Thickening of Brown Water Sludges by Dissolved-Air (Pressure) Flotation

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Abstract

A model describing the inter-relationships influencing dissolved-air (pressure) flotation systems, developed earlier for activated sludge, is verified using the settled sludge derived from the treatment of brown waters from the Table Mountain catchment area.

For efficient flotation, a cationic polyelectrolyte at an optimal dosage of 0,5 g kg^{-1} needs to be added to the brown water sludge.

The effluent from the flotation unit treating the settled sludge was comparable to that from the sedimentation basins. Float solids' concentrations up to 12% (120 g ℓ^{-1}) were achieved by flotation. This concentration is compared to a value of 2500 mg ℓ^{-1} achieved by the settled sludge after chemical addition and settling in the plant.

Taking the maximum flow handled by the plant as 18 $M\ell d^{-1}$, the water wasted with the sludge is approximately 450 m^3d^{-1}, i.e. 2,5% of the total flow. By applying flotation to thicken the waste brown water sludge to 12% would reduce the wastage to 9 m^3d^{-1}, i.e. 0,05% of the total flow. This has a further important implication when considering possible subsequent sludge disposal operations where the required handling capacity of such facilities is reduced by 98%.

Running costs associated with polyelectrolyte addition and power requirements of a flotation system to thicken the waste sludge from a plant treating brown waters would constitute a mere increase of 1,43% of the present overall running (chemical) costs.

Introduction

In the Cape Peninsula and Southern Cape coastal areas of South Africa, many of the catchment areas produce waters discoloured by the presence of humic and fulvic acids. At the Kloof Nek and Constantia Nek water treatment plants, treating the brown waters derived from the Table Mountain catchment area, the chemicals used to destabilize the colour compounds are 10 mg ℓ^{-1} sodium aluminate followed by lime to adjust the pH to 5,5 after subsequent addition of 50 mg ℓ^{-1} aluminium sulphate.

The overall treatment process consists of a brief conditioning period after coagulant addition, followed by settlement in horizontal flow hopper bottomed sedimentation tanks. The overflow from the sedimentation tanks is chlorinated, passed through rapid gravity sand filters, rechlorinated and stabilized before distribution. Backwash water from the filters is passed to the head of the plant.

Because of the gelatinous nature of the flocs formed in the process, the wasted sludge concentration from the bottom of the sedimentation tanks never exceeds approximately 2500 mg ℓ^{-1}. Therefore, for a plant treating, say, 20 million ℓd^{-1}, approximately 500 m^3d^{-1} water is wasted, i.e. 2,5% of the total flow. If the sludge were thickened to, say, a concentration of 10%, the corresponding wastage would be reduced to 12 m^3d^{-1}, i.e. 0,06% of the total flow. Perhaps of greater importance is the question of ultimate sludge disposal. Subsequent treatment processes would require a handling capacity of only 12 m^3d^{-1} compared to 500 m^3d^{-1}, the volume wasted without flotation.

It was the principal objective of this study to investigate the feasibility of thickening the waste sludge from plants treating brown waters by dissolved-air (pressure) flotation. A further objective was to verify a model describing the inter-relationships between process variables developed by Bratby and Marais (1975b) using activated sludge mixed liquor.

Apparatus

For the experiments conducted on the brown water sludge, a convenient mobile apparatus, shown in Fig. 1, was used. It consists essentially of a laboratory scale continuous flotation unit and a saturator unit (see Bratby and Marais, 1975a) mounted on a modified camping trailer.

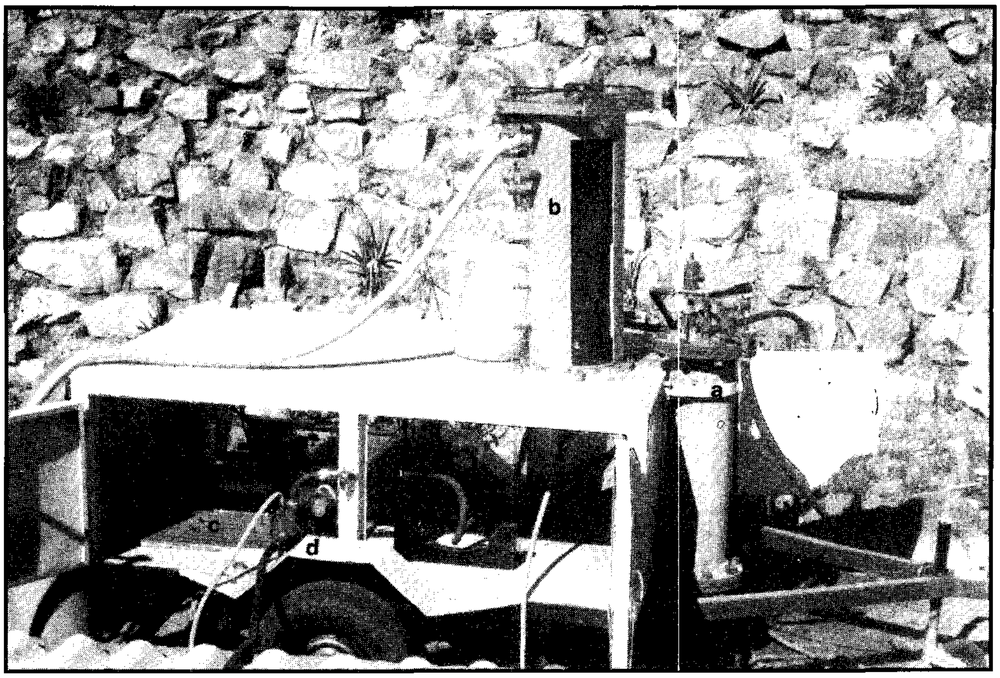

Figure 1
Laboratory scale (continuous) flotation apparatus (for obtaining data on both clarification and thickening) mounted on a trailer for ease in transportation.

To give an approximate idea of costs, the trailer alone has a cost in the order of R500. Together with the flotation and saturator units, the total cost (including a small compressor, pumps, etc.) is in the order of R1000.

The dimensions of the trailer are 1,2 m wide by 1,8 m long by 0,6 m deep. The top of the trailer is flat so that equipment is easily mounted. The two sides are fitted with doors so that the compressor, pumps and all piping, etc. may be installed within the trailer. The saturator is mounted on the front and secured to the trailer body with a metal strap, (a). The flotation unit, (b), is mounted on top of the trailer as shown. Two centrifugal pumps (c) and (d), each with capacity 10 to 50 ℓ min^{-1} and discharge heads 50 to 15 m, serve to provide the saturator and wastewater feeds. A small compressor, (e), which maintains the required pressure in the saturator, has a free air delivery of 28 ℓ min^{-1} at the maximum working pressure of 830 kPa. Saturator and wastewater feeds are monitored with the aid of rotameters.

Figure 2 shows the flotation unit in detail. Designed on the basis of the pilot scale flotation unit described by Bratby and Marais (1975b) it consists of two concentric cylinders approximately 1 m high. The outer cylinder has a diameter of 300 mm and is constructed of P.V.C. with two diametrically opposite clear perspex windows. The inner cylinder, with a diameter of 250 mm, is constructed of clear perspex. It is supported at the top of the outer cylinder and extends to within 20 mm from the base of the unit. The saturator feed is injected into the wastewater feed pipe at the bottom (a), and the blended stream is introduced tangentially to an inlet feedwell (b), 300 mm long. Liquid leaving the inlet feedwell at the top, travels down to the bottom of the unit and passes out via the annulus formed by the two cylinders, through one of the outlet pipes, (c). Whichever of these pipes, (c), is chosen as the outlet, determines the water level in the unit. Bubble-particle agglomerates, separated from the liquid flow path accumulate at the water level until they reach the top of the unit where they are removed by mechanical scraping. Vertical vanes, (d), extending from side to side of the inner cylinder prevent disturbance of the float when it is scraped.

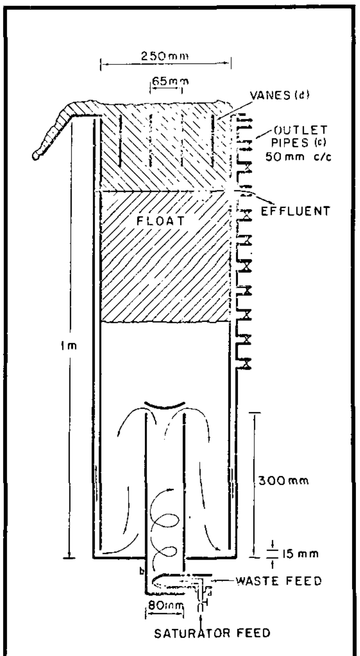

Figure 2
Laboratory scale continuous type flotation unit.

With the brown water sludge it was found necessary to add polyelectrolyte to effect efficient flotation (see later). Efficient mixing of the chemical into the wastewater stream was ensured by installing a rapid mix unit (illustrated in Fig. 3), designed in accordance with the principles set out by Vrale and Jorden (1971). This type of unit was found by Vrale and Jorden to offer the optimal form of rapid mixing when the destabilization mechanism is adsorptive in nature – particularly in the majority of cases where metal coagulants are used. They suggest that with polyelectrolytes, where the long chain molecules are adsorbed to adjacent colloids, the unit devised by them would probably offer the most efficient mixing – a factor all important when using polyelectrolytes.

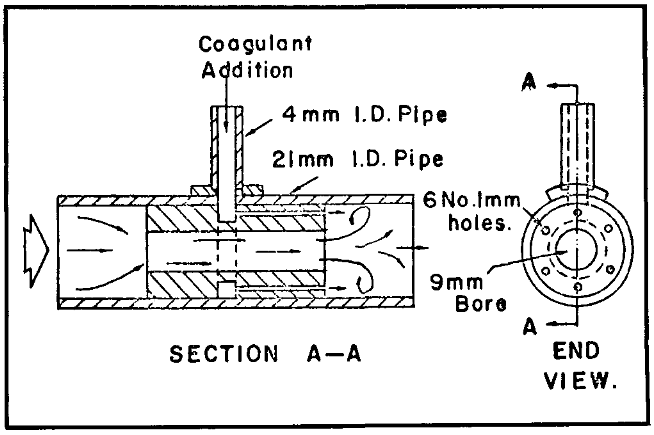

Figure 3
Rapid mixing unit for introduction of polyelectrolyte to wastewater feed (see Vrale and Jorden, 1971).

With the brown water sludge no flocculation time was required after polyelectrolyte addition – the rapid mix unit was installed on the wastewater feed line just prior to entry to the flotation unit. Polyelectrolyte was dosed by means of a peristaltic pump.

Flotation Characteristics of Brown Water Sludge

The sequence of experimentation adopted in these investigations is as described by Bratby and Marais (1976). Results are separated into two aspects: clarification and thickening.

Clarification

Figure 4 shows the results obtained from investigations into clarification by flotation applied to the brown water treatment sludge derived from the Kloof Nek treatment plant. A characteristic of this waste is the extreme fragility of the floc particles and a marked reluctance for re-formation after being sheared apart by the relatively turbulent conditions created by the precipitated bubbles in the flotation unit. For these reasons, to obtain efficient flotation it was found necessary to add small quantities of potable water grade cationic polyelectrolyte "Magnafloc LT 24". In the series of experiments described in this investigation, a dosage of 0,5 g kg^{-1} dry solids was found to be optimal. The concentration of the settled sludge after chemical addition and settling in the plant was consistently 2500 mg ℓ^{-1} and, therefore, the dosage of 0,5 g kg^{-1} dry solids corresponds to a concentration of 1,25 mg ℓ^{-1} applied to the sludge.

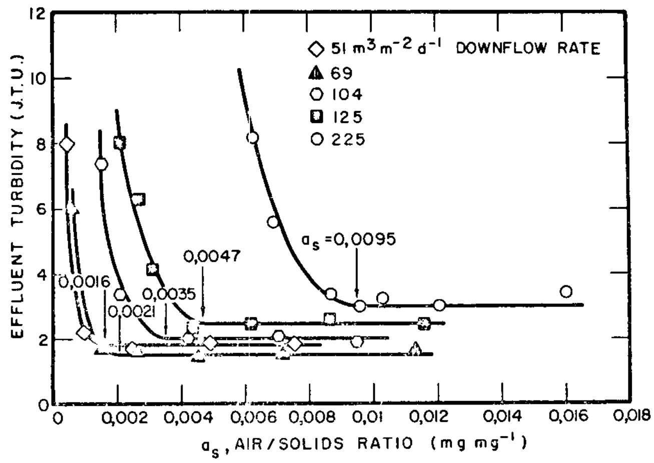

Figure 4
Determination of air/solids ratio, a_s corresponding to limiting downflow rates of 51,0; 69,0; 104,0; 125,0 and 225,0 m^3 m^{-2} d^{-1} respectively.

In constrast to these studies, Packham and Richards (1975) and Hyde (1975) found that with flotation applied to Thames river water no polyelectrolyte was required to effect efficient flotation. Metal coagulants only were required. Colour, turbidity and algal removals were as good as, if not better than plain sedimentation. This difference in results is probably due to the nature of the brown water as described above. Although bubble-particle attachment did occur to a certain extent when using alum alone, the union was relatively short-lived and, on the whole, the effluent was of very poor quality. This was the case with both the raw water after alum addition (and a lengthy conditioning period) and the settled sludge. It is unlikely that turbulence due to excessive flow rates was the cause of the unsatisfactory results since the same effect was evident even at very low flow rates into the flotation unit. A few experiments conducted with the raw water showed a marked improvement on polyelectrolyte addition, but the effluent quality still remained inferior. Experiments conducted on the sludge however, showed a dramatic improvement in effluent quality after flotation – comparable to gravity sedimentation.

This disparity in results was probably due to the relative contact opportunity between floc particles and precipitated bubbles in the flotation unit. With the raw water the suspended solids concentration was 60 mg ℓ^{-1}, whereas with the sludge it was 2500 mg ℓ^{-1}. Improved results with the raw water were attained by recycling a fraction of the float solids back to the inlet of the flotation unit thereby increasing the influent solids concentration to 200 mg ℓ^{-1}. The effluent quality in this case was as good as that from the sedimentation tanks.

It is possible that improved results with the raw water may have been attained if a longer contact period were provided between the rising bubbles and floc particles. With the unit shown in Fig. 2 the opportunity for bubble-particle attachment to occur was provided only in the inlet riser. Although this arrangement was more than adequate for the sludge, an improved design would probably effect efficient flotation with the raw water. Van Vuuren, Stander, Henzen, van Blerk and Hamman (1968) devised a system whereby the precipitated air feed is introduced together with the wastewater feed at the top of a chamber within the flotation unit. The precipitated air bubbles and wastewater particles are intimately mixed during their induced *downward* passage to the bottom of the chamber where they are allowed to rise within an outer concentric chamber before being introduced to the flotation area itself. With such a system ample contact opportunity between bubbles and particles is afforded.

Since the concentration of the raw water, after chemical addition, was consistently 60 mg ℓ^{-1} and taking the cost of "Magnafloc LT 24" as R2,50 kg^{-1}, the cost of the polyelectrolyte per million litres of raw water would be 7,5 cents. Taking the cost of aluminium sulphate as 10 cents kg^{-1}; sodium aluminate as 42 cents kg^{-1}; lime as 6,2 cents kg^{-1} and chlorine gas as 41 cents kg^{-1}, the present overall chemical cost at the mean concentrations used at the Kloof Nek plant is approximately R9,00 $M\ell^{-1}$ raw water. The polyelectrolyte cost, therefore, represents a mere increase of 0,83% of the present total chemical cost.

The sludge concentration of 2500 mg ℓ^{-1}, taken from the bottom of the settling tanks, illustrates the very poor dewatering capability of the alum sludge. Even when sludge was not withdrawn for 48 h at the plant, the sludge concentration was never greater than approximately 2500 mg ℓ^{-1}.

For each downflow rate applied (ranging from 51 to 225 $m^3m^{-2}d^{-1}$) the effluent turbidity (measured in JTU units) is shown for a range of air/solids ratios in Fig. 4. The air/solids ratio corresponding to the limiting downflow rate, v_L, was determined as the point from which the effluent turbidity increased significantly with decreases in a_s. Figure 6 also shows that the effluent suspended solids concentration, in mg ℓ^{-1}, is equal to approximately five times the effluent turbidity, in JTU.

Bratby and Marias (1975b) proposed a general relationship between v_L and a_s of the form

$$v_L = (K_1 a_s^{K_2} - K_3) \qquad m^3m^{-2}d^{-1} \qquad (1)$$

Figure 5 shows a plot of log v_L versus log a_s (values obtained from Fig. 4). It is seen that the general expression given by Eq. (1) found to be applicable to algal wastewater and activated sludge is verified for the alum sludge used in the investigations described here. In this case the constants K_1, K_2 and K_3 in Eq. (1) are, for v_L in $m^3m^{-2}d^{-1}$: 6500; 0,72 and 12,0 respectively. That is, the expression for the alum sludge is:

$$v_L = (6500\, a_s^{0,72} - 12) \qquad m^3m^{-2}d^{-1} \qquad (2)$$

where the settling velocity of an average particle at $a_s = 0$ is set at 12,0 m d^{-1} for linearity in Fig. 5.

In the full series of experiments described in this section, Fig. 6 shows that the effluent turbidity (at downflow rates less than v_L) had an average value of 2,94 JTU with a deviation of ± 1,8 JTU in the 90% confidence interval. The mean value of 2,94 JTU corresponds approximately to an effluent suspended solids concentration of 15 mg ℓ^{-1}.

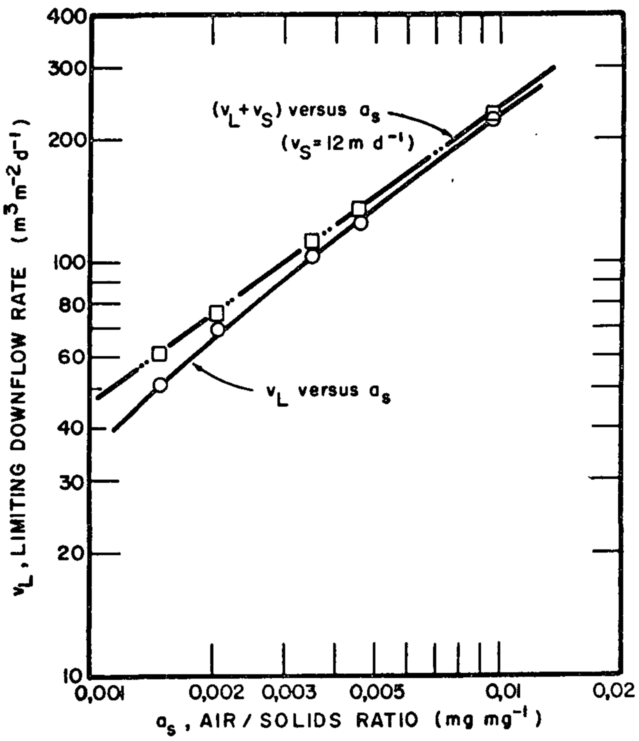

Figure 5
Linear relationship between $(v_L + v_S)$ and a_s. For brown water sludge v_S = *12m* d^{-1}.

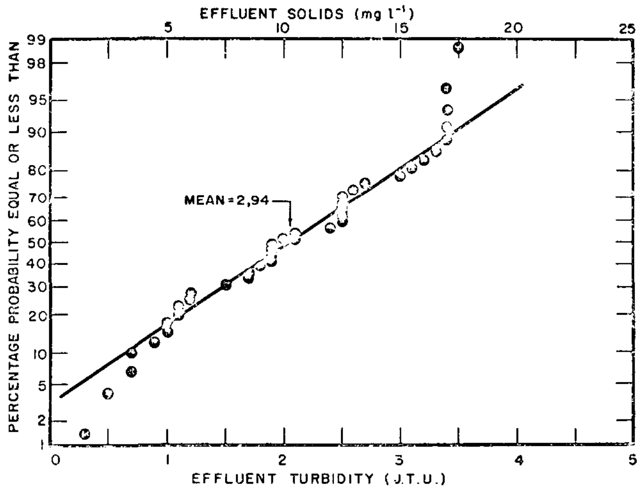

Figure 6
Effluent quality from flotation of brown water sludge. Mean turbidity = 2,94 J.T.U. Deviation = ±1,8 J.T.U. in the 90% confidence interval.

Thickening

In the experiments to determine the thickening characteristics of the brown water (alum) sludge by flotation, parameters Q_s, a_s and d_W were varied in the ranges 3 to 70 kg $m^{-2}d^{-1}$; 0,02 to 0,01 mg mg^{-1} and 20 to 90 mm respectively.

Experiments conducted by Bratby and Marais (1975b) with activated sludge showed that the air/solids ratio, a_s, had very little effect on the float solids concentration, C_F. With the brown water (alum) sludge, Fig. 7 shows that, similarly to activated sludge, varying a_s had very little effect on C_F.

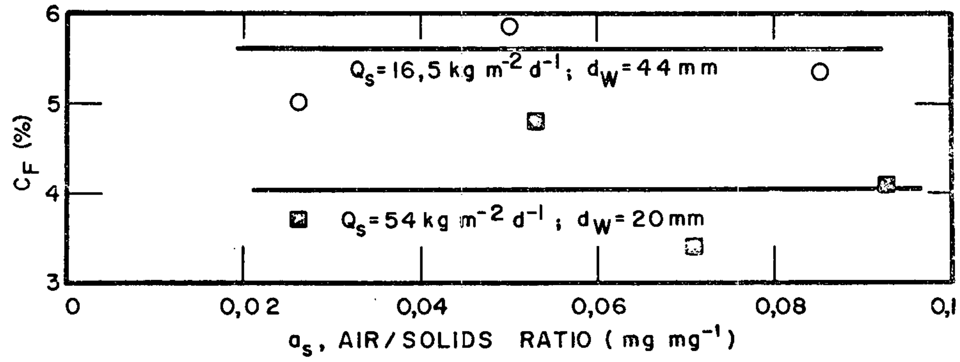

Figure 7
With the brown water sludge, the air/solids ratio, a_s, does not appear to affect the float solids concentration, C_F.

Figures 8 to 10 show that the relationship between solids loading rate, Q_s, and float solids concentration, C_F, is of the same nature as reported by Bratby and Marais (1975b) for activated sludge. That is, log Q_s versus log C_F plots as a straight line. Figure 11 shows the inter-relationship between C_F, Q_s and depth float above water level, d_W, to be as for activated sludge, of the form:

$$C_F = K_4 d_W^{K_5} Q_s^{-K_6} \quad (3)$$

where, for brown water alum sludge and C_F in %, d_W in m and Q_s in kg m^{-2}d^{-1}:

K_4 = 25,61; K_5 = 0,22 and K_6 = 0,28

i.e.

$$C_F = 25{,}61\ d_W^{0{,}22}\ Q_s^{-0{,}28} \quad \% \quad (4)$$

Figures 8 to 10 show that float solids concentrations of up to 12% (i.e. 120 g ℓ^{-1}) were achieved. By applying dissolved-air flotation to thicken the waste brown water sludge to 12%, therefore, would reduce the water wasted with the sludge by approximately 98%.

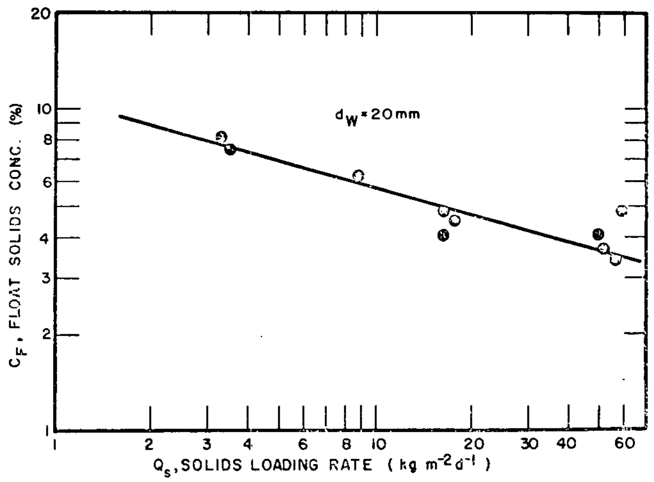

Figure 8
For brown water sludge, linear relationship exhibited between log Q_s and log C_F. Depth float above the water level, d_W= 2,0 cm.

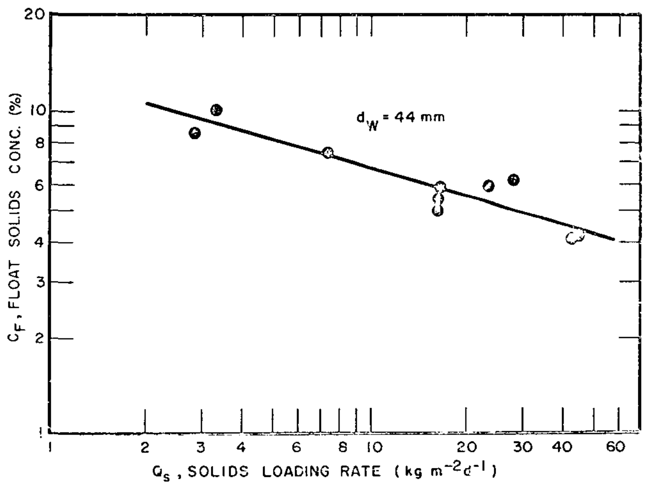

Figure 9
As for Fig. 8 but depth float above water level, d_W = 4,4 cm.

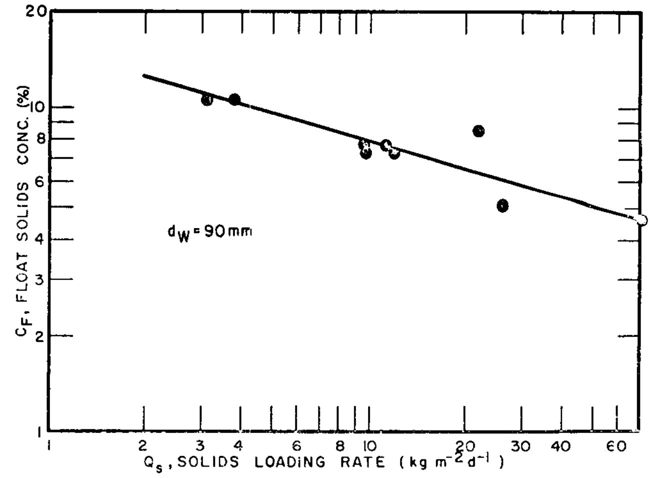

Figure 10
As for Fig. 8 but depth float above water level, d_W = 9,0 cm.

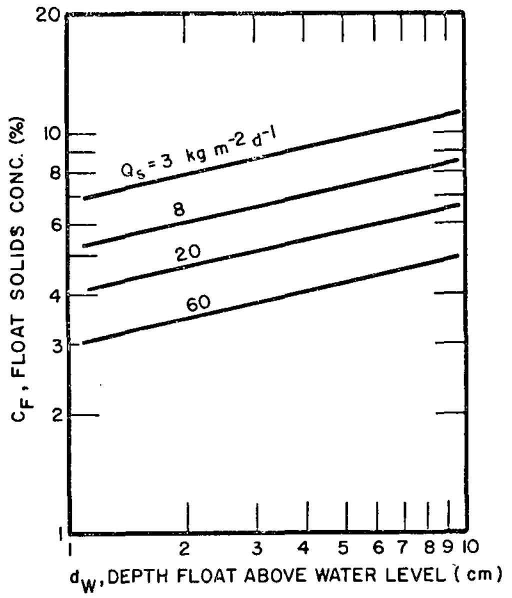

Figure 11
Linear relationship exhibited between log d_W and log C_F. Data taken from Figs. 8 to 10.

Although a_s does not affect the float solids concentration, C_F, it does influence the total depth of float solids ($d_B + d_W$) for a given depth of float above the water level, d_W. Figure 12 shows that the general relationship proposed for activated sludge by Bratby and Marais (1975b):

$$(d_B + d_W) = d_W(a_s^{K_7} + K_8)a_s^{-K_7} \quad \text{m} \quad (5)$$

is verified for this waste, with values for constants K_7 and K_8 as given below:

$$(d_B + d_W) = d_W(a_s^{0,64} + 1,39)a_s^{-0,64} \quad \text{m} \quad (6)$$

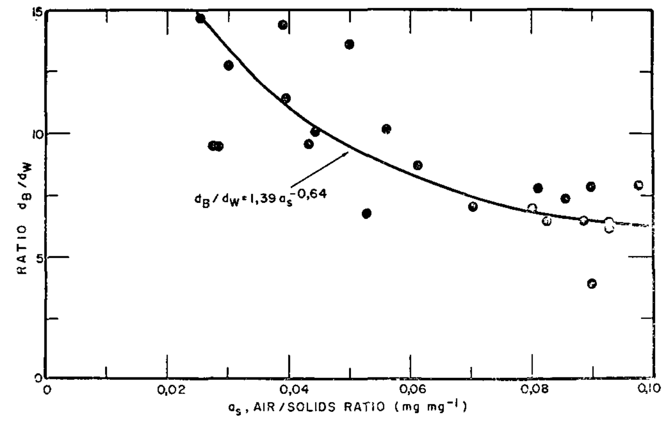

Figure 12
For the brown water sludge, the data exhibit a relationship between the air/solids ratio, a_s, and the ratio (depth float below water level) / (depth float above water level), d_B/d_W, of the form $d_B/d_W = 1,39/a_{s0,64}$

It is seen from Fig. 12 that for thickening, a_s should be stipulated as being not less than 0,04 since, below this value, the ratio d_B/d_W becomes uncertain (see Bratby and Marais, 1975b).

In summary, the three equations (1), (3) and (5) proposed for activated sludge are verified for the brown water (alum) sludge and it is possible to design a flotation unit for its dual functions of clarification and thickening. In this case the constants in the equations are: $K_1 = 6500$; $K_2 = 0,72$; $K_3 = 12$; $K_4 = 25,6$; $K_5 = 0,22$; $K_6 = 0,28$; $K_7 = 0,64$ and $K_8 = 1,39$.

Economic Optimisation of Process Variables

As pointed out by Bratby and Marais (1975b), Although Eqs. (1), (3) and (5) accurately reflect the behaviour of flotation applied to activated sludge and brown water sludge, there are certain complicating factors pertaining to design, that is, each one of the parameters influencing the flotation process is inter-related to the others. For an optimum solution in terms of cost an economic analysis is required delineating the interaction between capital costs and running costs and the influence of each parameter on these.

In the economic analysis carried out for the brown water sludge, parameters influencing clarification alone are first optimised. For this stage the float solids concentration achieved is not considered to be of importance. Secondly, for a range of specified float solids concentrations, C_F, a complete optimisation is presented. The basis of the economic analysis is that reported by Bratby and Marais (1975b).

Clarification

Figure 13 shows that irrespective of the wastewater flow rate, Q_{inf}, there is an optimal value of air/solids ratio, a_s, giving lowest total cost. For illustrative purposes results for only one wastewater solids concentration are shown. The optimum value of a_s is a function solely of the wastewater solids concentration, C_{inf}, as shown in Fig. 14, that is, for brown water sludge:

$$\text{Optimum } a_s = 0,2\ C_{inf}^{-0,47} \quad (7)$$

Equation (7) is of the same form as found for activated sludge by Bratby and Marais (1975b).

The value of a_s as determined by Eq. (7) is optimal in terms of total cost. With an influent solids concentration of 2500 mg ℓ^{-1} for example, the optimum value of $a_s = 0,005$. From Fig. 12, however, it is seen that below air/solid ratios of approximately 0,04, the total float depth (related to the ratio d_B/d_W) becomes uncertain. However, where the prime objective is clarification this is of little importance since the water level is set to a relatively high level (i.e. d_W has a low value) in the order of 20 mm below the top of the unit. In such instances the total float depth is minimal (less than 1,0 m).

For thickening applications the minimal practicable value of a_s does have importance (see later).

Having fixed a value for a_s, values for recycle ratio, r, and saturator pressure, P, may then be chosen to conform to the optimum a_s. The final combination of pressure and recycle ratio chosen will be dictated by the local availability of pumps and compressors.

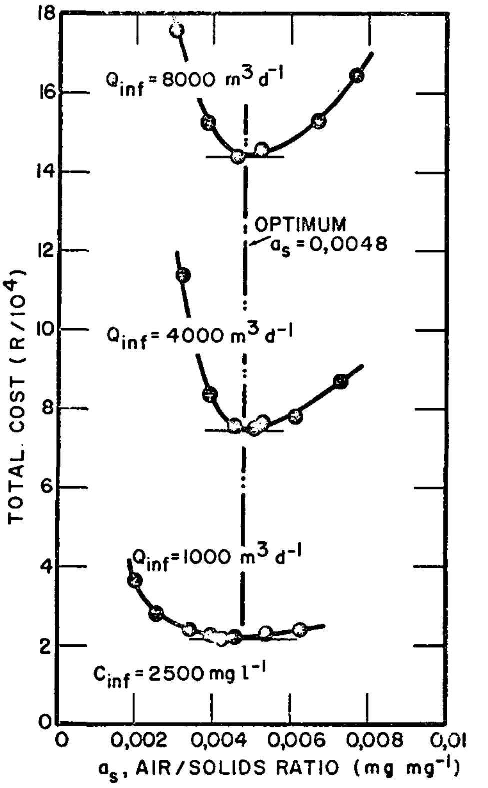

Figure 13
For the brown water sludge, irrespective of flow rate, there is an optimum air/solids ratio giving lowest total cost. For wastewater solids concentration, C_{inf} = 2500 mg l^{-1}, optimum a_s = 0,0048 mg mg^{-1}.

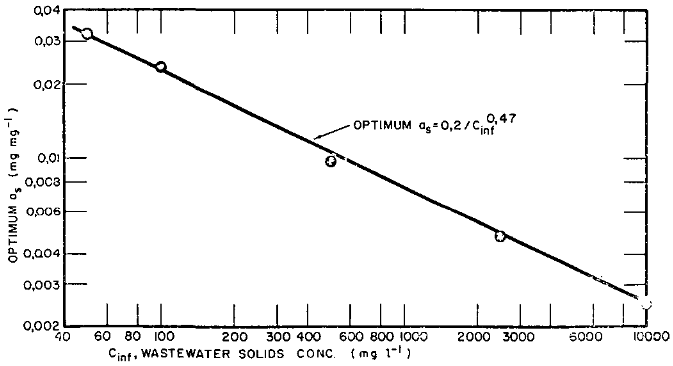

Figure 14
With the brown water sludge, for lowest total cost there is an optimum value of air/solids ratio, a_s, related solely to concentration, C_{inf}, as shown.

Total costs for clarification by flotation of brown water sludge

Figures 15 to 18 show total costs (capital plus capitalized running costs) for flotation applied to the Kloof Nek brown water (alum) sludge for solids concentrations, C_{inf}, ranging from 50 mg ℓ^{-1} (the concentration of the raw water after chemical addition) to 10 000 mg ℓ^{-1} (the maximum conceivable concentration attainable by the sludge after plain sedimentation). It should be noted that the sludge concentration withdrawn from the bottom of the sedimentation tanks was consistently 2500 mg ℓ^{-1}.

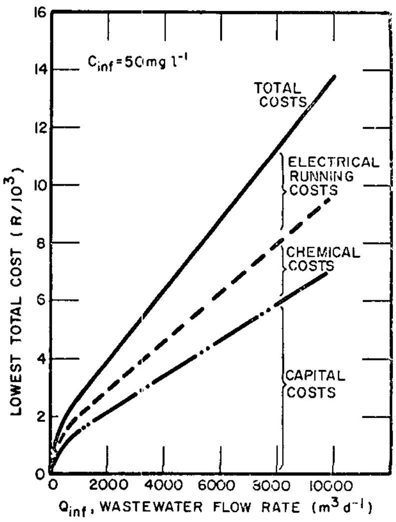

Figure 15
Lowest total costs (capital + running costs) for clarification by flotation applied to brown water sludge with solids concentration = 50 mg l^{-1}. Total costs approximately equally divided between capitalised running costs and capital costs.

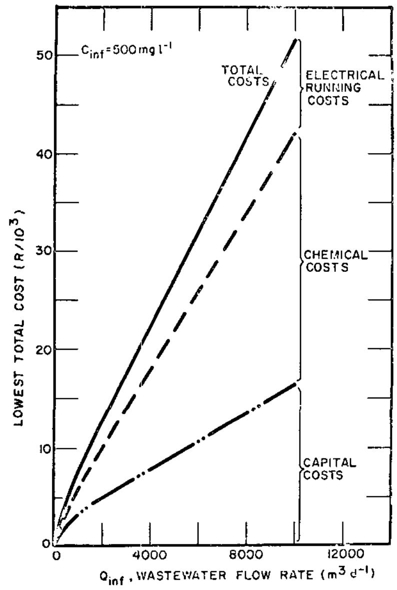

Figure 16
As for Fig. 15 but C_{inf} = 500 mg l^{-1}. Capitalised running costs associated with polyelectrolyte addition approx. equal to capital + electrical running costs.

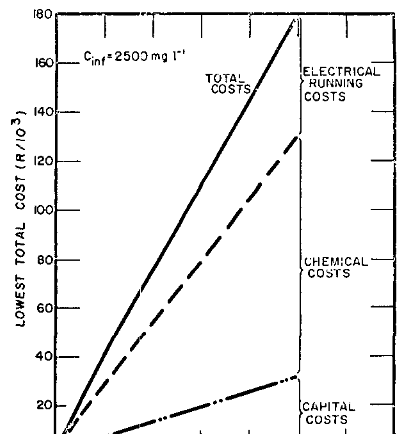

Figure 17
As for Fig. 15 but C_{inf} = 2 500 mg l^{-1}. Polyelectrolyte costs exceed capital + electrical running costs.

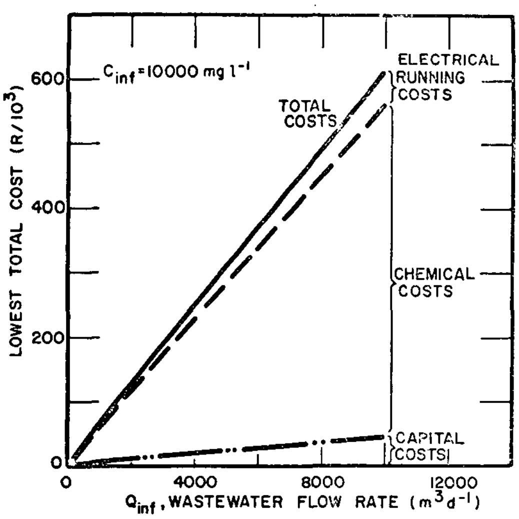

Figure 18
As for Fig. 15 but C_{inf} = 10 000 mg l^{-1}. Polyelectrolyte costs far exceed capital + electrical running costs.

It is seen from Figs. 15 to 18 that the total costs are split into three distinct fractions: capital costs, arising from the construction of the flotation unit, saturator unit, pump and compressor; chemical costs, arising from polyelectrolyte addition; and running costs, arising from the electrical power requirements of the pump and compressor. In general, the running costs associated with polyelectrolyte addition exceed both the capital costs and the electrical running costs. The predominance of chemical costs increases with increasing solids concentration. However, when compared with the overall chemical costs required for the routine running of the plant, polyelectrolyte costs are insignificant. For example, take an assumed annual average flow at the Kloof Nek plant as 10 Mℓ d^{-1} (i.e. 10 000 m^{3}d^{-1}) and a solids concentration of 50 mg ℓ^{-1}. At an overall chemical cost (alum + sodium aluminate + lime + chlorine) of R9,00 Mℓ^{-1}, the chemical cost per day is R90,00. This daily cost, accumulated over 20 years, amounts to an equivalent capitalized running cost of R376 790,00. From Fig. 15 it is seen that for flotation the additional capitalized cost of polyelectrolyte is R2 100,00 and the additional capitalized running cost incurred by electrical power consumption by the saturator pump and compressor is R3 000,00 i.e. a total of R5 400,00. Installation of a flotation unit at the plant to treat the whole raw water flow would, therefore, increase the overall running costs by a mere 1,43%.

Unlike activated sludge, it is difficult to include comparable costs for equivalent sedimentation tanks. In water treatment the wide variety of water qualities requires a wide range of design overflow rates. However, taking the Kloof Nek plant with a total sedimentation tank area of 930 m^2 at the approximate design flow of 14 000 m^{3}d^{-1}, an overflow rate of 15 m^{3}m^{-2}d^{-1} is computed. The capital cost of an equivalent circular settling tank, therefore, at a flow of 10 000 m^{3}d^{-1} is computed as follows.

Area of sedimentation tank required is given by:

Area = 10 000/15 = 667 m^2

$\therefore$ Diameter = $(\frac{4}{\pi}667)^{1/2}$ = 29 m

From Bratby and Marais (1975b):
Capital cost of tank = $330.29^{1,675}$
= R93 666,00.

From Fig. 15 it is seen that the total capital cost of a flotation system is only R7100 and the total cost (capital and capitalized running cost) is only R13 800. Installing a flotation system in lieu of a sedimentation tank, therefore, would effect a saving of 85% in the total cost. The diameter of 29 m required for a circular sedimentation tank may be compared to a diameter of approximately 6 m computed for a flotation unit.

It should be noted that the above comparison between flotation and sedimentation is based on the assumption that flotation applied to raw water is successful and produces an effluent quality comparable to sedimentation. As mentioned previously this was not the case. To produce an acceptable effluent quality it was necessary to recycle float solids back to the inlet of the flotation unit thereby increasing C_{inf} to approximately 200 mg ℓ^{-1}. However, the increased cost incurred by recycling float solids back to the influent would still be less than that for sedimentation. Figure 16 shows that by increasing influent solids to as high as 500 mg ℓ^{-1}, the capital cost for flotation is R17 000 and the total cost R52 000. This is still approximately half the estimated cost for sedimentation.

Thickening

In the design of a flotation system for a certain specified float solids concentration, the basic equations to be used are as given previously, i.e.

$$C_F = 25,61\, d_W^{0,22}\, Q_s^{-0,28} \quad \% \qquad (4)$$

and

$$(d_B + d_W) = d_W(a_s^{0,64} + 1,39)\, a_s^{-0,64} \text{ m} \qquad (6)$$

It is seen that for a particular float solids concentration, increasing or decreasing d_W decreases or increases respectively the area requirement of the unit. Further, for a particular d_W, the total depth of floats solids (d_B + d_W) is dependent on the air/solids ratio a_s.

On determining the dimensions of the unit for thickening, a check must be made to ensure that the area required for thickening is sufficient for clarification, as given by Eq. (2), i.e.

$$v_L = (6500\, a_s^{0,72} - 12) \quad m^3m^{-2}d^{-1} \qquad (2)$$

From the economic analysis it was found that the optimal values of a_s giving lowest total cost are related to the wastewater solids concentrations in a fashion identical to that for clarification alone, i.e. for brown water sludge:

$$\text{Optimum } a_s = 0,2\, C_{inf}^{-0,47} \qquad (7)$$

This indicates that for a particular wastewater flow rate and concentration, the running costs of the flotation system for any required float solids concentration are the same as those for an optimum design based on clarification. The capital costs, however, are significantly increased – depending on the float solids concentration achieved.

As pointed out in the section "Clarification" the optimum value of a_s as determined by Equation (7) needs to be modified for thickening applications where a high value of d_W is likely.

From Fig. 12, below a_s values of approximately 0,04 the ratio d_B/d_W becomes uncertain. In fact, for influent solids concentrations (C_{inf}) exceeding approximately 30 mg ℓ^{-1}, the optimum value of a_s as determined by Eq. (7) needs to be superseded by the minimal practicable value of 0,04 in all cases where thickening float solids to high concentrations is desired.

From the analysis it was found that irrespective of the wastewater flow rate and solids concentration there is an optimum value of d_W, giving lowest total cost, related solely to the desired float solids concentration, C_F. Figure 19 illustrates results for only one solids concentration, C_{inf}, and float solids concentration, C_F.

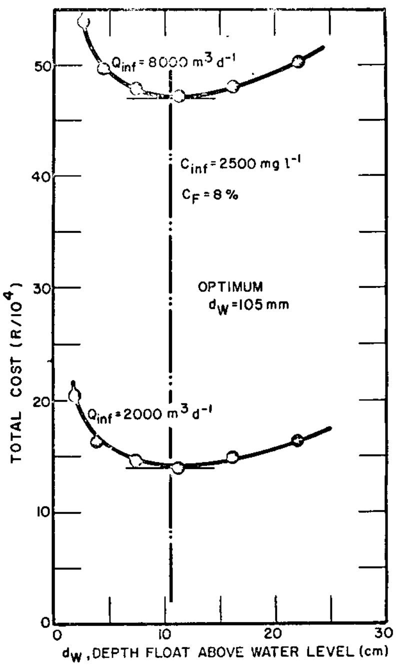

Figure 19

With brown water sludge, for any wastewater flow rate, Q_{inf}, and solids concentration, C_{inf}, there is an optimum value for depth float above water level, d_W, for lowest total cost. For C_F = 8%, optimum d_W = 10,5 cm.

Figure 20 shows that the relationship between float solids concentrations C_F and the optimum value of d_W is given for *brown water sludge*, by:

$$\text{Optimum } d_W = 0{,}013\ C_F \qquad \text{m} \qquad (8)$$

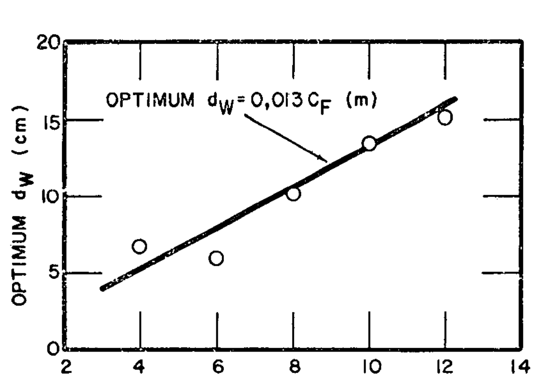

Figure 20

With the brown sludge, for lowest total cost there is an optimum value of float above water level, d_W, related to float solids concentration, C_F, as shown.

In the analysis for thickening the following provisos were stipulated:

(1) The total depth of the float solids should not exceed 2 m. Beyond this depth it is probable that the total depth of the flotation unit becomes excessive (greater than approximately 5 m) so that practical problems arise.

(2) The air/solids ratio should not be less than 0,04 for the brown water sludge (see Fig. 12). With air/solids ratios lower than this value the ratio d_B/d_W becomes uncertain and slight fluctuations in a_s may give rise to inordinate variations in the total depth of float solids.

Total costs for thickening by flotation of brown water sludge

Figures 21 to 23 show, for the brown water sludge, optimum (i.e. lowest) total costs for thickened float solids concentrations up to 12%. By designing a flotation system on the basis of clarification alone for solids concentrations of 60; 1000; and 2500 mg ℓ^{-1} respectively, float solids concentrations of 4; 3 and 2,5% respectively will be obtained. For higher float solids concentrations the total cost is increased as shown in Figs. 21 to 23.

Comparing the results obtained by Bratby and Marais (1975b) for activated sludge with those obtained for the brown water sludge, total costs for the latter are seen to be higher. For example, at a solids concentration of 2500 mg ℓ^{-1} and a flow rate of 7000 m^3d^{-1}, the total cost (capital plus capitalized running costs) of a flotation system to thicken activated sludge to a concentration of 6% was found to be R170 000,00. For the same conditions, the total cost for the brown water sludge is R320 000,00 (see Fig. 23). It is seen, however, that the brown water sludge includes a cost for polyelectrolyte which, for the above conditions, amounts to a capitalized cost of R100 000,00. The cost for flotation thickening applied to the brown water sludge, therefore, (excluding the polyelectrolyte cost) is 29% more than for activated sludge. This is probably due to the limit of 0,04 for the air/solids ratio stipulated for the brown water sludge as opposed to 0,02 for activated sludge, that is, running costs are higher for thickening the brown water sludge.

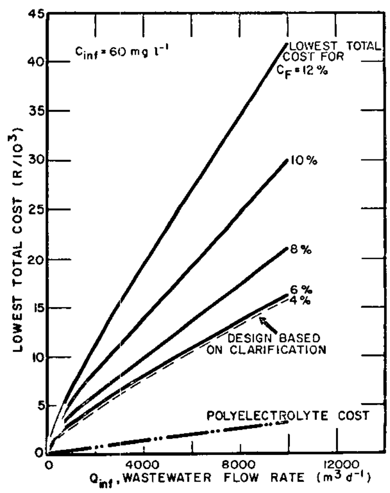

Figure 21

Lowest total costs (capital + capitalised running costs) for thickening brown water sludge by flotation (up to C_F = 12%). C_{inf} = 60 mg l^{-1}. For float solids concentrations below 4%, design based on clarification.

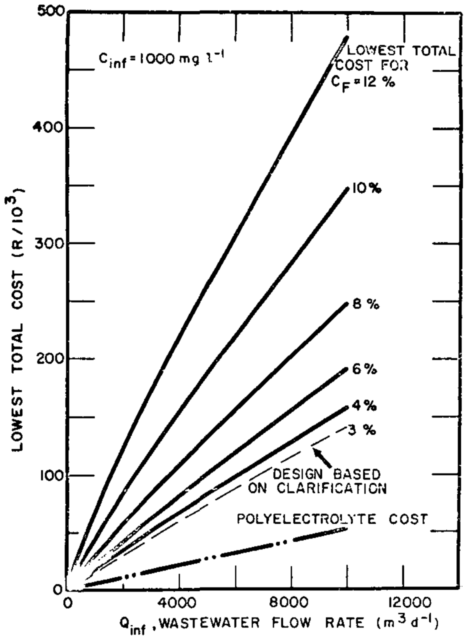

Figure 22
As for Fig. 21 but C_{inf} = 1000 mg l^{-1}. For float solids concentrations below 3% design based on clarification.

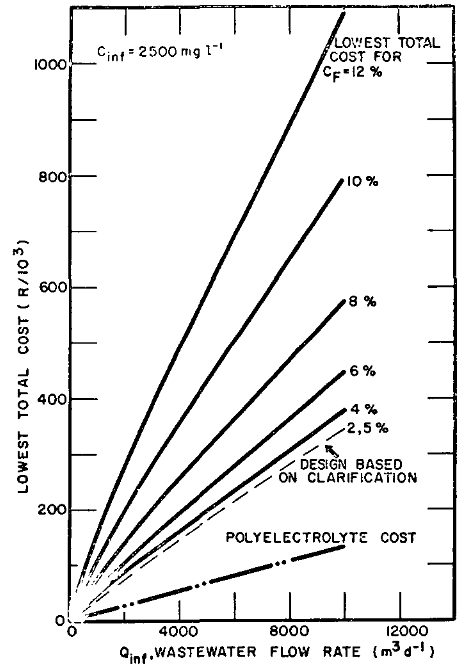

Figure 23
As for Fig. 21 but C_{inf} = 2500 mg l^{-1}. For float solids concentrations below 2,5%, design based on clarification.

Design Example

Flotation system for thickening brown water sludge to 10% (100 gℓ^{-1})

It is required to thicken the waste sludge from a waterworks treating the brown waters of the Cape Peninsula, South Africa. The raw water suspended solids concentration (after coagulant addition) is 60 mg ℓ^{-1}; the maximum flow handled by the plant is 18 Mℓd^{-1} and the settled sludge concentration is consistently 2500 mg ℓ^{-1}. Design a flotation system to thicken the waste sludge to 10% (i.e. 100 g ℓ^{-1}). Flotation experiments conducted with this waste have shown that a cationic polyelectrolyte at a dosage of 0,5 g kg^{-1} is required for efficient flotation. The calculation sequence is presented in step form below.

1. Maximum sludge draw-off to be thickened
   = 60.18000/2500 = 432 m^{3}d^{-1}.

2. Optimum value of air/solids ratio yielding lowest total cost is given by Eq. (7), i.e.

   Optimum $a_s = 0,2(2500)^{-0,47} = 0,005$.

   However, it is undesirable to operate at such a low air/solids ratio since the ratio d_B/d_W is uncertain. For this reason a value of a_s of 0,04 is chosen (see Fig. 12).

3. Optimum value for depth float above water level, d_W, is given by Eq. (8), i.e.

   Optimum $d_W = 0,013.10 = 0,13$ m

4. Solids loading rate, Q_s, required for a float solids concentration of 10% and a value of d_W = 0,13 m is given by Eq. (4), i.e.

   $Q_s = [25,61\ d_W{}^{0,22}/C_F]^{1/0,28}$

   $= [25,61.0,13^{0,22}/10]^{3,57} = 5,78$ kg m^{-2}d^{-1}

5. Area of unit required for thickening determined from value of solids loading rate, i.e.

   Area $= C_{inf}.Q_{inf}/Q_s = (2500.432/5,78)10^{-3} = 186,9$ m^2

6. At an air/solids ratio of 0,04, limiting downflow rate is given by Eq. (2), i.e.

   $v_L = (6500\ a_s{}^{0,72} - 12) = 628$ m d^{-1}

7. (a) Values of saturator pressure, P, and recycle ratio, r, are computed as follows

   $a_s = a_p r/C_{inf}$

   where a_p = mass of air precipitated per litre of pressure-saturated water (mg ℓ^{-1})

   $\therefore a_p r = a_s C_{inf} = 0,04.2500 = 100$

   i.e.

   $a_p = 100/r$

   From Bratby and Marais (1975a):
   $a_p = 0,195$ P
   therefore
   Pr = 100/0,195 = 512,8

   (b) From locally available pumps and compressors a suitable saturator pressure of 700 kPa is chosen. The value of recycle ratio is, therefore, given by r = 512,8/700 = 0,73.

   (c) The saturator pump has a discharge head equivalent to 700 kPa (i.e. a head of 71 m) and a flow rate, at this head, of 0,73.432 = 315 m^{3}d^{-1}.

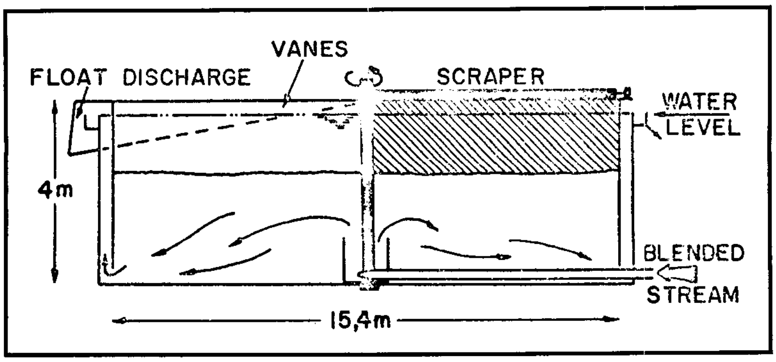

Figure 24
Recommended design for flotation unit for thickening wasted brown water sludge to 10% (100 g l^{-1}).

(d) The compressor unit must be capable of supplying a pressure of 700 kPa and have a free air delivery rate, FAD, of at least that given by:

$$FAD = a_p.rQ_{inf}/1300 = 100.432/1300 = 33{,}2\ m^3d^{-1}$$

8. Area required for clarification is given by:

$$\text{Area required for clarification} = 1{,}25\ (1 + r)Q_{inf}/v_L$$

$$= 1{,}25.1{,}73.432/628 = 1{,}49\ m^2$$

where 1,25 = factor of safety.

9. The area to be used for design is, therefore, the larger of the two, i.e. Area = 186,9 m^2.

10. $\text{Diameter} = (\frac{4}{\pi}\text{Area})^{1/2} = 15{,}4\ m$

11. Total depth float solids, $(d_B + d_W)$ given by Eq. (6), i.e.

$$(d_B + d_W) = d_W(a_s^{0,64} + 1{,}39)a_s^{-0,64}$$

$$= 0{,}13(0{,}04^{0,64} + 1{,}39)0{,}04^{-0,64}$$

$$= 1{,}55\ m.$$

12. Assuming a depth of 2,45 m for the clarification zone, then total depth of unit = 4 m.

13. Area of saturator = r Q_{inf}/1700 (see Bratby and Marais, 1975a)

$$= 0{,}73.432/1700$$

$$= 0{,}19\ m^2$$

$\therefore$ Diameter of saturator $= (\frac{4}{\pi}.0{,}19)^{1/2} = 0{,}49 =$ (say) 500 mm

14. Depth of packing must be 500 mm. For inlet arrangements etc. take total depth saturator = 1 m.

15. Final design information is, therefore:

Flotation Unit:
Diameter = 15,4 m
Depth = 4 m

Saturator:
Diameter = 500 mm
Depth = 1 m

Saturator Pump:
Discharged Head = 71 m
Flow Rate = 315 m^3d^{-1}

Compressor:
Working Pressure = 700 kPa
(Min) FAD = 33,2 m^3d^{-1}

16. A recommended final design is illustrated in Fig. 24.

Acknowledgements

This research was supported by the Water Research Commission of South Africa.

Gratitude is expressed to the following:

Cape Town City Council, for their co-operation when installing the mobile apparatus at the Kloof Nek Water Treatment Plant, Cape Province.

Allied Colloids (S.A.) (Pty) Limited, in particular Mr P. Peutherer, for donating a wide variety of polyelectrolyte samples.

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