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Determination of Discrete Particles Optimum Design Parameters for Surface Irrigation System Settling Basin

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Abstract Irrigated agriculture is faced with challenges that include sediment loading in the river basins and dams. The management of sediments in river basins and waterways has been an important issue for water managers throughout history. Water managers are faced with similar challenges caused by siltation of water reservoirs and irrigation water conveyance systems. As a copping strategy to counter the low irrigation application efficiency for surface irrigation systems, designs of settling tanks are typically oversized with an aim of having enough detention time for the sediment particles to settle. To settle discrete particles an optimum settling tank is important so as not to have problems over overdesigning and consequently costly projects. The optimum hydraulic design parameters for a settling basin were calibrated using a physical model prepared in the Civil Engineering laboratory at Jomo Kenyatta University of Agriculture and Technology. Using a dataset from a model a quartic equation $T = -1.5844x^4 + 42.341x^3 - 385.56x^2 + 1350.8x - 1188.8$ was developed to calculate turbidity drop in a small-scale settling basin when the flow rate is known**.** Finally, a quadratic equation $Y = 0.0906X^2 - 1.0975X + 5.7338$ was developed for calculating optimum surface area required for settling discrete particles for different flow rates. For this research the optimum design surface areas were Q_1 = 2.42m^2, Q_2 = 3.04 m^2, Q_3 = 3.75 m^2, Q_4 = 4.20 m^2 and Q_5 = 4.71 m^2 corresponding to 5.7, 8.7l/min, 9.9 l/min, 10.5 l/min and 11.1 l/min respectively.

Keywords Critical velocity, Discrete particle, Optimum, Settling basin, Turbidity

1. Introduction

Water quality control for irrigated agriculture requires that the sediments be controlled at their entrance point to the networks. This can be achieved by constructing a settling basin (SB) where better retardation of the sediments is realized by constructing large basins and cost not withstanding [1]. The objective of Sedimentation basins, which are hydraulic structures, designed and constructed where mostly there is water abstraction, is to remove most of suspended sediments that enters the intake by flowing water [2].

In Africa up to 85% of available water is used for agriculture [3]. The average rate of irrigation development in Sub-Saharan Africa countries between 1988 and 2000 was 43,600 ha/year [4]. The Food and Agricultural Organization estimates that irrigated land in developing countries will increase by 27% in the next 20 years, but the corresponding quantity of water expected to be available for agricultural production will only increase by a mere 12 % [5]. Going by these projections, an additional one million hectares of land will be under irrigation by 2025 [6]. However, irrigated agriculture is faced with challenges such as sediment loading in the river basins and dams.

For instance, Orange River in South Africa is rated among the most turbid rivers in the world due to sedimentation [7]. It is approximated that 1% of the world's water storage capacity in reservoirs is lost every year through sediment deposition. Actually, the life of reservoirs is usually reduced by the accumulation of sediment behind the upstream reservoir wall [8].

The management of sediments in river basins and waterways has been an important issue for water managers throughout the history as from the ancient Egyptians managing sediment on floodplains [9]. Currently, water managers are faced with similar challenges mainly resulting in siltation of water reservoirs [10]. For instance, the large Borken dam in South Wello in Ethiopia was constructed with multi-million dollars and silted within just one rainy season [11]. This was as a result of underestimation of the potential runoff and sedimentation during design stage [11]. In another case, [7] identified that Caledon River in South Africa carries the largest fine-mud suspended load, primarily from the erosion of Karoo sedimentary rock soils.

In Kenya, a research by [12] revealed that Gem-Rae irrigation scheme in Kisumu commissioned in 1985 had to be abandoned after 12 years of operation due to clogging of intake and main canals. In 1997 approximately 200m^3 of sediment were removed weekly from the intake [12]. This is a clear indication that sediment management is important if irrigation water distribution efficiencies are to be improved.

According to a study conducted in Murang'a County by [13] on smallholder irrigation projects, sprinkler clogging caused by sediments resulted in low coefficients of uniformity in water application. The issue of sediments entering irrigation pipelines and power canals has been a major problem confronting hydraulic engineers [14]. Therefore, sediment settling basins have been designed and constructed at river water intakes. The sedimentation tanks are designed to remove most of suspended sediments, which enters into the intakes by flowing water [2].

In Kenya, smallholder irrigation development is one of the key strategies for land use intensification with expected positive effects on rural incomes and poverty alleviation. About 20% (106,600 ha) of the potential irrigable land is already under irrigation where 50% (53,300) of this area is under smallholder irrigation [4], [13], [15]. Table 1 shows how the land under irrigation is utilized in Kenya per basin [16].

Table 1. Irrigation potential and Development by basins in Kenya

Basin	**Total Potential for Irrigation (Hectares)**	**Developed Area (Hectares)**	**Balance**
Tana	226,224	64,425	161,799
Athi	91,006	44,898	46,108
Lake Victoria basin	297,213	15,094	282,119
Kerio valley	101,753	9,587	92,166
Ewaso Ngiro	49,379	7,896	41,483
Total	**765,379**	**141,900**	**623,675**

Source: Leonard *et al.*, 2016

2. Sediment Settling Basins

Irrigation schemes are usually designed and implemented based on assumed efficiencies in water conveyance, water distribution and water applications. However, over time, physical changes in the irrigation schemes that include poor maintenance of structures, soil property changes and deterioration of equipment may alter these efficiencies [13].

The simplest method of sedimentation is to use rectangular tanks with horizontal flow through them.

The water with the particles in suspension is introduced at one end of the tank, then as the water flows to the other end of the tank settlement of particles in the water occurs. The aim of constructing a sedimentation tank is to allow a large proportion of the settling particles to reach the tank floor before the water is drawn at the outlet end of the tank [17]. Such horizontal flow tanks are usually built with a floor that slopes gently down to the inlet end to a hopper. The tank is fitted with a mechanism to scrape the sediment from the outlet end back to the inlet end and into the hopper from where it can be discharged hydraulically. In the design of such tanks detailed attention has to be given to the inlet and outlet ends to allow water flow from one end to the outlet end as uniformly as possible [14].

Settling basins are designed to retain water for a specified period of time. However, most settling basins built using these principles are oversized and and some not very effective because the critical design criteria are not considered or are inadequate [1]. While analyzing sediment settling and basin efficiency [18] was limited to only rectangular basins, with assumed uniform conditions across the full width of the basin. However, this condition is not always met as in practice, flow expansion and contraction happen in basin inlet and outlet zones.

The trapping efficiency of a settling basin is mainly governed by the geometry, general size and shape as the main dominating parameters [19]. Larger settling basins facilitate exclusion of more suspended sediment while the shape of a basin is very important for producing an even flow distribution in the basin [10], [2]. Even flow distribution is the key to maintaining optimum trapping efficiency and reduced turbulence. It is very important to have a properly designed inlet and outlet geometry to ensure that even flow distribution over the depth and width of settling basin exists [20].

A further consideration in the design of a sedimentation basin is the provision of adequate storage for settled sediment to prevent the need for frequent de-silting [10].

2.1. Principle of Sediment Particle Settling

In the settling principle, particle settling is achieved by detaining water long enough for the suspended sediment to settle from the water under the influence of gravity before the water is discharged to uncontrolled environment [21]-[22].

Settling of particles depends on two major factors namely: characteristics of the particle (Discrete particles and Flocculating particles) and concentration of the particles in suspension (Dilute or Concentrated) [23]. This is shown in Fig 1.

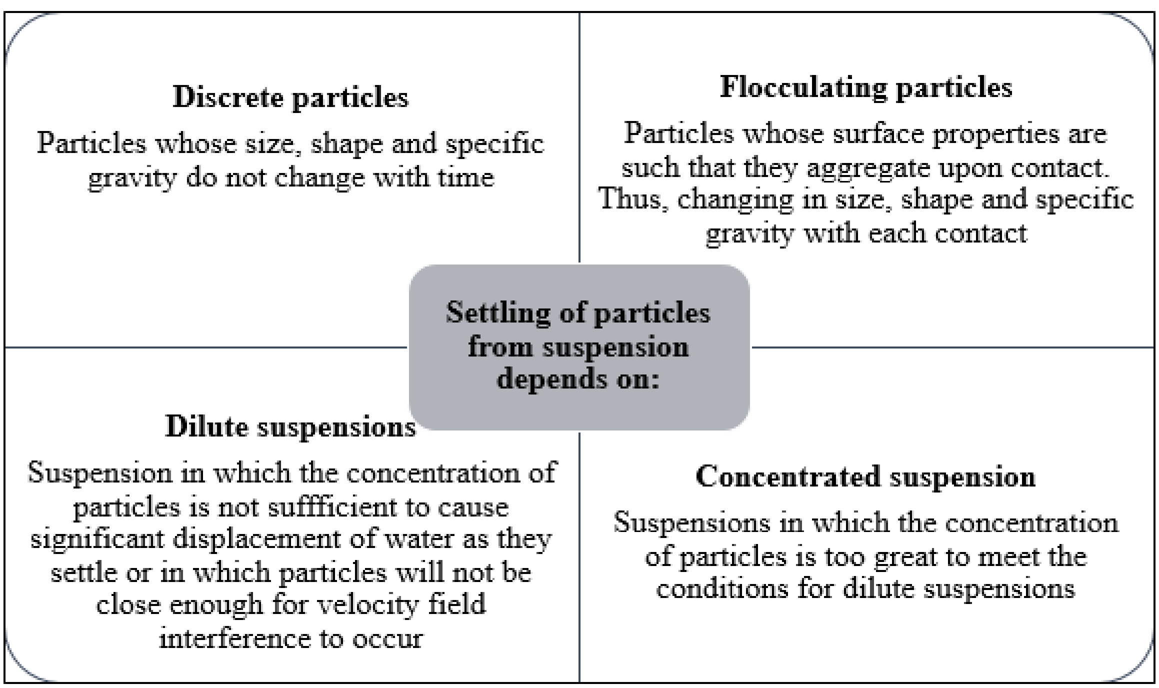

Fig 1. Factors affecting settling of particles

Suspended solids present in water having specific gravity greater than that of water tend to settle down by gravity as soon as the turbulence is retarded by offering storage. If a particle is suspended in water, it initially has two forces acting upon it [24]. The Equation representing the forces is presented in 1 and 2.
The forces of gravity, as shown in Equation 1

$$F_G = \rho_p g v_p \tag{1}$$

The buoyant force quantified by Archimedes as shown in Equation 2

$$F_B = \rho_w g v_p \tag{2}$$

Where:
F_G = Gravitation force (N)
F_B = Buoyant force (N)
ρ_p = Density of particle (kg/m^3)
V_p = Volume of particle (m^3)
g = Acceleration due to gravity (m/s^2)

If the density of the particle differs from that of the water, a net force is exerted and the particle is accelerated in the direction of the force. This net force becomes the driving force as shown in Equation 3 [25].

$$F_{net} = (\rho_p - \rho_w) g V_p \tag{3}$$

Where:
F_{net} = Net force (N)
ρ_p =density of particle (kg/m^3)
ρ_w =density of water (kg/m^3)
V_p = volume of particle (m^3)
g = Acceleration due to gravity (m/s^2)

Once the motion is initiated, a third force is created due to viscous friction. This force is called the drag force and quantified by Equation 4 [26]

$$F_d = C_D A_p \rho_w \frac{V_s^2}{2} \tag{4}$$

Where,
C_D = drag coefficient
A_p = Cross-sectional area of particle perpendicular to the direction of movement (m^2)
V_s = Settling velocity of the particle (m/s)

Since the drag force acts in the opposite direction to the driving force and increases as the square of the velocity, acceleration occurs at a decreasing rate until a steady velocity is reached at a point where the drag force equals the driving force [27].
Equation 5 gives the force balance for a discrete particle that is settling.

$$M_p \frac{dv_s}{dt} = F_G - F_B - F_D \tag{5}$$

After an initial transient period the acceleration $\frac{dv_s}{dt}$ reduces to zero and the settling velocity becomes constant, according to Equation 6 and 7 [28].

$$M_p \frac{dv_s}{dt} = 0 = F_G - F_B - F_D \tag{6}$$

$$0 = (\rho_p g V_p) - (\rho_w g V_p) - \left(C_D A_p \rho_w \frac{V_s^2}{2} \right) \tag{7}$$

Substituting Equation 7 reduces to Equation 8.

$$V_s = \sqrt{\frac{2g(\rho_p - \rho_w)}{C_D \rho_w A_p}} \tag{8}$$

Equation 2.8 is the settling velocity equation of discrete particle in any shape while the settling velocity for spherical particle as shown in Equation 9.

$$V_s = \sqrt{\frac{4(\rho_p - \rho_w) g d}{3 C_D \rho_w}} \tag{9}$$

The (C_D) used in Equation 9 is Newton's drag coefficient is a function of; flow regime around the particle (Flow rate) and particle shape.
For Spheres (C_D) is given in Equation 10.

$$C_D = \frac{24}{\mathrm{Re}} + \frac{3}{\sqrt{\mathrm{Re}}} + 0.34 \tag{10}$$

Where Re is Reynold's number, which is given by Equation 11 as:

$$\mathrm{Re} = \frac{V_s d}{V} = \frac{V_s d \rho}{\mu} \tag{11}$$

Hence in turbulent flow like in this case:

$Re \triangleright 10^4$
C_D=0.34-0.4

Substituting Equation 11 in 9, we get Equation 12, which is the equation for settling velocity of spherical discrete particles under turbulent flow conditions.
Equation 12.

$$V_s = \sqrt{\frac{10g(\rho_p - \rho_w)d}{3\rho_w}} \qquad (12)$$

Equation 12 is the settling velocity of spherical discrete particles under turbulent flow conditions.

2.2. Design Procedures for Settling Tanks

The design of current settling tank system is based on the principles of gravity settling. Gravity settling occurs in tanks of water with large cross-sectional areas where small influent and outward flows create a state of virtual quiescence in the system [29]. Discrete settling occurs in the systems with small particle concentrations where particle aggregation is negligible and settling occurs by natural forces. In discrete settling, the terminal velocity or settling rate of the particles are calculated using Stoke's law which assumes the rate depends only on the size of the particle, shape and density of the particle, viscosity and density of the fluid [24]. Equation 13 shows the Stoke's law.

$$U_t = S_o = \frac{d^2 \times g \times (\rho - \rho_f)}{18 \times \mu} \qquad (13)$$

Where:

d	= Particle diameter (m)
g	= Gravitational acceleration (m^2/s)
μ	= Fluid viscosity (Ns/m^2)
ρ	= Particle density (kg/m^3)
ρ_f	= Fluid density (kg/m^3)
U_t	= Terminal velocity (m/s)
S_o	= Stoke's settling rate of particle (m/s)

The overflow parameter is the crucial parameter in the design of the sedimentation tank and is generally chosen to be half of the value of the stoke's settling rate [20]. For a fixed influent rate, adequate particle removal only depends on the surface area of the tank as shown in Fig 2 [20].

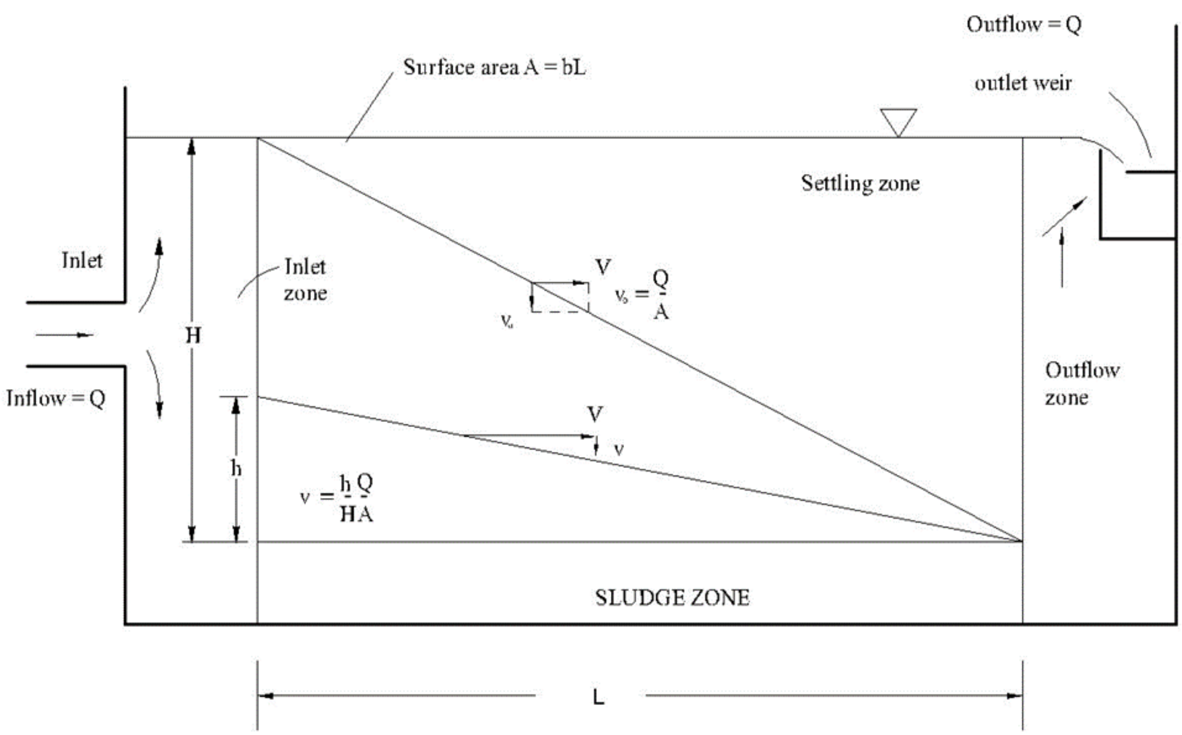

Fig 2. An ideal rectangular settling tank illustrating the settling of discrete particles

It is apparent that a particle will be settled at the sludge zone only if its settling velocity exceeds the water up-flow velocity. In this case the minimum up-flow is given by the flow rate divided by the surface area of the tank- Q/A, [30].

Where:

Q = Flow rate (m^3/s)

A = Surface area of the tank (m^2), given as WL (W= tank width, L= Tank Length)

From the geometry of the tank the time required for the particle to settle, t_o is given by;

$$t_o = \frac{H}{V_p} = \frac{L}{V_h} \tag{14}$$

Where:

t_o = Settling time (s)

H = Depth of the tank (m)

L = Length of the tank (m)

V_h = Horizontal particle velocity (m/s)

V_p = Terminal settling velocity in the vertical direction (m/s)

Since Vh = Q/WH, then Vp = Q/WL, rearranging and noting that surface area A = WL, then the terminal settling velocity will be given by.

$$V_p = \frac{Q}{A} \tag{15}$$

The design of settling tanks for a given flow rate (Q), involves the selection of the surface loading rate (Q/A), from which the required tank surface area may be calculated, and either tank depth (H) or detention time (t). The task of proportioning the tank once major parameters are chosen is based on the simple design charts based on the previous equations [31], [20].

This relationship shows that settling efficiency is independent of tank depth. This condition is only true if the forward velocity is low enough to ensure that the settled sediments is not scoured and re-suspended from the tank floor or the sludge zone [30]. This study was determine the critical settling velocities of discrete particles for various flow rates in a surface irrigation system. This would be used to design an optimum settling basin, which will eventually increase the overall efficiency of irrigation systems. Consequently, there will be improvement in yields from crop production because of increased available irrigation water leading to improved food security.

3. Materials and Methods

3.1. The Experimental Set up

A physical model of settling basin with a header tank was fabricated and used to run the various experiments (Fig 3). The study using the model was carried out in the Civil Engineering Laboratory at Jomo Kenyatta University of Agriculture and Technology (JKUAT).

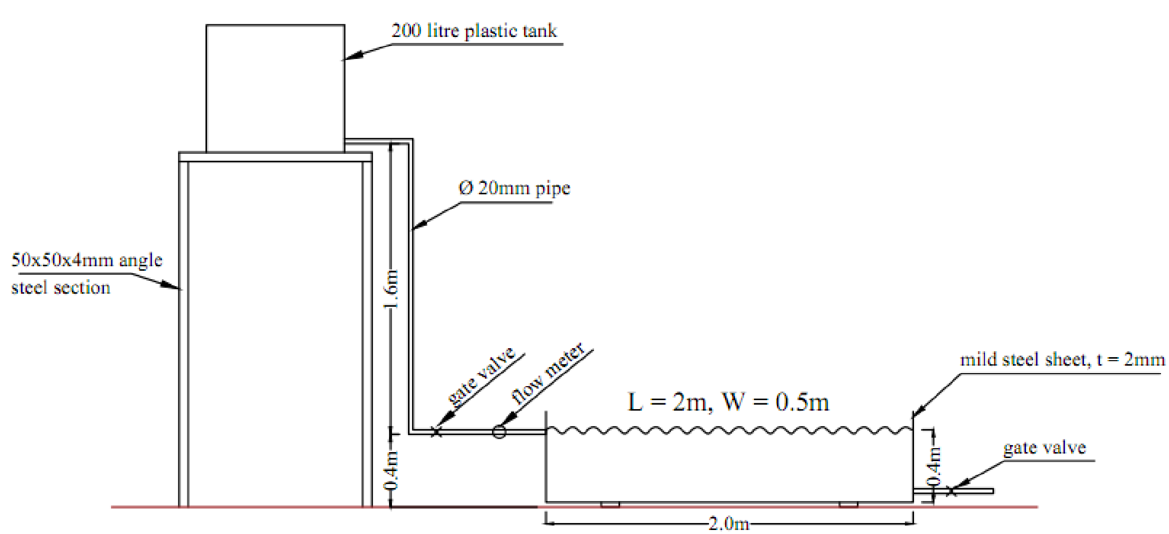

Fig 3. Schematic diagram for the experimental set up

Using settling velocity for a sand particle formula in Equation 15 [24] and a tank sizing ratio of 4:1 for linear measurements, the dimensions for fabrication of the model were taken as length of 2 m, breadth of 0.5 m and a height of 0.4 m. The inflow pipe used was of diameter 50 mm connected to a header tank at a height of 1.6 m with a measuring flow meter next to the control valve. In this height there was considerable minimum head for water to flow in the model settling tank by gravity.

The sand particles for preparing turbid water were passed through sieve no.100 with sieving mesh of 0.20 mm in diameter in order to achieve the discrete sand particles for the experiment. In practice the smallest particle expected to be settled in a settling basin is of diameter 0.2 mm for irrigation purposes [31]. However, particles less than 0.2 mm in size coagulate and settle as mass and others fall independently within the tank.

The turbidity of natural raw water varies from 10 to 500 NTU [32]. Hence, the experiment was carried out by making synthetic turbid water. About 350 grams of graded sand was added to 200 liters of clean water in the header tank. The suspension was stirred for one hour to achieve uniform homogeneous sample [33]. The turbidity of this homogenous solution was recorded as 465 NTU and then let in to the model settling basin at a pre-determined flow rate. The preparation of turbid water was repeated for 463, 424, 395 and 196 NTU water samples.

Different flow rates of 4.8 l/min, 5.7l/min, 8.7l/min, 9.9 l/min, 10.5 l/min, and 11.1 l/min were set using a gate valve and water released from the header tank into the model-settling basin. Turbidity levels were then monitored in the model tank and recorded for every 20 minutes over a period of 300 minutes for each flow rate. A time range of 20 minutes corresponded to a notable turbidity drop on the settled sediments. The flow rates were determined and regulated by the gate valve in order to realize the expected openings corresponding to a given flow. The gate valve was opened half-way per test progressively to achieve the flow rates.

The turbid water in the header tank was stirred after every 5 minutes to maintain uniformity and to ensure that the particles do not settle in the tank. This process was repeated for lower turbidities ranging from 180 to 250 NTU for flow rates of 8.7 l/min, 9.9 l/min, 10.5 l/min and 11.1 l/min respectively. The settling velocity of the particles was then computed using Equation 12.

4. Results and Discussion

4.1. Critical Settling Velocity

Table 2 presents a summary of the values of flow rates (Q_1, Q_2, Q_3, Q_4 and Q_5) measured at 4.8 l/min, 9.9 l/min, 10.5 l/min, 11.1 l/min and again 9.9 l/min with low turbidity value respectively. Retention time as particles were settling is represented by T_1, T_2 up to T_{16} where T_0 represents turbidity at zero (0) minutes and time increases at constant interval of 20 minutes.

During the experiment, it was noted that after five (5) hours the turbidity dropped significantly to levels acceptable for irrigation purposes, hence further collection of data was terminated. Monitoring of turbidity is important since currently no uniform guidelines that exist for acceptable turbidity values for irrigation water.

Table 2. Turbidity values at different time duration for different flow rates

	Turbidity Values (NTU)								
Flow Rate	T1(0)	T2	T4	T6	T8	T10	T12	T14	T16(300)
Q1	424.33	314.00	317.67	333.67	325.67	307.33	304.33	291.67	250.62
Q2	465.33	300.33	277.67	285.00	269.67	222.67	209.00	209.67	200.00
Q3	463.00	367.67	377.67	397.67	357.67	269.67	260.33	241.00	242.00
Q4	395.00	365.67	343.67	260.67	267.67	267.00	256.00	244.67	233.67
Q5	196.00	134.33	116.67	105.33	103.00	92.33	95.00	83.00	81.67

Table 3 shows the turbidity values for five different flow rates at time intervals of forty minutes. The turbidity drop indicates that as the discharge into the settling basin increases, the sediment settling increases leading to less sediment concentration in the water.

Table 3. Flow rate against sediment concentration

	TIME IN MINUTES								
FLOW RATE (l/min)	**0**	**40**	**80**	**120**	**160**	**200**	**240**	**280**	Turbidity drop
5.7	424.33	324.00	317.33	325.33	305.33	295.67	284.00	271.48	152.85
8.7	425.00	332.00	382.67	372.00	311.00	275.33	269.33	240.00	185.00
9.9	465.33	304.00	245.33	287.33	297.33	236.67	198.67	206.00	259.33
10.5	463.00	334.67	358.33	396.00	227.67	257.00	260.00	219.67	243.33
11.1	395.00	354.67	299.33	274.67	269.33	251.00	256.33	240.00	155.00

From design principles, settling velocity (Vs) is computed from the flowrate (Q) and the effective settling area of the tank (A). Equation 9 and 15 are applied to calculate the optimum dimensions for a settling tank.

The critical settling velocity was calculated and given in Table 4.

Table 4. Critical settling velocity for different flow rates

Flow Rate (l/min)	Particle settling velocity (m/s) Vp	Critical settling velocity (m/s) Vc	Removal Efficiency (%)
5.7	3.8 x 10-5	3.4 x 10-3	40
8.7	7.0 x 10-5	4.4 x 10-3	48
9.9	8.6 x 10-5	2.4 x 10-3	52
10.5	8.4 x 10-5	2.6 x 10-3	48
11.1	1.07 x 10-4	3.4 x 10-3	58

The critical settling velocity is the settling velocity of the particles that removes maximum amount of particles from the basin. As seen from Table 4 the critical settling velocities (0.0024 to 0.0044 m/s) are higher than the particle settling velocity (0.000038 to 0.00011 m/s) which is an indication that some particles settled faster at the sludge zone of the basin. In each flowrate the collection efficiency which is the settling efficiency was calculated and ranged from 40% (Q=5.7 l/s) to 58% (Q=11.1 l/s). The particle settling velocity is increasing with increase in flow rate from 0.000038 to 0.00011 m/s. The collection efficiency increased with increase in flowrate from 40% to 58%, which is an indication that as inflow rate is increased more sediment tend to settle at the bottom of the basin. A plot of turbidly against flow rate is shown in Fig. 4.

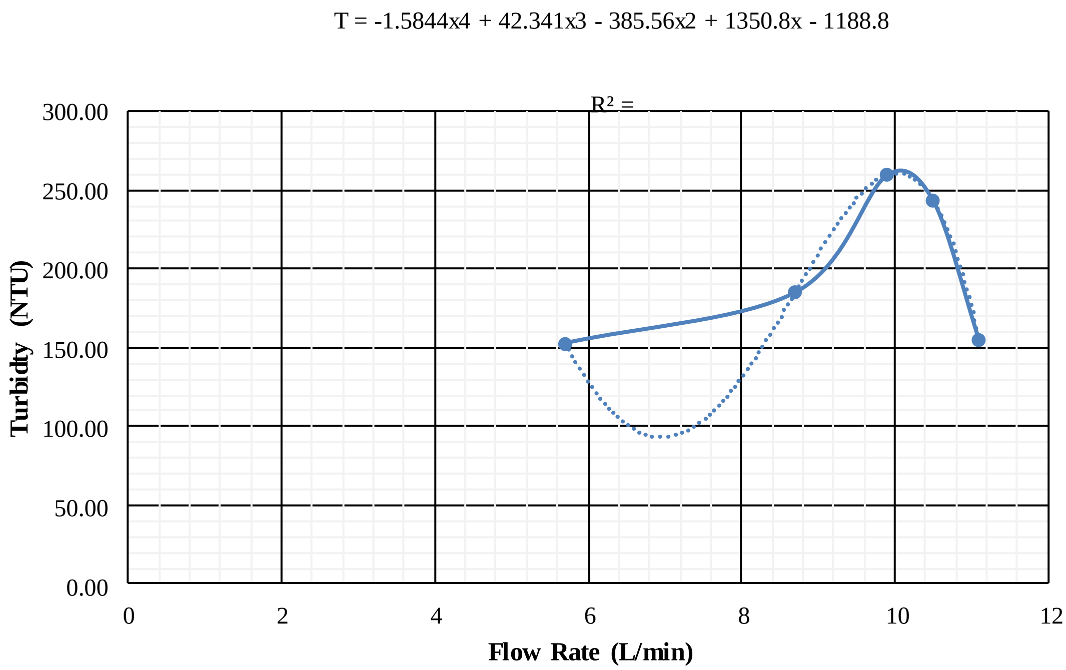

Fig 4. Flow rate against Turbidity drop

Fig 4 shows that as the flow rate increases, the turbidity drop also increases up to a critical flow rate of 9.9 l/min. After reaching this critical flow rate it starts to drop forming a curve with a polynomial curve behavior, which is deduced to quartic equation or biquadratic equation with an R^2 value of one. This indicates that the equation can be used to accurately calculate the expected turbidity drop in any turbid water with a given flow rates.

$$T = -1.5844x^4 + 42.341x^3 - 385.56x^2 + 1350.8x - 1188.8 \quad (16)$$

Where;

T = Turbidity value (NTU)

X = Flow rate (L/min)

Equation 16 shows the quartic equation for calculating turbidity in a given settling basin.

It is concluded that the sediment concentration at the sludge zone increased with increase in flow rate as shown in Table 3. This corresponds to what Martin B. [34] had found while researching on sediment load and sediment concentration prediction whereby the author observed that the highest sediment concentration was with high flows. Haiyan L. et. al [35] drew the same conclusion where flow rate significantly affected the concentration of heavy metals in water and Weixing M. et. al [36] found that there was increase turbidity and sediment density at the bottom of reservoirs during high inflow of storm runoff.

Artificial neural networks was applied to predict turbidity level of the water at different settling times ranging from 20 minutes to 280 minutes at interval of 20 minutes. The graph of predicted against the measured turbidity is presented in Fig 5.

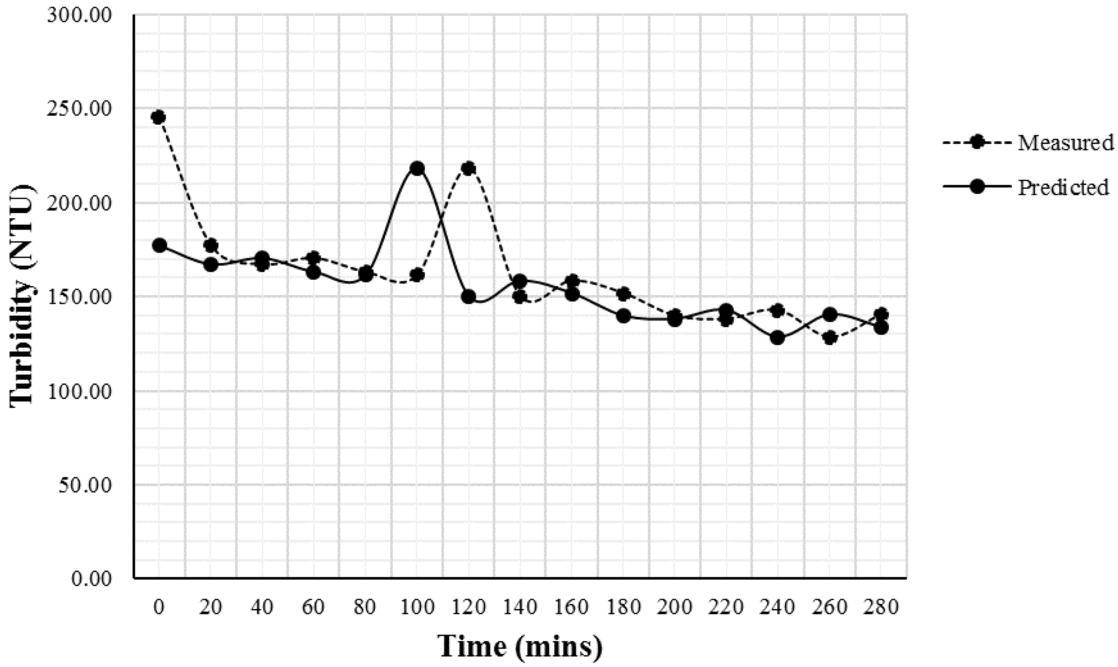

Fig 5. Measured and predicted turbidity

4.2. Determination of Optimum design parameters

Using Table 3 and Fig 5 the time for each turbidity drop was noted and Equation 16 was applied to calculate the optimum area required to settle the particle at a given flow rate. The depth of the tank was considered as 0.4 m as per the physical model but this can be varied according to the designs.

This means that with a settling tank of between 1.5675m^2 and 2.4225m^2 surface area the particles can be settled to the desired turbidity drop of 152.85 NTU. This process was down for three-flow rate that is 5.7l/min, 8.7l/min and 11.1 l/min. A graph of flow rate against optimum areas was then plotted as shown in Fig 6.

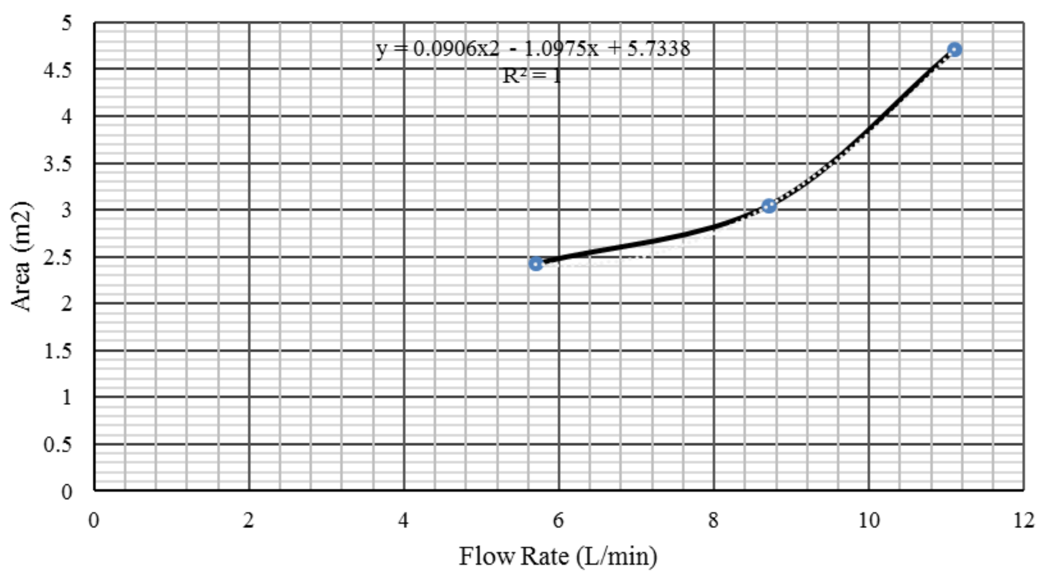

Fig 6. Flow rate against the optimum surface area

From Fig 6 a quadratic equation was developed for calculating the optimum surface area required to settle the particles and different flow rates. This is shown in Equation 17

$$Y = 0.0906X^2 - 1.0975X + 5.7338 \quad (17)$$

Where;

Y = Optimum Surface area of the tank (m^2)

X = Flow rate (L/min)

Using Equation 17, the optimum surface areas for the five flow rates were calculated and the results are presented in Table 5

Table 5. Optimum tank surface area for different flow rates

Flow Rate (L/min)	Turbidity drop (NTU)	Calculated Area (m^2)	Optimum Area (m^2)
5.7	152.85	2.50	2.42
8.7	185.00	2.08	3.04
9.9	259.33	1.92	3.75
10.5	243.33	2.08	4.20
11.1	155.00	1.72	4.71

5. Conclusions

1. Using stoke's law the critical settling velocities for discrete particles at different flow rates were calculated whereby it was found that the turbidity drop in a settling basin increased with the increase in flow rate.
2. A quartic equation $T = -1.5844x^4 + 42.341x^3 - 385.56x^2 + 1350.8x - 1188.8$ was developed which can be used to calculate turbidity drop in a small-scale settling basin when the flow rate is given (x).
3. Aquadratic equation $Y = 0.0906X^2 - 1.0975X + 5.7338$ was developed for calculating optimum surface area required for settling discrete particles for different flow rates.
4. The optimum areas for the five flow rates was calculated as; $Q_1 = 2.42m^2$, $Q_2 = 3.04\ m^2$, $Q_3 = 3.75\ m^2$, $Q_4 = 4.20\ m^2$ and $Q_5 = 4.71\ m^2$.

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