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Integral Water Treatment Plant (WTP) Design Guide for Resource Circularity in the Caribbean: Part 2

Kompleksowy przewodnik projektowania stacji uzdatniania wody (WTP) na potrzeby obiegu zamkniętego zasobów na Karaibach: Część 2

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Keywords: *Water Treatment Plant (WTP); Small Island Developing States (SIDS); potable water supply; contaminant removal; water quality standards; Caribbean; sustainable water management.*

Abstract
Small Island Developing States (SIDS) face increasing challenges in providing safe drinking water due to increasing demand, limited access to freshwater, and vulnerability to pollution. This paper presents the design and analysis of a conventional water treatment plant (WTP) designed for the Caribbean region with a capacity to treat and distribute 140,000 cubic meters of drinking water per day. The study describes a systematic approach to water treatment, including screening, coagulation, flocculation, sedimentation, flash filtration, and tertiary disinfection. The performance results show significant reductions in key contaminants such as coliform bacteria, Escherichia coli, turbidity, biochemical oxygen demand (BOD), and chemical oxygen demand (COD), bringing water quality within internationally accepted standards set by the World Health Organisation (WHO) and the United States Environmental Protection Agency (US EPA). The WTP proposal highlights practical engineering solutions to improve water security, protect public health, and ensure environmental sustainability in the Caribbean region.

Słowa kluczowe: *stacja uzdatniania wody (WTP), małe rozwijające się państwa wyspiarskie (SIDS), woda przeznaczona do spożycia, usuwanie zanieczyszczeń, standardy odnośnie jakości wody do spożycia, region Karaibów, zrównoważone zarządzanie wodą*

Streszczenie
Małe rozwijające się państwa wyspiarskie (SIDS) stoją przed coraz większymi wyzwaniami w zakresie zapewnienia bezpiecznej wody pitnej ze względu na rosnące zapotrzebowanie, ograniczony dostęp do słodkiej wody i podatność na zanieczyszczenia. Niniejszy artykuł przedstawia projekt i analizę konwencjonalnej stacji uzdatniania wody (WTP) zaprojektowanej dla regionu Karaibów, której wydajność wynosi 140 000 m3/d. W badaniu opisano systemowe podejście do uzdatniania wody, obejmujące cedzenie, koagulację, flokulację, sedymentację, filtrację i końcową dezynfekcję. Przedstawione wyniki pokazują znaczne zmniejszenie kluczowych zanieczyszczeń, takich jak bakterie z grupy coli, Escherichia coli, mętność, BZT5 i ChZT, dzięki czemu jakość wody spełnia międzynarodowe wymagania ustanowione przez Światową Organizację Zdrowia (WHO) i Agencję Ochrony Środowiska Stanów Zjednoczonych (US EPA). Przedstawione rozwiązania WTP podkreślają praktyczne podejście inżynieryjne mające na celu poprawę bezpieczeństwa wodnego, ochronę zdrowia publicznego i zapewnienie zrównoważenia środowiskowego w regionie Karaibów.

Introduction

Access to clean and safe drinking water remains one of the world's most pressing challenges, particularly in small island developing states (SIDS), where reliance on fragile ecosystems and limited freshwater resources makes communities highly vulnerable to water scarcity and pollution. According to the United Nations Environment Programme (UNEP), approximately half of all wastewater globally is discharged untreated into rivers, lakes and oceans, posing significant risks to human health and aquatic ecosystems. In the Caribbean, this situation is exacerbated by rapid urbanisation, industrial growth and climate-related stresses, which are placing increasing pressure on existing water infrastructure. The consequences of inadequate treatment are severe, ranging from outbreaks of water-borne diseases to long-term environmental degradation. The World Health Organisation (WHO) estimates that more than 840,000 people die annually from water-related diseases, many of which are preventable through effective treatment and monitoring. This reality highlights the urgent need for robust and sustainable water treatment solutions that can ensure a reliable supply of drinking water in SIDS. This paper presents the design of a conventional but scalable water treatment plant (WTP) for the Caribbean, capable of treating and distributing 140,000 cubic meters of drinking water per day. The plant includes various treatment stages including primary, secondary and tertiary processes, designed to meet stringent environmental and health standards. By addressing common contaminants such as coliform bacteria, E. coli, turbidity, BOD and COD, the proposed design provides a replicable framework to ensure access to safe water, reduce dependence on external sources and advance sustainable water resource management across the West Indies.

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Secondary Treatment Stage

Filtration Processes

The secondary treatment stage primarily involves filtration as illustrated in **Figs 1-5**. This process utilises one or two layers of granular media with varying particle sizes. Water is allowed to flow through these filters under the influence of gravity. The filtration process captures smaller particles that were not removed during the clarification stage. As the filter media becomes clogged with trapped particles, it is necessary to clean the filters. This is achieved through a backwashing process, where a portion of the treated water is forced upward through the filter bed. The upward flow dislodges the accumulated particles, which are then carried away for further treatment [36].

Design Considerations

The design calculations for the filtration system can be found in Appendices 10 and 11. A total of 20 filters will be installed, with 17 operational at any given time. The remaining three filters will serve as spares to ensure continuous operation during maintenance or backwashing periods. Each filter is designed to handle a flow rate of 8235.3 [unit of flow rate, e.g., m^3/h or gallons per minute].

Water Infrastructure Design Process Stage 3, Sample guide for the calculations [REFERENCES-64, 65, 66, 67, 68, 69] (Appendix 6)

Calculations for filter size: Minimum number of filters $= 2.7\sqrt{Q}$ (1)

Q = volumetric flow rate $= 140000\ m^3/day = 36.984$ million gallon/day

So, Filter number $= 2.7\sqrt{36.984} = 16.42$, say 17 nos. filter

20 constructed overall, 1 out of action when cleaning (back washing) and 2 in cases if a breakdown occurs.

Finding the flowrates through one filter

Volumetric flowrate of 17 *Nos. Filters* $= 140000\ m^3/day$

Thus, Volumetric flow rate of each filter $(Q_c) = 8235.294\ m^3/day$

Now, Filter velocity (F_v) which lies between $100\ m^3/day \sim 200\frac{m}{day}$, $150\ m^3/day$ is chosen. Hence, the cross-sectional area of filter required, A =

$$\frac{Q_c}{\text{Filter velocity } (F_v)} = \frac{8235.294\frac{m^3}{day}}{150\frac{m}{day}} = 54.9m^2 \qquad (2)$$

The Dimensions of filter: Breadth = 5m, Length = 11m

Typical Filters are made up of 2 layers, calculated by working of a standard example:

Anthracite layer depth is 1500mm, and sand layer depth is 300mm.

Using a ratio of 300:1500 = 0.2, *Therefore: First layer (Anthracite):* 900mm depth

Effective size = 1.4mm, *Second layer – Sand:* 180mm, Effective size = 0.6mm

Head Loss Pressure in the filter chamber when the granular media is clean:

Calculated using the Carmen – Kozeny Equation:

$$h/L = \frac{k\mu(1-\varepsilon)^2}{g\rho\varepsilon^3(6\theta d)2v} \qquad (3)$$

Where,

h = head loss in depth of bed, L

g = acceleration of gravity

k = dimensionless Kozeny constant with a value of about 5 for most filtering conditions

ε = porosity

V = superficial velocity about the bed = flow rate/bed area

μ = absolute viscosity of fluid

ρ = mass density of fluid

Calculations to find Backwash flow for anthracite bed expansion by 35% and how much the sand layer will expand under this flow.

$$L_E = L_F + (0.35L_F) \qquad (4)$$

$$L_E = 0.9 + (0.35 \times 0.9) = 1{,}215m$$

where,

L_F = First filter layer depth (Anthracite) in m

L_E = First filter layer effective depth (Anthracite)

$$\varepsilon_E = 1 - [L_F L_E \times (1 - \varepsilon_F)] \qquad (5)$$

$$\varepsilon_E = 1 - [0.9 \times 1.215\ (1 - 0.495)] = 0.626$$

where,

ε_E = Effective porosity (Anthracite) in %

ε_F = Porosity of Anthracite in %

$$\beta = g \times \rho_w(\rho_p - \rho_w) \times d \times \varepsilon_E{}^3 \times \mu^3 \qquad (6)$$

β = 9.81 × 999.103(1500 − 999.103) × 0.00143 × 0.626³ × 0.001139², **β = 2547.34**

where,

g = acceleration of gravity

ρ_w = Density of water

ρ_p = Density of sand

d = Partical (Anthracite)diameter

ε_E = Effective porosity (Anthracite) in %

μ = Dynamic viscocity (Anthracite)

$$R_e = -k_v(1 - \varepsilon_E)2k_l + 12k_l k_v{}^2(1 - \varepsilon_E)^2 + 4k_l\beta \qquad (7)$$

R_e = −227.5(1−0.626)2 × 4.4 + 12 × 4.4 × 227.5² (1−0.626)2 +(4 × 4.4 × 2547.34), *Re* = 16.248

where,

R_E = Reynold's number

k_v = The flow coefficient

k_l = The flow coefficient (liquid)

Hence the Backwash Velocity can be computed by equation:

Backwash Velocity (V) = [$\mu \times R_e$] / [$p_w \times d$]

V = [0.001139 × 16.248] ÷ [3600 999.103 × 0.0014], V = 47.63 *m/hr*

With the backwash velocity calculated, the bed expansion of the sand needs to be calculated:

$$X = \mu \times v \times g(\rho_p - \rho_w)d^2 k_v + (k_l \times \rho_w \times v \times d \times \mu) \qquad (8)$$

X = (0.001139 × 47.63 × 9.81(2650−999.103)0.0006² × 112.5 + (2.25 × 999.103 × 47.63 × 0.0006 × 0.001139), X = 0.165

where,

X = Bed expansion of sand

V = Backwash Velocity

g = acceleration of gravity

ρ_w = Density of water

ρ_p = Density of sand

d = Partical (sand)diameter

μ = Dynamic viscocity (sand)

$$Y = k_v \times \mu \times V \times g(\rho_p - \rho_w)d^2$$

Y = 112.5 × 0.001139 × 47.6336003 × 9.81(2650 − 999.103)0.0006², Y = 0.0972 (9)

where,
Y = Bed expansion of Anthracite
V = Backwash Velocity
g = acceleration of gravity
ρ_w = Density of water
ρ_p = Density of sand
d = Partical (sand)diameter
μ = Dynamic viscocity (sand)

$$\varepsilon_E = \sqrt{X} + X^2 + Y^3 3 + \sqrt{X} - X^2 + Y^3 3 \quad (10)$$
$$\varepsilon_E = \sqrt{0.165} + 0.165^2 + 0.097^3 3 + \sqrt{0.165} - 0.165^2 + 0.097^3 3$$
$$\varepsilon_E = 0.553$$

where,
ε_E = Effective porosity (sand) in %

$$L_E(\text{sand}) = L_E(\text{Anthracite}) - \varepsilon_{E1} - \varepsilon_E\ (\text{sand}) \quad (11)$$
$$L_E(\text{sand}) = 1.215 - 0.415 - 0.553 = 0.247\text{m}$$

where,
L_E = First layer effective depth (sand)
ε_{E1} = Porosity of sand
ε_E = Effective porosity (sand) in %

So,
The Bed Expansion = $L_E L_F - 1\ 100\%$ = 0.235 × 0.18−1 100%
= 30.786% (12)

Therefore, maximum bed expansion of both layers total with a velocity of 47.63m/hr = (0.18m × 0.30786) + (0.9m × 0.35) = 0.37m

Therefore, the gullet wall should not be at least 370mm high to stop granular material escaping the section. The water troughs must also be higher than the bed expansion or the granular material will be sent away as sludge.

Wash through designs calculations: Height of from rested material,
H_o: $(0.75L + P) < H_o < (L + P)$,
L = *Thickness of granular media* = 1.08m,
So, P = *Height of trough* =?

To calculate the Height of trough (P):
No. Troughs = $Q_{max}\, Q_{Trough}$ (13)
Q_{Trough} = 47.63 × 11 × 54,
Q_{Trough} = 654.913 m^3/hr

Extrapolating the B = 533mm and the W ≈ 319mm (See Appendix 6 – Graphical interpolation with flowrates and widths)

P = W + *B*2, P = 319*mm*+ 533*mm*, P = 0.852*m*
[(0.75 × 1.08m) + 0.852m] < Ho < (1.08m + 0.852m)
1.3955m < Ho < 1.6655m

Therefore, H_0 must lie between the two values, 1.66m has been chosen.
This value also needs to be higher than both the head loss and bed expansion, which are 0.294m and 0.37m respectively, which 1.66m is.
The next check is the minimum trough height equation:
Min trough height = Q_{max}1.4*B*2/3 + Freeboard
Min trough height = (2619.65 × 1.4 × 0.533*m*)/3 + min of 0.05m
Min trough height = 1.033m < 1.66m therefore check met.

The next step is to confirm the spacing between the troughs, S:
S = [*Length of chamber*] / [*No* of *Troughs*], S = 12*m* ÷ 4, S = 3*m*
[1.5Ho < S < 2H_0]

(1.5 × 1.66m) < S < (2 × 1.66m), 2.49m < S < 3.32m, 2.49m < 3m < 3.32m therefore check is met.

The running time before each filter needs to be cleaned is 24hr, 1.33 hr apart from one another to allow backwash clear well to refill. Thus, the back wash time: Time for backwash to reach top of trough = 1.66 + 1.0847.63

T = 0.058hr = 3.48min, Therefore, total running time must be greater than 3.48min. Using ratios from design criteria at typical water treatment plant to create a ratio: Time for backwash to reach top of trough = 2.5+1.845 = 0.096 hr = 5.733 minutes

The *Total Running Time* = 5.733*min*/15*min*= 0.382, Total running time for our water treatment plant: 3.48/ 0.382 = 9.1*min*. With an estimated Back wash water capacity = 2% – 5% of total flow in 24hr, Hence, the backwash volume is stored in a clear well and will supply all the filter beds: V = (Backwash period) × (Total Filter Area in operation at one time) × (Filter Rise Rate), V = (9.1min) × (55m²) × (47.63*m/hr* 60*min*), where V = 397.314m³ 0.02<397.3148235.294<0.05, 0.02<0.0482<0.05 which is feasible for the design.

The settling tanks dimensions: Height above trough top = 3.06m & One complete tank to supply 10 of the filters at any given time with backwash: 397.314m³ × 10 = 3973.14m³, Dimensions = 3973.14 ÷ 124.1*m* = 32.016*m*², With circular base – Radius = 3.2m, Entrance Pipe area into each filter: 1.6217 = 0.0953*m³/s*, At a velocity of 1.5m/s: 0.09531.5= 0.0635m², Radius = √0.0635π = 0.142m, Q outflow = ×55=0.728m³/s, Velocity = 1.5m/s, Area = 0.7281.5 = 0.485m², Radius = √0.485π = 0.393m

Table1: Typical Design criteria for filters at Conventional Water Treatment Plants [36]

Tabela 1: Typowe kryteria projektowe filtrów w konwencjonalnych stacjach uzdatniania wody [36]

Parameter	Units	Value
Filter Type	-	Conventional, Dual-Mode
Flow Control		Influent weir split, constant level
Number	-	12
Interior Dimensions	m.m	4.3 x 11.6 x 2 cells
Media surface area (total)	m²	1200
Maximum available head	M	2.5
Filtration rate (at water treatment plant design flowrate) One Filter Offline All Filters in service	 m/h m/h	 15 13.75
Filter media Top layer Type Depth Effective size Uniformity coefficient Specific gravity	 ---- M Mm ---- ----	 Anthracite 1.5 1.0 <1.4 1.5~1.6
Bottom layer		
Type	----	Sand
Depth	M	0.3
Effective size	Mm	0.5
Uniformity coefficient	----	<1.4
Specific gravity	----	2.65
Backwash criteria		
Maximum rate	m/h	56
Normal rate	m/h	45
Duration	Min	15

The filter system used for this particular facility is known as a rapid sand filter, which was chosen over slow sand filters. A slow filter produces a better result in terms of cleanliness, however the rapid filter, combined with the designed primary treatment makes it a superior method. The list of its advantages grow longer when you take into account that it takes a maximum of 2 hours for water to pass through, compared with several days. The water will enter each filter via a pipe of 284mm in diameter, where the water will spill into a gullet which carries the same purpose as that used in the flocculation and sedimentation. The added advantage of the gullet is to restrict damage to the granular material bed it studied by Frostburg [15].

The water can now begin filtrating down through the coarse material. The filter rate is a taken value from a range (100 *m/hr*~200 *m/hr*), the choice was 150 *m/hr* (mean). For the flow rate entering each individual filter, the surface area of the filter will be 55m² (breadth = 5m and length = 11m). The actual filter is made up of two layers of differing materials or dual media. The advantage to having multiple types of material compared with one is because the filters need less cleaning which leads to longer runs, greater filtration rates and are able to handle higher turbidity levels [11]. When using two layers, the first layer must be larger in particle size then the second layer. This therefore means the medium which creates the larger voids sits on top. The reason for this is if the smaller voids made up the top layer, the second layer would become negligible as all contaminants would be caught in the initial granular layer. Any pollutants that did pass through would easily filtrate through the second layer. As well as wasting finances on an unnecessary layer, it would also reduce the efficiency of the filter. This is due to the run length before backwash is required will reduce as the filter will clog at a more common rate. With the larger particle on top, the material used must be less dense the second layer. This is because when the backwash system is activated it will force each granular material off the ground. If the top layer were to be heavier, it would sink faster and over time the layers would swap their starting positions. A common material to meet these demands is crushed anthracite, which is a carbon-based material. This makes for a very hard and strong material which will be able to withstand the eroding environment it is placed under. Anthracite thickness is designed at 0.9m. The second layer is a typical sand layer, which meets the requirements of being a smaller and heavier particle. The depth of the sand layer is to be 0.18m. With these materials in place, the minimum level of the water in the chamber can be calculated. This value is also referred to as the hydraulic head of the filter when the material is completely clean (free of any pollutants). This value is found to be 0.294m above the anthracite layer [9,10].

Beneath the layers of course material, lies what is known as a Filter underdrain system. The system works by the filtered water passing through the black plate of the filter underdrain segments knowns as the Integrated Media Support (IMS) cap. The water is then either pumped to the tertiary treatment process or stored for when the filter material needs cleaning by backwash [5,8]. The filter underdrain system generally uses a combination of water and air to back wash, however, to maintain simplicity and reduce the cost by not adding an air pump. The actual segments are designed themselves to cope with the environment they are being positioned into. This also will reduce constructions costs when the filter is built. It is also far more flexible for what surface it can be used on. Before the overall levelness of the base surface cannot be more than ± 3.2mm, where with this system allows for a ± 6.35mm. They are light weight therefore easy to handle, as they are made from high-density polyethylene which is very strong against physical forces and corrosion. It is also resistant to calcification, the build-up of calcium salts. The IMS cap previous mentioned also has a positive impact. Gravel was the original choice for supporting the filter media weight, now it is no longer required due to the cap. This reduces the cost of material, which needs to be replaced where the IMS cap is merely removed, cleaned and returned [12,25,34,36].

One of the systems biggest advantages is the large backwash flow distribution. In conventional methods, with the use of nozzles, they are spaced far apart which leads to some areas of the filter not being cleaned which leads to what is known as dead space. Typical filtration maintenance methods release the backwash water from points which are much closer together. Therefore, more of the filter is reached and cleaned. This also means less water is needed to clean the sand/anthracite, as well as doing it at a faster rate. All of these facts combined means longer run times and with that money saved through energy consumption reduction [23](See figure 1,2)

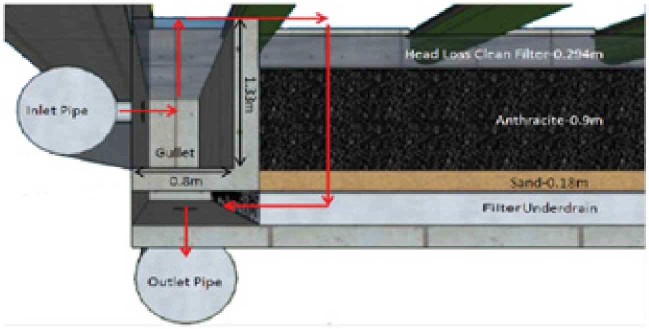


Fig. 1. Cross-section of rapid filter, red arrows represent flow direction

Rys. 1. Przekrój poprzeczny filtra pośpiesznego, czerwone strzałki wskazują kierunek przepływu

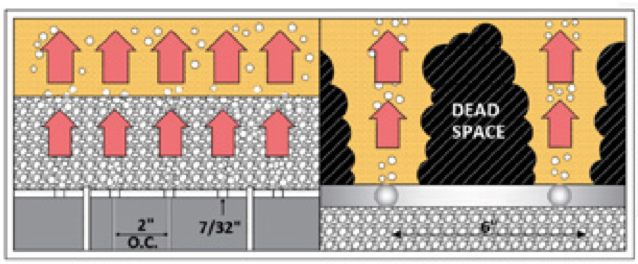


Fig. 2. Filter underdrain cleansing mechanisms and benefits

Rys. 2. Mechanizmy czyszczenia drenażu filtra i korzyści

The backwash velocity was found to be 47.63m/hr, which lead to the anthracite bed to expand by 35% and the sand by about 31%. Therefore, the total increase in the filters thickness is 0.37m, hence the wall between the filter and the gullet must be greater than this value or granular medium will be lost. 0.45m was chosen with the extra 0.08m acting as a safety factor [2,22].

The backwash water is stored is a single tank that is connected to all the filters to be able to clean any one of them when required. The minimum tank size was to be able to supply a single filter with backwater, which came to 397.314m^3 of water. This value lies within the required range of 2-5% of the total filtered water in a single run period. A single run period is the time between each filter is to be cleaned, which in this case is 24 hours. However, laser levels are located above the water surface in case the level rises faster than predicted and backwashing can be put into action earlier. To be sure there will be no delay, the tank was designed to hold enough water to clean 10 filters (3973.314m^3) at the same time in the off chance such a number requires clean at any given point. Therefore, the tank (circular) will have a diameter of 6.4m and stretch across the filters entire length. Backwash water is forced into the filters that require the cleaning, with the pressure coming through the use of gravity which requires minimal support from a pump. This is done by the tank being built higher than the maximum hydraulic head pressure that the any filter chamber will reach, 3.06m.

Once the backwash process begins, the level in the chamber will rise taking the trapped pollutants with it. To remove the now free contaminants, troughs are in place across the length of the chamber. The reason for having the troughs is so the pollutants that rise in the far corners of the chamber can be collected. The troughs dimensions and positioning can be seen in, of which all values have been calculated (Appendix 10). Note the barriers on both sides of every trough which prevent filtration materials from entering the trough and being taken away [4,18](See Figure 3,4,5).

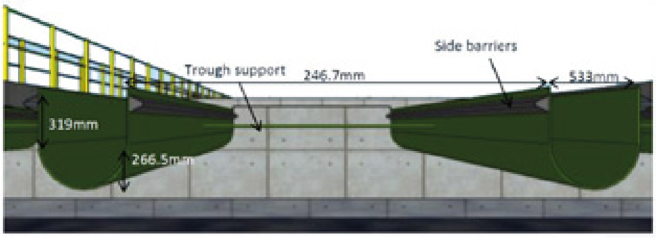


Fig. 3. Backwash troughs with dimensions

Rys. 3. Koryta do płukania wstecznego z wymiarami

The water will spill into the troughs taking the water to a parallel chamber which takes the discharge away for further treatment. Each chamber will take the overflow from a maximum of 2 filters.

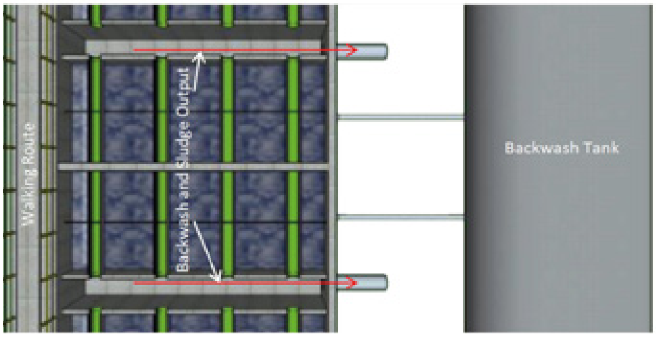


Fig. 4. Birds eye view of sludge outlet points

Rys. 4. Widok z lotu ptaka na punkty odpływu osadu

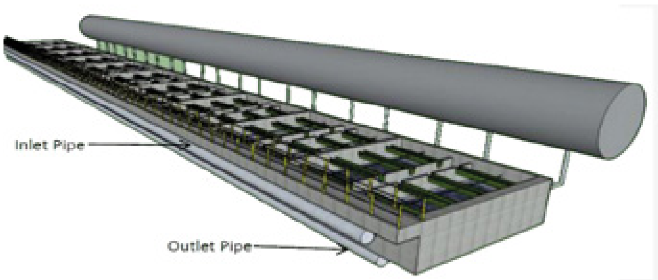


Fig. 5. All 20 filters, backwash water tank and inlet/outlet pipes

Rys. 5. Wszystkie 20 filtrów, zbiornik wody do płukania wstecznego oraz rury wlotowe/wylotowe

Tertiary Treatment

Sludge Management

The sludge, a concentrated mixture of solids and water, is collected from various stages of the treatment process, including the sedimentation tanks and the backwash water from the filters. Effective sludge treatment is crucial to recover valuable water and minimise environmental impact. The sludge is typically thickened and dewatered to reduce its volume and water content. The resulting dewatered sludge can then be disposed of in landfills or beneficially used in agriculture as a soil conditioner, leveraging its nutrient content [29,32].

Design Considerations: Natural Treatment Systems

To further enhance the treatment process and minimise environmental impact, a natural treatment system, specifically a reed bed, can be implemented [2,22]. Reed beds and constructed wetlands provide a sustainable and cost-effective solution for sludge treatment (See Fig 7). Reed beds harness the power of natural microorganisms and plants to purify the sludge. The use of reed wetland will be in place to clean the sludge before the treated water is sent back into the treatment facility, entering just before the sedimentation tank. The advantages to using wetlands are numerous and varied. It is highly effective in reducing the sludge volume, wet sludge reduced by 90% through drying and mineralisation which also removes organic matter by 25% and almost 100% after filtration through numerous layers. The wetland vegetation, particularly Bulrush (Scirpus spp.), is highly effective in removing a wide range of pollutants, including organic matter, nutrients (nitrogen and phosphorus), heavy metals, and pathogenic bacteria [6,13].

The use of vegetation to clean sludge means no chemicals, reducing cost and more importantly, protecting the environment. There is a massive cost saving on energy as the treatment is done my natural microorganisms with only small pumps need to transfer the treated water back to the facility. There are no odour concerns as mineralisation happens in aerobic way. Less CO_2 is produced when compared with other sludge cleaning techniques. There is a reduction in the cost of spreading and transporting for two reasons. The first reason is that the use of vegetation typically reduces the sludge volume by a third when compared with alternative mechanical methods. It also allows for sludge to reach a height of 1.2-1.5m which equates to around 10 years of treatment, meaning transport is needed after long time periods to remove the sludge. The vegetation which can be used is the Bulrush (Scirpusspp) as this plant can grow to 5ft tall, and is a very efficient remover of nutrients, oils, heavy metals (copper, nickel and zinc) and most importantly, hazardous bacteria such as *E-Coli* and Salmonella [19,24].

Disinfection and Chlorination

The chlorine is brought on site as a liquid gas, where it is stored in a metal container. The metal chamber is a suitable location as long as the Cl_2 remains dry. The container itself is to be stored inside as well. This is to protect the tank from the elements, particularly heat which

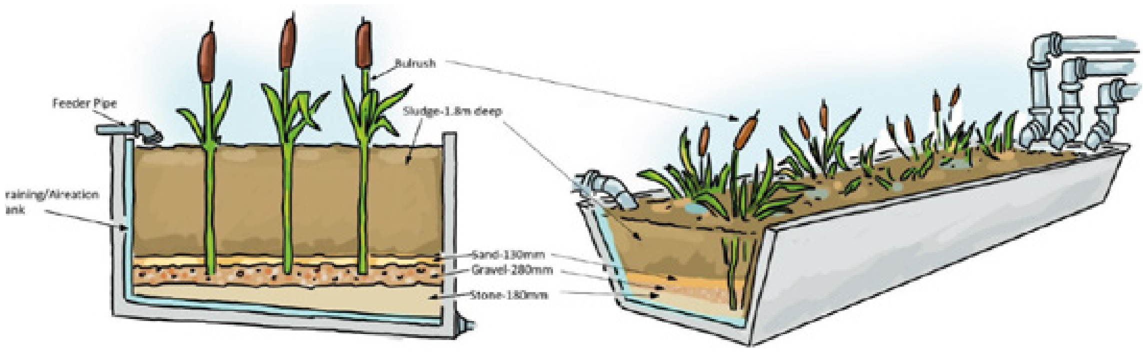


Fig. 6. *A Cross-sectional view of engineered/constructed wetlands (lefthand side) and water circularity entry points from wastewater and sludges entering constructed wetlands (righthand side)*

Rys. 6. Przekrój poprzeczny sztucznych/oczyszczalni hydrobotanicznych (po lewej stronie) oraz punkty wejścia obiegu wody ze ścieków i osadów wpływających do zbudowanych oczyszczalni hydrobotanicznych (po prawej stronie) could increase pressure and become a risk. Therefore, the stores must be detached from the rest of the plant for precautionary reasons. The store must have very efficient floor ventilation (60 air changes per hour); this is because Cl_2 is heavier than air. The evaporator converts the liquid Cl_2 into a gas. The chlorinator, where the Cl_2 gas is stored before being injected into the water. The effluent passes through the point where the flow is recorded. This data is passed on to the chlorinator which will control the required dosage of Cl_2 needed for the current flow rate. The injector takes the Cl_2 from the chlorinator and injects it into the water. The amount injected into the water is 5 $\frac{mg}{L}$, a value which meets the specified requirements of the WHO. This value is calculated by finding the chlorine demand of the water (how much Cl_2 is need for the number of toxins) and adding it to the residual quantity wanted. The Cl_2 residual quaintly is crucial for maintaining the waters cleanliness as it prevents pathogens from re-breeding and infiltrating the water once again; a level of 0.5mg/L is the target value (WHO). The injected chlorine reacts with the water and the following chemical equation takes place [3, 27]:

$$Cl_2 + H_2O \rightarrow HOCL + H^+ + Cl^- \tag{14}$$

Chlorine diffuser is where the initial mixing takes place. The water passes through a row of chambers before passing through a permeable membrane. Within this membrane is a single10mm hollow shaft of which the chlorine gas is injected out of by a series of holes. The current comes in contact with the Cl_2 before it then passes through rapid turbine which mixes the water and chemical far more thoroughly [1]. The chlorine contact basin is of a certain specified volume of which the flow is detained in the basin for a quantified time. This time will be on the 30minutes mark (WHO) and is the period required for the chemical to effectively break down microorganisms. In the design, 3 tanks will be in operation (another being cleaned) of which all must hold 972m³ of water. The Width/Depth ratio must be 1:1 and the Length/Width ratio must be greater than 40:1, to have the most impact. Therefore, the dimensions are 2m for depth and width, and the length is 243m. 243m is too long for a single straight line, therefore baffles will be employed. 7 baffles will be constructed which leaves a tank length of 34.725m, and an overall width of 15.20m which is far more practical in terms of land space needed. Due to the length being reduced and the need for corners, 1m diameters semi-circles have been placed at each turning point. The reason being is that they will help the flow around the sharp bends more efficiently. Liquid Sulphur Dioxide tank. Sulphur Dioxide's purpose is to control the amount of Cl_2 residual that exits the facility and distributed to the public. It is the most efficient substance for a facility this size, where sodium bisulphate is a harder compound to control in such magnitudes. The following chemical reaction takes place:

$$SO_2 + H_2O \rightarrow HSO_3{}^- + H^+ \tag{15}$$

$$HOCL + HSO_3{}^- \rightarrow {}^-Cl^- + SO_4{}^{2-} + 2H^+ \tag{16}$$

It is well understood residual Cl_2 is crucial to maintain treatment, however it should not exceed the 0.5 $\frac{mg}{L}$ limit or the Cl_2 can become toxic and have been known to be carcinogenic. Therefore, SO_2 controls the level that resides in the water, and requires about 1.0 $\frac{mg}{L}$~1.2 $\frac{mg}{L}$ of SO_2 (a reducing agent) to remove 1.0 $\frac{mg}{L}$ of chlorine. Another evaporator, turning the liquid compound into a gas as it is easier to inject. Sulphonator, carries out the same parameters as the chlorinator. Cl_2 residual analyser, which reads the amount of Cl_2 that is still in the water, if too higher (>0.5 $\frac{mg}{L}$) this information is fed to the Sulphonator, and it will automatically calculate how much SO_2 is need to reduce the residual to 0.5 $\frac{mg}{L}$. If the chlorine levels are too low, this data is feedback to the chlorinator which will recalibrate how much chlorine is injected into the water [20,21,31].

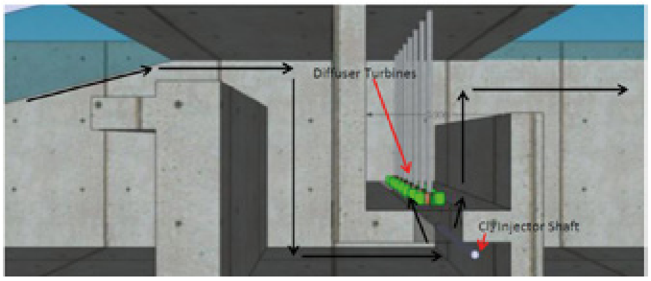


Fig. 7. Chlorine Diffuser [23]

Rys. 7. Dyfuzor chloru [23]

Fig. 8. Chlorination Tank and baffles

Rys. 8. Zbiornik chlorowania i przegrody

An injector, the same as the one used for the chlorine. Sulphur Dioxide diffuser is where the injected gas has an effect on the effluent water. Exactly the same mechanism can be in place that is used for the chlorine diffuser (**Fig.10**). Just after the water has been reduced, the residual level is re-analysed to confirm the correct amount remains. If not re-calibrations to the entire system can be made without the need to operatives who are expensive and slow [30,35].

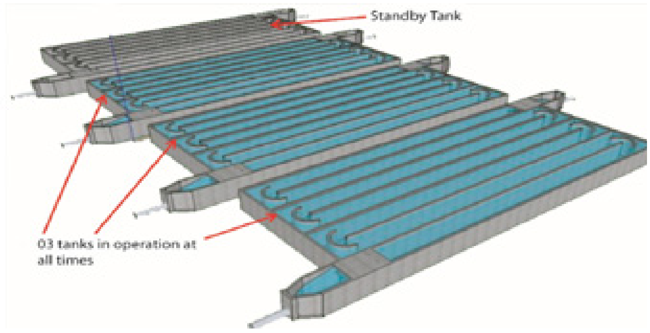


Fig. 9. All four (4) chlorination tanks for disinfection within the Water Treatment Plant (WTP)

Rys. 9. Wszystkie cztery (4) zbiorniki chlorowania w stacji uzdatniania wody (WTP)

Bacteria and other potentially harmful microorganisms may be present in the filtered water after it leaves the filter unit. In order to eradicate bacteria and other microorganisms and thereby avoid waterborne illnesses, disinfection is required. There are several techniques involved in disinfection. Chlorine is now especially frequently used to disinfect water. It is reasonably safe to use, dependable, and reasonably priced [7,17].

Water demand = 140,000 m^3/day
Required chlorine and residual chlorine are 0.36 and 0.2 mg/L, respectively (17)
Chlorine demand = 0.36 mg/L – 0.2 mg/L = 0.16 mg/L
Consumed chlorine = 0.36 mg/L × (1/ 10 6) × 1400,000 × 1000 = 47.5 kg/day
The time required to complete the disinfection performed in a storage tank is 0.5 h (18)

Q = Volume / time Volume = Q × time
Volume = 140,000 m^3/day × (1/24) × 0.5 h = 2916 m^3
Using effective depth of 4 m and length (L) = 2× width (W)
A = 1,250 m^3 / 4 m = 729.16 m^2
L × W = 729.16 m^2
W × W = 729.16 m^2
W^2 = 364.58 m2W = 19.09 m
and L = 2 × 19.05 = 38 m
Velocity = distance / time
Velocity = 38 m/0.5 h = 76 m/h = 0.0211 m/s (19)
Finding the Depth (H):
The volume of the tank is given by:
V=L×W×HV = L \times W \times HV=L×W×H
Substituting the values:
2916=38×19.09×H2916
H=4m

Storage and pumping

Water can be stored in storage tanks or delivered to customers via high lift pumps once the last phases of treatment are finished. After that, depending on the needs of the household, it can be used as drinking water. Figures 20 and 21 display the specifics of the pumping and storage tank.

Q avg. = 140,000 m^3/day = 5833 m^3/h = 1.62 m^3/s Using v = 1.5 m/s (20)
A= $\frac{1.62}{\frac{\pi}{4}0.75^2} = 3.5 < 2$ *then more than three pumps is needed*

Three pumps are used; the two first pumps is working, and the third one is on standby. The forth pump is used during maximum demand.

Each pump should be able to handle a portion of the peak hourly demand.

Storage tanks

The total flow rate is: Q_{avg}=1.62 m^3/s
Using the velocity formula: Q=A×v
where: v=1.5m/s and A is the required cross-sectional area of the pump pipe
A=Q/v=1.62/1.5=1.08m^2
The storage tank should be able to hold a portion of the daily demand, typically 25-50% of the average daily flow. Assuming 50% storage, the required volume is:
V_{tank}=0.5×Qavg=0.5×140,000=70,000 m^3
Since there are two (2) tanks, each tank will have a volume of:
Vtanks=70,000/2=35,000 m^3
A rectangular tank has a volume given by:
V=L×W×HV = L \times W \times HV=L×W×H

where:
- L = length (m)
- W = width (m)
- H = height (m)

Now, let's assume some practical dimensions while keeping the height reasonable (e.g., H = 6m for ease of maintenance and structural feasibility):
35,000=L×W×6
L×W=5,833.33 m^2L
Let's assume a width of 50 m, then:
L=5,833.33/50=116.67 mL =116.67 m
- Length: 117 m (approx.)
- Width: 50 m
- Height: 6 m

The proposed WTP for Small Island developing states should include primary, secondary, and tertiary treatment processes and parallel treatment trains as shown in **Fig. 10**.

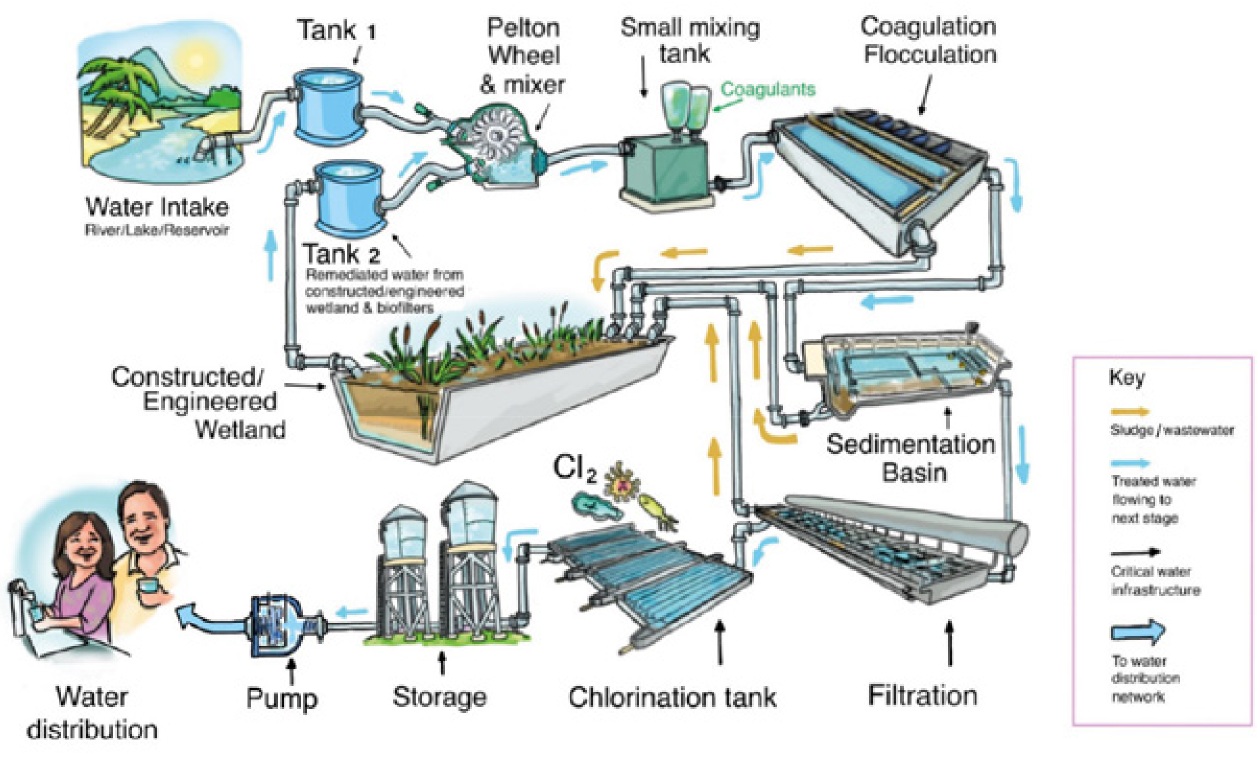


Fig.10: Schematic of Urban Water Treatment Plant for Small Island Developing States.
Rys. 10: Schemat miejskiej stacji uzdatniania wody dla małych rozwijających się państw wyspiarskich.

Table 2: Summary of the WTP design outline and adapted from[14]

Tabela 2: Podsumowanie projektowania WTP na podstawie [14]

No. Unit	Description	No. of units	Shape/ configuration of units	Dimensions of units
1	Intake	2	circular	Diameter=7m Depth=10m
2	Coagulation	2	circular	Area=314.16 m^2 Depth= 19m
3	Flocculation	1	Rectangular	Width=10m Length=10m Depth=5m
4	Clarification	1	Rectangular	Width=15m Length=97.2m Depth=4m
5	Filtration	17 nos. filter	Rectangular	Width=5m Length=11m Depth=1.5m
6	Disinfection	3	Rectangular	Width=14m Length=38m Depth= 2
7	Storage tanks &pumping	Tank=2	Rectangular	Length: 117 m (approx.) Width: 50 m Height: 4 m
		Pumps=4	circular	Q=5833m^3/hr V=1.62m/s D_{pipe}=0.75m

CONCLUSION

Before embarking on the feasibility study, design, and construction of a water treatment plant in the Caribbean, several crucial prerequisites must be addressed. Firstly, potential water sources need to be identified, whether it be surface water like rivers and lakes, groundwater from aquifers, or seawater. Secondly, a thorough understanding of sustainable abstraction rates is vital to avoid depleting precious water resources. This involves careful assessment of river flow, groundwater levels, salinity variations, and seasonal fluctuations, particularly in vulnerable locations like Jamaica and the Bahamas. Finally, a comprehensive grasp of the hydrological system is essential, including baseflow dynamics and the impact of rainfall and stormwater runoff on both the quality and availability of water resources.

The Caribbean region faces numerous challenges that can complicate the development and operation of water treatment plants. Political influence often interferes with the water sector, hindering effective planning and management. Unsustainable development practices, such as unplanned housing, deforestation, and improper agriculture, contribute significantly to pollution. Mining activities, both legal and illegal, can further degrade water quality. Additionally, malfunctioning wastewater treatment plants in countries like Aruba and Trinidad and Tobago discharge untreated effluent into natural water systems, causing long-term pollution and contamination.

Engineering firms and contractors working in the Caribbean must consider these unique challenges alongside the typical factors involved in water treatment plant development. High turbidity events, particularly in Trinidad and Tobago, can overwhelm conventional treatment processes. The variability of power supply across different islands poses challenges for reliable plant operation. Limited land availability in Small Island Developing States (SIDS) restricts the development of large-scale facilities. Finally, the heavy reliance on North America and Europe for equipment and services can create vulnerabilities in the supply chain. Careful consideration of these factors is essential for designing and constructing sustainable water treatment works in the Caribbean. A water treatment plant capable of supplying with 140,000 m3 per day of water per day has been designed as a case-study for Caribbean treatment works. This facility is tailored to effectively remove specific pollutants present in the raw surface water source. By adhering to stringent global water quality standards, particularly those set by the [relevant regulatory authority, e.g., US Environment Agency, the plant ensures the delivery of safe drinking water. The design incorporates state-of-the-art technologies to optimise treatment efficiency and minimise environmental impact. In today's world, engineers face the dual challenge of maintaining high-quality water supply while reducing the carbon footprint of water treatment processes. This plant is a step towards achieving this goal, as it is designed to operate with minimal energy consumption and reduced chemical usage. Once the water has been purified, it will be distributed to the surrounding communities, fulfilling the plant's primary objective of providing a reliable and sustainable water supply.

Appendix

Appendix 1 – Table 3 showing Typical Design Criteria for Horizontal-Shaft Paddles and Vertical-Shaft Turbines [16,26]

Załącznik 1 – Tabela 3 przedstawia typowe kryteria projektowe dla łopatek o osi poziomej i turbin o osi pionowej [16,26]

Design Parameter	Units	Horizontal Shaft with Paddles	Vertical-Shaft Turbines
Velocity Gradient, G	s^{-1}	20-50	10-80
Tip speed, Maximum	m/s	1	2-3
Rotational Speed	rev/min	1-5	10-30
Compartment Dimensions (Plan) Width Length	m m	3-6 3-6	6-30 3-5
Number of Compartments	No.	2-6	4-6
Variable-speed Drives	-S	AC Drives (Variable Frequency Drives – VFDs): Power Rating: Typically, in the range of 10 to 1000 kW, depending on the size of the paddlewheel. Voltage: Common voltages include 208V, 480V, and 600V. Current: Varies depending on the power rating and voltage. Frequency: Typically, 0 Hz to 60 Hz or 50 Hz. Speed Range: Often designed for a wide speed range, typically 10% to 100% of base speed. Efficiency: Typically, above 95%.	Doubly-Fed Induction Generators (DFIGs): Power Rating: Typically, in the range of 100 kW to several MW. Voltage: Medium voltage (e.g., 6.6 kV, 11 kV). Current: Varies depending on power rating and voltage. Frequency: Matches grid frequency (50 Hz or 60 Hz). Speed Range: Can operate over a wide range of speeds, typically ±30% of synchronous speed. Torque: High torque at low speeds, suitable for variable wind conditions. Efficiency: Typically, above 95%.

Appendix 2 – Table 4 showing the Design Criteria for Paddle wheel Flocculator [16]

Załącznik 2 – Tabela 4 przedstawia kryteria projektowe dla flokulatora z kołem łopatkowym [16]

Parameter	Unit	Value
Diameter of wheel	m	3-4
Paddle Board Section	mm	100 x 150
Paddle Board Length	m	2-3.5
A_{paddle} boards / Tank Section Area	%	< 20
$C_d = 1.16$	L/W=1 L/W=5 L/W=20 L/W >>20	$C_d = 1.16$ $C_d = 1.20$ $C_d = 1.50$ $C_d = 1.90$
Paddle tip speed	m/s m/s	Strong Floc, 4 Weak Floc, 2
Spacing between paddle wheels on same shaft	m	1
Clearance from basin walls	m	0.7
Minimum basin depth	m	1m greater than diameter of paddle wheel
Minimum clearance between stages	m	1

Appendix 3 – Table 5 showing Settling Velocities of Selected Floc Types

Załącznik 3 – Tabela 5 przedstawia prędkości osadzania wybranych typów kłaczków

Floc Type	***Settling Velocity at 15°C***	
	metre/hour	***Feet/minute***
Small Fragile Alum floc	2-4.5	0.12-0.24
Medium-sized Alum floc	3-5	0.18-0.28
Large Alum Floc	4.0-5.5	0.22-0.30
Heavy Lime Floc (Lime Softening)	4.5-6.5	0.25-0.35
Iron hydroxide Floc (Fe^{3+})	2-4	0.12-0.22
Polyaluminum Chloride (PACl) floc	2-4	0.12-0.22

Appendix 4 – Table 6 showing Typical Design criteria for horizontal-flow rectangular tanks [REF 32]

Załącznik 4 – Tabela 6 przedstawia typowe kryteria projektowe dla zbiorników prostokątnych z przepływem poziomym [REF 32]

Parameter	Units	Value
Type	-	***Horizontal-Flow Rectangular Tank***
Minimum number of tanks	Unitless	2
Water Depth	m (ft)	3-5 (10-16)
Length-to-Depth ratio, Minimum	Dimensionless	15:1
Width-to-Depth Ratio	Dimensionless	3:1-6:1
Length-to-Width Ration, minimum	Dimensionless	4:1-5:1
Surface Loading Rate (overflow rate)	m/h (gpm/ft^2)	1.25-2.5 (0.5-1.0)
Horizontal mean-flow velocity (at maximum daily flow)	***m/min (ft/min)***	0.3-1.1 (1-3.5)
Detention Time	h	1.5-4
Launder weir loading	m^3/m.h (gpm/ft)	9-13 (12-18)*
Reynolds number	Dimensionless	>10^{-5}
Bottom slope for manual sludge removal systems	m/m	1:300
Bottom slope for mechanical sludge scrapper equipment	m/m	1:600
Sludge collector speed for collection path	m/min(ft/min)	0.3-0.9 (1-3)
Sludge collector speed for the return path	m/min(ft/min)	1.5-3 (5-10)

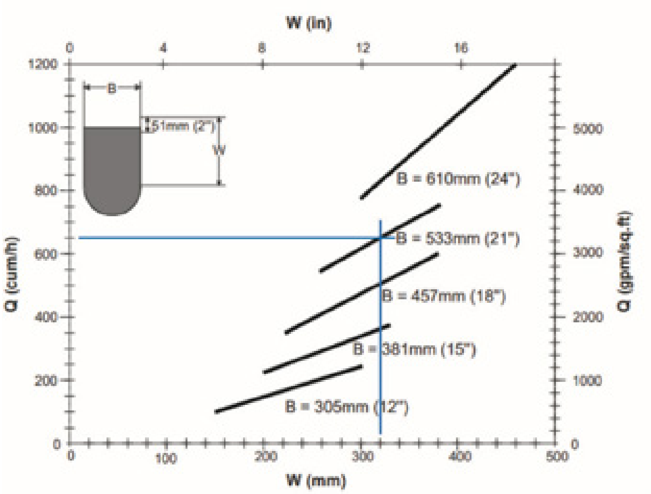
Appendix 5 – Table 7 showing head loss characteristics of conventional water filtration media (Anthracite and Sand)

Załącznik 5 – Tabela 7 przedstawia charakterystykę strat ciśnienia w konwencjonalnych złożach filtracyjnych (antracyt i piasek)

Head Loss in filter	*Anthracite*	*Sand*
Depth, m	0.9	0.18
Dimensionless coefficient, k	6	5
Dynamic viscosity, μ (kg/ms)	0.001139	0.001139
Porosity ε (%)	0.495	0.415
Gravity, g (m/s^2)	9.81	9.81
Density ρ (kg/m^3)	1400 – 1690	1400-1500
Shape Factor Ø	0.6	0.7
Particle Diameter, d (m)	0.0014	0.0006
Filter Velocity v (m/s)	0.0017361	0.001736
h_L(m)	0.1167	0.1784
h_L Total (m)	0.2943	

Appendix 6 – Graphical interpolation with flowrates and widths

Załącznik 6 – Interpolacja graficzna z natężeniami przepływu i szerokościami



Appendix 7 – Table 8 showing the Design criteria for filters at typical Water Treatment Plant [Ref 28]

Załącznik 7 – Tabela 8 przedstawia kryteria projektowe dla filtrów w typowej stacji uzdatniania wody [Ref 28]

Parameter		Value
Filter Type	-	Conventional, Dual-Mode
Flow Control		Influent weir split, constant level
Number	-	12
Interior Dimensions	m.m	4.3 x 11.6 x 2 cells
Media surface area (total)	m^2	1200
Maximum available head	m	2.5
Filtration rate (at water treatment plant design flowrate) One Filter Offline All Filters in service	 m/h m/h	 15 13.75
Filter media		
Top layer		
Type	----	Anthracite
Depth	m	1.5
Effective size	mm	1.0
Uniformity coefficient	----	<1.4
Specific gravity	----	1.5~1.6
Bottom layer		
Type	----	Sand
Depth	m	0.3
Effective size	mm	0.5

Uniformity coefficient	----	<1.4
Specific gravity	----	2.65
Backwash criteria		
Maximum rate	m/h	56
Normal rate	m/h	45
Duration	min	15

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