unwanted impurities are removed.

Primary Stage

The primary stage in the following design is made up of three parts, coagulation, flocculation and a horizontal sedimentation tank. In some cases, the primary clarifiers can be up-flow in configuration as well. Table 3 shows how much of the sampled containments will be removed in this stage.

Table 3 Enhancing Water Quality after Primary Treatment Stage [6, 18,46]

Tabela 3. Poprawa jakości wody po etapie wstępnego oczyszczania [6, 18,46]

Contaminant	Case-Study Quantity	Expected Removal Efficacy	Expected Discharge/ Effluent
Total Coliforms	8000 CFU/100mL	94-95 %	480-400 CFU/100mL
Escherichia Coli	2400 CFU/100mL	75-82%	600-432 CFU/100mL
Turbidity	25 NTU	90 %	2.5 NTU
Biochemical Oxygen Demand (BOD)	1840 mg/L	25-30 %	1380-1288 mg/L
Chemical Oxygen Demand (COD)	3512 mg/L	10-15 %	3160-2985 mg/L
Potential Hydrogen (pH)	7.0-8.0	-	7.0-8.0

Secondary Stage

The second stage is the use of a rapid filter which is the conventional follow up to sedimentation. **Tables 3 and 4** represents its effectiveness.

Table 4 Designing Effective Secondary Treatment Systems for Post-Primary Water Treatment Processes. The Role of Engineering Design in Improving Post--Secondary Treatment Stages Water Quality [12,21,53]

Tabela 4 Projektowanie skutecznych systemów oczyszczania wtórnego dla procesów oczyszczania wody po etapie oczyszczania wstępnego. Rola projektowania inżynieryjnego w poprawie jakości wody na etapach oczyszczania wtórnego [12,21,53]

Contaminant	Case-Study Quantity	Expected Removal Efficacy	Expected Discharge/ Effluent
Total Coliforms	400-480 CFU/100mL	90-95 %	40-48 or 20-24-CFU/100mL
Escherichia Coli	600-432 CFU/100mL	30-40%	420-302 or 360-260 CFU/100mL
Turbidity	2.5 NTU	90 %	0.25 NTU
Biochemical Oxygen Demand (BOD)	1380-1288 mg/L	85-90 %	207-193 or 138-129 mg/L
Chemical Oxygen Demand (COD)	3160-2985 mg/L	70-80 %	948-896 or 632-597 mg/L
Potential Hydrogen (pH)	7.0-8.0	-	7.0-8.0

Tertiary Stage

The final stage in most treatment works, commonly the use of chlorine disinfection is implemented, which has already been identified to be the most historical and provide maximum efficacy in the Caribbean with great effectiveness, when it comes to environmental methodologies of ridding water resource from the smallest concentrations of pollutants and organic contaminants. Effective method to rid the water of the smallest to pollutants. **Table 5** represents its effectiveness.

Table 5. esigning Effective Secondary Treatment Systems for Post-Secondary Water Treatment Processes. The Role of Engineering Design in Improving Post--Tertiary Treatment Stages Water quality [12,21,53]

Tabela 5. Projektowanie skutecznych systemów oczyszczania końcowego dla procesów oczyszczania wody po etapie oczyszczania wtórnego. Rola projektowania inżynieryjnego w poprawie jakości wody na etapach oczyszczania końcowego [12,21,53]

Contaminant	Case-Study Quantity	Expected Removal Efficacy	Expected Discharge/Effluent
Total Coliforms	40-48 or	99 %	0.4-0.48 or 0.2-0.24 CFU/100mL
Escherichia Coli	420-302 or 360-260 CFU/100mL	99%	4.2-3.02 or 3.6-2.6 CFU/100mL
Turbidity	0.25 NTU	90 %	0.025 NTU
Biochemical Oxygen Demand (BOD)	207-193 or 138-129 mg/L	25-30 %	155-145 or 97-90 mg/L
Chemical Oxygen Demand (COD)	948-896 or 632-597 mg/L	10-15 %	853-806 or 569-507 mg/L
Potential Hydrogen (pH)	7.0-8.0	-	7.0-8.0

After each stage, primary through to tertiary, the water must be periodically tested for microbiological and physiochemical quality to confirm that each phase is fulfilling its requirements in the removal of pollutants from the water. This is to accommodate any process changes at each of the following sections of the treatment works / water treatment plant, to re-calibrate dosage of it such as increase the dose of chlorine injected.

Results and Discussion

Water treatment processes & water plant design

Inflow and Inlet Pipes

A single inlet pipe brings in water from a local water source. From this case study, the inflow pipes must be able to accommodate flows withstand a flows of 140,000 m^3/d or 1.62 m^3/s., withstanding an fluctuations in flowrates, as well as provide adequate capacity to accommodate changing demands (dry wet seasons)The optimum pipe diameter is determined by designing the entire system to allow the flow to run at a uniform velocity. The velocity of the water must lie between the ranges of 0.8 m^3/s to 4 m^3/s, thereby providing acceptable speeds from stage-to-stage. Therefore, the plant will be designed to cope with a constant velocity of 1.5 m^3/s. This velocity should be maintained in all pipes connecting each stage of the facility. The reason for adopting this particular velocity is based on economic feasibilities and pricing. Though a slightly slower velocity will require larger pipes, it will not need hugely expensive pumps to maintain this flow. On the other hand, gravity systems can be looked at based on associated cost with pumping. With the flow rate and the velocity known, the area of this initial pipe can be calculated, which is found to be 1.08 m^2. Therefore, the pipe will have a radius 0.586 m. Though this pipe size is large, it can be split when entering the treatment phases as larger volumes of water are harder to treat then smaller ones, hence a reduction in pipe diameter. In designing inflow and inlet pipelines, one can consider two (2) pipelines in parallel. The water flow can be maintained by the pipe being laid upon a gradient; this means less pumps required to induce the required flow. However, a pump should be installed at the accurate position within the hydraulic system when flowrates are not being met, such as a reduction in the water sources head pressure. This can occur in a particular dry spell which can lead to a reduction in the hydraulic head. A reduced hydraulic head means a smaller force driving the water down the pipe at the required velocity, hence the need of a pump to maintain the required optimal flow. The inlet pipe from the source of water must have adequate capacity to accommodate as well the ecohydrological abstraction rates in order to operate and maintain a constant supply. Optimum pipe diameters are typically determined using standard hydraulic formulate in the design process, thereby indicating an acceptable inlet velocity, which ought to be in the range of 0.8 m/s to 4.9 m/s thereby facilitation of plant designs with constant flow velocities at 1.5 m/s that mentioned by Haydar et al. [19]. Inflow and inlet pipes at water treatment plants require sufficient flow pressure to ensure adequate water supply for the treatment processes, which varies based on factors like treatment processes, plant capacity, pipe design, elevation changes, and hydraulic head.

Intake:

The intake structure's primary purpose is to assist in the safe removal of water from the water source and to release that water into the withdrawal conduit, also known as the intake conduit, which allows it to flow up to a WTP. A raw water gravity pipe directs the water into the wet well (intake). The following is a description of the average discharge (Q avg.) that was used in the intake design:

Q avg. = 1.4 m^3/s

when four pipes were used to convey raw water.

Q per one gravity pipe = 0.694 / 4 = 0.35 m^3/s.

Velocity inside the gravity pipe = 1 m/s.

Area (A) = Discharge (Q) / velocity (v)

$$\frac{0.35 m^3/s}{1 m/s} = 0.35\ m^2$$

$$\frac{\pi D^2}{4} = 0.35\ m^2 \tag{1}$$

Diameter (D) of each raw water gravity pipe = 0.47–0.5 m
No. of wells = 4, circular wells were preferred Detention time (t) = 20 min

$$\text{Discharge} = \frac{VOLUME(V)}{TIME(t)} \quad (2)$$

$Q = 1.4\ m^3/s = 84\ m^3/min$
Q for 4 wells = 21 m^3/min
$V = Q \times t = 21 \times 20 = 420\ m^3$ The bottom of the well is located at 1.5 m below of lower water level (LWL) Effective depth of the intake well = 10 m, the Area of the well = 420 / 10 = 40 m^3 , To find the diameter of the circular well section:

$$A = \frac{\pi D^2}{4} =$$
$$D = 7m$$

The plan of the intakes is shown in **Fig 2**:

The design of the suction pipe is as follows: Q = 0.35 m^3 /s V = 1.5 m/s. The cross-sectional area of the suction pipe is A = Q / v = 0.35 / 1.5 = 0.23 m^2

$$D = \sqrt{\frac{4 \times 0.23}{\pi}} = 0.29 \quad (3)$$

(Use D = 0.3 m)

Q back washing =1/3 Q = 0.35 / 3 = 0.056 m3 /s. Velocity of water in backwashing pipe = 3 m/s
Cross-sectional area of the pipe A = Q / v = 0.17 / 3 = 0.056 m^2
On the basis of the data provided the high-water level (HWL) is 6.3 m, the LWL is 3.4 m. Well bottom level = LWL - 1.5 = 3.4−1.5=1.9 m. Total well depth from HWL = HWL - Bottom Well Level = 6.3−1.9=4.46m. Effective well depth given as 10 m, so the total depth is confirmed.
Final Answer: Strainer diameter = 1.24 m, Depth of well = 10 m
The following criteria are considered:
Total discharge (Q) = 1.4 m^3 /s , To design one strainer: Total discharge / 4 = 0.35 m^3 /s, Velocity through strainer (v) = 0.15 m/s

$$A = \frac{Q}{V} = \frac{0.35}{0.15} = 2.33 m^2$$

If the area of strainers is 50% of the total area: Gross Area = 2 × Area of the strainers (holes)= 2 × 2.33 m^2= 4.66 m^2

Diameter of the hole = 12 mm, Area of one hole = $A = \frac{\pi}{4} \times \frac{12^2}{1000} =$

Area of hole = 0.000113 m^2, Number of the hole = $\frac{2.26}{0.00013} = 20000$,

To find the strainer diameter: $\frac{Gross\ Area}{2\pi rh}$ Height of the rectangular shape strainer with closed end h = 1.2 m, Strainer Diameter (D) = $\frac{4.66}{3.14 * 1.2} = 1.23$ m

Primary: Once the water is conveyed or pumped into the plant through the inlet pipe, the first stage of treatment, primary, begins with screening before going on to coagulation, flocculation, and sedimentation.

Screening Process: Screening is the initial stage of the treatment which removes the large debris from the water intake such as wood or plastics. It can also be done at the intake section, prior to pumping or utilising gravity flow systems. Screenings at water utility intakes are usually collected and trucked to landfill sites in the West Indies. Typically, screening is done by a metal mesh with bar spacing of 1-60mm which prevents the debris from traveling any further. The mesh is then cleaned mechanically, with the removed debris bagged before being transported out of the plant to either be incinerated or at a much lower cost dumped at a landfill that mentioned by Guida et al. [17].

Design Considerations: There is a number of mechanical equipment that can be used in the removal of large waste form the intake flow. The screening design is known as a belt screen process (**Fig.3**) which works by the water running through the mesh, 1mm, which the debris is collected and slowly moved upwards where once it reaches the peak of the belt, it is cleaned the debris is typically sent to landfill.

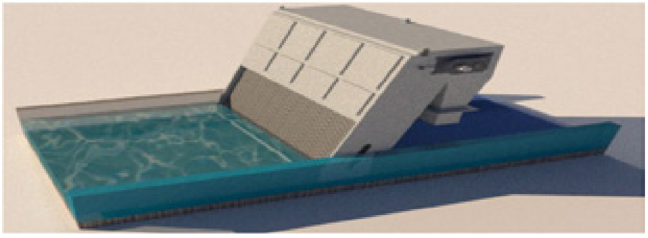

Fig. 3: Typical Belt Press with Screening Mechanisms at the inlet stages of the Water Treatment Plant

Rys. 3: Typowa prasa taśmowa z mechanizmami cedzenia na wlocie do stacji uzdatniania wody

The design of this WTP or WTW is based around the mechanisms of conventional water treatment processing and being adaptable, to be able to fit within any give inflow channel. In this case it can be fitted to fit within a single steel tank with six (6) side by side placed at an opening of a single outlet pipe which transports water for further treatment. The reason for collecting the water in a single storage tank is that the water will slow down significantly, increasing the efficacy of the screen and reducing the harm such a force of water could cause to the tiny wire mesh.

The belt-press system can be designed to create a number of steps or ledges which combined with the small mesh size, increase the efficiency of the belt in catching contaminants. These systems can be found at desalination plants in the Caribbean and the generation of waste sludge again is conveyed to landfills. The double

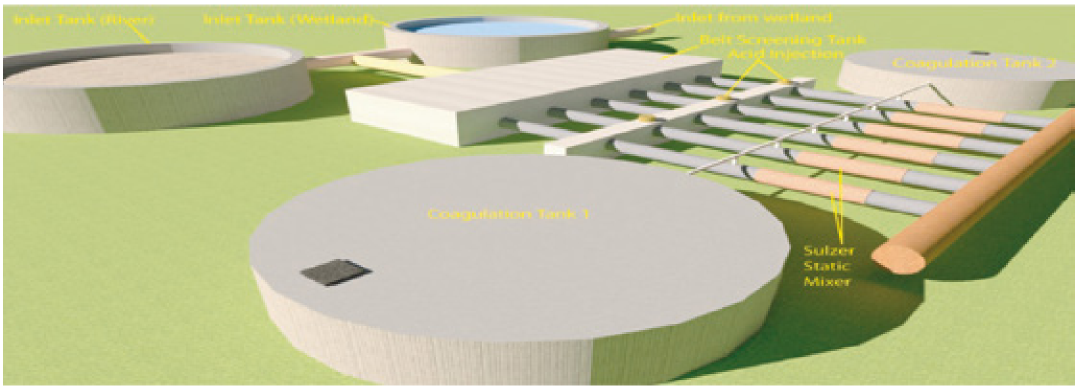

Fig. 2. *View of the Inlet pipe, Screening Tank, coagulation, and rapid mixers.*

Rys. 2. Widok rury wlotowej, zbiornika z kratą, koagulacji i mieszania szybkiego.

catchment method means less material is forced through the mesh by the force of the water. Once the belt reaches the peak (the top) and is well clear of the water level, the steps flatten to allow for the cleaning of debris from the mesh to be easier. The cleaning process involves a combination of 'teeth' and a powerful jet of water which work together to scrape the meshes clean [32]. Once the belt is clean it should continue down the back of the mechanism and once again submerge into the effluent. The rate which the belt completes a cycle depends on the amount of waste that is being picked up by the mesh. Therefore, the rate the belt moves must be adaptable which is where the use of lasers becomes applicable. The laser system calculates the amount of vegetation for example being picked up and will increase the belt cycle speed (or decrease depending) to maintain efficiency. This also reduces the need for a 24-hour operative, minimalising cost. The advantage to using this product is corrosion resistance, which reduces the cost of maintenance. The odour of the water is contained within the tank. All connection points such as bearings and bolts are above the water mark which again limits the cost of maintaining the product [26].

Coagulation Process

The primary treatment process uses the weight of particles to concentrate them to a specific location (bottom of a clarifying tank), see **Fig 4-5**. From there, it can be easily collected for removal. Common water and environmental engineering knowledge dictate that heavy particles will sink to the bottom of any basin in a short time period. For example, particles that are 1mm in diameter will take 6 seconds to drop 1 metre. However, simply leaving gravity to do its work will have a little effect on smaller particles, those that are 0.01μm for example will take almost 600 years to drop a single metre at best Ali and Budari [2]. This is where coagulation comes to hand. It works by adding coagulant chemical substances such as Aluminium Sulphate: ($Al_2(SO_4)_3$);Ferric Chloride: $FeCl_2$ or in some cases Polymers (large molecules with varying chemical structures, often based on organic compounds like acrylamide. Their specific chemical formulae can vary greatly depending on the type of polymer used). that works to neutralise the suspended components which are of a negative charge. The reason for neutralising their ions is because beforehand they are repelled from each other, now they are capable of sticking together, creating larger and therefore heavier particles. These heavier elements will have a reduced settling time and are known as floc [29].

Design Process

The coagulant to be used is common in most plants around the world, aluminium sulphate ($Al_2(SO_4)_3$) 14.3.H_2O. It is generally the most effective in turbidity's greater the 100 NTU which is the case for this water source for our case study. It is also most effective at reasonable temperatures such as 25°C, the average for freshwater reservoirs and rivers in the Caribbean that mentioned by Tota-Maharaj and Tota-Maharaj [35]. It is also the easiest substance to work with and requires no coagulant aids. However, for it to be effective it must

Fig. 5: Aerial View of Paddles (2.8m × 20m)
Rys. 5: Widok z lotu ptaka na łopatki (2,8 m × 20 m)

be placed in an environment where the pH levels are in-between 6.5-7.2, where the current water level is 8.0 [33]. To reduce the pH level of the surface water being treated, sulphuric acid (H_2SO_4) is added before the coagulation stage for testing to determine optimal dosages of ($Al_2(SO_4)_3$) 14.3.H_2O to have an effective impact. The required amount of acid was calculated (**Appendix 1**) to be 0.126 m^3/day (0.00000146 m^3/s) for the entire flow entering the plant. The design however dictates for 6 pipes parallel to one another, of which 4 will be operational at all times. Therefore, there will a single acid tank to supply 2 of the pipes with the acid in order to reduce the pH to 7.0. Each tank will need to hold a week's worth of H_2SO_4 to supply there 2 pipe requirements, leaving a required volumetric flowrate of 0.441 m^3/s.

Water treatment process calculation: 01

Sulphuric Acid (H_2SO_4) dosing calculations (Appendix 1)

H_2SO_4 volume requirement calculated with an average pH of influent water$_1$ = 8.0
The Molarity (M) of input water$_1$ = 1×10^{-8}, Volume (V) of input water$_1$ = 140,000,000 L/day, pH of injected acid = 1.0 & Molarity (M) of injected acid = 1×10^{-1} = 0.1 mol/L &
Volume of injected acid = (V_{acid}), Required pH of output water$_2$ = 7.0
Required morality of output water$_2$ = 1×10^{-7} mol/L & Volume of output water$_2$ = (V_{acid}) L/day + 140,000,000 L/day.
To find (V_{acid}), use following formula and rearrange:

$$(M_{acid}) + (V_{acid}) + Mwater_1\ Vwater_1 = Mwater_2\ (+Vwater_1) \quad (4)$$

(2) $0.1\ (V_{acid}) + [(1 \times 10^{-8})(140{,}000{,}000)] = 1 \times 10^{-7}\ ((V_{acid}) + 140{,}000{,}000)$,
Therefore, V_{acid} = 126 L/day = 0,126 m^3/day = 0,00146 m^3/sec

Initial Water Resources Storage: Three (3) tanks to supply 6 pipelines for 1-week treatment, or 2 tanks to supply the 4 pipelines to keep the plant running, providing a continuous 24/7 water supply is crucial for maintaining public health, promoting economic development, and preserving ecological balance. This necessitates robust water infrastructure, efficient water management, and sustainable water resource utilisation.

a) b)

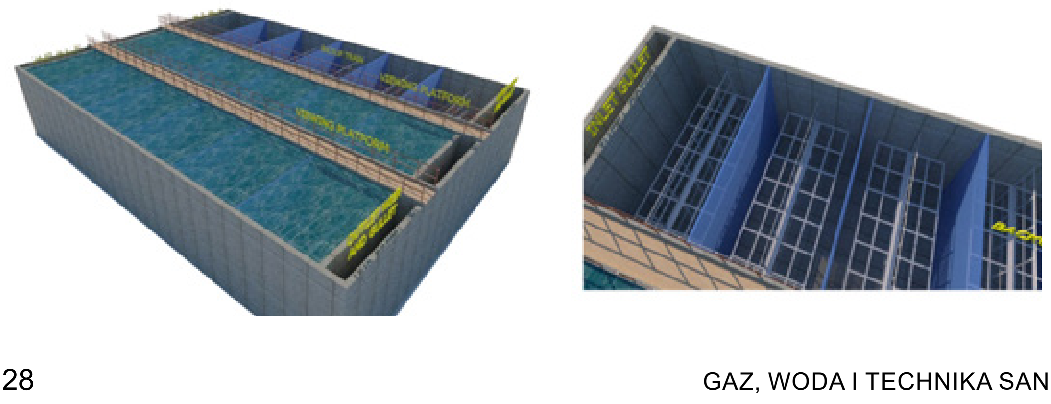

Fig. 4a: *Complete view of flocculation tank and train system for Water Treatment Plant Fig.* **4b**: *Flocculation tank and train system (Aerial View of Paddles)*
Rys. 4a: Widok na komorę wolnego mieszania (flokulator) dla stacji uzdatniania wody Rys. 4b: Zbiornik flokulacji (widok z lotu ptaka na łopatki)

$$(V_{acid})/week/tank = 0.126\ m^3\ s \times 2\ tanks\ 4\ pipes \times 7days = 0.441\ m^3/s \quad (5)$$

The reason for having 3 tanks, of which only 2 will be injecting acid at any time is to allow unpressured time to clean, maintain and refilling the tanks without disrupting operations. It will also mean that if there are any unscheduled delays to acid tanks or pipes there are backups at hand. Stainless steel tanks are typically used as they are effective against corrosion, and the pump used to inject the required quantity into the water is known as a magnetic-drive pump [4,8].

With the water quality expecting to be dropped to the required pH level, the $(Al_2(SO_4)_3)$ 14.3.H_2O can be added. The quantity required varies depending on the quality of the water. A jar test lets the plant operators know the most effective coagulant quantity in terms of creating the most amount of floc. Since no such test were carried out to support the design, the required amount chosen lying within a range, 10-150 mg/L & 30 mg/L was chosen, though quite low, with a high turbidity level, the chemical/coagulant will not have to work hard to be effective in bringing already close particles together [34]. The coagulant must be able to feed 4 of the 6 pipes at the same time. In this case only two tanks will be needed, one on either side of the 6 pipes. Each tank will be able to supply any of the 6 pipes with their required dose. This is so when one pipe is being cleaned, the other can maintain operations. Therefore, each tank must hold the required coagulant for the entire flow. The mass of $(Al_2(SO_4)_3)$ 14.3.H_2O needed (Appendix 2) per day is 4,200 kg, a huge quantity which require a huge tank hence each tank will be designed to hold a single hour's worth of coagulant before it needs to be refilled. The tank is to be circular with a radius of 10m and a height of 19m. The tanks will be built underground to protect the chemical from the elements as well as making re-filling the tanks throughout the day far easier. The tank can be built using steel, concrete or engineering bricks which are more durable to the chemical environment they will be used for storage [39].

Water treatment process calculation: 02

Coagulant mass and concentration requirement (Appendix 2)

The Mass of Alum coagulant needed =
30 mg × 100,0000 × 140,000,000 = 4200 kg/day/tank
Therefore, 4,200 kg needed every day per tank.
The volume of tank designed to hold a single hour of alum:
Hence, volume required per hour = 140,000 m^3/sec, Thus, 24*hrs* = 5833.333 m^3/hr & Base area = π × 102 A = 314.16 m^2, Height =18.57m ~ 19m.

With this dosage and concentration of the coagulation known, the Danckwerts intensity of segregation can be calculated. This value comes to 2.584×10^{-5} (I_s). This value represents how effective the coagulant will be in mixing evenly with the water after rapid mixing. The efficiency is dictated by how close the Is value is to zero, the closer it is the more effective the final mixture will be [38].With the mass of Alum $(Al_2(SO_4)_3)$ 14.3.H_2O known and the amount of acid need to reduce the pH, it needs to be added to the flow along with the acid. Once it enters the flow stream all three components will be mixed evenly by the use of a rapid mixer. The method used is known as an in-line or static mixer which works by fitting numerous blades inside a pipe segment. These blades do not move (hence the term static), it is in fact the high turbulence that is created due to the metal plates that mixes the flow effectively. The advantages to such a system are that it requires fewer parts, takes up less space, is very reliable and can withstand up to 100 million litres a day each.

For the rapid mixer to be effective, the flow and all its added chemicals must reside within the mixer for a total of 3.903 seconds (**Water treatment Process calculation 3**). Therefore, this value determines the length of the pipe required for a mixer with an already known diameter, 0.586m (equal to the pipe running into it). The mixer will be made up of five segments attached end to end, with each section having a length of 0.879m that mentioned by Twort et al. [38] and Aker Solutions [1].

Water treatment process calculation: 03

Designing inline mixer design (Appendix 3)

For the Intensity of segregation, the closer to 0, the better mixing processes achieved:

$$Xa = Ca \text{ dose coagulant} = 30 \text{ mg/L coagulant} \quad (6)$$

Concentration of chemical in feeder stream (C_a):
$(Al_2(SO_4)_3)$ 14.3.H_2O = molecular weight of 600 mg/mmol, Alum source specific gravity = 1.335, Alum strength = 48.5% as alum, Concentration = 1.335 × 1000359.313 mgL × 48.5 mg dry alum 100 mg wet alum = 647707.6462 mg/L
647707 × mmol alum 600 mg alum × 2 mmol Almmol alum × 27 mg Al mmol Al=58293.688 mgL of Al
X_a= 3058293.688 = 0.000515
Assuming a Coefficient of variation of time is 5%:
σ_m= 0.05 x 0.000515 = 2.575 $\times 10^{-5}$ σ_u= 0.000515(1−0.000515) = 0.023 l_s= 2.575 x 10 − 50.0232=1.288 × 10^{-6}, Q = 140000 m^3/day = 0,405 m^2/sec, Area of Pipe = 0.405 1.5 m/s =0.27 m^2
Diameter = √4 × 0.27 π = 0.586m, The number of mixers required per pipeline needs to be found:
$\sqrt{1_s}$ = 1.288 × 10^{-6} = 0.001135
0.001135 = $\sigma_m \sigma_u L/D$

Where σ_m & σ_u, are a pre derived values, known as the mixing ratio, in this case 0.21, is the length/diameter of mixer which is aimed to be 1.5, Log 0.001135 = (log0.21) L/D, L/D = 4.35, , Therefore, 5 lengths of static pipe are needed for this WTP case study.

At this stage of the WTP design the mixing time is required, typically looking for a value of below 10 seconds for in line mixers.

Velocity (v) = 0,405 m^3/sec[π × $(0,586)^2$ × 3] =1.126 $^{m}/_{sec}$
Time (T_m) = 5 × 1,5 × 0,586 × 1.126 = 3.903 sec

Flocculation Process

While coagulation combined with rapid mixing is effective in forming flocs for larger particles (over 1 micrometre), it is less efficient for smaller particles. To encourage the aggregation of these smaller particles, a flocculation chamber is employed. This chamber uses slow-moving mechanical mixers to gently agitate the water. This gentle stirring promotes particle collision, enabling the formation of larger, heavier and settle able flocs [11].

Design Considerations

The single pipe from the coagulation phase in this WTP case study, can be divided into three separate pipes, each leading to a flocculation tank. The water enters each tank through a gullet, a narrow channel designed to slow the water flow. This controlled release reduces turbulence, allowing for the gentle mixing provided by the mechanical turbines. The minimal volume of the gullet maximises turbulence within the channel, preventing pollutants from settling and requiring less frequent cleaning. Three (3) primary mechanical mixer types are applicable: horizontal paddles, vertical turbines, and hydraulic flocculation. Horizontal paddles are often favoured due to their ability to form larger flocs, reliability, zero head loss, and efficient mixing of large volumes with fewer shafts. Detailed calculations and design tables are provided in Appendices 4, 5, and 6 [30], [24]. The proposed system will operate with three trains, two actively treating water for 25 minutes, while the third undergoes maintenance or cleaning. Each train's inlet pipe, capable of handling 0.81 m^3/s, requires a diameter of 0.828 m to maintain a flow velocity of 1.5 m/s. The six chambers in each train, each holding 202.55 m^3 of water, will be equipped with two horizontal paddles. These paddles, with a height of 4 m and a clearance of 0.5 m from the top, bottom, and sides, will have a depth and length of 5 m. The width of each chamber, determined by the paddle configuration, is 8.1 m. With 0.7 m clearance from the walls and 1 m between paddles, each paddle will measure 2.85 m in length. Each paddle will have four wings, each equipped with three equally spaced 100 mm wide boards. This design adheres to standards by ensuring the total board area is less than 20% of the compartment's surface area. The optimal rotation speed for the flocculation mechanisms is 2.27 revolutions per minute, falling within the design range of 1-5 rpm. Visual representations of the design created using Google Sketch UP 2019, AutoCAD 2018 and Adobe Illustrator CS are included it mentioned by Crittenden et al. [9] and Sulzer [31].

Water Infrastructure Design Process Stage 1, Sample guide for the Depth of Basin (Appendix 4)

The Depth of basins/clarifiers or sedimentation tanks can also be determined by the diameter of paddles (3m ~ 4m) + 0.5m clearance, Basin depth = 4m + (0.5m x 2) = 5m

Two (2) flocculates will be in use at all times where another is in place for when 1 is broke down or being cleaned for periodic maintenance purposes. Therefore, 70million litres are running through each flocculate per day. With an estimated Flowrate entering each tank:

$$\frac{1.62\ m^3/s}{2} = 0.81\ m^3/s \ \text{\& The area} = \frac{0.81\ m^3/s}{1.5\ m/s} = 0.54m^2, \text{Radius} = \sqrt{\frac{0.54}{\pi}} = 0.414m$$

Volume per compartment (v):
70 ml/day × 1000 m^3 ml × (detention time: 1440 min or 1 day) × Trains × Stages per train
Detention time typically 20-45 minutes, use 25mins & Trains =1 respectively., Stages per train range between 2-6, 6 to be used.
Volume per compartment (v) = 202.55 m^3 & Min length = basin depth = 5m, Area = VBD = $\frac{202.55m^3}{5m}$ = 40,51 m^2 & Width = $\frac{40.51m^2}{5m}$ = 8,1 m, Side clearance = 0.7 either side, So, Paddle length = 8.1 – (0.7 × 2) = 6.7m, Paddle length must range between 2m ~ 3.5m, However, 6.7m is too long.
Therefore, two (2) paddles must be used, which must be spaced 1m apart. Hence, the Paddle lenght = $\frac{6.7m-12m}{2}$ which is within the 2m ~ 3.5m range.
Depth = 5m, Length = 5m & Width = 8.1m. Paddle wheel number = 2 at 2.65m in length.
Board number = 4, Panels per board = 3 at 100mm width each, equally spaced.
The ratio of Area of boards: Area of compartment < 20%
The Area of board (0.1m × 2.65m) × 12 boards / 2 paddle wheels at 8.1m (width) × 5m (basin depth) = 3.926% < 20%

Power and Rotation calculations:

Power needed to reach velocity gradient design value:

$$P = G^2 \mu v \tag{5}$$

Here,
G = Velocity gradient which must range between $20s^{-1}$ – $50s^{-1}$, use 35 s^{-1}
μ = Dynamic viscosity of water at 15° C is 0.001139 kg/m.s
V = Volume of mixing compatment = 202.55. m^3
P = power of mixing input to compartment = 35^2 × 0.001139 × 202.55 = 282.61 kgm^2/s^2 = 282.61 J/s

So, Power per paddle:

$$P = \frac{1}{2}\rho C_D A_p [V^3(\text{Inside paddle}) + V^3(\text{Middle paddle}) + V^3(\text{Outside paddle})] \tag{6}$$

Here, Ap = projected area of paddle = 6.84m^2
Paddle Lenght & Width ratio check = $\frac{2.85}{0.15}$ = 19
C_D = drag coefficient on paddle (for turbulent flow), unitless
So, use table value (C_D) = 1.5 (as 19 is close to 20)
ρ = density of fluid = 999.103 kg/m^3

$$\text{Velocity of each paddle} = \frac{r2\pi N(0.75)}{60s/min} \tag{7}$$

Where,
r = Distance, Centre of shaft to centre of board
N = Shaft rotational speed, rev/min
0.75 = Approx. speed of paddle in water

Now,
r_{inside} = 0.667 – 0.12 = 0.547m
r_{middle} = 1.333 – 0.12 = 1.213m
$r_{outside}$ = 2.0 – 0.12 = 1.88

$$P = \frac{1}{2}\rho C_D A_p [V^3(\text{Inside paddle}) + V^3(\text{Middle paddle}) + V^3(\text{Outside paddle})] \tag{8}$$
$$P = \frac{999.102 \times 1.5 \times 6.84}{2} \times \left(\frac{2\pi N(0.75)}{60\ s/min}\right)^3 \times (0.547^3 + 1.213^3 + 1.88^3)$$
$$282.61 = 5125.39\ (4.85 \times 10^{-4} N^3)(8.588)$$
$$N = 2.268\ \text{rev/min}, \text{(which is within the design range 1 − 5)}$$

Sedimentation The Settling Process

The sedimentation clarifier is a crucial stage in the wastewater treatment process, where the fruits of the preceding steps come to fruition. The flocculated particles, formed through chemical and mechanical processes, are introduced into a large settling tank (See **Fig 7**). Under the influence of gravity, these heavier-than-water particles gradually settle to the bottom of the tank. As the sludge accumulates on the tank floor, it is periodically removed. This sludge undergoes further treatment to reduce its volume and harmful content. The treated sludge can then be disposed of in a landfill or repurposed for agricultural applications. The clarified water, now relatively free of suspended solids, is recycled back into the treatment process, typically just before re-entering the sedimentation tank. This recirculation helps to maintain optimal operating conditions and enhance treatment efficiency [16].

Design Considerations

Two (2) primary configurations of sedimentation tanks are commonly employed: horizontal and circular. For this specific design, a horizontal tank has been selected to effectively achieve the required contaminant removal. The tank's design incorporates a gullet, similar to the flocculation tank, to minimise turbulence. This design feature is essential to prevent the incoming flow from disrupting the settled solids, ensuring optimal sedimentation. To facilitate efficient sludge removal, a chain-and-flight mechanism will be implemented [16, 10], which ensures the reliable and continuous extraction of sludge from the tank floor. A detailed analysis of the design calculations can be found in Appendix 7.

Water Infrastructure Design Process Stage 2, Sample guide for the calculations [47-50] and (Appendix 5)

With a maximum daily flow, Q_{max} = 1.62 m^3/s

Hence, two (2) basins will be in use whilst a third is being cleaned and in place for a potential break down of one of the others. Inlet and entry points

The average density of water (ρ) at 15-20°C = 999.103 kg/m^3

The Width of tank is designed around length of flights used in chain flight scraper:

A 15m wide tank, with 3m ~ 5m long flights side by side can be selected. Using depth design criteria: 3m ~ 5m and choosing the mean which is 4m.

The Particle sizes are assumed as a jar test cannot be carried out; the smallest alum floc is assumed using design table. The reason for using the smallest as it will have the longest detention time; therefore, a tank designed to this time spec will almost guarantee the removal of all floc.

The Settling rate is 2 metre drop every hour.

Area of all settling tanks combined = QVc= 1.62 23600s = 2916 m^2

Length requirement = A / (No.Basins) (width)= 2916 / 2 × 15 = 97.2m

Width (w) = 15 m & depth of tank (dt) = 4m

Note, the use of flight by chain sludge removal method, does not work at a length greater than 60m, therefore, 2 will need to be set up. The Length to width ratio is 97.2:15 which is equal to 6.48:1 > which is greater than the minimum requirment of a 5:1 ratio.

Length to depth ratio is 97.2: 4 which is equal to 24.3:1 > which is greater a minimum requirment of a 15:1 ratio.

Typical detention times in basins / clarifiers range from 1.5 hours to 4 hours.

Assuming, a standard detention time (D_t) of 2 hours per tank.

Detention Time = Volume / Flowrate (Q)

Horizontal flow velocity (V_f) = $\frac{Q}{A} = \frac{1.62m^3/s}{120m^2}$ = 0.0135 m/sec or 0.81 m/min which is between the range of 0.3 and 1.1m/min. Horizontal-flow velocity (V_f) = QA = (1.62 m^3/s) / Area (15m × 4m) × 2 basins = 0.81m/min find references

Now, what is known as the Reynold and Froude numbers need to be found. Reynolds number must be below 20000, if not this signifies high turbulence in the flow.

$$\text{Reynolds number } (R_e) = \frac{\rho V_f R_h}{\mu} \quad (9)$$

Where,

ρ = Density of fluid = 999.103 kg/m^3

V_f = Horizontal flow velocity = 0.0135 $\frac{m}{s}$ or 0.81 m/min

R_h = Cross sectional area / Wetted perimeter = 4m × 15m) + 15m + (2 × 4m) = 2.609m

μ = dynamic viscosity of the fluid = 0.001139 kg/m.s

Reynolds number (R_e) = $\frac{999.103 \times 0.0135 \times 2.609}{0.001139}$ = 30895.44

As the design standard states Re cannot exceed 20000, hence, 30895 is far too high. Therefore, the applications of baffles will be needed to be constructed, running in a longitudinal fashion or varying configuration according to the water and environmental engineers' preference. Two (2) baffles would be sufficient in this water treatment works. However, this would change the chain scraper design, and therefore three (3) baffles would be recommended in the design, separating the scrapers from each other (Crittenden et al., 2005) [10].

This design change only changes the hydraulic radius value:

Rh = Cross sectional area/ Wetted perimeter = 4m × 5m + 5m + (2 × 4m) = 1.538m

Reynolds number (R_e) = $\frac{999.103 \times 0.0135 \times 1.538}{0.001139}$ = 18212.797 < 20000 (Okay)

Froude's number must be greater than 10^{-5}, if not, the horizontal flow is not the dominate force in the tank.

Froude number (Fr)

$$= \frac{V_f^2}{gR_h} \quad (10)$$

$$\frac{(0.0135\frac{m}{s})^2}{9.81\frac{m}{s^2} \times 1.538m} = 1.21 \times 10^{-5} > 10^{-5} \quad \text{(design criteria reached)}$$

So, exit weir size = $\frac{0.81\,m^3/s}{1.5\,m/s}$ = 0.54 m^2 & $\frac{0.54m^2}{15m}$ = 0.036m height

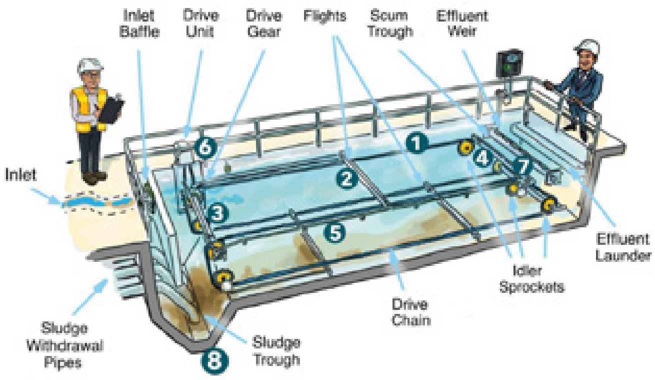

Fig. 6: Horizontal-flow rectangular sedimentation tank within the Water Treatment Plant (WTP)

Rys. 6: Prostokątny zbiornik sedymentacyjny z przepływem poziomym na stacji uzdatniania wody (WTP)

Where:

1. Collector chain link which is designed specifically to any individual settlement tanks specifications, unlike other companies that work with a single mould.
2. Scraper flights, which skim floc that floats on the surface and scraps dense sludge off the bottom. Made almost completely of stainless steel, other than the pints that make contact with the base and water surface which are made of easily replaceable rubber. The rubber allows for flexibility which makes it more effective at catching all solids.
3. Shafts, which like the links, are designed to the settlement tank in question.
4. Tensioning system, in most designs to tighten the chain, the tank must be emptied. In this system, it can be done as the treatment process is under way from outside the tank.
5. Stainless steel rails, which the flights slide along as they clean the tank.
6. Motor turning the shaft and maintaining the chain and flights cycle. It must be at a slow cycle, as anything to rapid will create turbulence and lift settled particles off the base.
7. Scum canoe which collects the surface floc. They remove the floc by the use of an electrical actuator rather than a hydraulic method as less energy is needed.
8. Sludge hooper, where the settled sludge is forced into, before being sucked away with the use of pumps. The pumps work automatically when the waste gets to a certain weight. The Hooper sides at a 60° to discourage the particles rising.

The number of basins is the same as with the flocculation process, 2 in operation and a third waiting in the wings when needed (**Figs 7-9**). It is designed to allow the smallest floc sizes to settle which have a settling rate of 2m/hr. Generally, the size of floc would be found by jar tests, however as this is not possible the design takes into account the worst-case scenario. The depth of each tank will be 4m and the length of each compartment is 97.2m. A chain by flight cleaning method is not effective beyond the 60m mark; therefore 2 have been placed within the tank. The width of the tank is to be 15m; however, it is to be divided into 3 segments of what are known as baffles. This is due to the Reynold and Froude's numbers, which depict turbulence, did not meet the standards for a 15m wide chamber; hence 5m baffles are in place each with a scrapper as described by the Minnesota Rural Water Association [25].

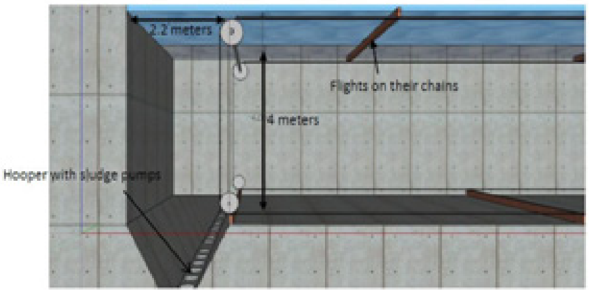

Fig. 8a: Chain and flight cleaning mechanism
Rys. 8a: Mechanizm czyszczący łańcuch i ślimak

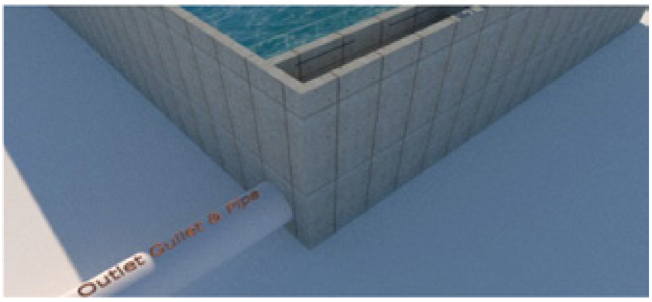

Fig.8b. Flow Outlet from the clarifier / settling tank
Rys. 8b. Wylot przepływu z osadnika / zbiornika sedymentacyjnego

When the water reaches the end of the tank it will pass through a weir which maintains the flow required without the water level rising and flowing over the sides. The weir is 0.06m deep and 3.00m long between each buffet and represented in **Fig.8**.

Once the water has passed through the weir it collects in a further gullet before being pumped out by a single pump which transfers the flow to the secondary stage (Fig. 8). An overall view of the system can be seen in Fig.9, note the single empty tank ready and waiting for any mechanical breakdowns that a plant may face on a daily basis.

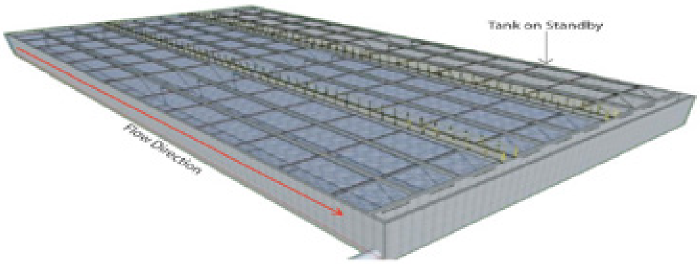

Fig. 9. *Overview of Sedimentation Tanks*
Rys. 9. Zbiorniki sedymentacyjne

Conclusion:

This study highlights the critical importance of designing water treatment plants (WTPs) that are both technically robust and adaptable to the unique conditions of Caribbean small island developing states (SIDS). By integrating conventional primary, secondary, and tertiary treatment processes with innovative design considerations, the proposed WTP model is able to meet stringent international standards and ensure safe drinking water for approximately 140,000 cubic meters per day. The findings indicate that untreated or inadequately treated water continues to be a significant source of environmental degradation and public health risk. However, through systematic approaches such as coagulation, flocculation, sedimentation, rapid filtration, and disinfection with chlorine, contaminants including coliforms, E. coli, turbidity, and organic pollutants can be effectively reduced to safe levels. This structured approach not only protects human health, but also contributes to the water cycle, sustainability and long-term resilience to climate-induced pressures. As a result, the proposed WTP scheme provides a replicable and scalable framework that can be adopted across the Caribbean and beyond. Its emphasis on efficiency, resilience and adaptability ensures a sustainable supply of drinking water, reduces dependence on external sources and is aligned with global goals for integrated water resources management. Continued investment, innovation and adherence to regulatory standards will be critical in transforming such schemes into operational infrastructure that protects communities and ecosystems alike.

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