Settling Characteristics of Kut Dam Water

by

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January, 1990

DECLARATION

I hereby declare that this research work was carried out and written by me, and that it has not been presented neither in whole nor in part for a higher degree in this university or other universities. Any material or information other than mine has been duely referenced.

Siedoma, Lebura	Date

CERTIFICATION

This thesis entitled "Settling Characteristics of Kubanni Dam Water" by Lebura SIEDOMA meets the regulations governing the award of the degree of Master of Science (Water Resources and Environmental Engineering) of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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ABSTRACT

This study investigates the settling characteristics of the raw water of Kubanni Dam at the main campus of Ahmadu Bello University, Zaria. The investigation included plain sedimentation and sedimentation of coagulated water samples using aluminium sulphate (alum) as coagulant. Coagulation and flocculation tests were performed by the batch method in beakers (Jar tests), while the sedimentation tests were performed using settling column.

The results obtained indicate that plain sedimentation is not adequate for the turbidity removal from the raw water of the dam, but coagulation and flocculation processes are necessary, An optimum alum dose of 50mg/L is required to coagulate the water during the dry season and 70mg/L during the rainy season. An average turbidity removal of 85.596 was obtained from the sedimentation of the coagulated water samples as compared to 26% obtained from the plain sedimentation of the water samples.

Data from the settling of the coagulated water samples are used to design new sedimentation basins and for up-grading the existing sedimentation basins of the University water works. Four rectangular tank units are suggested, each of dimensions 21m x 5.5m with effective depth of 3m. Also four circular tanks could be used each of diameter 12m and effective depth of 3m. Using the experimental data the existing four basins (square in plan-8.5 x 8.5m and effective depth of 5m) are up-graded by

adjusting the volumetric flow rate. Two additional basins are recommends to achieve turbidity removal of 85.5% instead of the 70% removal being achieved presently.

The structural design of the basins and the estimated cost are also presented.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF SYMBOLS

The symbols are presented as they appear in the text.

K_s	-	Smoluchowski first order rate constant.
G	-	Root-mean-square velocity gradient.
ϕ	-	Volume fraction of dispersed phase.
t	-	Time.
E	-	Coefficient of turbulent diffusion.
α	-	a constant.
C	-	Concentration of particles.
C_o^1	-	Initial concentration of particles.
V_s	-	Settling velocity of a particle.
V_o	-	Surface overflow rate.
R	-	Fraction of particles removed by a settling basin.
$F(V_s)$	-	Commulative settling velocity distribution.
V_a	-	Velocity of approach in a suspension.
S	-	Volumetric solid fraction of a suspension.
n_p	-	Coefficient that identifies basin performance.
Q	-	Volumetrict flow rate.
A_s	-	Surface area of a tank.
F_w	-	Force due to gravity.
F_B	-	Buoyant force due to liquid.
F_D	-	Frictional or drag force
M	-	Mass of particle

ρ_s	-	Mass density of particle.
ρ_L	-	Mass density of liquid.
A_p	-	Projected area of particle in direction of motion.
V_p	-	Volume of particle.
C_D	-	drag coefficient.
g	-	Acceleration due to gravity
d	-	diameter of particles.
N	-	number of contacts per unit volume and unit time made by particles.
n	-	number of particles.
S_p	-	Specific gravity of particle.
ν	-	Kinematic viscosity.
H	-	depth of settling zone of a basin.
L	-	length of basin.
b	-	width of basin.
h	-	Any depth of liquid in a basin.
	-	absolute viscosity.
V_h	-	Horizontal velocity in a basin.
C_o	-	Portion of particles with settling velocity less than V_o.
M^1	-	bending moment.
M_u	-	Ultimate resistance moment.
h^1	-	depth of concrete section.

d^1 - effective depth of tension reinforcement

f_s - tensile stress in reinforcement.

A_{s1} - Area of steel.

f_{cu} - Characteristic concrete cube strength.

f_y - Characteristic strength of reinforcement.

b^1 - width of concrete section.

x - Neutral axis depth.

z - Lever arm of concrete section.

CHAPTER 1

INTRODUCTION

1.1 DESCRIPTION OF THE PROCESS

Sedimentation as applied to sanitary engineering processes may be defined as the removal of suspended solid particles from a suspension through gravity. For the solid particles to settle out they must be heavier than the medium in which they are suspended.

When the solid particles are separated from the suspending liquid by gravitation and natural aggregation, the process is described as plain sedimentation. However, the settling velocities of finely divided and colloidal particles under gravity alone may be so small that ordinary sedimentation is not practical. It has therefore been necessary to use procedures which agglomerate the smaller particles into larger aggregates which then have sufficient settling velocities. When chemicals are added to induce or hasten aggregation of finely divided and colloidal particles in a suspension, the process is called coagulation, and the chemical used is called coagulant.

Significantly different settling characteristics are associated with different types of suspension of solids. Fundamental to the process of sedimentation is the consideration of the character of the suspended matter to be removed. The characteristics of the particles to be considered are not only the settling velocities but also the question of whether or not they retain their individualities as they settle.

Also, the character of the fluid motion including the momentary fluctuations from the average motion at every point in the fluid mass is an important consideration. Besides these factors, all secondary motions, such as convection or displacement currents and fluctuations which are characteristics of turbulent flow, should be considered if the fluid motion is to be completely defined.

1.2 Historical Background

Sedimentation has been used for generations to clarify liquors and to concentrate solids in many widely diversified fields. It is the most commonly used process in the field of water and waste water treatment.

The history of sedimentation, is the history of man's quest for pure water. Ancient literature and archaeological evidence record man's relentless search for pure water. Man's earliest standards of water quality were few - freedom from gross turbidity, taste, and colour. While his present criteria of purity have become more complex and certainly more quantitative, the principles, methods, and materials for purifying water have remained remarkably similar to that of the earliest recorded date of about 2000 B.C. But except for the developments of the last few decades, the history of water treatments is largely a record of empiricism and a description of an art rather than a science, Cohen and Hannah (1971).

The clarification of water by sedimentation is a natural process, and evidence of its purposeful utilization reaches us from antiguity. In the ancient Egypt, the turbid flood waters of the Nile were diverted to stilling basins and allowed to settle before distribution for domestic use. In the ruin of carthage, the remains of large rains water storage and settling reservoirs, erected for public supply have been found. In Rome, settling reservoirs for improvement of water quality were built before the christian era. Around 2000 B.C., the Minoans constructed small settling basins in the drainage system, Hauslin and Tebbutt (1978). Although the date of the earliest sedimentation tank is unknown, it is clear that the process has long history in treatment.

When after many centuries, watertight household vessels became available, man observed that water could be more or less clarified by storage in containers or by filtration through porous receptacles or sand layers, leaving the objectionable sediment behind. After a lapse of still many more centuries, man discovered that sedimentation could be aided by adding a precipitant or coagulant. This recognition that coagulant aided both sedimentation and filtration, and the use of great variety of organic and inorganic materials marked the beginning of modern treatment of water. Moreover, for the first time, man was not constrained to accept the quality of water that nature provided, but rather was able to modify it to meet his demands for increasing higher standards of quality.

1.3 Objectives of the Study

The varied nature of settling characteristics of suspensions makes continous experimentation necessary for new plant design, and improvement of plant operation. Laboratory settling experiments - usually specific for a particular water or wastewater - are conducted to determine the settling characteristics of the suspended solid particles, and the results used for both plant design and improvement of plant performance.

There is no record to show that the settling characteristics of the raw water for the Ahmadu Bello University campus water treatment plant were determined before the plant was designed and brought into operation. The objectives of this study, therefore, are: to determine the settling characteristics of the raw water of Kubanni dam, which is the source of water for ABU water treatment plant; to use the results obtain to design new sedimentation basins; and also to use the results for the upgrading of the sedimentation basins of the ABU water treatment plant.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Coagulation and Flocculation

Coagulation describes the effect produced by the addition of a chemical (Coagulant) on a colloidal dispersion resulting in particle destabilization due to reduction of the forces tending to keep the particles apart. Rapid mixing is important to obtain uniform dispersion of chemical and to increase the opportunity for particle contact. The entire process occurs in a very short time, probably less than a second.

The second stage of the formation of settleable particles from destabilized colloidal-sized particles is termed flocculation. In contrast to coagulation, where the primary force is electrostatic or inter-ionic, flocculation occurs by a chemical bridging or physical enmeshment mechanism. Flocculation is obtained by gentle and prolonged mixing which converts sub-microscopic coagulated particles into visible suspended particles. At this stage the particles are large enough to settle rapidly under the influence of gravity, Clark and Viessman (1965).

p^H is an important variable that affects the coagulation process. There is a p^H range for any given water within which good coagulation and flocculation occurs in the shortest time with a given coagulant dose. The extent of the p^H range is affected by the type of coagulant used and by the chemical composition of the water, as well as by the concentration of the coagulant, Vaughn (1976).

Alum (Aluminium sulphate, $Al_2(SO_4)_3 \cdot 18H_2O$) is by far the most commonly used coagulant. Iron salts can be used as well and in some instances have advantages over alum. A significant advantage of iron salts over aluminium is the broader p^H range for good coagulation. For alum the p^H range for good coagulation is fould to be quite narrow, ranging from about 6.0 to 7.8; the range for ferric sulphate is about 5.5 to 6.8. Coagulation within the optimum p^H range enhances economic use of chemical and may be reflected in the high quality of the water treatment plant effluent, Cohen and Hannah (1971).

Alum is used in this study because it is the coagulant being used at the Ahmadu Bello University Campus water treatment plant. It is also readily available in large quantities, and less costly compared to iron salts. One disadvantage of iron salts is that they might increase the iron content of the water being treated.

Packham (1962) described a method of studying the Coagulation of dilute mineral suspensions with aluminium sulphate, which substantially reduce p^H various with Coagulant dose, but which allows the p^H to be varied independently of the coagulant dose. The objective of the study was to establish whether or not the nature of the suspended solids giving rise to turbidity has a significant effect if the p^H is controlled. A variety of minerals likely to be present in river water and samples of suspended solids isolated from rivers were studied. A fixed suspended solids concentration of 50mg/l was used, this being of the same order as the average suspended solids concentration in the local rivers where the study was carried out. According to Packham, the concentration of alumimium sulphate required to coagulate the 50mg/l suspensions was clearly affected to a marked degree by the p^H. The coagulant concentrations required were least in the neutral p^H range, suggesting that the hydrolysis products of alumimium have an important role, and that the alumimium ion itself plays no direct part in the mechanism. The results obtained were largely independent of the nature of solids in suspension.

The settling behaviour of floc particles produced in coagulation of colloidal suspension in water treatment operations is recognized as being substantially dependent upon the nature of the colloids, coagulant, p^H and mixing. However, it is impractical sometimes to adjust one or more of these parameters to obtain desired settling behaviour. In such cases the application of coagulant aids may provide the best or only solution. Coagulant aid often promote settling when the sedimentation characteristics of flocs formed by normal coagulation procedure are poor. There are two categories of coagulant aids. The first includes those for which substantially increase in floc size occurs as a result of sorption and enmeshment of particles by long - chain polymeric molecules. The second includes clay-type minerals that affect an increase in the density of floc particles. However, certain coagulant aids, for example, activated silica, may be included in both categories, and thus no clear line of demarcation can actually be drawn between them Posselt et al (1968).

Birkner and Morgan (1968) reviewed a mathematical theory for the coagulation kinetics of colloidal suspensions published by Smolunchowski in 1917. Kinetic equations were presented to describe the rate of change in total particle concentration after repulsive forces between similarly charged double layers surrounding the particles were reduced sufficiently so that interparticle collisions were not hindered. According to Birkner and Morgan, Smoluchowski developed two kineticequations; the first described the kinetic process when interparticle collisions occurred through Brownian diffusion of the individual colloidal particles. The second described the process when a laminar shear gradient causes particle transport at a point in fluid.

For a dilute suspension of monodispersed particles, the initial rate of change in total particle concentration in a laminar shear field was expressed as

$$\frac{dN_t}{dt} = -K_s N_t \qquad 2.1$$

where

$$Ks = \frac{4G\phi}{\pi} \qquad 2.2$$

in which ks is the theoretic smolunchowski first order rate constant, is the volume fraction of the dispersed phase. and G is the root - mean - square velocity gradient associated with the laminar flow.

Equation 2.1 is valid only during the initial phase of the flocculation reaction when it can be assumed that particle collisions occur between pairs of particles of the same size. As the flocculation reaction proceeds, the total particle concentration decreases, consequently, the assumption of collisions between particles of equal size is no longer valid and equation 2.1 can no longer be used to describe the kinetic process.

A number of investigators have studied the effect of chemical - addition times on coagulation. Nickel (1945) stated that in organic-colour coagulation the addition of lime or some other type of alkali should be made after the addition of alum. He further stated that with coagulation for turbidity removal, any p^H adjustment should be made well ahead of the coagulant addition in order to obtain the most economical results.

According to Moffett (1968), all p^H affecting chemical, such as lime, soda ash, acid, and chlorine, should be added upstream from the rapid mixer to ensure complete dissolution and mixing. He also stated that the p^H - affecting chemicals should be balanced so that the optimum p^H for coagulation is maintained after the addition of alum.

Kawamura (1973) studied the effects of the sequence of chemical addition on both turbidity and colour removal. According to Kawaamura, the colloidal particles would be stabilized if lime were added before or with alum. This stabilization would be the result of the adsorption of calcium, hydroxide, and carbonate ions on the colloidal particles. He concluded that lime should be added between 10 to 20 seconds after the addition of alum.

Griffith and Williams (1972) performed coagulation tests with varying dilution of alum. Dilutions used in their study of water from two rivers varied from 1.5 to 14.7 percent alum. Their results indicated that dilution did not inhibit coagulation but actually aided it. A general trend toward better turbidity removal with more dilute alum solutions was found. They felt that when mixing a small flow of alum solution with a much larger flow of raw water, a larger stream of a dilute alum solution would disperse more rapidly and uniformly than would a small stream of concentrated alum solution.

Jeffcoat and Singley (1975) studied the effect of alum concentration and chemical-addition times on coagulation. They used a synthetic turbid water which was made by mixing distilled water, kaolinite suspension, and a highly coloured lake water. The synthetic water was designed to stimulate a typical surface water with an alkalinity of 50mg/l as calcium carbonate, an organic colour of ten units, and a turbidity of 40 Jtu. The synthetic water had a p^H of 7.6. In order to study the effects of alum dilution and sequences of chemical addition on coagulation, the optimum alum dosage and p^H were first determined. These optimum conditions were ascertained by performing a series of jar tests. From these preliminary tests the alum dosage and p^H that produced the best results were chosen as the optimum conditions and used in the study. They concluded from the study that optimum p^H for coagulation increased when the alum dosage increased. Better turbidity removal occurred with dilute alum solutions, and better turbidity removal occurred when lime was added before alum.

The accurate and precise control of coagulation is the most important step in the operation of water treatment plant. For a raw water of rapidly changing characteristics, coagulation control is also the most difficult step in plant operation. The performance of the sedimentation process depends in considerable measure on the effectiveness of the coagulation - flocculation process, Hudson (1981).

2.2 Sedimentation

In 1889 Seddon was concerned about some definite knowledge of the laws under which the sediment in suspension passes out. Although a number of theories to explain the phenomenon had been put forward before that time, Seddon was worried that no effort was made to substantiate or validate any of them by a careful system of experiment and observation. In an attempt to have a good knowledge of the sedimentation process he carried out a series of experiments and observations using muddy water in a jar. Seddon (1889) concluded that settlement was simply due to the action of gravity on the sediment.

One of the earliest and most widely known attempts to set up a mathematical theory of settling in a sedimentation basin was that made in 1904 by Allen Hazen. The theory devised by Hazen was made possible by the realization that the processes which take place during sedimentation are extremely complex and difficult to discuss in their entirety. Hazen assumed some simplier conditions than those which actually exist, thus simplifying the complicated state of the problem. He distinguished between quiescent and turbulent flow and restricted his analysis to suspensions of discrete particles having a single settling velocity. Hazen demonstrated that the extent of the basin removal of discrete particles in a quiescent basin is dependent on the surface overflow rate and not on the depth of the basin.

However, the validity of Hazen's analysis of sedimentation in turbulent basin was questioned by some investigators, including Cordoba-Molina et al (1978), mainly because it was said to be based on incorrect assumptions. One of these assumptions is that of infinite turbulence which is incompatible with the implicit assumption that no sediment is picked up from the bottom of the basin. Nevertheless, Hazen's paper was said to contain something of value toward the development of a rational theory of sedimentation.

Slade (1973) investigated sedimentation in quiescent and turbulent basins. The purpose of the investigation was to continue Hazen's analysis, and to develop a workable theory which takes into account the variation in hydraulic value of the constituent elements of the actual sediment, as well as the variation in the degree of turbulence of the basin. Slade produced a number of mathematical equations two of which he considered most important and could be used to predict how different kinds of sediments will settle under different tank conditions.

These two equations are as follows:

$$Br = tr\ (D_{r+1} - D_r) \qquad 2.3$$

in which $r = 1, 2, \ldots, n.$

$$Ba = \sum_{r=1}^{s} Br\left(1 - \frac{a}{krtr}\right) + \sum_{r=s+1}^{n} Bre^{-\frac{a}{tr}} \quad Bc \qquad 2.4$$

where Br = quantity of solids settling with an average velocity, Ba = quantity of solids remaining in suspension at the end of time, a; Bc = quantity of solids in colloidal state; a = time during which sedimentation occurs, t = time of subsidence; s = a number of parts less than n; D = a derivative of Ba.

According to Slade, equation 2.3 gives the relation between the distribution of hydraulic properties of the sediment and its settling characteristics in a quiescent tank. By means of this equation the distribution of hydraulic values of a sample of sludge may be determined by observing its settling behaviour in a glass cylinder, for instance knowing this distribution, its settling characteristics in an actual tank may be predicted by equation 2.4.

Employing modern concepts of turbulence, Dobbins (1944) studied the settling of discrete particles having single settling celocity in the presence of turbulence. He developed an equation for transient concentration profile during settling. The equation is as follows:

$$C = A^{-vshE^{-1}} + (Co-A)e^{-0.5vshE^{-1}}$$

$$X \sum_{n=1}^{\infty} \frac{2\,\alpha_n^2\,Vse^{-(\alpha_n^2 + 0.25V_s^2E^{-1})t}}{\left(\alpha_n^2E-\frac{Vs^2}{4E}\right)\left[\left(\alpha_n^2+\frac{Vs^2}{4E^2}\right)H+\frac{Vs}{E}\right]}Yn \qquad 2.5$$

in which A = concentration at h = 0 and t = 0; Vs = settling velocity of particles; h = elevation above surface of bed material; Co = initial concentration of particles; E = coefficient of turbulence diffusion; t = time of settling; = a constant; n = an integer; e = base of natural system of logarithms; Yn = a function of depth, h; H = depth of flow in the tank.

Although Dobbins' model is considered to be important, it has been found to be of limited application mainly because the equations are said to be too cumbersome and complicated for practical use.

Camp (1946) extended Hazen's result for quiescent settling to the case of discrete particles having a settling velocity distribution. He developed a model that continues to be employed when continuous sedimentation basins are designed using quiescent settling data. Camp also extended the mathematical solution obtained by Dobbin in his model to the case of a continuous-flow settling tank. A modified removal formula according to the effect of eddy diffusion was deduced as follows:

$$R = 1 - 8\left(\frac{V_sH}{2E}\right)^2 e^{i} \times \sum_{n=1}^{\infty} \frac{\alpha_n^2 H_n e^{j}}{\left[\left(\frac{V_sH}{2E}\right)^2 + \alpha_n^2 + \frac{2V_sH}{2E}\right]\left[\left(\frac{V_sH}{2E}\right)^2 + \alpha_n^2\right]^2} \qquad 2.6$$

in which $i = V_sH/2E$, $j = -(i^2 + {}_n^2)\frac{V_s}{2iV_o}$; and 1, 2, ..., n are successive real postive roots of the transcendental equation

$$2 \cot = \frac{\alpha}{\frac{V_sH}{2E}} - \frac{\frac{V_sH}{2E}}{\alpha} \qquad 2.6\ (a)$$

Other paramenters are as defined in equation 2.5.

EL-Baroudi (1969) observed that one of the main discrepancies in tank performance from the ideal behaviour is that real flows are turbulent and encounter a certain degree of mixing or eddy diffusion. According to El-Baroudi, the mathematical expressions obtained by Dobbin (1944) and extended by Camp (1946), are handicapped in their applications. This was because of the inavailability of the characteristic of the mixing coefficient for a given tank. El-Baroundi attempted to obtain a characteristic mixing coefficient from the analysis of tracer dispersion curves of settling basins. The formula he obtained for the observed transient tracer dispersion curve of settling tanks is as follows:

$$C(t) = \frac{Co}{2} e^{k(ls-vt)+k^2Et} (\operatorname{erfc}Z_1 - \operatorname{erfc}Z_2) \qquad 2.7$$

He provided graphical solution to approximate this equation by excluding the term erfcZ2. In equation 2.7, $C(t)$ = Concentration of tracer or particles at given time, t; Ls = length of settling zone, erfc = error function; V = velocity of flow; Z, and Z2 are parameters, $Z1 = (Ls-vt)/2\sqrt{Et} + k\sqrt{Et}$; $Z2 = Ls/2\sqrt{Et} + k\sqrt{Et}$; k = parameter defined as $1/_{LI}$ or V_I/Am; where L_I = length of in let zone; VI = volume of inlet zone; and Am = effective or modal sectional area of the tank and = Q/Vm; in which Q = flow rate; and Vm = effective or modal velocity of flow.

Cordobo-Molina et al (1978) attempted to show that despite their apparent difference in approach, the models developed by Hazen (1904), Camp (1946), and Dobbins (1944), lead in the limit to identical results. Working on the basis of Dobbins' equations for transient concentration profile during settling, they developed simplified statements that were then checked by model testing in a circular centre-feed clarifier. The relations developed are:

$$Rq = 1 - \frac{1}{Vo} \int_0^{Vo} F\ (Vs)\ dVs \qquad 2.8$$

and

$$Rt = \frac{1}{Vo} \int_0^{\infty} e^{-Vs/vo} \left[1 - F(Vs)dVs\right] \qquad 2.9$$

in which Rq = fraction of particles removed during quiescent settling; Rt = fraction of particles removed during turbulent settling; F(Vs) = Cumulative settling velocity distribution; and Vo = surface overflow rate.

Cordobo-Molina et al (1978) found that mathematical models for quiescent sedimentation and for sedimentation in the presence of turbulence provided upper and lower boundary lines that fit the observed data quite well. As the overflow rate of the system increased, the data closely approached the values for the model for turbulent conditions. It was concluded that the turbulent model provided a conservative design relationship for continuous flow settling tanks using quiescent settling data.

The behaviour of discrete and flocculent suspensions under upflow conditions was investigated by Brown and La Motta (1971). They used a simplified model tank which was filled with water and also contained a fixed volume of spherical particles supported at rest on a porous plate. An upward flow was introduced through the tank which resulted in an approach velocity. When the particles are in suspension or imminent suspension the approach velocity is expressed as follows:

$$Va = Vs(1-s) \qquad 2.10$$

in which S is the volumetric solid fraction of the suspension.

Rebhum and Argaman (1965) reported the results of a tracer test study to determine the hydraulic efficiency of a sedimentation basin. The experimental results show that in basin with inserted baffles, the effective flow was 70% mixing and 30% plug flow. Because of the similarity between the flow curves they obtained and those reported in literature, Rebhum and Argaman concluded that in most sedimentation basins, more than 60% of the flow is composed of mixing, and the remainder being plug flow.

On the basis of tracer studies, Clark and Viessman (1965) contended that in general, long narrow rectangular basins are superior to radial-flow circular basins. Circular basins, according to them exhibit unstable, poorly shaped flow waves. However, the unstable flow-wave associated with the radial-flow basin may not be a valid index of efficiency of that type of settling vessel, they concluded.

In a theoretical investigation, Chiu (1974), found that a flow separation occurs in the radial-flow centre feed clarifiers, but not in rectangular tanks. This separation, according to him, results in eddy currents that reduce the efficiency in the removal of settleable solids. In comparison, flow conditions in horizontal-flow rectangular tanks were considerably more stable.

One approach for estimating the effect of currents on the performance of a sedimentation tank is that devised by Hazen (1904) and elaborated by Fair et al (1971). Using the concept of longitudinal change in treatment response, Fair et al (1971) expressed the proportion of particles with settling velocity V_s, that will be removed in the basin as follows:

$$\frac{C}{C_o} = \left(1 - \frac{n_p V_o}{Q/A_s}\right)^{-\frac{1}{n_p}} \qquad 2.11$$

in which n_p is the coeffici nt that identifies basin performance. From a mathematical analysis of longitudinal mixing in settling tanks, it was found that the value of the coefficient n_p could be approximated by the ratio of the difference between the mean and modal flowing-through periods to the mean flow-through period of the detention time distribution curve.

A detailed review of literature by Hansen (1967) reveal that there were many attempts to apply the shallow depth sedimentation principles recognized by Hazen as early as 1904. It has been observed that the lack of acceptance of tray settling in water and wastewater treatment systems was due to the reluctance of design engineers to depart from previous practice in size and shape of basins. According to Culp and Hsiung (1969), the attempts to use wide, shallow trays met with only a limited success because of two major problems: the unstable hydraulic conditions encountered with wide, shallow trays; and the minimum tray spacing which was limited by the vertical clearance required for mechanical sludge removal equipment. To overcome these problems Culp and Hsiung used small diameter tubes which provide theoretically excellent conditions for efficient sedimentation. They demonstrated the use of these small diameter tubes and were successful.

2.2.1 Types of Sedimentation

Particles settle from suspension in different ways depending on the concentration of the suspension and the characteristics of the particles. Fitch (1958) identified four distinct types of sedimentation which reflect the concentration of the suspension and the flocculating properties of the particles. They are described as follws:-

<u>Type - 1 Sedimentation</u>: This is the settling of a dilute suspension of discrete particles. The particles settle as individual entities with no significant interaction with neighbouring particle. This type of sedimentation occurs when grit and sand particles settle out of a suspension.

<u>Type - 2 Sedimentation</u>: This is the removal from a dilute suspension of flocculent particles. In this type of sedimentation, heavier particles having large settling velocities overtake and coalesce with smaller, lighter particles to form still larger ones with increased rates of subsidence. This type of sedimentation can be observed during the settling of coagulated water, and suspended solid particles in untreated wastewater in primary settling facilities.

<u>Type - 3 Sedimentation</u>: This occurs in suspensions of intermediate concentration where the particles are sufficiently close together so that interparticle forces are able to hold them in fixed positions relative to each other. As a result the particles subside as a large mass rather than as individual particles. This type of sedimentation is also called "zone settling", and occurs in secondary settling facilities used in conjunction with biological settling facilities.

Type - 4 Sedimentation: This occurs when the concentration of the suspension is such that the particles actually make contact with each other, and a structure of compacting mass results. Further settling can occur only by compression of the structure. This type of sedimentation is also called "compression", and usually occurs in the lower layers of a deep secondary settling facilities, and in sludge thickening facilities.

It has been suggested that during sedimentation process, it is common to have more than one type of settling occuring at a given time, and it is also possible to have all four types occuring at the same time. Clark and Viessman (1965) and Metcalf and Eddy (1979).

2.2.2 Settling of Discrete Particles

When a discrete particle is released in a still liquid it moves vertically downward if its density is greater than that of the liquid. The particle will accelerate until the frictional drag of the liquid on the particle approaches the value of the impelling gravity. After this balance of forces, the vertical velocity of the particle with respect to the liquid at rest will be substantially constant. This terminal velocity is known as the settling velocity of the particle.

The settling of the particle has been described by the equations of classic mechanics. By Newton's second law, the downward acceleration of the particle is given by Webber (1972) as follows:

$$M \frac{dv}{dt} = F_W - F_B - F_D \qquad 2.12$$

where M = the mass of the particle;

V = settling velocity of particle; t = time;

F_W = force due to gravity = $(\rho_s V_p\, g)$;

F_B = bouyant force due to liquid = $(\rho_l V_p\, g)$;

F_D = fractional or drag force = $(C_D A_p \rho_l V_s/2)$;

When the values of F_W, F_B, and F_D, are substituted in equation 2.12, and noting that at terminal velocity the acceleration of the particle, $(^{dv}/_{dt})$, is zero, the settling velocity of the particle can be shown as

$$V_s = \sqrt{\frac{2g(\rho_s - \rho_l)}{C_D \rho_l} \left(\frac{V_p}{A_p}\right)} \qquad 2.13$$

where = density of particle; = density of liquid;

g = acceleration due to gravity; C_D = drag coefficient; A_p = projected area of particle in direction of motio = ($d^2/_4$ for spheres); V_p = volume of particle = ($d^3/_6$). Thus $V_p/A_p = 2d/_3$, where d is the diameter of the particle. Replacing V_p/A_p with $2d/_3$ in equation 2.13 gives

$$V_s = \sqrt{\frac{4}{3} \frac{g}{C_D} \frac{(\rho_s - \rho_l)\, d}{\rho_l}} \qquad 2.14$$

The drag coefficient C_D has been found to be a function of Reynolds number, Re. For Re = 0.5 (region of viscous settling), $C_D = 24/Re$, and since $Re = Vsd/\mu$, the corresponding settling velocity of the particle is given by

$$V_s = \frac{g(\rho_s - \rho_L)d^2}{18\mu} \qquad 2.15$$

Equation 2.15 is known as Stoke's law, in which μ is the absolute viscosity of the liquid.

For turbulent settling (Re 10^3) the drag coefficient has been found to be approximately constant at a value of about 0.4. Thus the corresponding settling velocity of the particle is given by

$$V_s = 1.82\sqrt{\left(\frac{\rho_s - \rho_L}{\rho_L}\right)dg} \qquad 2.16$$

Equation 2.16 is known as Newton's law.

Discrete particle equations assume that the particles are spherical. But as noted by many investigators, including Camp (1946), particles to be removed from water and wastewater by sedimentation are seldom spherical and usually quite irregular in shape. The influence of irregularity in shape upon the drag has been found to be greater as the velocity increases.

2.2.3 Settling Flocculent Particles

Particles in relatively dilute suspensions do not always behave as discrete particles, but will coalesce or flocculate during sedimentation. Flocculation in a settling basin is attributed to the differences in the settling velocities of the particles, whereby faster settling particles overtake and coalesce with slower particles. It is also attributed to velocity gradients in the liquid which cause particles in a given region of higher velocity to overtake those in adjacent stream paths moving at slower velocity.

When flocculation occurs, settling velocities are sually increased. It is difficult to obtain a correlation between the rate of change of settling velocities and the rate of flocculation. This is because when particles coalesce, size, density, and shape, are all changed. The rate of flocculation has been found to be a function of the number of contacts between particles in unit time. Hence, the number of contacts per unit time in unit volume is used as a measure of the rate of flocculation.

Camp (1946) and Fair et al (1971) have shown that the number of contact N, per unit volume and time can be estimated from the size and number of spherical particles. If a unit volume of a liquid contains n_1 particles of diameter d_1 and settling with velocity v_1, and n_2 particles of diameter d_2, and settling with velocity v_2, the number of contacts per unit volume and unit time is given as

$$N = n_1 n_2 \frac{\pi}{4} (d_1 + d_2)^2 (v_1 - v_2) \qquad 2.17$$

When the particles have equal density and the consideration is limited to viscous settling, equation 2.15 is substituted for the difference in settling velocities, $v_1 - v_2$ in equation 2.17. Hence equation 2.17 becomes.

$$N = n_1 n_2 \frac{\pi g (S_p - 1)}{72V} (d_1 + d_2)^3 (d_1 - d_2) \quad 2.18$$

where S_p is the specific gravity of the particles, and V is the settling velocities, thus there will be no union of particles having the same settling velocity. It is also proportional to the relative number of contact, N/n, and therefore directly proportional to the concentration of particles.

2.2.4 Sedimentation in an ideal Basin

An ideal basin has been defined as a hypothetical settling tank in which settling takes place as in a quiescent settling container Camp (1946). An ideal rectangular continuous - flow basin for unhindered settling has the characteristic that the direction of flow is horizontal and the velocity is the same in all parts of the settling zone. In an ideal rectangular basin the paths of all discrete particles are straight lines, and all particles with the same settling velocity will move in a parallel path. The settling pattern will be the same in all longitudinal sections of the settling zone.

The ideal basin comprises of four zones as shown in figure 2.1. These are the inlet zone, settling zone, outlet zone, and sludge zone.

Figure 2.1 assumes a uniform dilute suspension flowing into the ideal basin with a horizontal velocity V_n. All particles having settling velocities equal to or greater than Vo will be removed from the suspension. Also, all particles settling with velocity Vs which is less than Vo

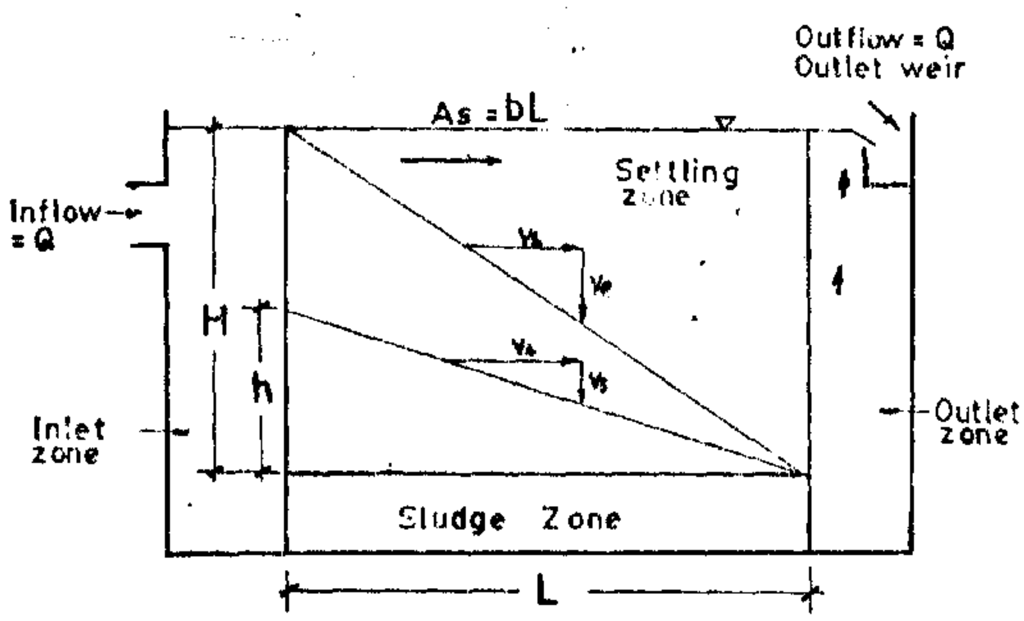


Figure 2.1 Di· .ete Particles Settling in an Ideal Rectangular Sedimentation Tank (Clark and Viessman, 1965)

will be removed if they enter the basin at depths equal to or less than $h<H$. The ratio of such particles that will be removed from the suspension is h/H. From similar triangles, this removal ratio is

$$R = \frac{h}{H} = \frac{Vs}{Vo} = \frac{bl\,Vs}{Q} = \frac{Vs}{Q/As} \qquad 2.19$$

where $bL = As$ is the surface area of the settling zone of width, b, and length, L. Q is the flow rate, and $Vo = Q/As$ is the overflow rate or discharge per unit surface or floor area of the basin. The overflow rate is defined as the settling velocity of particles which are just removed in an ideal basin if they enter the basin at the surface, Clark and Viessman (1965).

The efficiency of an ideal basin may be estimated by making use of equation 2.19 in conjunction with figure 2.2. The concentration of all the particles in the unsettled liquid is taken to be unity. Since all particles with settling velocities equal to or greater than Vo will be totally removed, the removal of such particles is 1-Co, where Co is the portion of particles with settling velocities less than Vo. For each size particle with a settling $Vs<Vo$, the portion removed would be Vs/Vo. When considering various sizes in this category, the percentage removal of these particles will be

$$r = \int_0^{Co} \frac{Vs}{Vo}\,dc \qquad 2.20$$

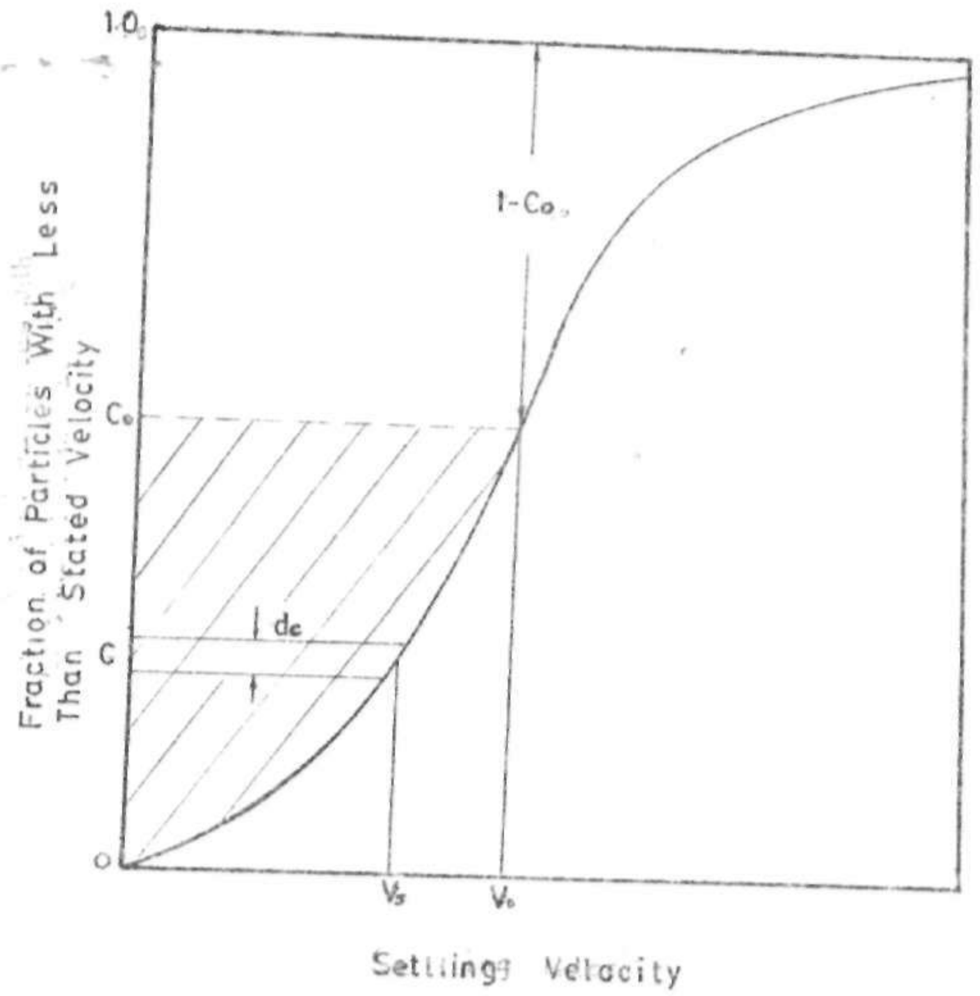


Figure 2.2 Typical Settling-Velocity Curve For discrete Particles (Camp, 1946)

and the overall removal will therefore be

$$R = (1-Co)+\frac{1}{V\sigma}\int_{0}^{Co} Vsdc \qquad 2.21$$

The second term in equation 2.21 is usually determined by a graphical integration of a settling analysis curve such as given in figure 2.2. This involves the shaded portion of the figure.

From equation 2.19, it may be said with respect to settling of discrete particles in an idea basin that for any given discharge, the removal of suspended particles is a function of surface overflow rate and independent of the depth of the basin. Also the removal of particles which settle at velocity $Vs < Vo$ is inversely proportional to the surface overflow rate, Vo. In vertical flow tanks, however, only particles with settling velocities equal to or greater than Vo will be removed.

For flocculent particle settling, the increasing velocity brought about by coalescence results in a curvilinear settling path. The opportunity for particle contact increases as the depth of the basin increases, so that the removal of suspended particles depends not only on the clarification rate but on the depth as well.

The removal efficiency in an ideal basin for a flocculent suspension can be calculated from the information provided in figure 2.3, which is obtained from Laboratory settling analysis. The smooth curves are called the "Iso-concentration curves", and represent the maximum settling path for the indicated removal.

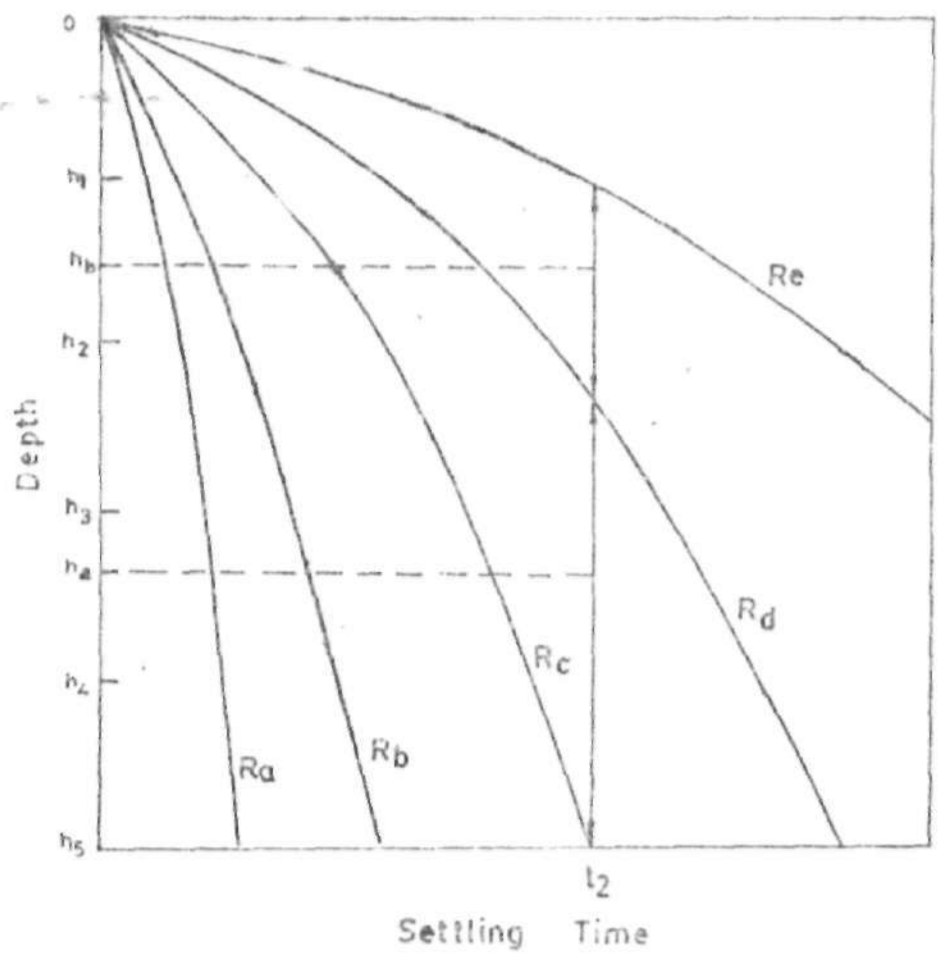


Figure 2.3 Typical Settling Curve for Flocculent Particles (Webber, 1972)

Referring to figure 2.3, suppose a tank of depth h_5 has an overflow rate of Vo. At time t_2, Vo = h_5/t_2. All particles having settling velocities equal to or greater than Vo will be removed from the suspension, and particles with smaller velocities will be removed in proportion to Vs/Vo. R_c percent of the particles have had average subsidence velocities Vo or greater, and will be completely removed from suspension. The remaining particles between Rd and Rc have settled at an average velocity of ha/t_2, and those between Re and Rd have settled at an average velocity of h_b/t_2. An approximation for total overall removal is given by

$$R = Rc + \frac{ha(Rd-Rc)}{t2Vo} + \frac{h_b(Re-Rd)}{t2Vo} \qquad 2.22$$

Removals in basins with different overflow rates and depths can be calculated in a similar manner.

2.2.5 Types of Sedimentation Tanks

Sedimentation basins may vary from a simple excavation in the ground to an elaborate multi-storey structure of steel or concrete. They may be square, rectangular, or circular in plan.

There exists a considerable variation in the size of sedimentation basins. The depth of basin may vary from 2 to 4.5m, 3.0m being a preferred value. Circular tanks are often 30m in diameter, but the size may vary between 10 and 60m. Rectangular tanks are often constructed with a length of 30m, but lengths up to 90 meters are sometimes used.

Square tanks are generally smaller, a side length of 21 meters is common. The width of mechanically cleaned, rectangular tanks is often controlled by the available length of the scrapers. Common length - to - width ratios employed in design are from 3:1 to 5:1. The diameter of mechanically cleaned, circular tanks is governed by the structural requirement of the trusses supporting the scrapers.

Except for steep - sided sludge hoppers, the bottom of most settling tanks slopes gently. Bottom slopes may lie close to 1% for rectangular tanks and 8% for circular or square tanks. The detention time of sedimentation basins may vary from less than one hour to more than a day. But since the greater part of the removal occurs during the first portion of the period, and since water is to be later subjected to further treatment, basins are designed usually for a detention period of 3 to 4 hours, Clark and Viessman (1965), Fair et al (1971), and Webber (1972).

CHAPTER 3

LABORATORY INVESTIGATION

3.1 Material

The experimental unit is shown in figure 3.1. It consists of a cylindrical, transparent perspex column of 0.15m in diameter. The depth is 2.62m and has 0.15m deep hopper bottom and open at the top. Five sampling ports are provided at equally spaced intervals of 0.5m and also an outlet at the bottom to remove settled solids and to drain the column. The column is supported on a wooden base. The raw water samples were obtained from the Kubanni dam reservoir at the main campus of Ahmadu Bello University, Zaria. The chemical used for the coagulation process was alum (aluminium sulphate, $Al_2(SO_4)_3.18H_2O$). Other apparatus included the multiple stirrer jar test apparatus.

3.2 Method

Two series of experiments were carried out including plain sedimentation of the raw water samples, and sedimentation of the coagulated and flocculated water samples. These experiments were conducted in the Environmental Engineering, Ahmadu Bello University, Zaria, during the months of April, May and July, 1988. These were the periods toward the end of the dry season, the beginning of the rainy season, and the peak of the rainy season, respectively.

3.2.1 Plain Sedimentation

The plain sedimentation experiments were carried out by mixing the raw water samples in a container to maintain uniform concentration of suspended solid particles. The

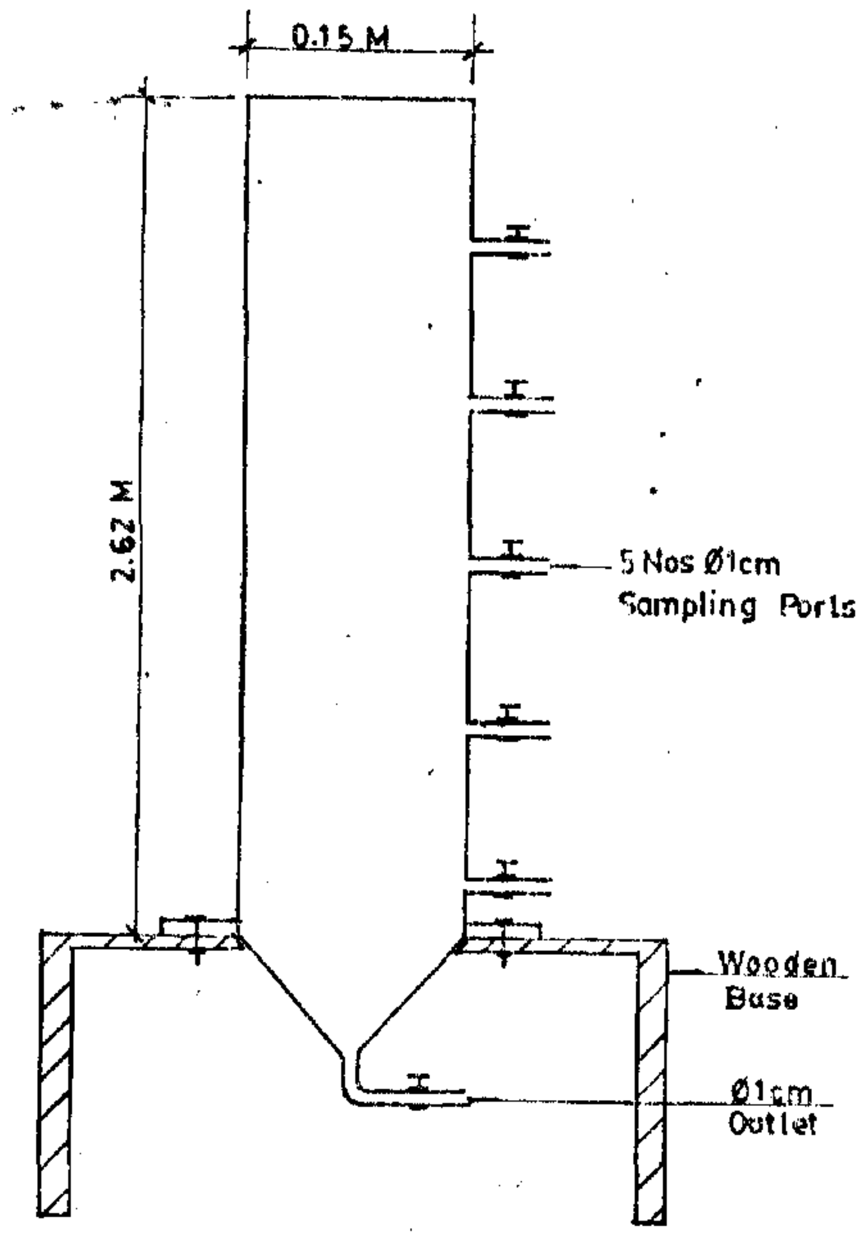


Figure 3.1 Settling Column

water samples were initially analysed for turbidity which was measured by the Nephelometric turbidimeter model DRTIDO, and the results expressed in Nephelometric turbidity unit (NTU). The suspension was then poured into the settling column and allowed to settle under quiescent conditions. Samples were withdrawn from the sampling ports at time intervals of 20,40,60,80,100,120, and 150 minutes and analysed for turbidity. A total of ten test runs were conducted.

3.2.2 Coagulated and Flocculated Sample Settling Experiments

The sedimentation of the coagulated and flocculated samples in the settling column was preceded by coagulation and flocculation tests (jar tests) using the multiple stirrer jar test apparatus, and alum as coagulant. The purpose of the jar tests was to determine the optimum alum dose to be used for the coagulation of the raw water samples, and also to determine the optimum p^H value for the optimum alum dose.

The jar tests were conducted according to a standard method presented by Jackson and Sheiham (1981). For the determination of the optimum alum dose 1000-ml samples of the raw water were measured into 1000-ml beakers and placed on the multiple stirrer unit. The sample in each beaker was subjected to rapid stirring (100 rev./min) for one minute and alum solution added. The quantities of the alum solution added were such that varied the alum concentration in the samples between 10 mg/l and 100 mg/l in 10 mg/l increments. Rapid stirring was maintained after the alum addition for another one minute, then followed by slow stirring (20 rev./min) for 15 min. The samples were then allowed to settle for 30 min. and the supernatant decanted and analysed for turbidity.

To determine the optimum p^H value, 1000-ml raw water samples were measured into the 1000-ml beakers and little quantities of calcium hydroxide (lime) and sulphuric acid added. The alkali acid additions were such that varied the final p^H over the range between 5.5 and 8.5 in 0.5 p^H increments. The test was repeated using the optimum alum dose obtained from the previous test in all the beakers. As in the previous test, after the samples had settled for 30 min. the supernatant was decanted and analysed for turbidity.

Having determined the optimum alum dose and the optimum p^H value, the coagulated and flocculated sample settling experiment was conducted. First the initial turbidity of the was was determined as before. The optimum alum dose obtained from the jar tests was used to coagulate a measured volume of the raw water. The coagulated anf flocculated waster was poured into the settling column and allowed to settle under quiescent conditions. Samples were withdrawn from the sampling ports at time intervals of 20,40,60,80, 100,120 and 150 min. and analysed for turbidity. A total of ten test runs were conducted.

CHAPTER 4

RESULTS

4.1 Analysis of Results

4.1.1 Plain Settling

The graphs of plain sedimentation experiments are presented in figures 4.1 (a) and (b). These are graphs of settling velocity versus fraction of turbidity remaining at the sampling points due to particles with less than stated velocity. The settling velocities are computed by dividing each depth by the settling times. The data used in plotting the graphs are presented in tables 4.2 (a) and (b) in appendix B.

The data obtained from the xperiments are contained in tables 4.1 (a) to (n) in appendix A. These data are the turbidities remaining at the depths and indicated time intervals. The average values are in tables 4.1 (f) and (m), while the average values expressed as fractions of the raw water sample turbidity are in tables 4.1 (g) and (n). Data from these tables are used for the computation of the data in tables 4.2 (a) and (b) in appendix B, which are used for the plotting of the graphs in figures 4.1 (a) and (b).

4.1.2 Coagulation and Flocculation Experiments

The graphs for the coagulation and flocculation experiments (Jartests) are presented in figures 4.2 (a) to (d). Figures 4.2 (a) to (c) are graphs of alum dose versus the turbidity remaining in the water after 30 min. settling time. The raw water p^H values as indicated on the graphs are 7.8, 7.9, and 7.9 respectively for figures 4.2 (a), (b)

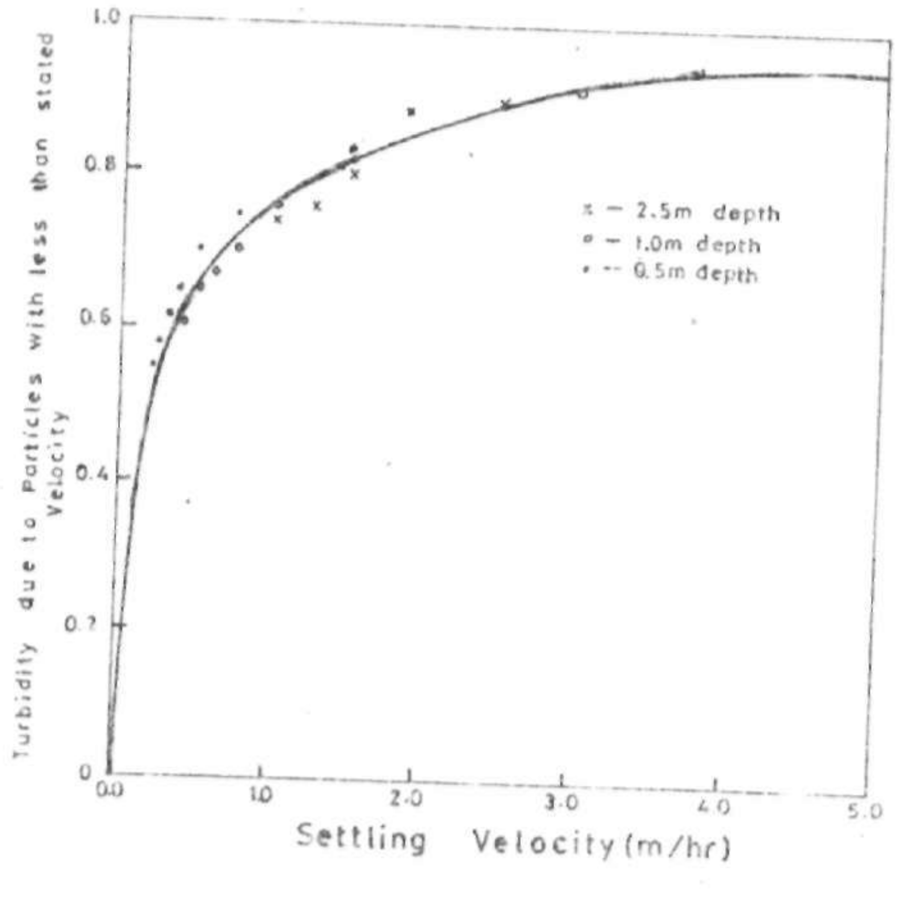


Figure 4.1(a) Settling Velocity Curve for Discrete Particles (Dry season Water Samples, April, 1988)

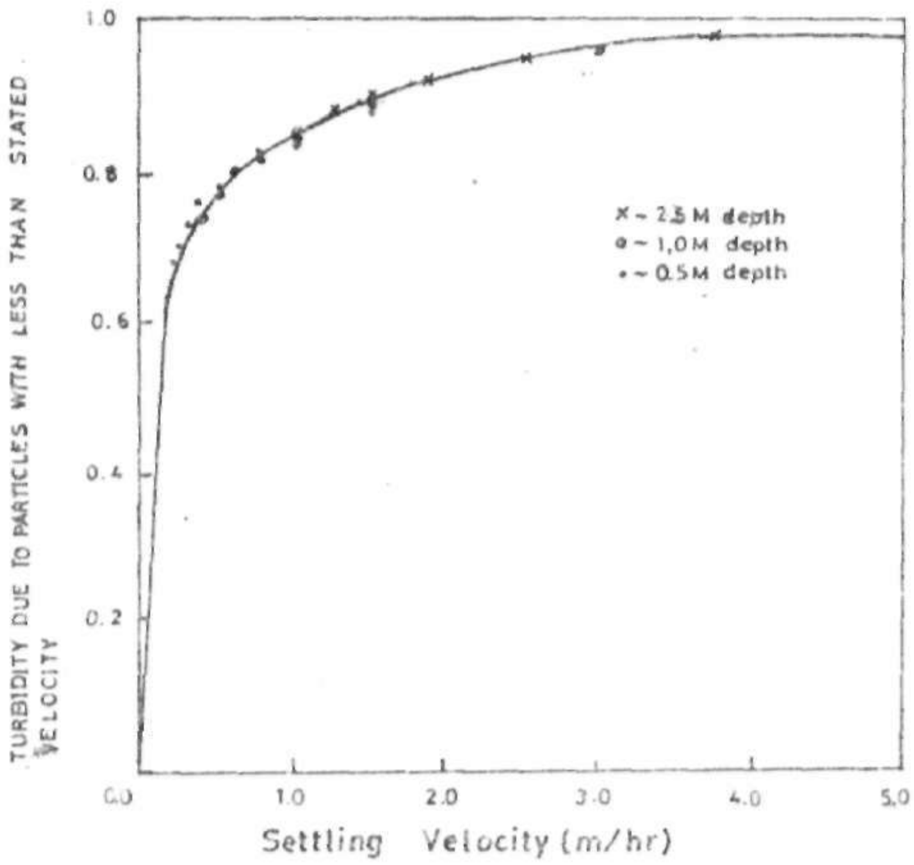


Figure 4.1(b) Settling Velocity Curve For Discrete Particles (Rainy Season Water Samples, May, 1988)

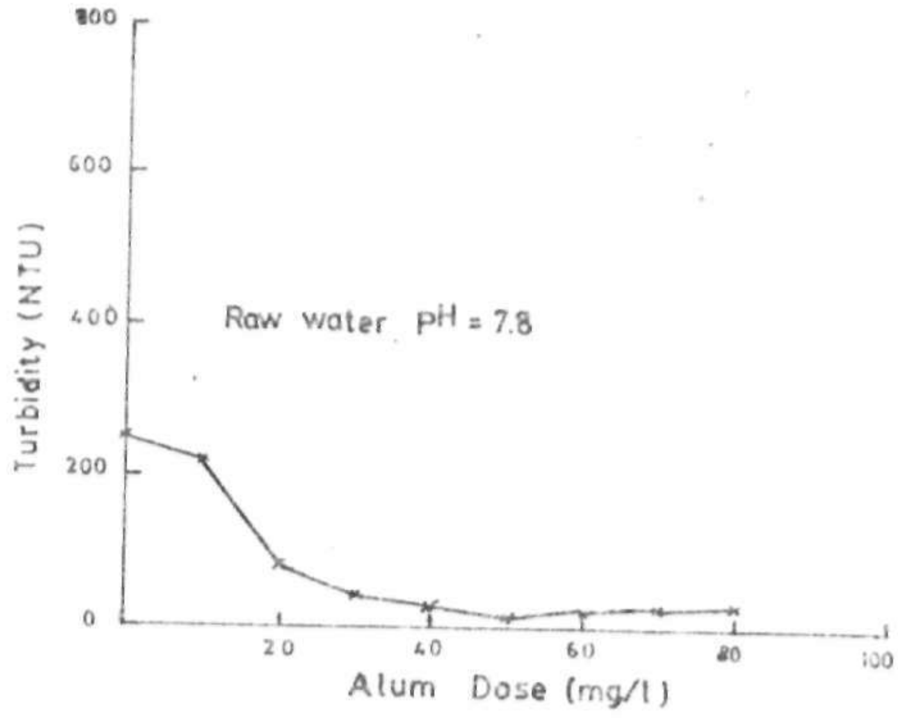


Fig. 4.2(a) Jar Test Curve For Optimum Alum Dose (Dry Season Water Samples. April,1988)

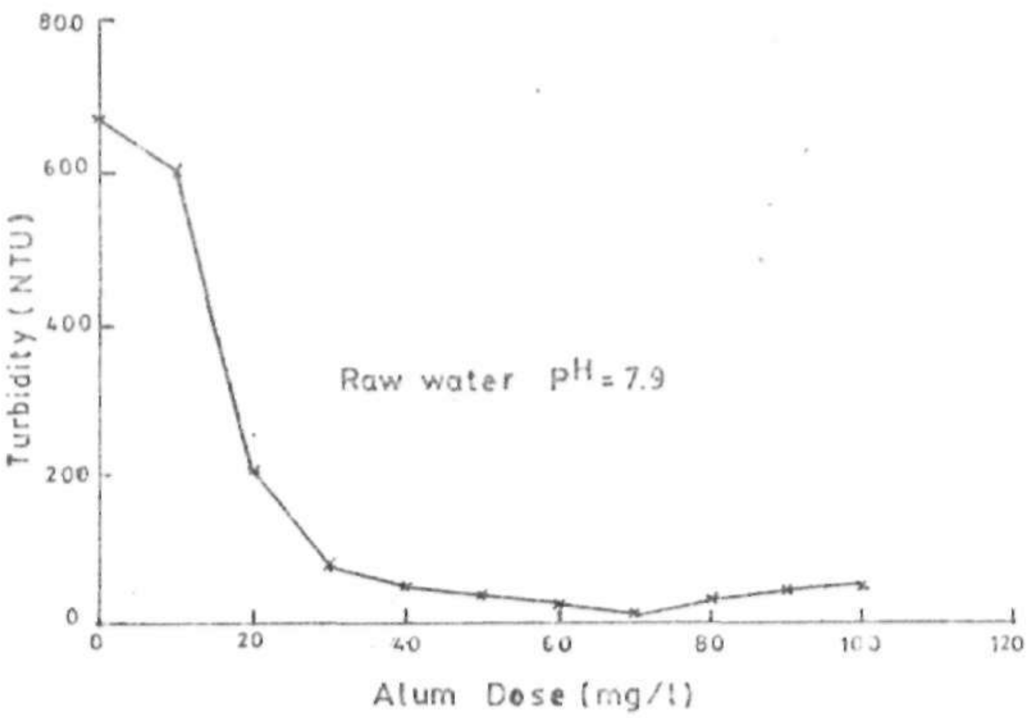


Figure 4.2(b) Jar Test Curve for Optimum Alum Dose (Rainy Water Samples, May, 1988)

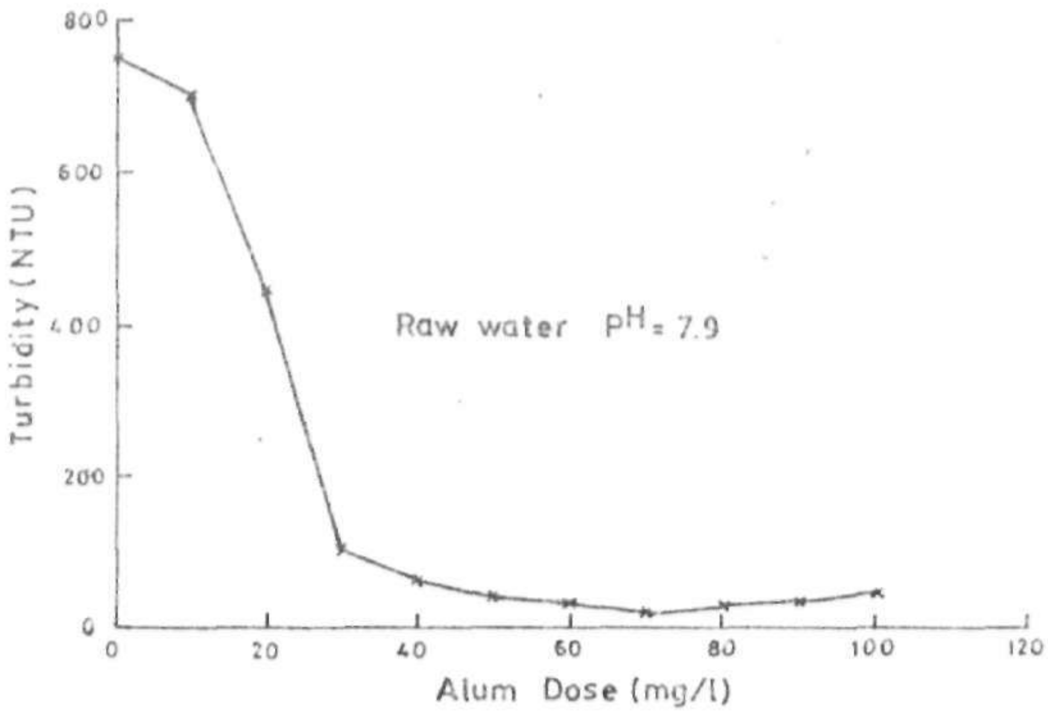


Figure 4.2(c) Jar Test Curve for Optimum Alum Dose (Rainy Season Water Samples, July, 1988)

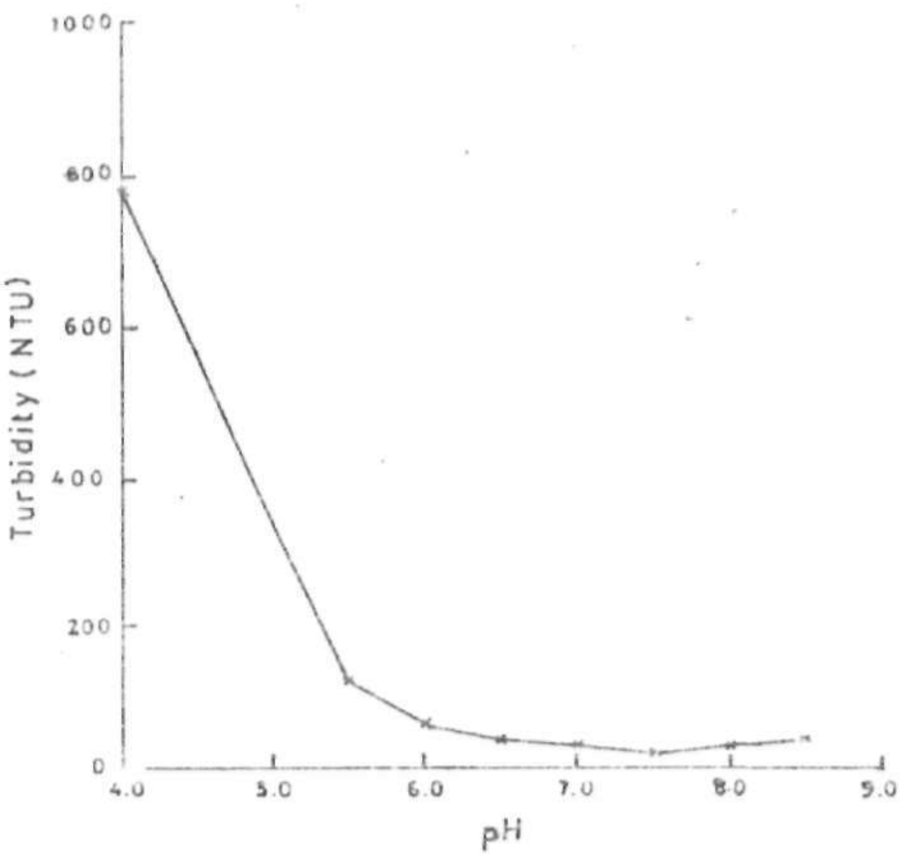


Figure 4.2(d) Jar Test Curve For Optimum pH (Rainy Season Water Sample, July, 1988)

and (c). Figures 4.2 (d) is a graph of p^H values versus the turbidity remaining in the water after settling time of 30 min., and is for a constant alum dose of 70 mg/l.

Tables 4.3 (a) to (d) in appendix D contain the data obtained from the experiments. Data in tables 4.3 (a) to (c) are the turbidities remaining in the water after the settling time for the different doses of alum applied, while data in table (d) are for different p^H values. Data in these tables are used for plotting the graphs in figures 4.2 (a) to (d).

4.1.3 Coagulated and Flocculated Sample Settling Experiments

The percentage turbidity removals obtained from the experiments are plotted against their respective depths and settling times, and smooth curves - the iso-concentration curves - are drawn to connect points of equal removals. These are presented in figures 4.3 (a) and (b). Making use of the iso-concentration curves and equation 2.22, the data in tables 4.5 (a) and (b) in appendix F are computed for tank depth of 2.5m. Data in these tables are the percentage turbidity removals at different detention times and overflow rates. Figures 4.4 (a) and (b) are graphs of percentage turbidity removal as a function of detention time, and as a function of surface overflow rate. These graphs are plotted using the data in tables 4.5 (a) and (b) in appendix F.

The data obtained from the coagulated and flocculated sample settling experiments, and from which the above

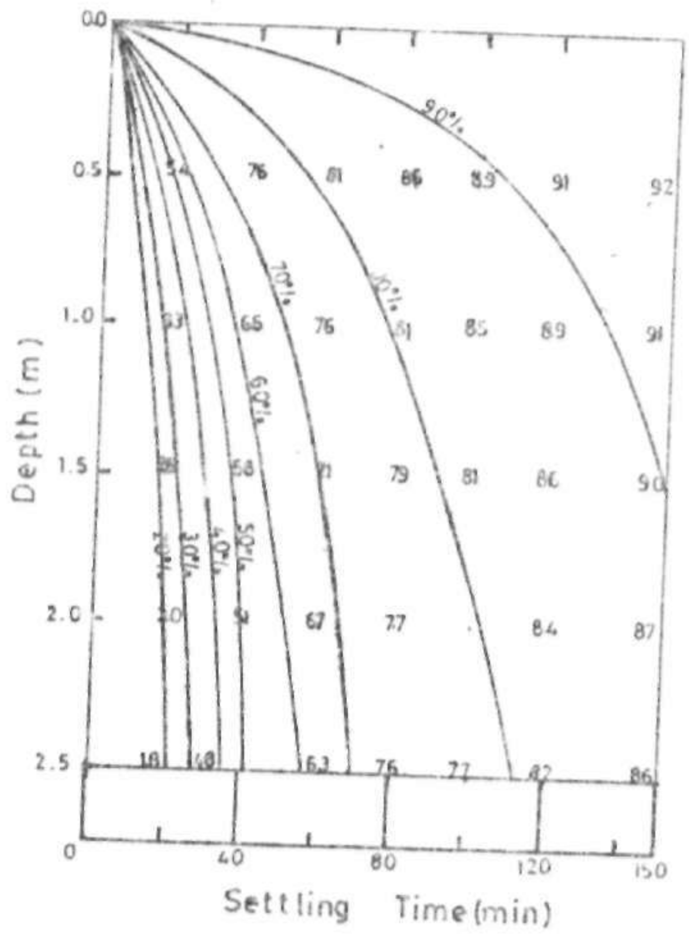


Figure 4.3(a) Settling Curves for Flocculent Particles.
(Dry Season Water Sample, April, 1988)

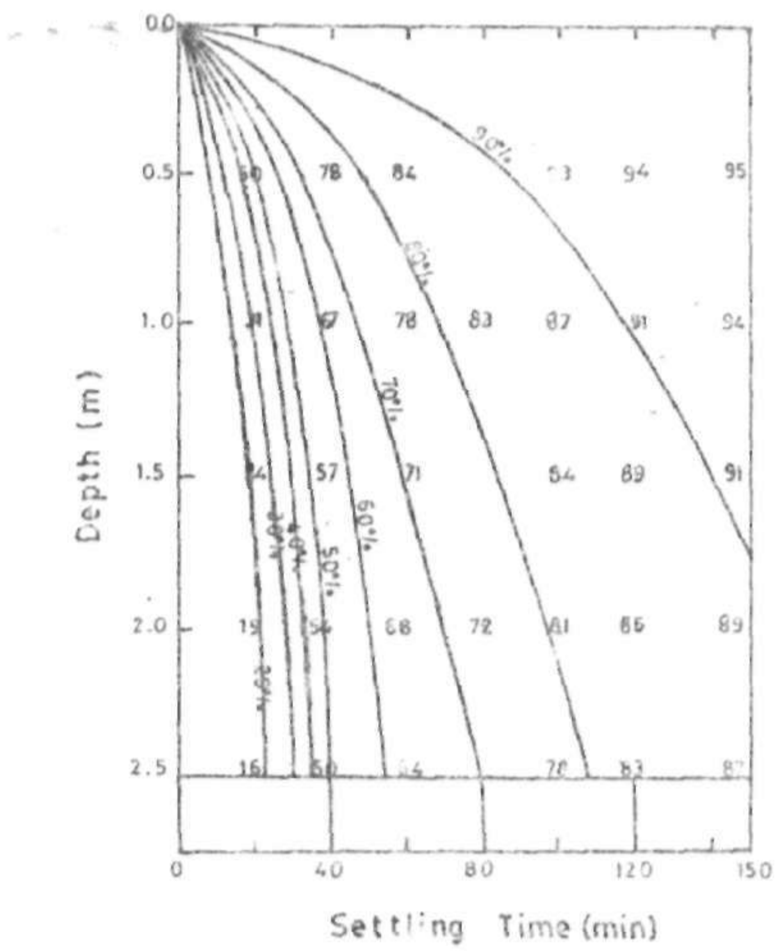


Figure 4.3(b) Settling Curves for Flocculent Particles.
(Rainy Season Water Samples, May, 1988)

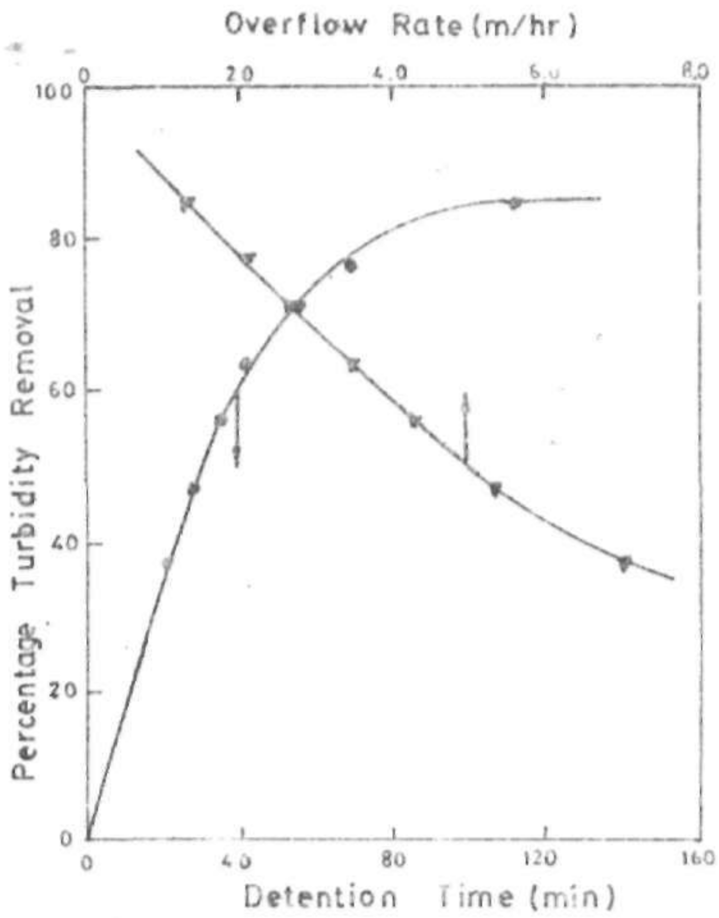


Figure 4.4(a) Detention Time / Overflow Rate Vs Percentage Turbidity Removal (Dry Season Water Samples)

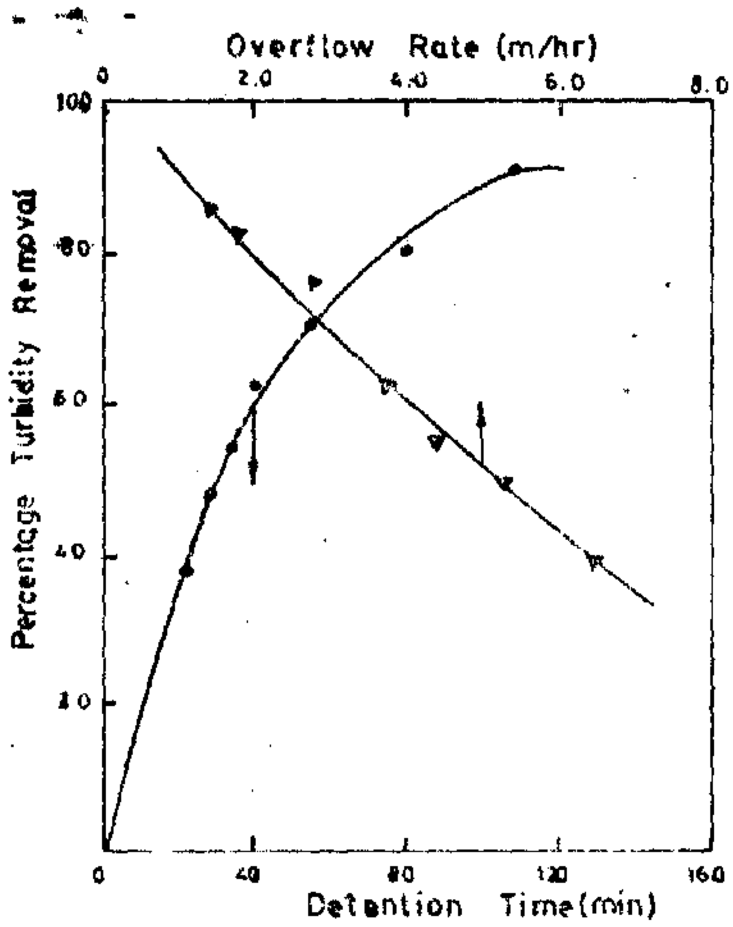


Figure 4.4(b) Detention Time/Overflow Rate Vs Percentage Turbidity Removal (Rainy Season Water Samples)

graphs are plotted, are contained in tables 4.4 (a) to (n) in appendix E. These data are the turbidities remaining at each sampling depth and the indicated time intervals. Tables 4.4 (f) and (m) contain the average values, while table 4.4 (g) and (n) contain the percentage turbidity removals computed using the average turbidity removals and the raw water turbidity.

4.2 Discussion of Results

4.2.1 Plain Sedimentation

Calculations for percentage turbidity removal from the curves in figure 4.1 indicate that no appreciable turbidity removal is achieved from plain sedimentation of the raw water. As an example, for an overflow rate of 1.5m/hr (common value used for settling basins), the turbidity removal is 29.5% from the curve in figure 4.1 (a), and 22% from the curve in figure 4.1 (b), giving an average removal of 26%. These removals are computed using the curves and equation 2.21. Details of the calculations are presented in appendix C. From these results, it is obvious that plain sedimentation is not adequate for the turbidity removal from the raw water of Kubanni dam at Ahmadu Bello University Campus, Zaria.

4.2.2 Coagulation and Flocculation Experiments

It was found that the optimum alum dose increased with increase in raw water turbidity. When the average raw water turbidity was 245NTU, the optimum alum dose was found to be 50mg/l during the dry season. However, during the rainy season, the raw water turbidity increased to an average of 697 NTU and the optimum alum dose was found to be 70mg/l. The optimum p^H value determined at the optimum alum dose of 70mg/l was found to be 7.5, which is within the optimum range (6.0 to 7.8) for effective alum coagulation.

4.2.3. Coagulated and Flocculated Samples Settling Experiments

Analysis of the results from the coagulated and flocculated sample settling experiments as presented in figure 4.4 indicates maximum turbidity removals of 85% during the dry season, and 86% during the rainy season, at detention times of 110 min. and 106 min. respectively. The corresponding overflow rates are 1.45m/hr and 1.4m/hr, respectively. Thus, an average of 85.5% turbidity removal was achieved for an average detention time of 108 min., and corresponding overflow rate of 1.425 m/hr.

It is evident from these results that coagulation and flocculation processes are necessary for the turbidity removal from the raw water of Kubanni dam at Ahmadu Bello University Campus, Zaria. For design purposes, the overflow rate and detention time obtained are incorporated with appropriate safety factors.

CHAPTER 5

DESIGN OF SEDIMENTATION TANKS

Based on the laboratory settling analysis data, it was concluded in chapter four that plain sedimentation will not be adequate for the turbidity removal from the Kubanni dam water. The design of the settling basins, therefore, will be based on the data from coagulated and flocculated settling experiments.

5.1 Calculation of Coagulant dosage

From the laboratory coagulation and flocculation experiments, during the dry season, optimum alum dose obtained was 50 mg/l, and during the rainy season, it was 70 mg/l.

Daily volumetric flow rate = 10560 m^3/day

Daily alum dosage required by the treatment plant during the dry season

$$= 10560 \times 10^3 \times 50$$

$$= 528 \times 10^6 \text{ mg}$$

$$= 528 \times \text{kg/day}$$

One bag of alum weighs 50 kg, therefore number of bags required

$$= \frac{528}{50}$$

$$= 10.56 \text{ or } 11 \text{ bags per day.}$$

Daily alum dosage required furing the rainy season

$$= 10560 \times 10^3 \times 70$$
$$= 739.2 \times 10^6 \text{ mg}$$
$$= 739 \text{ kg/day.}$$

Number of bags required

$$= \frac{739}{50}$$
$$= 14.78 \text{ or } 15 \text{ bags per day.}$$

5.2 Design of Tanks

From the experiments, for 85.5% turbidity removal

Surface overflow rate = 1.425/hr

Detention time = 108 min.

Depth of tank = 2.5m.

Assume safety factors of 1.5 for the surface overflow rate and 1.75 for detention time

Therefore, design data become

Surface overflow rate = 1.425/1.5
= 0.95m/hr

Detention time = 108 x 1.75
= 3 hrs

Assumed daily flow rate (based on ABU water treatment)

= $10560 m^3$/day.

Four units of sedimentation basins are to be provided.

Daily flow rate per unit $= 2640 m^3/day$

$$= \frac{\text{flow rate}}{\text{overflow rate}}$$

$$= \frac{2640}{24 \times 0.95}$$

$$= 116 m^2$$

Effective depth of basin $= \frac{\text{flow rate x detention time}}{\text{surface area}}$

$$= \frac{2640 \times 3}{24 \times 116}$$

$$= 3m.$$

5.2.1 Rectangular basin

Assume length-to-width (L:b) ratio to be 4:1

$$b = \left(\frac{116}{4}\right)^{\frac{1}{2}} = 5.39m$$

$$L = 5.39 \times 4 = 21.54m$$

Use $L = 21m$ and $b = 5.5m$

A longitudinal slope of 3% from the shallow outlet end toward the deeper inlet end is to be provided. Thus the depth will vary from 3m at the outlet end to 3.65m at the inlet end (figure 5.1).

Volume of basin $= 21 \times 5.5 \times 3 + \frac{1}{2}(21 \times 5.5 \times 0.65)$

$$= 346.5 + 37.5$$

$$= 384 m^3$$

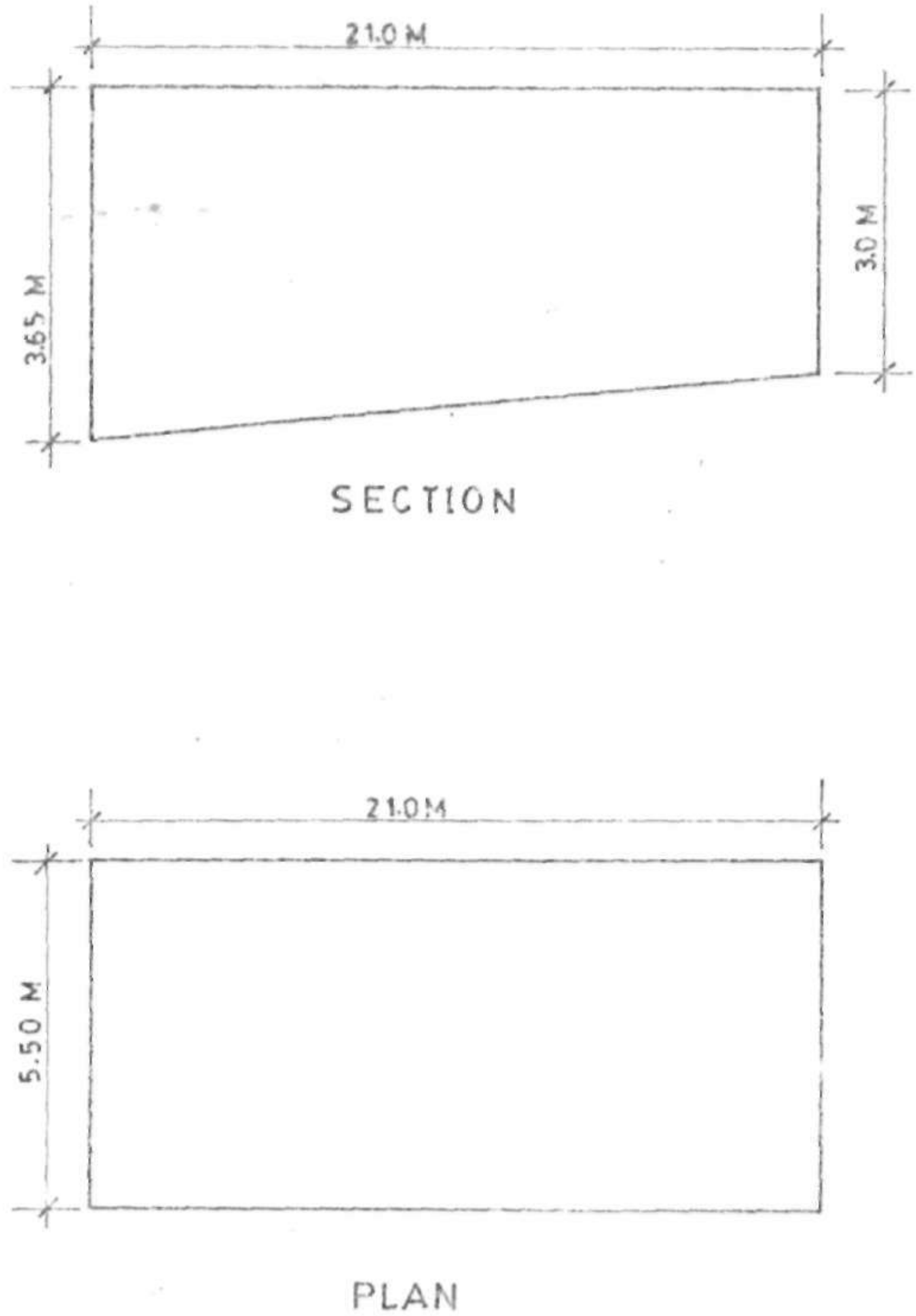


Figure 5.1 Rectangular Sedimentation Tank

$$\text{Detention time} = \frac{\text{Volume}}{\text{flow rate}}$$

$$= \frac{384 \times 24}{2640}$$

$$= 3.5 \text{ hrs.}$$

5.2.2 Circular basin

$$\text{Diameter of basin, D} = \left(\frac{116 \times 4}{\pi}\right)^{\frac{1}{2}}$$

$$= 12\text{m.}$$

A hopper bottom with angle of slope of 12° will be provided. Thus, the depth will vary from 3m at the periphery to 4.3m at the center (figure 5.2).

$$\text{Volume of basin} = \pi \times 6^2 \times 3 + \frac{\pi}{3}(6^2 \times 1.3 - 1.5^2 \times 0.3)$$

$$= 339 + 48$$

$$= 387\text{m}^3$$

$$\text{Detention time} = \frac{\text{Volume}}{\text{flow rate}}$$

$$= \frac{387 \times 24}{2640}$$

$$= 3.5 \text{ hrs.}$$

5.2.3 Upgrading of ABU Water Works Sedimentation basins

There are four units of square sedimentation basin in the ABU water treatment plant. The sketch of the basin is shown in figure 5.3, and the treatment plant layout is shown in appendix G. Field analysis of the raw and settled water samples during this study indicated that

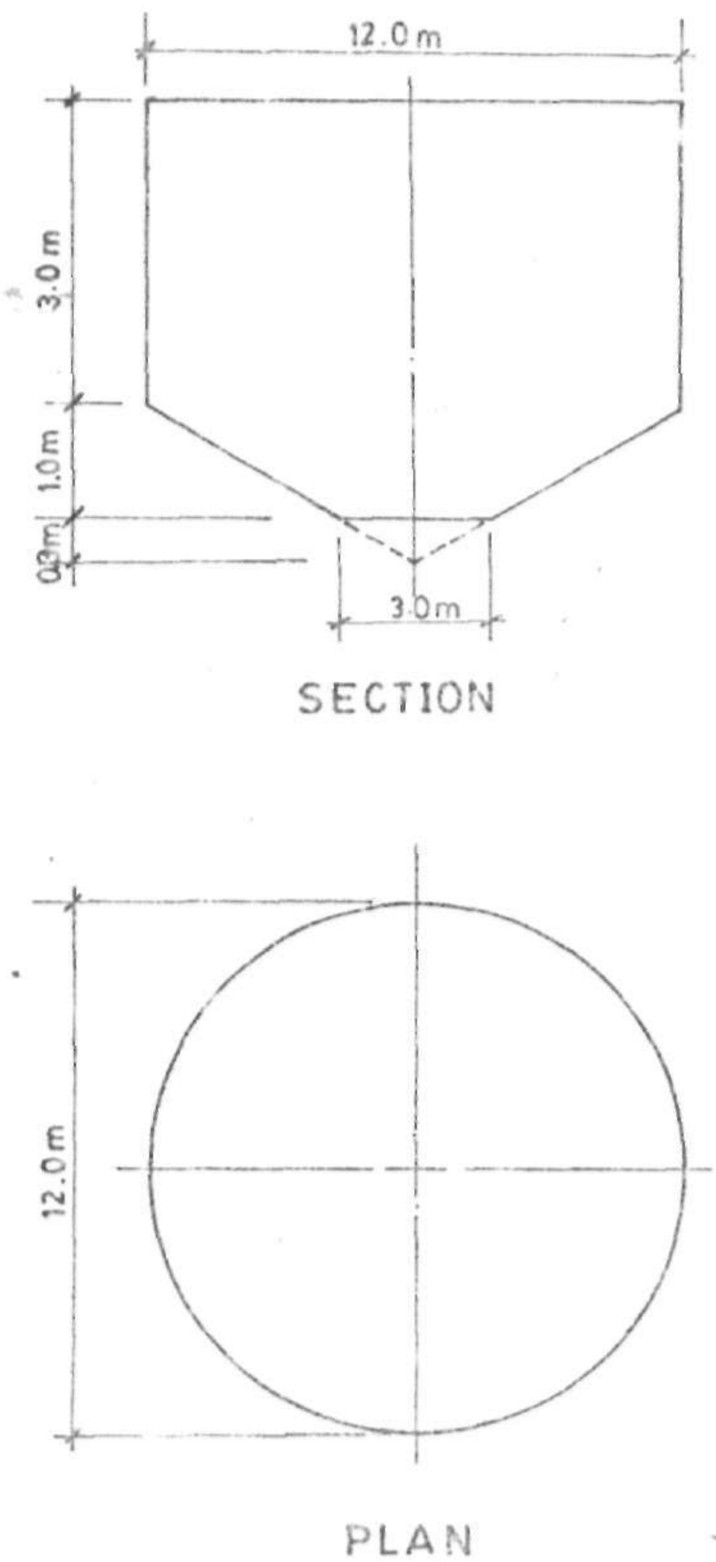


Figure 5.2 Circular Sedimentation Tank

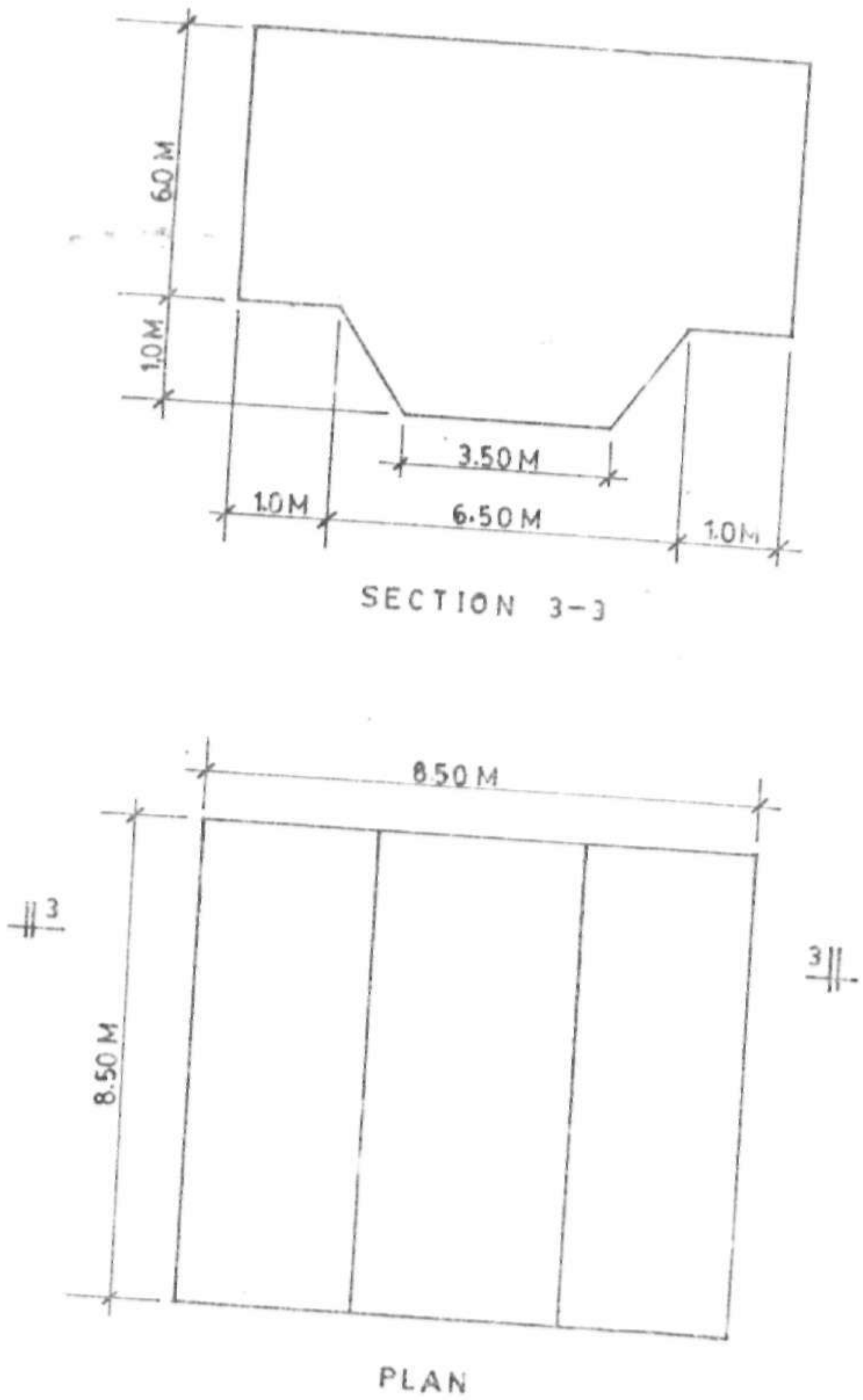


Figure 5.3 ABU Water Treatment Plant Sedimentation Basin

the basins provide an average efficiency of 70% turbidity removal. The analysis data is presented in appendix I.

The purpose of the upgrading is to achieve the 85.5% turbidity removal as obtained from the laboratory settling analysis using the same source of raw water. Since the dimensions of the basins are fixed, the upgrading can be achieved by using the surface overflow rate obtained from the settling analysis to adjust the volumetric flow rate into the basins. Detailed design parameters of the basins are as follows:

Surface area of basin	=	8.5mx8.5m
Effective depth of basin	=	5.0m
Volume of basin	=	$385m^3$
Daily flow rate	=	$2640m^3$/day
Surface overflow rate	=	1.5m/hr
Detention time	=	3.5 hrs
Upgrading overflow rate	=	0.95m/hr
Flow rate = 8.5^2x0.95	=	$69m^3$/hr
	=	$1656m^3$/day
Total flow rate	=	$10560m^3$/day
Number of basins required	=	$\frac{10560}{1656} = 6.3$

Six basins are to be provided. This means that two new basins will be constructed in addition to the existing four.

Flow rate to each basin $= \frac{10560}{6}$

$= 1760 m^3/day.$

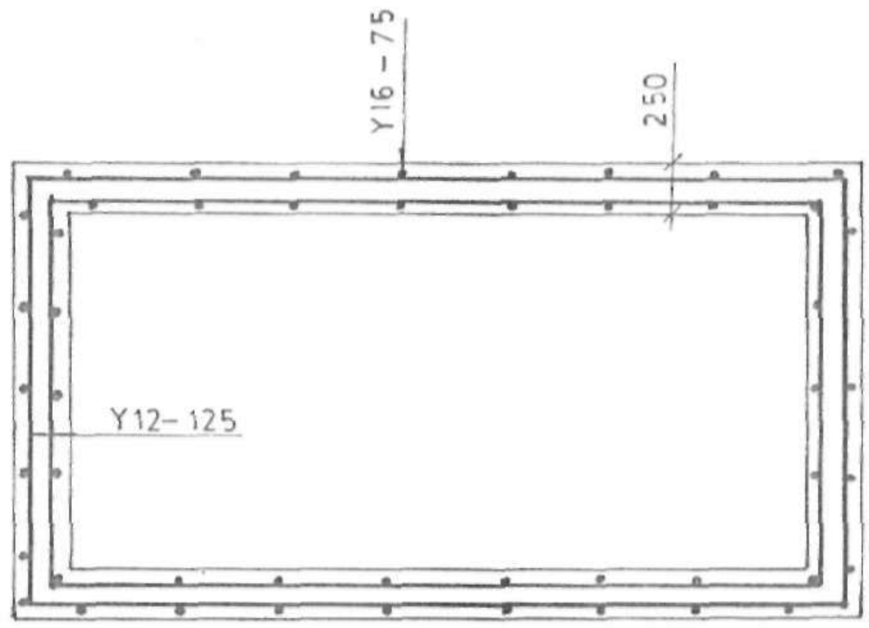
Overflow rate $= \frac{\text{flow rate}}{\text{surface area}} = \frac{1760}{8.5^2 \times 24} = 1.0 m/hr$

Detention time $= \frac{\text{Volume}}{\text{flow rate}} = \frac{335 \times 24}{1760}$

$= 5$ hrs.

5.3 Structural design of Sedimentation Tanks

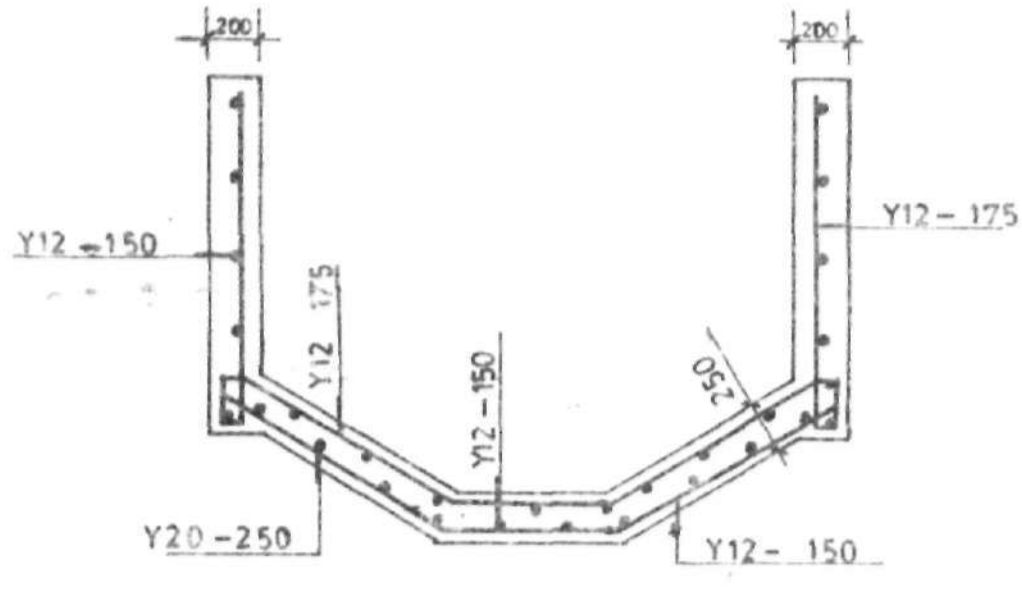
Reinforcement detailing for the rectangular, circular, and square tanks are shown in figures 5.4, 5.5, and 5.6, respectively. The detailed design calculations for the three sedimentation basins are presented in appendix H.



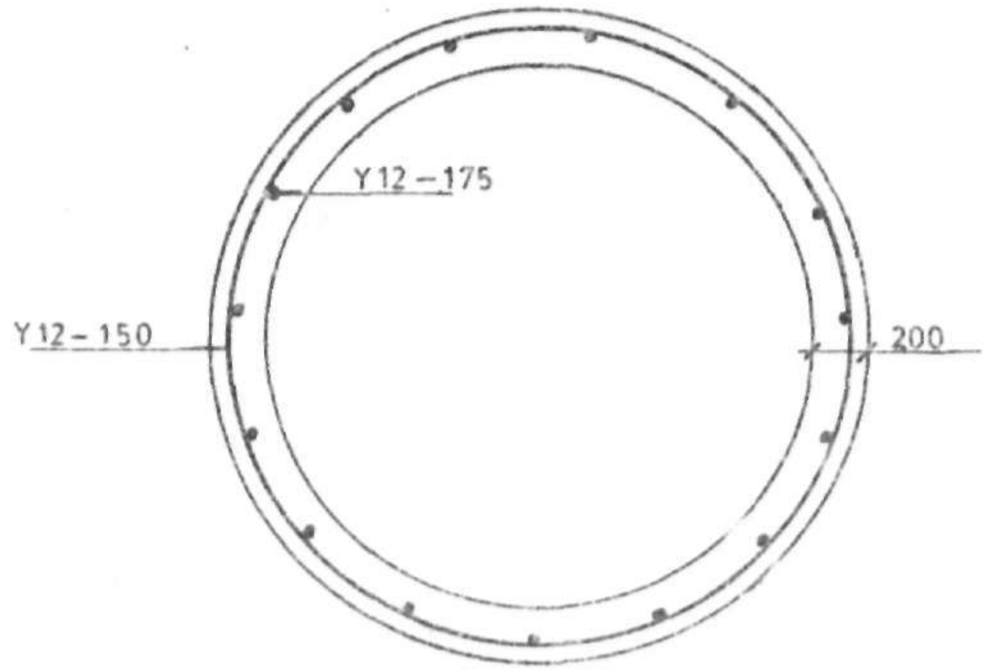
HORIZONTAL SECTION

VERTICAL SECTION

FIGURE 5.4 REINFORCEMENT DETAILS FOR RECTANGULAR TANK

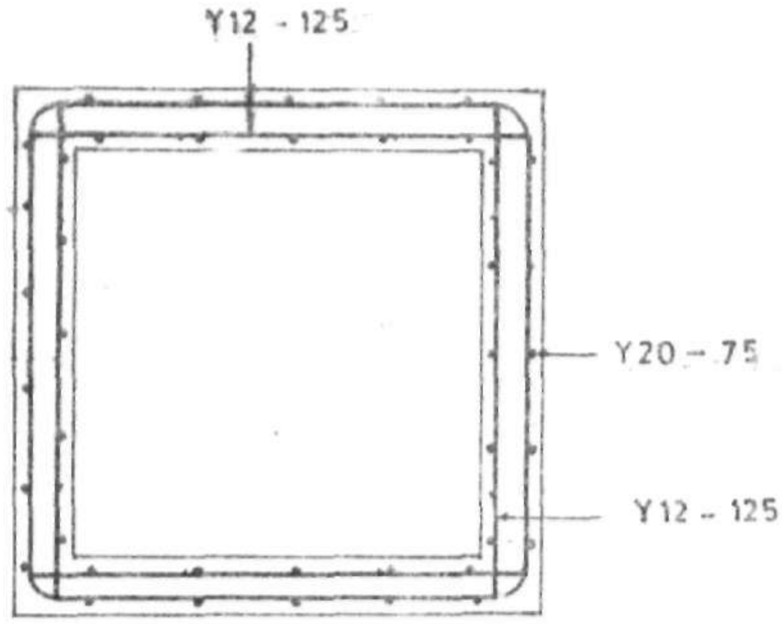


SECTION

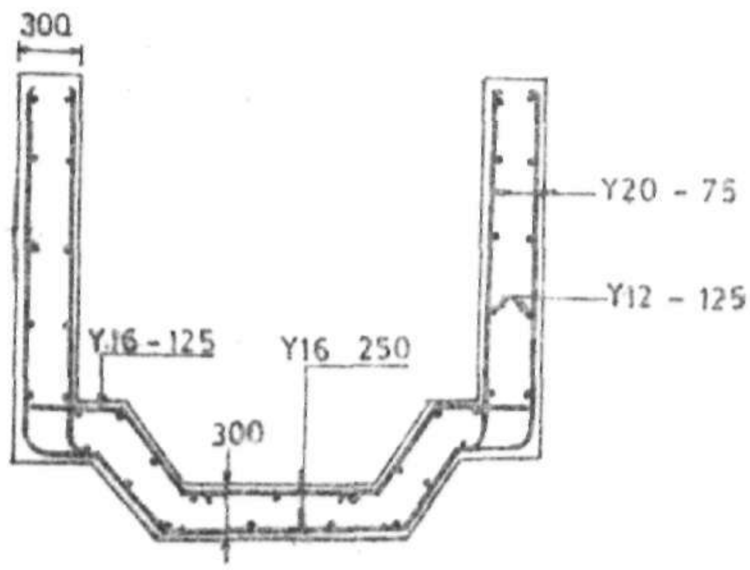


PLAN

Figure 5.5 Reinforcement Details For Circular Tank



HORIZONTAL SECTION



VERTICAL SECTION

Figure 5.6: Reinforcement Details for Square Tank

CHAPTER 6

COST ANALYSIS

6.1 Materials

The cost of any construction is made up of the cost of materials, plant and labour. For the construction of reinforced concrete sedimentation tanks, the two basic materials required are steel reinforcement bars and concrete. Only the cost of these two materials are estimated herein.

Unlike steel bars which can be obtained from the local distributor, concrete has to be produced on site, and the production requires some degree of control in order to obtain the desired quality of both fresh and hardened concrete.

6.1.1 Unit Cost of Concrete

The components of concrete are cement, fine and coarse aggregates, and water. The volume of fully compacted concrete is obtained by a simple calculation using the absolute volume method, which assumes that the volume of the compacted concrete is equal to the sum of the absolute volumes of all components. It is usual to calculate the quantities of components to produce one cubic meter of concrete, Neville (1981), and Dhir and Jackson (1980).

Thus, if W, C^1, A, and A_2 are the required weights of water, cement, fine agrregate, and coarse aggregate, recpectively, for the cubic meter of fresh concrete, we have

$$\frac{W}{\rho} + \frac{C^1}{\rho\gamma_c} + \frac{A}{\rho\gamma_a} = 1 \qquad 7.1$$

where ρ is the mass density of water, γ with the appropriate suffix represents the specific gravity of each material. $A = A_1+A_2$, $\gamma_c = 3.15$, and $\gamma_a = 2.55$.

Using a minimum coarse aggregates of size 20mm, ordinary portland cement, and concrete mix design of 1:2:4, $A/C^1 = 6$, and $W/C^1 = 0.5$. This design has been found to give average quality concrete with minimum site supervision. From equation 7.1

$$C^1 = \frac{1}{\frac{0.5}{1000} + \frac{1}{3150} + \frac{6}{2550}} = 315 \text{ kg}$$

Thus;

$$W = 0.5 \times 315 = 158 \text{ kg}$$

$$A_1 = 2 \times 315 = 630 \text{ kg}$$

$$A_2 = 4 \times 315 = 1260 \text{ kg}$$

In the local market

Cost of one bag (50kg) of cement	= N20.00
Cost per kg of fine aggregate	= N 0.02
Cost per kg of coarse aggregate	= N 0.065
Total cost of cement = $\frac{315}{50} \times 20$	= N126.00

Total cost of fine aggregate = = 630x0.02 = ₦12.60

Total cost of coarse aggregate = 1260x0.065 = ₦81.90

Therefore, cost of one cubic meter of concrete = ₦220.50

6.1.2 Unit Cost of steel bars

High tensile steel bars are sold in lengths of 12m, the quoted unit prices are for this length.

Cost per length of 12mm steel bars = ₦25.00

Cost per length of 16mm steel bars = ₦42.00

Cost per length of 20mm steel bars = ₦75.00

The unit costs of steel and concrete are used for the calculation of total cost estimates of these materials for the rectangular, circular, and square tanks, and presented in tables 6.1, 6.2, and 6.3, respectively.

Table 6.1: Estimate cost of steel and concrete for rectangular tank

DESCRIPTION	UNIT OF MEASURE	QUANTITY	UNIT PRICE ₦	TOTAL ₦
12mm steel bars	m	4178	25.00	8,704.17
16mm steel bars	m	3460	42.00	12,110.00
20mm steel bars	m	1570	75.00	9,812.50
Sub-total				30,626.67
Concrete	m^3	100	220.50	22,050.00
Grand total				52,676.67

Table 6.2: Estimate Cost of steel and concrete for circular tank

DESCRIPTION	UNIT OF MEASURE	QUANTITY	UNIT PRICE ₦	TOTAL ₦
12mm steel bars	m	4840	25.00	10,083.33
20mm steel bars	m	4085	75.00	25,531.25
Sub-total				35,614.58
Concrete	m^3	72.44	220.50	15,973.02
Grand total				51,587.60

Table 6.3: Estimate Cost of steel and concrete for square tank

DESCRIPTION	UNIT OF MEASURE	QUANTITY	UNIT PRICE ₦	TOTAL ₦
12mm steels bars	m	3860	25.00	8,041.67
20mm steel bars	m	6097	75.00	38,106.25
Sub-total				46,147.92
Concrete	m^3	101.61	220.50	22,405.01
Grand total				68,552.93

CHAPTER 7

SUMMARY AND CONCLUSION

From the study the following summary and conclusions are drawn.

Different suspensions have different settling characteristics. Consideration of the character of the suspended matter to be removed from a suspension is fundamental to the process of sedimentation.

The varied nature of settling characteristics of suspensions makes necessary the conduct of settling experiments for a particular water or wastewater for the purpose of new plant design and improvement in the performance of existing plants.

Results obtained from the study indicate that plain sedimentation will not be adequate for the treatment of the water of Kubanni dam at ABU main Campus, Zaria. Coagulation and flocculation is necessary.

Results obtained also indicate that an optimum alum dose of 50mg/l is required for the coagulation of the raw water of Kubanni dam during the dry season of the year, 70mg/l is required furing the rainy season. Field investigation showed that ABU water works uses an average alum dose of 27mg/l during the dry season and an average of 36mg/l during the rainy season.

A maximum turbidity removal of 85.5 percent was obtained from the laboratory settling experiments at an overflow rate of 1.425m/hr and detention time of 108 minutes. Field investigation during the study showed that the average treatment efficiency of AEU waterworks sedimentation basins is 70 percent turbidity removal.

The data obtained from the study are used for the design of new sedimentation basins and for up-grading of the existing sedimentation basins of the University water works.

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APPENDIX A

Table 4.1 Turbidity data for plain sedimentation at indicated depths and times. Dry season water samples, April, 1988.

(a) Raw water turbidity = 268 NTU

Depth (m)	Time (Min.)						
	20	40	60	80	100	120	150
0.50	220	202	189	177	169	158	151
1.00	245	221	204	190	184	177	163
1.50	255	234	220	208	200	198	188
2.00	257	243	227	217	208	200	194
2.50	260	252	236	225	214	205	196

(b) Raw water turbidity = 262 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	214	193	177	167	158	149	140
1.00	235	211	194	180	171	167	155
1.50	249	225	209	199	180	185	175
2.00	251	232	221	208	199	193	183
2.50	254	242	232	217	203	196	186

(c) Raw water turbidity = 245 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	202	184	170	158	151	136	133
1.00	226	200	187	169	162	156	150
1.50	236	197	189	179	176	176	167
2.00	238	223	210	197	188	182	174
2.50	240	233	223	205	195	184	178

(d) Raw water turbidity = 250 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	210	191	181	165	160	145	142
1.00	237	209	196	178	173	165	157
1.50	243	221	205	195	190	183	179
2.00	245	234	219	206	201	191	185
2.50	248	244	228	213	207	194	189

(e) Raw water turbidity = 256 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	213	191	178	165	158	143	14[illegible]
1.00	237	209	194	178	170	165	155
1.50	247	202	204	196	188	184	176
2.00	249	232	219	206	197	191	183
2.50	252	243	230	215	204	196	186

(f) Average raw water turbidity = 256 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	212	192	179	166	159	148	141
1.00	236	210	195	179	172	166	156
1.50	246	223	207	197	189	184	177
2.00	248	233	220	207	200	192	184
2.50	251	243	230	215	205	195	187

(g) Average turbidity as fraction of raw water value

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	0.83	0.75	0.70	0.65	0.62	0.58	0.55
1.00	0.92	0.82	0.76	0.70	0.67	0.65	0.61
1.50	0.96	0.87	0.81	0.77	0.74	0.72	0.69
2.00	0.97	0.91	0.86	0.81	0.78	0.75	0.72
2.50	0.98	0.95	0.90	0.84	0.80	0.76	0.73

Rainy Season water samples, May, 1988

(h) Raw water turbidity = 680 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	600	564	531	517	500	475	460
1.00	650	600	568	556	542	522	501
1.50	663	638	6[illegible]9	585	562	551	542
2.00	670	654	630	600	590	568	557
2.50	680	665	647	625	613	598	581

(i) Raw water turbidity = 665 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	587	556	523	506	489	467	452
1.00	640	594	561	548	539	512	490
1.50	655	628	604	574	558	539	531
2.00	660	641	624	596	584	558	545
2.50	665	655	635	614	604	586	564

(j) Raw water turbidity = 668

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	593	553	520	506	484	466	452
1.00	642	596	559	548	534	515	496
1.50	657	630	603	573	556	541	536
2.00	663	643	623	596	583	561	550
2.50	668	657	635	612	600	588	568

(k) Raw water turbidity = 637

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	591	554	521	510	487	469	458
1.00	644	596	563	550	531	518	450
1.50	657	630	600	576	552	545	436
2.00	663	643	623	596	583	561	550
2.50	673	657	637	616	600	592	574

(1) Raw water turbidity = 660 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	586	549	516	502	480	464	451
1.00	635	590	560	543	529	508	488
1.50	649	620	595	568	548	535	528
2.00	654	630	614	590	574	556	541
2.50	660	649	627	609	594	582	559

(m) Average raw water turbidity = 669 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	589	555	522	508	488	468	455
1.00	642	595	562	549	535	515	495
1.50	656	627	602	575	555	542	535
2.00	662	642	622	595	582	562	549
2.50	669	656	636	615	607	589	569

(n) Average turbidity as fraction of raw water value

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	0.88	0.83	0.78	0.76	0.73	0.70	0.6[illegible]
1.00	0.96	0.89	0.84	0.82	0.80	0.77	0.74
1.50	0.98	0.94	0.90	0.86	0.83	0.81	0.80
2.00	0.99	0.96	0.73	0.89	0.87	0.84	0.82
2.50	1.00	0.98	0.95	0.92	0.90	0.88	0.85

APPENDIX B

Table 4.2: Plotting data for plain sedimentation

(a) Dry Season Water Samples, April, 1988

Time (Min.)	Settling velocity (m/hr.)	Fraction of turbidiy due to particles with velocity less than indicated
	(0.50m depth)	
20	1.50	0.83
40	0.75	0.75
60	0.50	0.70
80	0.38	0.65
100	0.30	0.62
120	0.25	0.58
150	0.20	0.55
	(1.0m depth)	
20	3.00	0.92
40	1.50	0.82
60	1.00	0.76
80	0.75	0.70
100	0.60	0.67
120	0.50	0.65
150	0.40	0.61

	(1.5m depth)	
20	4.50	0.96
40	2.25	0.87
60	1.50	0.81
80	1.13	0.77
100	0.90	0.74
120	0.75	0.72
150	0.60	0.69
	(2.0m depth)	
20	6.00	0.97
40	3.00	0.91
60	2.00	0.86
80	1.50	0.81
100	1.20	0.78
120	1.00	0.75
150	0.80	0.72
	(2.50m depth)	
20	7.50	0.98
40	3.75	0.95
60	2.50	0.90
80	1.88	0.84
100	1.50	0.80
120	1.25	0.76
150	1.00	0.73

(b) Rainy Season Water Samples, May, 1988

Time (Min.)	Settling Velocity (m/hr.)	Fraction of turbidity due to particles with velocity less than stated
	(0.5m depth)	
20	1.50	0.88
40	0.75	0.83
60	0.50	0.78
80	0.38	0.76
100	0.30	0.73
120	0.25	0.70
150	0.20	0.68
	(1.0m depth)	
20	3.00	0.96
40	1.50	0.89
60	1.00	0.84
80	0.75	0.82
100	0.60	0.80
120	0.50	0.77
150	0.40	0.74

	(1.50m depth)	
20	4.50	0.98
40	2.25	0.94
60	1.50	0.90
80	1.13	0.86
100	0.90	0.83
120	0.75	0.81
150	0.60	0.80
	(2.0m depth)	
20	6.00	0.99
40	3.00	0.96
60	2.00	0.93
80	1.50	0.89
100	1.20	0.87
120	1.00	0.84
150	0.80	0.82
	(2.5m depth)	
20	7.50	1.00
40	3.75	0.98
60	2.50	0.95
80	1.88	0.92
100	1.50	0.90
120	1.25	0.88
150	1.00	0.85

APPENDIX C

Turbidity Removal Calculations for Plain Sedimentation

(a) Dry season water samples

Assumed overflow rate, $V_o = 1.5$m/hr

From figure 5.1, fraction of turbidity, C_o, with Velocities $V_s < V_o$ is $C_o = 0.825$.

Using equation 3.10, the total removal is given by

$$R = (1 - C_o) + \frac{1}{V_o} \int_0^{C_o} V_s dc$$

$$= (1-0.825) + \frac{1}{1.5} \int_0^{0.825} V_s dc$$

Graphical integration of the area under the curve in figure 5.1 yields

$$\int_0^{0.825} V_s dc = 0.18$$

$$\therefore \quad R = (1-0.825) + \frac{1}{1.5}(0.18)$$

$$= 0.175 + 0.12 = 0.295$$

$$= 29.5\% \text{ removal}$$

(b) Rainy season water samples

Assumed overflow rate, $V_o = 1.5$m/hr

From figure 5.2, fraction of turbidity, C_o, with Velocities $V_s < V_o$ is $C_o = 0.895$

Using equation 3.10, total removal is

$$R = (1 - Co) + \frac{1}{Vo}\int_{o}^{Co} Vs\,dc$$

$$= (1-0.895) + \frac{1}{1.5}\int_{o}^{0.895} Vsdc$$

Graphical integration of the area under the curve in figure 5.2 yields

$$\int_{o}^{0.895} Vsdc = 0.172$$

$$\therefore R = (1-0;895) + \frac{1}{1.5}(0.172)$$

$$= 0.105 + 0.115 = 0.22$$

$$= 22\% \text{ removal}$$

Average removal from the two calculations in

$(29.5+22)/2 = 25.75$ or 26%.

APPENDIX D

Table 4.3: Jar test results

(a) Optimum alum dose for dry season water samples.
April, 1988 (raw water turbidity = 250 NTU)

Alum dose (mg/l)	10	20	30	40	50	60	70	80
Turbidity (NTU)	220	80	40	30	15	25	28	32

(b) Optimum alum dose for rainy season water samples
May, 1988 (raw water turbidity = 670 NTU)

Alum dose (mg/l)	10	20	30	40	50	60	70	80	90	100
Turbidity (NTU)	600	208	80	50	40	25	15	35	45	50

(c) Optimum alum dose for rainy season water samples. July, 1988 (raw water turbidity = 750 NTU)

Alum dose (mg/l)	10	20	30	40	50	60	70	80	90	100
Turbidity (NTU)	700	440	100	60	40	35	20	30	35	45

(d) Optimum p^H at optimum alum dose of 70mg/l for rainy season water samples (raw water turbidity = 780 NTU)

p^H	5.5	6.0	6.5	7.0	7.5	8.0	8.5
Turbidity (NTU)	120	60	40	35	20	30	40

APPENDIX E

Table 4.4: Turbidity data for flocculent settling at indicated depths and times.
Dry season water samples, April, 1988.

(a) Raw water turbidity = 260 NTU

Depth (m)	Time (Min.)						
0.50	20	40	60	80	100	120	150
1.00	110	59	50	39	30	25	23
1.50	176	101	71	53	49	37	27
2.00	192	119	81	55	52	40	34
2.50	196	126	88	61	58	48	37

(b) Raw water turbidity = 259 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	106	58	48	57	20	23	21
1.00	157	83	59	49	39	30	23
1.50	170	99	70	52	46	35	24
2.00	185	116	81	56	51	39	32
2.50	189	124	86	59	56	45	33

(c) Raw water turbidity = 220 NTU

Depth (m)	Time (Min.)						
	20	40	60	80	100	120	150
0.50	104	53	43	32	23	18	16
1.00	150	76	51	41	30	21	17
1.50	169	94	64	46	40	28	20
2.00	180	109	72	50	46	34	26
2.50	185	117	80	52	49	38	28

(d) Raw water turbidity = 231 NTU

Depth (m)	20	40	60	80	100	120	150
0.50	108	54	44	33	24	19	17
1.00	153	79	54	43	31	22	19
1.50	174	95	64	46	42	29	22
2.00	187	112	73	50	47	35	28
2.50	191	118	82	53	50	39	31

(e) Raw water turbidity = 255 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	107	56	45	43	25	20	18
1.00	156	82	57	46	36	26	21
1.50	171	96	66	48	43	31	22
2.00	186	114	78	54	49	37	30
2.50	189	120	84	55	52	41	31

(f) Average raw water turbidity = 245 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	107	56	46	35	26	21	19
1.00	155	81	56	46	35	26	21
1.50	171	97	67	49	44	32	23
2.00	186	114	77	53	49	37	30
2.50	190	121	84	56	53	42	32

(g) Average % turbidity removal

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	54	76	81	86	89	91	92
1.00	33	65	76	81	85	89	91
1.50	26	53	71	79	81	86	90
2.00	20	51	67	77	79	84	87
2.50	18	48	63	76	77	82	86

Rainy season water samples, May, 1988

(h) Raw water turbidity = 660 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	322	142	102	70	44	37	31
1.00	446	213	143	111	85	61	40
1.50	492	281	190	139	105	75	62
2.00	522	300	209	184	128	100	74
2.50	546	325	197	236	147	112	86

(i) Raw water turbidity = 641 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	325	140	105	73	49	40	31
1.00	443	215	141	112	87	59	40
1.50	489	276	189	133	104	72	57
2.00	523	299	205	182	125	99	70
2.50	541	325	233	195	141	112	85

(j) Raw water turbidity = 636 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	324	143	104	70	45	39	33
1.00	444	212	141	109	84	57	38
1.50	489	276	186	134	104	71	57
2.00	524	296	206	180	120	96	70
2.50	542	323	232	194	140	110	84

(k) Raw water turbidity

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	321	144	101	69	41	38	33
1.00	447	211	143	108	81	57	38
1.50	491	278	185	137	102	70	59
2.00	521	295	207	180	121	95	72
2.50	543	321	231	193	143	108	83

(l) Raw water turbidity = 640 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	323	141	103	73	46	41	32
1.00	445	214	142	110	83	56	39
1.50	489	274	185	132	100	69	55
2.00	520	295	203	179	121	95	69
2.50	538	321	228	191	139	108	82

(m) Average raw water turbidity = 645 NTU

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	323	142	103	71	45	39	32
1.00	445	213	142	110	84	58	39
1.50	490	277	187	135	103	71	58
2.00	522	297	206	181	123	97	71
2.50	542	323	232	194	142	110	84

(n) Average % turbidity removal

Depth (m)	Time (Min)						
	20	40	60	80	100	120	150
0.50	50	78	84	89	93	94	95
1.00	31	67	78	83	87	91	94
1.50	24	57	71	79	84	89	91
2.00	19	54	68	72	81	85	89
2.50	16	50	64	70	78	83	87

APPENDIX F

Table 4.5. Design calculations for 2.5m tank

(a) Dry Season water samples, April, 1988

Time (Min)	% Turbidity Range	Av.Depth of range (m)	Vs (m/hr)	Vo (m/hr)	$\frac{V_s}{V_o}$	% Turbidity Removal
21.2	0-20	2.50	7.08	7.08	1.00	20.00
	20-30	1.77	5.01		0.71	7.10
	30-40	1.00	2.83		0.40	4.00
	40-50	0.70	1.98		0.28	2.80
	50-60	0.50	1.42		0.20	2.00
	60-70	0.30	0.85		0.12	1.20
	70-80	0.15	0.42		0.06	0.60
	80-90	0.05	0.14		0.02	0.20
			Total Removal			37.90
28	0-30	2.50	5.36	5.36	1.00	30.00
	30-40	1.77	3.79		0.71	7.10
	40-50	1.07	2.29		0.43	4.30
	50-60	0.75	1.61		0.30	3.00
	60-70	0.47	1.01		0.19	1.90
	70-80	0.23	0.49		0.09	0.90
	80-90	0.08	0.17		0.03	0.30
			Total Removal			47.50

34.6	0-40	2.50	4.35	4.35	1.00	40.00
	40-50	1.77	3.07		0.71	7.10
	50-60	1.10	1.91		0.44	4.40
	60-70	0.67	1.16		0.27	2.70
	70-80	0.33	0.57		0.13	1.30
	80-90	0.12	0.21		0.05	0.50
			Total Removsl			56.00
42.2	0-50	2.50	3.55	3.55	1.00	50.00
	50-60	1.77	2.52		0.71	7.10
	60-70	1.03	1.46		0.41	4.10
	70-80	0.47	0.67		0.19	1.90
	80-90	0.17	0.24		0.07	0.70
			Total Removal			63.80
56	0-60	2.50	2.68	2.68	1.00	60.00
	60-70	1.83	1.96		0.73	7.30
	70-80	0.83	0.89		0.33	3.30
	80-90	0.27	0.29		0.11	1.10
			Total Removal			71.70

70.8	0-70	2.50	2.12	2.12	1.00	70.00
	70-80	1.33	1.13		0.53	5.30
	80-90	0.40	0.43		0.16	1.60
			Total Removal			71.70
112	0-80	2.50	1.43	1.34	1.00	80.00
	80-90	1.37	0.73			5.40
			Total Removal			85.40

Table 4.5 (b) Rainy season water samples, May, 1988

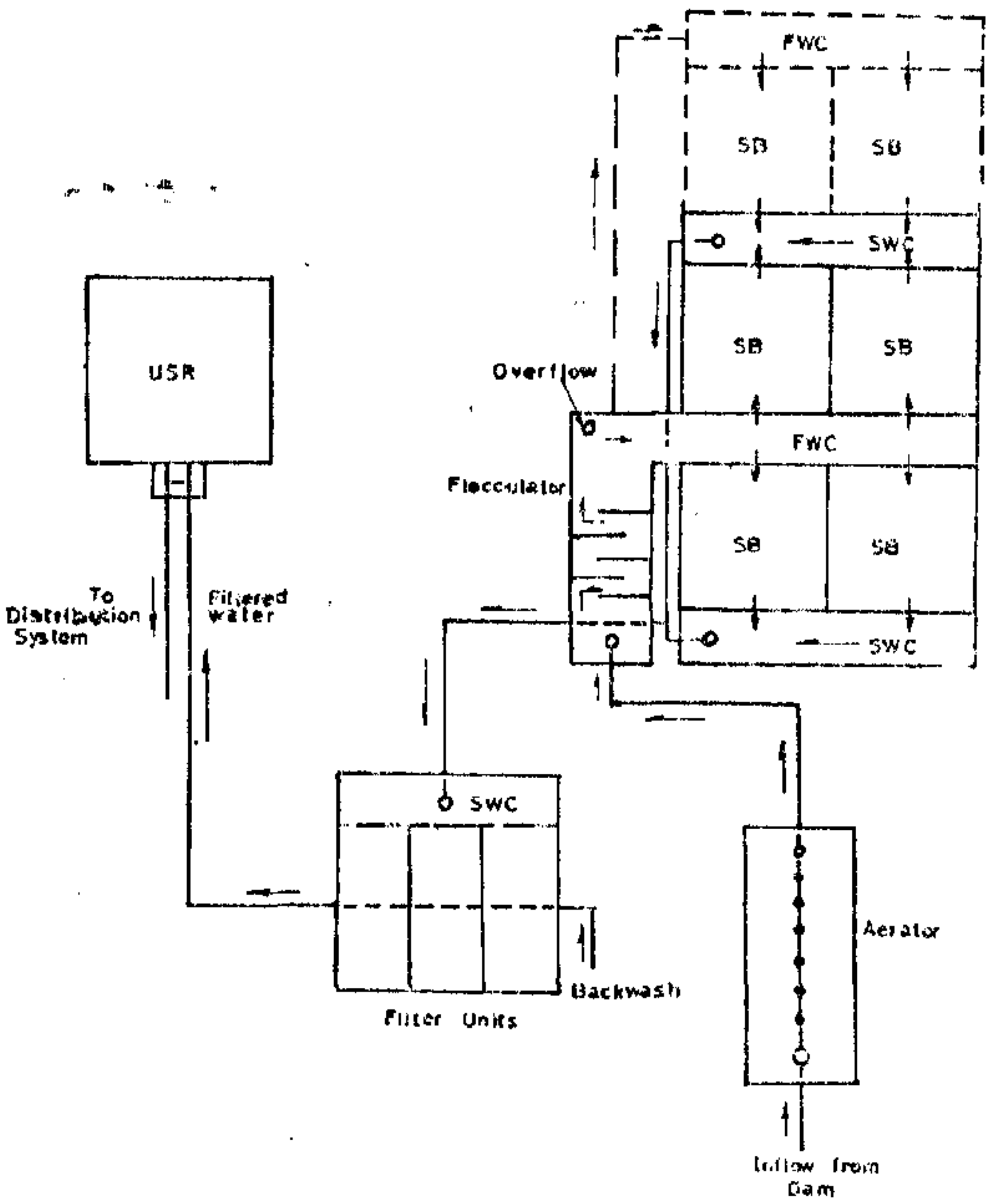
Time (Min)	% Turbidity Range	Av.Depth of Range (m)	V_s (m/hr)	Vo (m/	$\frac{V_s}{V_o}$	% Turbidity Removal
22.4	0-20	2.50	6.70	6.70	1.00	20.00
	20-30	1.83	4.90		0.73	7.30
	30-40	1.07	2.87		0.43	4.30
	40-50	0.70	1.88		0.28	2.80
	50-60	0.47	1.26		0.19	1.90
	60-70	0.32	0.86		0.13	1.30
	70-80	0.20	0.54		0.08	0.80
	80-90	0.10	0.27		0.04	0.40
			Total Removal			38.80

28.2	0-30	2.50	5.32	5.32	1.00	30.00
	30-40	1.80	3.83		0.72	7.20
	40-50	1.23	2.62		0.49	4.90
	50-60	0.73	1.55		0.29	2.90
	60-70	0.47	1.00		0.19	1.90
	70-80	0.30	0.64		0.12	1.20
	80-90	0.17	0.36		0.07	0.70
			Total Removal			48.80
34.2	0-40	2.50	4.39	4.39	1.00	40.00
	40-50	1.57	2.75		0.63	6.30
	50-60	1.00	1.75		0.40	4.00
	60-70	0.60	1.05		0.24	2.40
	70-80	0.37	0.65		0.15	1.50
	80-90	0.17	0.30		0.07	0.70
			Total Removal			54.90
40	0-50	2.50	3.75	3.75	1.00	50.00
	50-60	1.50	2.25		0.60	6.00
	60-70	0.87	1.31		0.35	3.50
	70-80	0.50	0.75		0.20	2.00
	80-90	0.23	0.35		0.09	0.90
			Total Removal			62.40

54.3	0-60	2.50	2.76	2.76	1.00	50.00
	60-70	1.67	1.85		0.67	6.70
	70-80	0.83	0.92		0.33	3.30
	80-90	0.23	0.25		0.09	0.90
			Total Removal			70.90
80	0-70	2.50	1.88	1.88	1.00	70.00
	70-80	1.83	1.37		0.73	7.30
	80-90	0.73	0.55		0.29	2.90
			Total Removal			80.20
108	0-80	2.50	1.39	1.39	1.00	80.00
	80-90	1.50	0.83		0.63	6.00
			Total Removal			86.00

APPENDIX G

A.B.U. WATER TREATMENT PLANT LAYOUT



FWC = FLOCCULATED WATER CHANNEL
SWC = SETTLED WATER CHANNEL
SB = SEDIMENTATION BASIN
USR = UNDERGROUND SERVICE RESERVOIR
——— DIRECTION OF FLOW
— — — PROPOSED ADDITION

APPENDIX H

Structural Design Calculations for Sedimentation Tanks

Exposure Class B, Minimum Cover to Steel reinforcement = 40mm, Concrete grade 25, reinforcement strength 425.

Rectangular Tank

Tank dimension: 21 x 5.5 x 4.65m

Average water depth: 3.3m

Walls

Bending moment, $M^1 = 9.81 \times 3.3^3 = 59$ KNm/m

Consider thickness of wall. $h^1 = 250$mm

Effective depth, $d^1 = 250-50 = 200$mm

(a) Ultimate Limit State

$$\frac{Mu}{b^1 d^{12} fcu} = \frac{1.6 \times 59 \times 10^6}{1000 \times 200^2 \times 25} = 0.094$$

$$Z/d^1 = 0.864 \text{ (from std table)}$$

$$Z = 200 \times 0.864 = 172.8\text{mm}$$

$$A_{s1} = \frac{M_u}{0.87 f_y Z} = \frac{1.6 \times 59 \times 10^6}{0.87 \times 425 \times 172.8} = 1477\text{mm}^2$$

Provide Y20-200 (1570mm^2)

(b) Serviceability Limit State

$$\frac{100A_{s1}}{b^1d^1} = \frac{100 \times 1570}{1000 \times 200} = 0.78$$

$X/d^1 = 0.381$ (from std table)

$X = 0.381 \times 200 = 76.2mm$

Reinforcement tensile stress, fs is

$$fs = \frac{M^1}{As_1\,(d^1 - \frac{x}{3})} = \frac{59 \times 10^6}{1570(200-\frac{76}{3}} = 215N/mm^2$$

which is greater than the $130N/mm^2$ allowable for a Class 3 member and steel area must be increased.

Minimum $As_1 = \frac{215}{130}x1570 = 2597mm^2$

Provide Y16-75 ($2680mm^2$) main steel

Provide Y12-125($905mm^2$) distribution steel

Floor

$h^1 = 300mm$, $d^1 = 300-50 = 250mm$

Floor to be divided into four panels of 5.5x5.3 by supporting beams.

Weight of slab + water = 0.3x24+9.81x3.3

= 39KNm/m

Over the support beams, $M^1 = 0.1x39x5.3^2$

= 109KNm/m

At mid-span, M^1 $= 0.08x39x5.3^3$

= 87KNm/m

(a) Ultimate Limit State

At mid-span, $\frac{Mu}{b^1 d^{12} fcu} = \frac{1.6 \times 87 \times 10^6}{1000 \times 250^2 \times 25} = 0.089$

$Z/d^1 = 0.872$, $Z = 0.872 \times 250 = 218mm$

$As_1 = \frac{Mu}{0.87 f_y Z} = \frac{1.6 \times 87 \times 10^6}{0.87 \times 425 \times 218} = 1727mm^2$

Provide Y20-175 ($1800mm^2$)

Over the beams

$As_1 = \frac{1.6 \times 109 \times 10^6}{0.87 \times 425 \times 208} = 2268mm^2$

Provide Y20-125 ($1510mm^2$)

(b) Serviceability Limit State

$\frac{100As_1}{b^1 d^1} = \frac{100 \times 2510}{1000 \times 250} = 1.0$

$^x/_{d^1} = 0.418$, $X = 0.418 \times 250 = 104.5mm$

Steel stensile stress, $fe = \frac{109 \times 10^6}{2510 \left(250 - \frac{104.5}{3}\right)}$

$= 202 \; N/mm^2$

which is greater than $130N/mm^2$ allowable,

steel to be increased.

Minimum $As_1 = \frac{202}{130} \times 2510 = 3900mm^2$

Provide Y20-75 ($4190mm^2$)

Provide Y12-125 ($905mm^2$) transverse steel.

Circular Tank

Diameter of tank = 12m

Depth of water = 3m

Consider h^1 = 200mm, d^1 = 200-50 = 150mm

$$\frac{H^2}{Dh^1} = \frac{3^2}{12 \times 0.2} = 3.75$$

Hoop tension = 0.5x0.316x9.81x3x12

= 56 KN.

$$As_1 = \frac{1.6x56x10^3}{130} = 689mm^2$$

Provide Y12-150 (754mm^2)

Norminal reinforcement to be provided to carry bending moment at base of wall

As_1 = 0.003x1000x200 = 600mm^2

Provide Y12-175 (645mm^2)

Hopper bottom

h^1 = 250mm, d^1 = 250-50 = 200mm

Weight of water = 9.81x3.65 = 36 KN/m^2

Weight of slab = 0.25x25 = 6 KN/m^2

hoop tension $= \frac{12}{2}$ (36x1.38m12^o+6cos12^o)

= 6(36x1.3x0.21+6x0.98)

= 94 KN/m of sloping side

$$As_1 = \frac{1.6x94x10^3}{130} = 1157mm^2$$

Provide Y20-250 (1260mm^2)

Transerve steel $= 0.003 \times 1000 \times 250$

$= 750mm^2$

Provide Y12-150 ($754mm^2$).

Square Tank

Dimension: 8.5x8.5x6m with hoppe bottom

Depth of water = 5.0m

Walls

At base of wall, $M^1 = 0.089 \times 9.81 \times 5^3$

$= 102$ KNm/m

Consider $h^1 = 300mm$, $d^1 = 300-50 = 250mm$

(a) Ultimate Limit State

$$\frac{Mu}{b^1 d^{12} fcu} = \frac{1.6 \times 102 \times 10^6}{1000 \times 250^2 \times 25} = 0.10$$

$$^Z/d^1 = 0.854, \; Z = 0.854 \times 250 = 214mm$$

$$As_1 = \frac{1.6 \times 102 \times 10^6}{0.87 \times 425 \times 214} = 2063mm^2$$

Provide Y20-150 ($2090mm^2$)

(b) Serviceability Limit State

$$\frac{100As_1}{b^1 d^1} = \frac{100 \times 2090}{1000 \times 250} = 0.84$$

$$^X/d^1 = 0.392, \; X = 0.392 \times 250 = 98mm$$

reinforcement tensile stress

$$fs = \frac{102 \times 10^6}{2090\,(250 - \frac{98}{3})} = 225N/mm^2$$

which is greater than 130 N/mm² allowable

Minimum $As_1 = \frac{225}{130} \times 2090 = 3617 mm^2$

Provide Y20-75 ($4190 mm^2$)

Provide Y12-125 transverse steel

Base

$h^1 = 300mm, \; d^1 = 250mm$

Weight of water and slab $= 0.3 \times 24 + 9.81 \times 6.85$

$= 74 \; KN/m^2$

At the Corners

$$M^1 = \frac{74 \; (2 \times 3.5^3)}{12 \; (2.3.5)} = 76 \; KNm/m$$

Free bending moment

$$= \frac{74 \times 3.5^2}{8} = 113 \; KNm/m$$

Positive bending moment at mid-span

$$= 113 - 76 = 37 \; KNm/m$$

$$\frac{Mu}{b^1 d^{12} fcu} = \frac{1.6 \times 37 \times 10^6}{1000 \times 250^2 \times 25} = 0.04$$

$$Z/d^1 = 0.947, \; Z = 0.947 \times 250 = 237mm$$

$$As_1 = \frac{1.6 \times 37 \times 10^6}{0.87 \times 425 \times 237} = 676 mm^2$$

Provide Y16-250 ($804 mm^2$)

At the Corners

$$\frac{Mu}{b^1 d^1 fcu} = \frac{1.6x76x10^6}{1000x250^2x25} = 0.08$$

$Z/d^1 = 0.887$, $Z = 0.887x250 = 222mm$

$$As_1 = \frac{1.6 \times 76 \times 10^6}{0.87x425x222} = 1481mm^2$$

Provide Y16-125 ($1610mm^2$)

Provide Y12-125 transverse steel.

APPENDIX I

Efficiency of ABU water treatment plant
Sedimentation basins

Table 5.1 (a): Dry season water samples

Raw water turbidity (NTU)	Settled water turbidity (NTU)	% turbidity Removal
245	69	72
230	64	72
240	74	69
255	76	70
235	68	71
Average % turbidity removal		71

(b) Rainy season water samples

Raw water turbidity	Settled water turbidity	% turbidity Removal
650	195	70
638	204	68
652	189	71
645	206	68
640	204	68
Average % turbidity removal		69

Overall average % turbidity removal

$$= \frac{71 + 69}{2} = 70\%$$