

" THE DEVELOPMENT OF AN
ANCILLARY TEXTILE INDUSTRY WASTE
TREATMENT PROCESS " .

BY

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September 1971.

SYNOPSIS.

The objective of this thesis is twofold —

- (1) to design the unit processes for treating the waste from a specific industrial plant using the best practicable technology, and
- (2) to critically examine the theory of these unit processes for utilization in design.

The industrial waste was derived from a synthetic button and textile trimmings factory producing polyester and casein buttons. The effluent has a high pH and COD; contains metallic poisons; and is highly coloured. It exceeds the limitations for effluent quality promulgated by the Cape Town Municipality before discharge to the sewers. A certain measure of pretreatment before discharge to the sewers was therefore required.

The treatment process eventually selected consists of neutralization; flocculation; sedimentation; sand filtration of the sludge; and atmospheric drying of the sludge.

Each of the proposed unit operations was investigated experimentally to obtain data for optimum design:—

Optimum flocculation occurred when the effluent was neutralized from a pH of 11 to a pH of 5; then flocculated with 500ppm of aluminium sulphate; and sufficient sodium aluminate added to bring the final pH value within the narrow range of pH=5,8 to pH=6,0. The sequence of chemical additions, and the pH limits, were found to be crucial.

No unusual pattern emerged from the sedimentation process experiments.

Experiments on the sand filtration of the sludge indicated that constant

head filtration was superior to falling head drainage in general, confirming the theoretical prediction in this respect. The sand filter loading rate was optimum when the sludge slurry was fed to a depth of 10-15 cm at constant head for approximately one day. Atmospheric drying could then proceed until the sludge is spadeable, at a moisture content of less than 8,5 g water/g dry solids. This required a total period of seven days on the sand filter. Experiments on the atmospheric drying of the drained sludge on the sand beds at constant temperature, humidity and air velocity, indicated that there was a significant deviation between theoretically predicted drying times and experimental observations.

A design chart is developed for simulating the operation of the settling tank with variable flow rates and influent solids concentrations. In addition, two modified filtration loading rate equations are proposed, the one taking into account the resistance of the filter media (which is significant at low sludge depths and concentrations), and the other one in which the filter media resistance is negligible (at higher sludge concentrations and at higher sludge depths).

Modifications of ideal drying theory equations are proposed in order to estimate the "constant" drying rate period, and the "critical" moisture content (at which value, the falling rate of drying period commences). The modifications account for the variation in these "constants", and appear to be exponential functions of the solids-loading to the bed.

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1. THE DEVELOPMENT OF AN ANCILLARY
TEXTILE INDUSTRY WASTE TREATMENT
PROCESS

1.1. Introduction.

In 1971, the Cape Town Municipality promulgated regulations (15) controlling the discharge of waste water to the sewers of the city. The effect of these regulations is to protect the sewage works against the introduction of any waste which may injuriously affect the purification process, and thus prejudice the disposal of the treated sewage effluent, should it not meet with the requirements imposed in terms of the Water Act, Act No. 54 of 1956.

These regulations are of particular concern to industrial plants generating waste waters. These industries are now compelled to control the quality of their effluents by either modifying the process streams, or pre-treating the effluents. This thesis deals with the solution of this problem for an ancillary textile industry.

1.2. Description of the Manufacturing Process.

The industry described here is unusual in that its main activity is the production of synthetic buttons and trimmings for the clothing industry. At present, both polyester and casein-formaldehyde buttons and trimmings are manufactured. Essentially the process consists of the following operations:-

Polyester raw material is admixed with a catalyst with or without added inert filler. The fluid mass is cast into moulds and cross-polymerisation allowed to proceed under precisely controlled conditions. The casein raw material, dissolved in an aqueous alkaline medium, is precipitated out

of solution; admixed with inert filler; and compressed and dried in moulds. Thereafter case-hardening is provided by means of a formaldehyde solution treatment, and further drying.

The cast buttons, etc., are further processed by machining in specially-tooled lathes to produce various designs and surface effects. The buttons, buckles and other trimmings are given a surface polish by tumbling in a slurry of 200-mesh pumice in hexagonally shaped tumbler drums. Finally, the polished products are given a variety of surface finishes by dyeing in a large number of vats in an assortment of colours, both with and without chemical or solvent surface etching. All the products are then thoroughly rinsed and dried. (See Photograph 1).

As a result of the appreciable number of processes and combinations of processes, a large variety of both organic and inorganic chemicals are used and these give rise to variation in the composition and quality of the waste effluent from the processing plant.

The effluent from the plant is highly alkaline (pH range normally 10,5 - 11,5) and toxic towards biological degradation processes. It also contains high concentrations of organic biodegradable material and is toxic towards aquatic life. It is thus a serious pollution hazard.

The local regulations ⁽¹⁵⁾ require inter alia that before a waste can be discharged to the sewers, the pH value must be within the range 5,5 - 12,0 and hexavalent chromium limited to a maximum of 50 mg/l (as CrO_3 ; i.e. 26 mg/l as Cr). In addition, the Council is empowered to impose certain restrictions as it may deem necessary, and in the case of this particular plant, the restrictions included limiting the effluent pH value to a maximum of 9,5; reduction of the intensity of the colour; and

limiting the Chemical Oxygen Demand (COD) to less than 1500 mg/l.

Failure to comply with an acceptable quality of effluent can lead to closure of the processing plant.

The current installation has been treated as a large-scale pilot plant in aiding the formulation of a practical solution to the problem. It is of prime economic consideration that the evolved treatment process shall be relatively simple, and low-cost in installation and operation.

1.3. Survey of the Effluent Problem

A rational investigation of the waste water problem requires information about the following aspects:-

- (1) chemicals used in the various processes;
- (2) waste streams generated by the various processes and their polluting constituents; and
- (3) magnitudes of the flow and concentration of pollution in the various streams, and their variability.

The nature of the processes involved suggests a combination of textile industry wastes; and chemical processing, plastics and dye-house wastes.

1.3.1. Chemicals used in the Processes.

A survey of the known chemical compounds found on the premises showed the presence of the following raw materials:-

A. Raw Base Materials

1. Polyester resin, being a maleic acid/propylene glycol/styrene mixture which is polymerised and cross-polymerised in situ in the presence of an organic peroxide catalyst. The styrene vehicle and diluent is also polymerised in this reaction.

2. Casein.
3. Sodium Hydroxide flake, and bulk 40% solution.

B. Solvents

1. Acetone.
2. Methyl Ethyl Ketone.
3. Methylene Chloride.
4. Thinners (hydrocarbon/ketone based).
5. Benzyl Alcohol.

C. Acids

1. Glacial Acetic Acid.
2. 90% Formic Acid.

D. Oxidation/Reduction Chemicals

1. Potassium Dichromate.
2. Sodium hypochlorite, 10% solution.
3. Hydrogen Peroxide (100 volume strength).

E. Textile Dyeing Chemicals

1. Calcolene Oil (employed in polyester dyeing).
2. Levelling Agents (essentially anionic, non-ionic and cationic detergents).
3. Multitude of organic dyestuffs (e.g. "Phthalocyanine Blue"; "Solochrome Black"; "Diamond Chrome Fast"; "Carbolan"; etc.).
4. Titanium Dioxide pigment and filler.
5. Calcium Carbonate pigment and filler.

F. Polishing and Cleaning Compounds

1. Pumice Powder (200 mesh).
2. Teepol (anionic detergent).

1.3.2. Waste Streams.

All processes continue batch-wise. The only interruptions are for vat cleaning between different dyeings among 16 different dye vats. (cf. Photograph 1 for position of vats). All chemical wastes, vat washings and floor sweepings are discharged down a sloping floor towards a common open channel along the wall and behind the dye vats; there is thus no question of isolation of selected pollution streams. A removable steel grid prevents large objects from blocking the channel outlet. The channel discharges directly into the sewer through an external drain at ground level.

1.3.3. Concentration and Variability of Waste Stream.

A 24-hour composite sample of effluent was collected at the external drain. Sampling was carried out at half-hourly intervals. An analysis gave the following constituents and concentration of pollution:-

TABLE 1.I

Effluent Analysis (24-hour Composite Sample)

1. pH :	11.65
2. Total Solids:	4864 mg/l
3. Settleable Solids:	Negligible
4. COD :	13556 mg/l
5. 4-hour O ₂ absorbed (OA) from N/80 Permanganate, 27°C.:	1625 mg/l

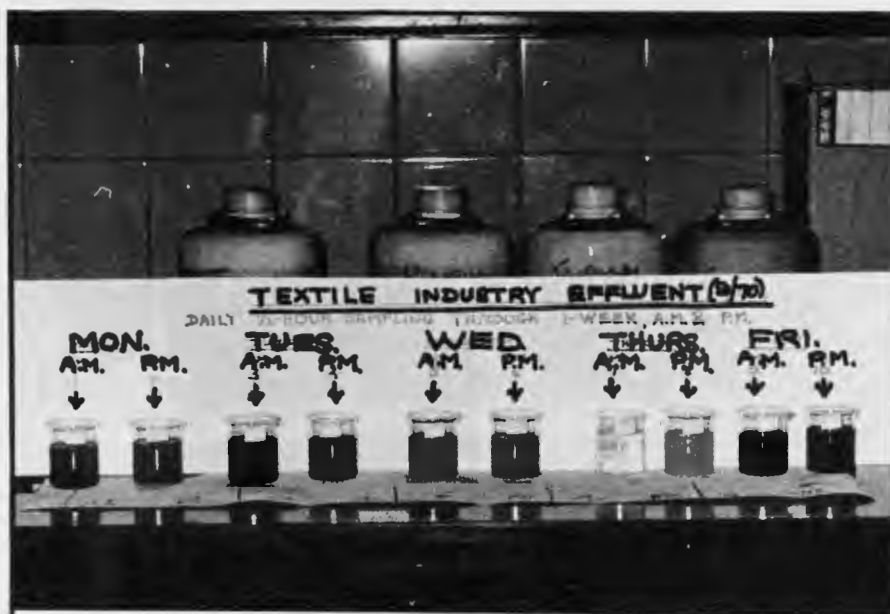
Effluent Analysis (24-hour Composite Sample) (Continued)

6. Ammoniacal Nitrogen ($\text{NH}_3\text{-N}$):	7,1 mg/l
7. Total Nitrogen (N):	16,8 mg/l
8. "Grease" (Hexane Solubles):	686 mg/l
9. Total Chromium (Cr):	1,52 mg/l
10. Hexavalent Chromium (Cr):	0,84 mg/l
11. Copper (Cu):	0,27 mg/l
12. Zinc (Zn):	Negligible
13. Manganese (Mn):	Less than 1 mg/l
14. Calcium (Ca):	24 mg/l
15. Magnesium (Mg):	7 mg/l
16. Chloride (Cl):	190 mg/l
17. Sulphate (SO_4):	48 mg/l
18. Hydroxide (OH):	416 mg/l
19. Carbonate (CO_3):	85 mg/l
20. Physical appearance:	Murky dark red emulsion
21. Odour:	Typical organic/phenolic.

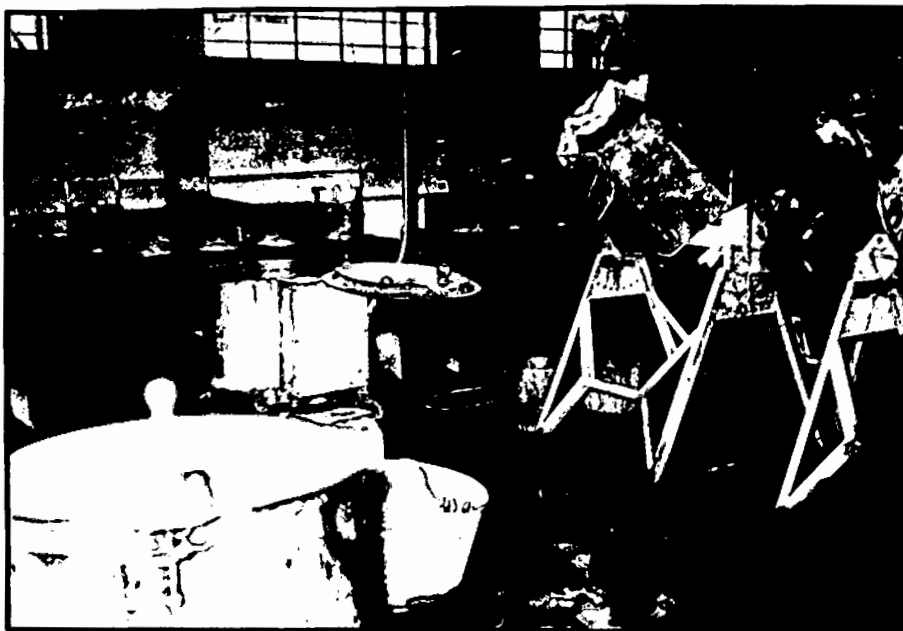
To determine the variability of the polluting substances, samples were collected half-hourly throughout a 9 hour working day over a period of a five day week. Samples collected in the mornings, and those collected in the afternoons, were combined separately to provide 5 morning samples and 5 afternoon samples respectively. The associated flow rate was not recorded, but it was observed that it was fairly uniform throughout the day. Records of daily water meter readings for the processing section only were however available, and showed relatively small variation from day to day.



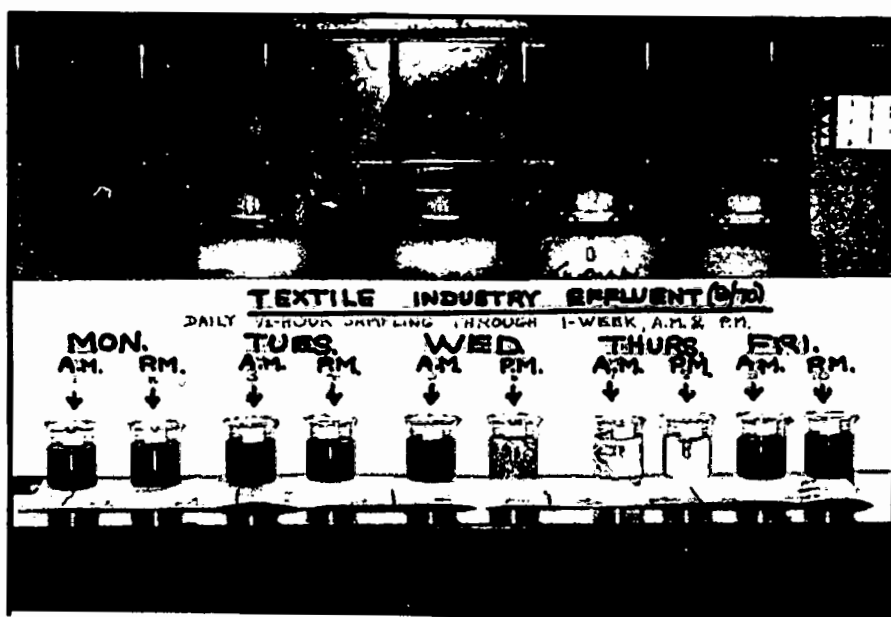
Photograph 1: General view of Processing Area.
(Hexagonal Polishing Tumbler on right;
Dye Vats against rear wall).



Photograph 2: The Half-Daily Variation in appearance of
Effluent over one week (From left to right,
Monday a.m. to Friday p.m.).



Photograph 1: General view of Processing Area.
(Hexagonal Polishing Tumbler on right;
Dye Vats against rear wall).



Photograph 2: The Half-Daily Variation in appearance of
Effluent over one week (From left to right,
Monday a.m. to Friday p.m.).

The following comparative analyses were carried out:

TABLE 1.II

Half-Daily Effluent Analyses

Sample	Colour	pH	COD (mg/l)	Chloride (mg/l as Cl)	Daily Flow (m ³)	Temperature (°C.)
1. Monday a.m.	Dark yellow	11,25	3074	896		22
2. Monday p.m.	Yellow	11,40	2971	358	7,96	22
3. Tuesday a.m.	Black	3,05	4145	1032		22
4. Tuesday p.m.	Grey	6,80	2139	717	7,32	22
5. Wed. a.m.	Red/Black	10,81	15607	673		24
6. Wed. p.m.	Blue/grey	11,25	2704	403	8,82	22
7. Thurs. a.m.	White	11,15	568	1287		23
8. Thurs. p.m.	Blue/grey	11,40	680	1755	10,23	23
9. Friday a.m.	Red	3,10	4287	1272		23
10. Friday p.m.	Very dark red.	3,48	2970	487	11,41	23
Arithmetical Means:			3915	888		

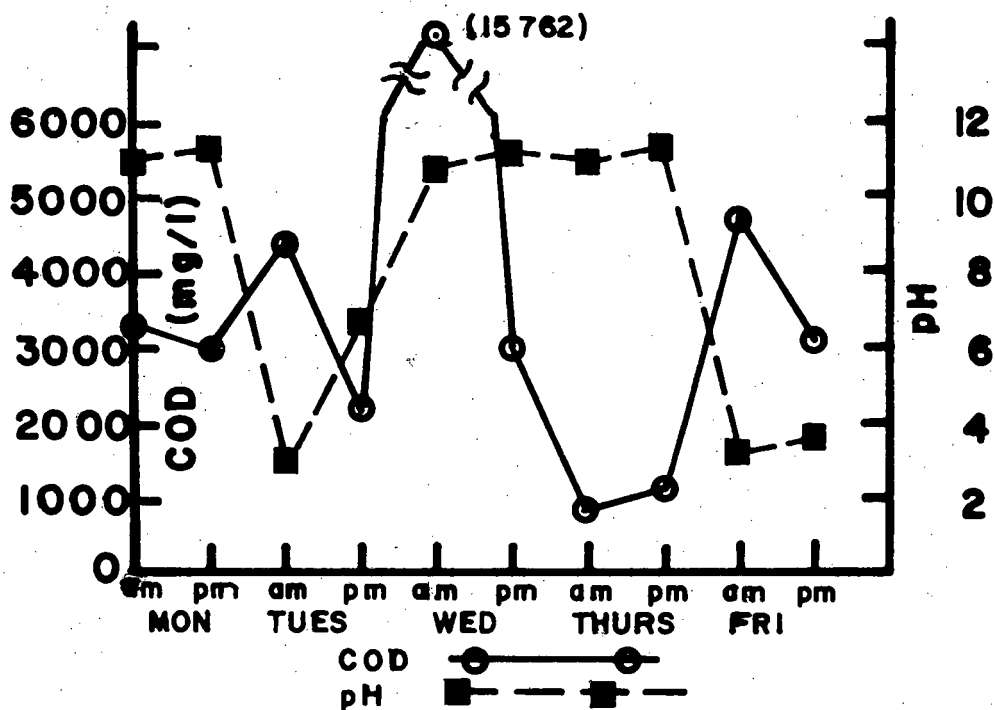
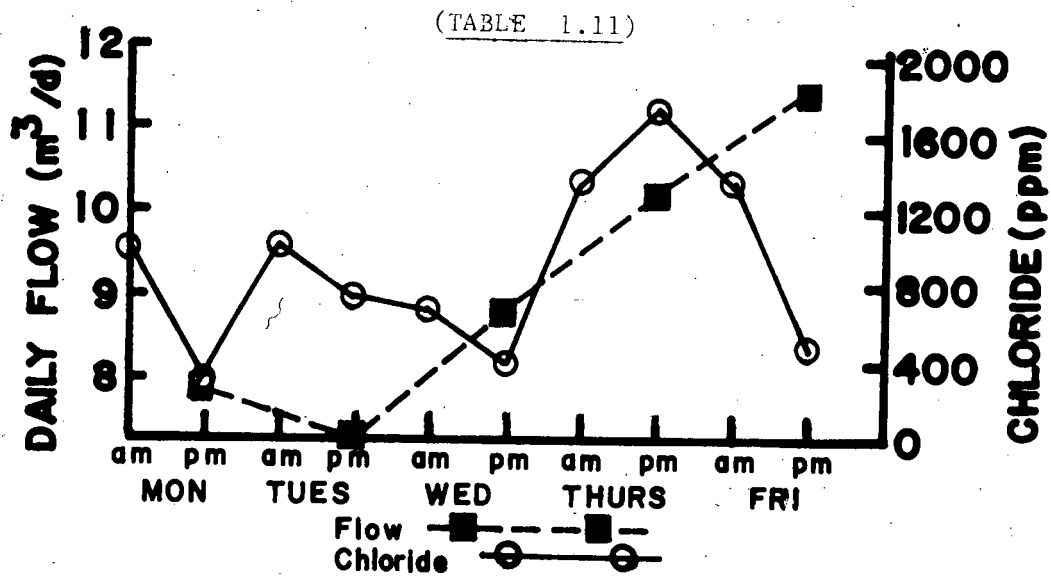
Photograph 2 shows the appearance of each of the 10 effluent samples.

1.3.4. Evaluation of Results.

The half-daily fluctuations in flow, pH, COD and chloride are plotted on a time basis in Fig. 1. (COD values shown are uncorrected for chloride).

Graphical statistical analyses on cumulative probability paper (Fig. 2) show that the chloride and daily flow are approximately normally distributed, whereas in contrast the pH probability plot shows the presence

FIG. I: HALF-DAILY FLUCTUATION OF PARAMETERS.



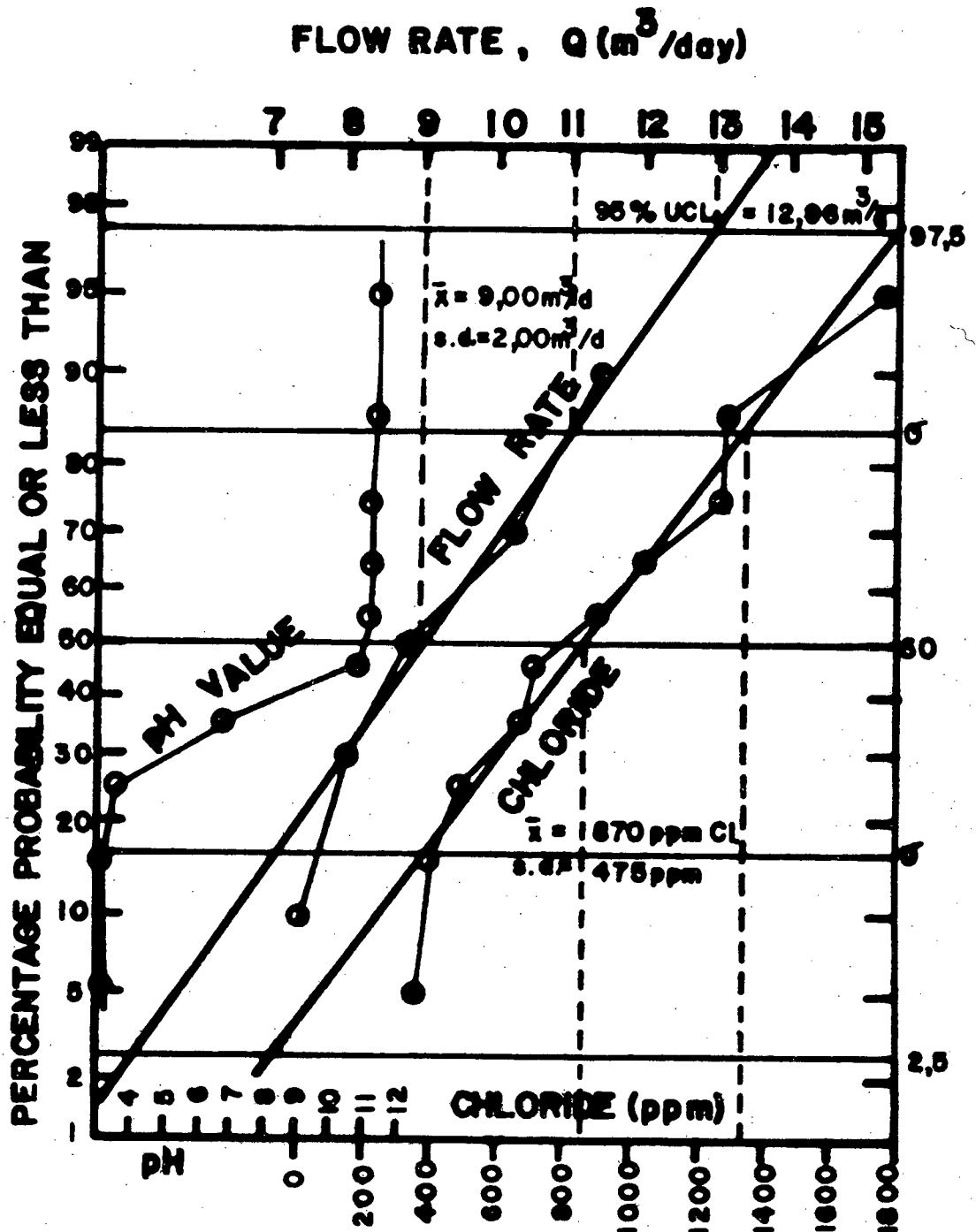


FIG. 2. DISTRIBUTION OF HALF-DAILY pH, CHLORIDE & FLOW.

(TABLE 1.II)

of 2 distributions. On enquiry, the lack of statistical equilibrium in the pH values became apparent. On the Tuesday and Friday mornings, and on Friday afternoon, when the pH values were 3 to 3,5 respectively, sulphuric acid had been discharged to the drain in a crude attempt to neutralize the illegal effluent.

The COD values are log-normally distributed (Fig. 3), and it can be noted that the expected upper 95% confidence limit of 15200 mg/l is in good agreement with peak values actually found (13556 mg/l and 15607 mg/l respectively).

The statistical analyses yield the following results:-

	Flow (m ³ /d)	COD (mg/l)	Chloride (mg/l)
1. Arithmetic Mean, $\bar{x}$:	9,00	3795 ($\bar{x}_g = 2600^*$)	870
2. Standard deviation, S:	440	2387 ($S_{\text{geometric}}$)	475
3. Upper 95% Confidence Limit:	12,96	15200	1800
4. Lower 95% Confidence Limit:	5,09	400	nil

Note: * $\bar{x}_g$ = Geometric mean value.

The values for the COD indicate that considerable variation is to be expected and stresses the need for a balancing collection tank prior to any treatment process, to "smooth" the processing, rather than "on-stream" continuous treatment. The flow distribution provides an estimation of expected average and peak flow rates and their ratio for the present plant, and also for the intended expanded plant.

The normal pH value of the effluent is higher than that permitted for disposal to the sewer. The average COD value is in excess of the required maximum concentration, and the effluent is highly coloured. Treatment of the process waste water is therefore necessary.

1.4. Proposed Unit Operations.

The variability of the composition of the effluent indicates the need for a balancing tank (section 1.3.4.). This tank should be of sufficient capacity to collect a reasonable and manageable quantity of effluent, such as one day's flow. It should have some means of mixing, such as a simple paddle mixer, to homogenize the contents.

(1) Neutralisation.

A comparison of the analyses and the requirements of the regulations, indicates that after mixing the effluent must be neutralised. Most of the time, this will require sulphuric acid to reduce the pH value. However, if the pH value requires to be raised, an addition of caustic soda will suffice. Neutralization can conveniently be effected in the balancing tank. It is necessary to have a similar second tank to collect the effluent flowing during the neutralization operation in the first tank.

(2) Flocculation and Coagulation.

To reduce the COD and colour of the effluent, a flocculation process may succeed. Simultaneously, it may be possible to reduce the chromate content. For this unit operation, it is necessary to investigate experimentally which chemical or combination of chemicals, will bring about a flocculation. Iron and aluminium salts are normally used in the flocculation of coloured and turbid waters, and it is appropriate to attempt flocculation with these common treatment reagents in the case of the waste water. Flocculation requires a particular minimum dosage of chemicals, and in addition, the growth of the floc and its subsequent coagulation is best effected in a particular pH range. The conditions

for optimum flocculation are determined by comparative experiments, in which the dosage and pH are separately varied. Flocculation of the neutralized effluent can be carried out batch-wise, or preferably on a semi-continuous basis.

In a semi-continuous procedure, the neutralized effluent from a batch is fed continuously through "on-stream" chemical dosers to the flocculation/sedimentation tank. The rate of dosing remains constant for each neutralized batch.

In a batchwise flocculation, dosing can be accomplished in the balancing tanks. However, each tank must have sufficient capacity to allow time for floc growth and coagulation in the one tank, while the other is receiving the flow.

(3) Sedimentation and Thickening.

After flocculation, the supernatant has to be separated from the coagulated floc. This operation is performed in a sedimentation tank, in which the floc is allowed to settle and thicken, or concentrate.

In the semi-continuous flow operation, the clarified supernatant overflows from the sedimentation tank to the sewer, while the thickened sludge may be drawn off continuously or intermittently from the bottom of the tank.

In batch operation, the flocculated effluent may be pumped to a separate sedimentation tank having a capacity equal to that of the balancing tank, and the floc allowed to settle and thicken. Alternatively, settling and thickening may proceed in the balancing tank, provided that the tank is designed as a sedimentation tank and the tank bottom is designed for removal of sludge.

In semi-continuous operation, a smaller sedimentation tank will be required than that necessary for batch operation, to obtain a desired

sludge thickening.

(4) Dewatering and Drying of Flocculated Sludge.

The thickened sludge from the sedimentation process still contains a high percentage of liquid. This liquid must be removed as much as possible (i.e. "dewatered") to reduce the bulk of the thickened sludge, and then the solids can be dried and conveniently handled for final disposal.

Dewatering can be accomplished by filtration (or drainage) on a sand filter. The wet sludge is fed onto a sand bed, where the solids are retained and the clear filtrate is underdrained to a sewer. This constitutes a filtration operation. As solids are deposited on the sand bed, the filtration resistance offered by the sludge cake itself increases.

There are three ways of feeding the thickened sludge onto the filter beds:-

- (i) The depth of slurry on the sand bed is maintained constant for a particular length of time ("constant head filtration"), and then allowed to drain without further addition of slurry ("falling head drainage").
- (ii) The bed of sand is filled to a particular depth with slurry, and allowed to drain (falling head drainage only).
- (iii) The bed may be topped-up intermittently with the feed.

The depth of dewatered sludge increases progressively until the filtration resistance offered by the sludge cake reduces the filtration rate to a low level, when the cake is allowed to drain completely.

In the first method of operation, the mass of sludge solids increases progressively, but the depth of the slurry remains constant and the rate of filtration decreases. At some stage the slurry feed will have to be

diverted to a second sludge sand filter. Thus, to determine the minimum area necessary to dewater the sludge, the optimum depth of slurry feed is required.

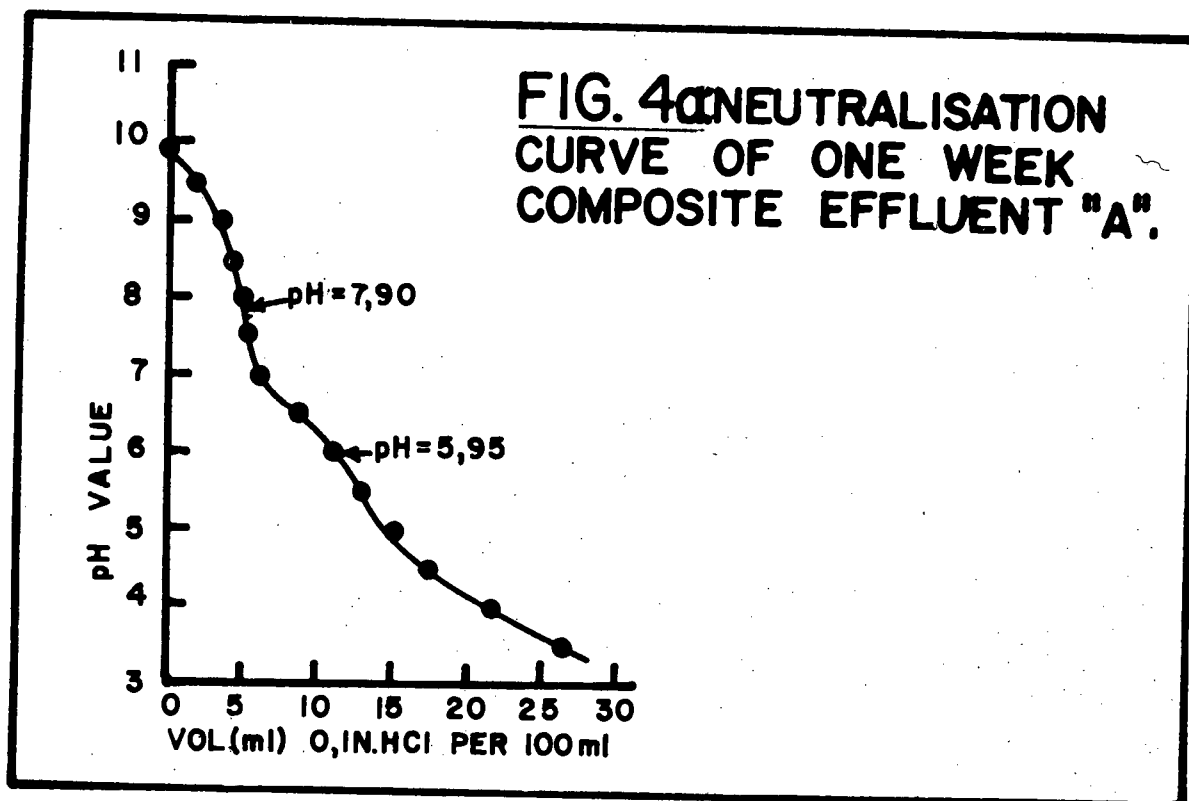
(5) Sludge Drying.

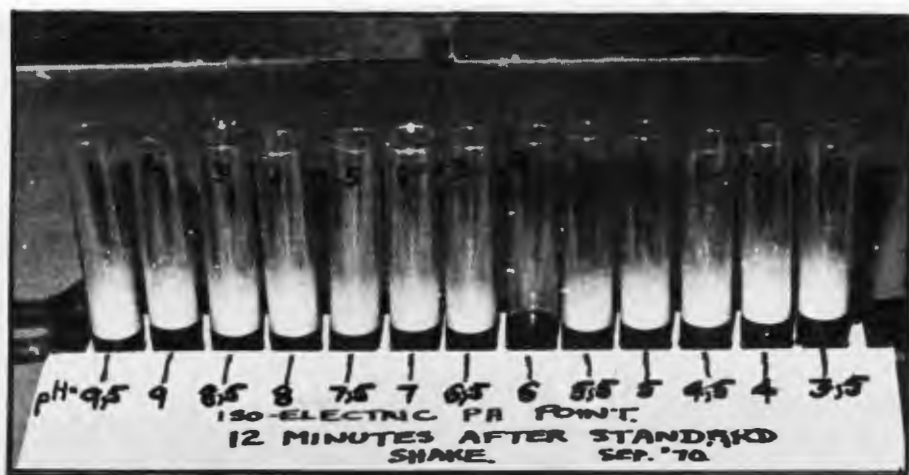
Exposure of the draining sludge to the atmosphere causes evaporation of some of the water. Drying rates are controlled by climatic and atmospheric conditions, which are to a degree unpredictable. If the rate of drying is the controlling element, the area of sand beds for filtration will be dependent on the drying characteristics, rather than on the filtration characteristics. Thus, in this case, the unit operation of atmospheric drying is closely associated with that of dewatering by sand filtration.

2. INVESTIGATION OF UNIT OPERATIONS.

2.1. Flocculation pH

A plot of the neutralization curve of the composite effluent (Fig. 4a) shows two points of inflection; one at a pH value of 7,9; and the other, not as clearly defined, at a pH value of 5,95. The shape of the titration curve appears to be similar to that obtained by titrating a weak base with a strong acid, which raised the thought that it may indicate the "iso-electric pH" for the colloidal matter present in the effluent. However, many of the colloidal particles among the dyestuffs present may exhibit various iso-electric pH values, and it therefore seemed necessary to establish if there was some composite "iso-electric" point, which may suggest a suitable flocculating pH. An initial attempt to find an iso-electric point for the colloidal matter where auto-coagulation might occur in the combined effluent, showed no clear-cut precipitation and separation in a reasonable time. However, it was noted that as the pH value was lowered in 0,5 unit decrements, when each sample was shaken an equal number of times by inverting, it was seen that there appeared to be a definite gradient in the head of foam produced above the liquid level. The foam layer broke readily at $\text{pH} = 6,0$ whereas there was a definite increase in the height of the foam and the time taken for the foam to breakdown as the pH value increased above 6,0 or decreased below 6,0. Shaking equal volumes of effluent at different pH values simultaneously, the foaming effect is clearly demonstrated in Photograph 3. This phenomenon was interpreted as indicating a pH value where the anionic and cationic surfactants in the effluent neutralised one another.





Photograph 3: The effect of pH on the Foaming Stability of the Effluent.



Photograph 4: Appearance of Effluent after Flocculation and settling one hour.



Photograph 5:
Comparison of Supernatant (left) and Sludge (right) after Sedimentation.

Experiments on flocculating the colloidal matter (see below)

confirmed that this pH was indeed an important one, and that the optimum pH value for flocculation with aluminium salts (aluminium sulphate alone, or in conjunction with sodium aluminate) was within the narrow range of pH = 5,8 to 6,0. In this pH range the floc grew rapidly and settled readily with minimum frothing difficulties, while the resultant supernatant was pale in colour and clear. Photographs 4 and 5 show the appearance of the supernatant and sludge after this treatment.

Based on previous experience for the treatment of an alkaline textile industry waste, the Niers Modified Flocculation Process ⁽¹⁾ was applied to the waste. Ferrous sulphate was added as the flocculating agent, followed by aeration at various pH ranges from pH = 11 to pH = 3. A heavy black floc formed, but on settling, the supernatant remained colloidal and had an inky blue-black colour.

It thus appeared that aluminium sulphate, either alone or with the subsequent addition of sodium aluminate, was the most successful in producing a readily settleable floc, and was of more interest for further investigation.

To optimise the process, a program of comparative flocculation was put into action:-

- (i) The concentration of aluminium sulphate was maintained constant while the pH was incrementally decreased from pH = 7,0 to pH = 3,0 until the conditions for best flocculation were obtained.
- (ii) At the selected pH value, the concentration of aluminium sulphate was varied until the coagulation time was a minimum.
- (iii) A dose of aluminium sulphate smaller than that found in (ii) was added to slightly acidic solutions, and both the pH and aluminium

sulphate dosage increased by the addition of sodium aluminate.

Fig. 4 b shows these flocculation test results graphically

(Data in Section 4.1).

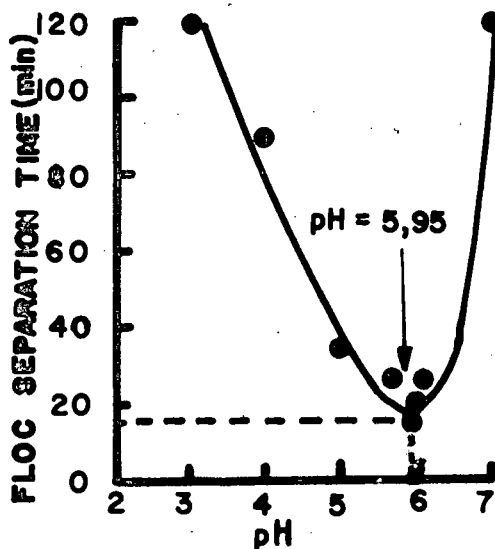
The flocculation process eventually selected was as follows:-

1. Acidify effluent with 10% m/v sulphuric acid solution to pH = 6,5. Add one-half again of the volume of acid required to reach pH = 6,5. (pH drops to approximately 4 to 5).
2. Add 500 ppm aluminium sulphate hydrate in the form of a 10% m/v solution and rapid-mix for a few minutes.
3. Add 10% m/v sodium aluminate solution rapidly and immediately after mixing in the aluminium sulphate, until the pH value is in the range 5,8 - 6,0 (usually about 500 - 800 ppm).
Rapid-mix for 2 - 3 minutes, and then stir slowly for 20 minutes and allow to settle.

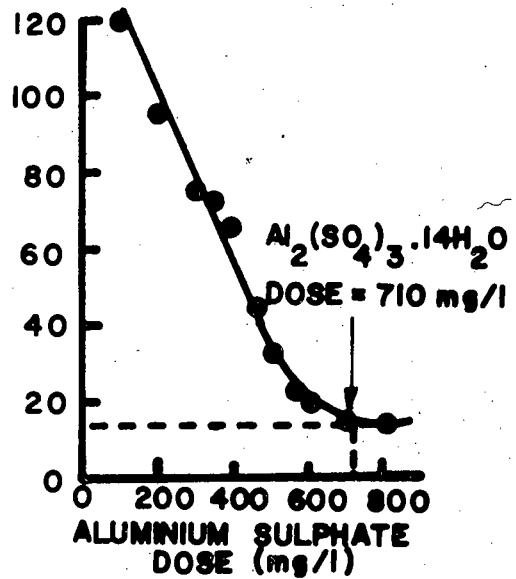
A comparative analysis of the original effluent and the clear supernatant after flocculation gives an indication of the efficacy of the process:-

	<u>Raw Effluent</u>	<u>Flocculated Supernatant</u>
1. COD (mg/l):	3193	1600
2. pH value:	10,32	5,87
3. Hexavalent Chromium (As Cr) (mg/l):	0,52	Not detectable
4. Trivalent Chromium (As Cr) (mg/l):	0,84	Not detectable
5. Colour:	Murky dark red	Clear Pale pink

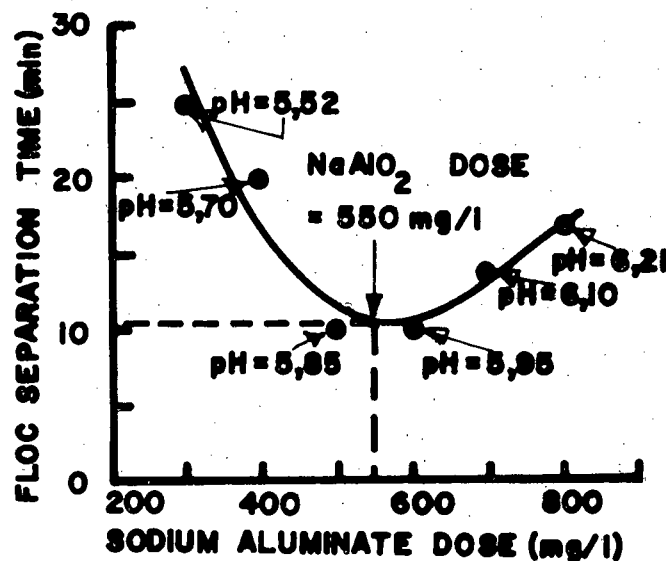
Reduction in COD = 50%



(i) Aluminium Sulphate Dose = 500 mg/l.



(ii) pH Value = 5.90.



(iii) 500mg/l Aluminium Sulphate Added At pH=5.

FIG. 4b: FLOCCULATION AT (i) CONSTANT ALUMINIUM SULPHATE DOSAGE, (ii) CONSTANT pH, & (iii) CONSTANT ALUMINIUM SULPHATE DOSAGE AT pH=5, FOLLOWED BY SODIUM ALUMINATE DOSAGE.

(Section 4.1., Appendix)

Of particular importance was the fact that the hexavalent chromium was reduced in the acidification stage, when all the chromium (1,36 mg/l) appeared to be present as trivalent chromium. The latter was completely removed with the sediment in the flocculation stage.

In addition, in the short time lag after acidification, free chlorine was liberated from the hypochlorite in the waste. Apparently, an oxidation/reduction reaction occurred with part of the oxidisable organic matter present, for no hypochlorite ion was detected in the settled supernatant after treatment, although it was invariably present in the raw effluent.

The estimated water consumption in excess of that employed in the processing plant is 25% (this water being used in ablution, urinals, toilets, dishwashing, etc.), which reduces the treated effluent COD to a value somewhat below 1000 mg/l (and less than 1500 mg/l) before discharge in the Municipal sewer. Thus there appears to be no necessity for further treatment before final discharge.

2.2. Sedimentation and Thickening.

Having established the conditions for optimum flocculation, it is necessary to determine the settling and thickening characteristics of the floc. The theoretical principles and experimental procedures were those adapted from the work of Eckenfelder (2a, 3), Wills (4), Talmage and Fitch (7), Kynch (16) and Fair and Geyer (5). Other procedures are possible (6a), but require the precise evaluation of physical constants such as size and density of the flocculating and settling particles.

During the sedimentation of a floc, there may be essentially two types of settling (2a) process:

(a) Flocculent settling, which occurs when coalescence of the settling particles results in an increase in the settling velocity. The settling rate increases with time, and this results in a curvilinear settling path (Fig. 6).

With flocculent settling in a continuous process, the overflow rate and retention time of the settling tank are both significant. This type of settling is common in water treatment and primary settling in waste water treatment.

(b) Zone settling, which occurs when the floc particles are so concentrated that the settling is hindered by the appreciable upward displacement of fluid. The mass of flocculated sludge settles as a blanket, with a distinct interface between the top of the settling sludge and the supernatant.

Zone settling occurs when the solids concentration exceeds approximately 500 mg/l, and is common in the settlement of activated sludge particles and where heavy doses of flocculent chemicals are used. This type of settling

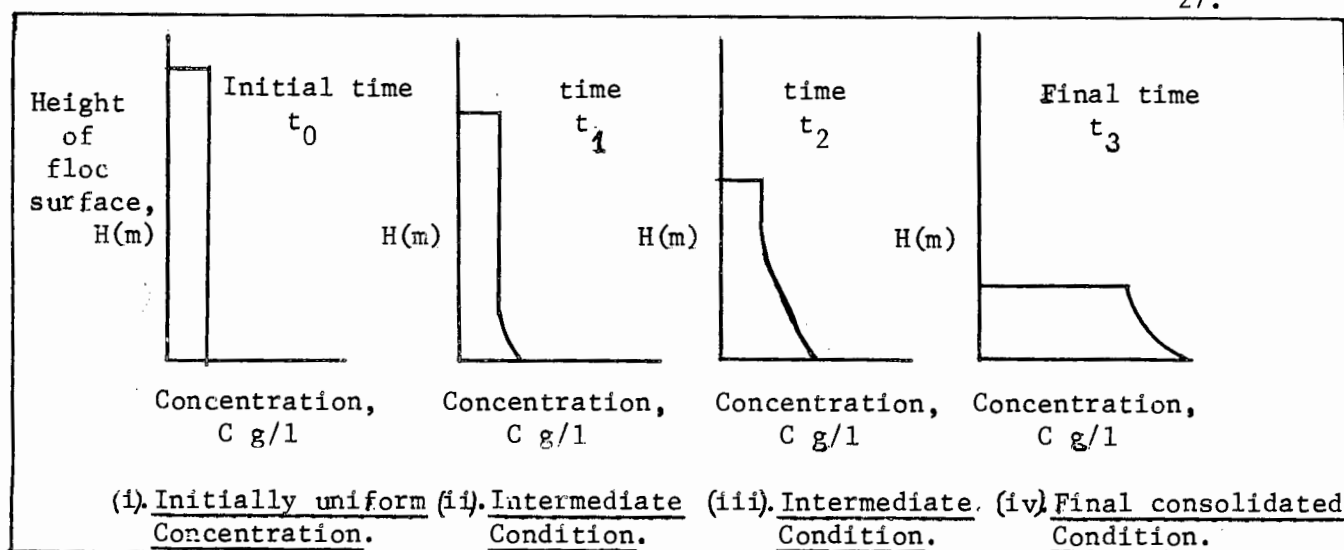
occurs in the treatment of textile wastes, such as that investigated in this thesis, and will therefore be considered in detail.

Kynch ⁽¹⁶⁾ presented a mathematical analysis of batch settling tests without analyzing the relationship between batch settling tests and continuous thickeners. To apply the Kynch method to continuous thickening, Talmage and Fitch ⁽⁷⁾ reviewed that part of the mechanism of continuous thickening which is pertinent to the problem.

In the initial stages of zone settling, the flocs fall through the liquid without pressing on layers of floc below. The quantity of solids which can settle through a unit horizontal cross-section in unit time (i.e. the "flux", $\text{kg/m}^2/\text{hr}$) is equal to the product of the settling rate (m/hr) and the volumetric solids concentration (kg/m^3). In a continuous thickener, the solids must be able to subside through any concentration layers between the concentrations of feed and underflow at least as rapidly as they are fed to the unit. Otherwise, a layer of whatever concentration limits the solids-handling capacity will form and act as a barrier. If insufficient area is present to handle the solids, such a barrier layer would build up. All solids in excess of the amount which could fall through this zone would eventually have to overflow the thickener. Therefore, a thickener must have sufficient area to allow the solids to subside through whichever concentration layer has the least solids-handling capacity.

In a batch settling test, as the particles approach the bottom of the container, the settling rate approaches a small but finite value, and then decreases abruptly to zero because of the barrier presented by the bottom of the container. When the floc particles reach the bottom of the column, they must start accumulating and this causes a local increase in their concentration. Concentration layers with lower solids-handling

capacities (due to their higher local concentrations) than the initial concentration layers, propagate up from the bottom of the vessel and appear eventually at the upper boundary of the settling floc. These concentration layers with limited solids-handling capacity rest upon other such layers below them and compress them. A compression or consolidation of the thickened sludge commences, and this results in a concentration gradient from the bottom of the column of thickened sludge to its top surface. The propagation upwards of concentration layers with limited solids-handling capacity causes an ever-decreasing settling rate, so that there is a transition between the initial zone settling condition and the compression condition. Eckenfelder (2a) described this transition as a separate zone in the zone settling process. Kynch⁽¹⁶⁾ described how in a settling dispersion initially at a uniform concentration, as the layers of higher concentration propagated from the bottom reach the surface, the concentration at the surface begins to increase and the initial constant settling rate of the surface begins to decrease. The change in concentration from one concentration layer to the next one below it may be very small (i.e. "continuous"), or it may be finite (i.e. "discontinuous"). This results in a series of concentration discontinuities. The whole adjustment of concentration which occurs when a dispersion settles can then be described as a series of small concentration discontinuities propagated through the fluid. In the compression zone, the thickened floc continues to settle at a reduced rate, and is consolidated until the minimum depth and maximum concentration is reached. The following diagrams show the floc height-concentration relationships at different times in a batch settling test:-



During consolidation, compression causes liquid to be squeezed out of the floc and released upwards into the clarified supernatant, thereby concentrating or thickening the sludge.

Kynch ⁽¹⁶⁾ showed how the settling rate and concentration of any capacity - limiting concentration layer which may exist can be determined from the variation in settling rate observed in a batch settling test. For the purposes of an analysis of the thickening process, the application of the Kynch theory used by Talmage and Fitch ⁽⁷⁾ will be considered:

Kynch postulated that the settling velocity, $V \text{ (m/hr)}$, of a particle is a function only of the local solids concentration, $C(\text{kg/m}^3)$, around the particle, or mathematically,

$$V = f(C).$$

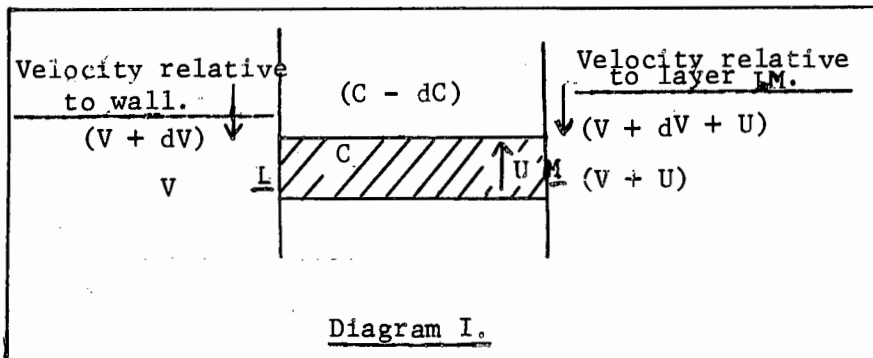
The function is not defined and may change in any manner as the concentration changes. The relationship between the two can be deduced from observations on the fall of the top boundary of the dispersion.

In a batch test starting at uniform concentration, all the solids start settling at a uniform velocity, since $V = f(C)$. As the settling solids begin to collapse against the bottom of the vessel, they must pass through all concentrations between the initial concentration and that of the deposited solids. If, at any of these intermediate concentrations, the solids-handling capacity is less than that at the lower concentration

occurring immediately above it in the vessel, a zone of such intermediate concentration must start building up, since the solids cannot pass through it as fast as they are settling down into it.

Kynch showed that the rate of upward propagation of each such constant concentration zone is constant.

Consider the infinitely thin layer LM (diagram I, below) at the upper boundary of such a zone, having a concentration, $C \text{ kg/m}^3$, originating at the bottom at zero time and moving upward at a velocity of $U \text{ m/hr}$. The solids settling into this layer come from a layer having a concentration of $(C - dC) \text{ kg/m}^3$ and a settling velocity with respect to the vessel of $(V + dV) \text{ m/hr}$, but with respect to the layer of $(V + dV + U) \text{ m/hr}$.



The concentration of solids settling out of this layer will be C with a settling velocity of V with respect to the vessel and $(V + U)$ with respect to the layer. Since the concentration of the layer is constant, the quantity of solids settling into the layer must equal the quantity of solids settling out of the layer, and a material balance can therefore be made.

If the cross-sectional area of the layer LM is $A(\text{m}^2)$, then at time $t \text{ (hr)}$, the mass of solids settling into the layer is $(C - dC)At(V + dV + U) \text{ kg}$, and the mass of solids settling out of the layer is $CAt(V + U) \text{ kg}$. Therefore,

$$(C - dC)At(V + dV + U) = CAt(V + U) \quad (1).$$

Simplifying,

$$C \cdot dV - V \cdot dC - dC \cdot dV - U \cdot dc = 0.$$

Solving for U, and ignoring second order differential terms,

$$U = C \frac{dV}{dC} - V \quad (2).$$

Since, according to Kynch's assumption $V = f(C)$, by differentiation,

$$\frac{dV}{dC} = f'(C),$$

and substituting in (2),

$$U = C f'(C) - f(C) \quad (3).$$

Since C is constant for the layer in question, $f(C)$ and $f'(C)$ have fixed values and therefore U must also be constant by equation (3).

The constancy of U may now be used to determine the solids concentration of the layer at the upper boundary of the settling floc. Let C_0 (kg/m^3) and H_0 (m) be the initial concentration and height respectively, of a column of floc in a batch settling test in a column such as that in Fig. 5. The total mass of solids in this column of floc is then $C_0 H_0 A$ (kg).

When any capacity-limiting concentration layer reaches the floc/water interface, all solids in the column must have passed through it, since it was propagated up from the bottom of the column. If the concentration of this layer is C_2 (kg/m^3), and it reaches the interface at time t (hr) with a settling velocity V_2 (m/hr), then the mass of solids having passed through this layer, $C_2 A t_2 (V_2 + U_2)$ kg, must equal the total mass of solids in the column. Thus, equating the two expressions for the total mass of solids in the column,

$$C_0 H_0 A = C_2 A t_2 (V_2 + U_2) \quad (4).$$

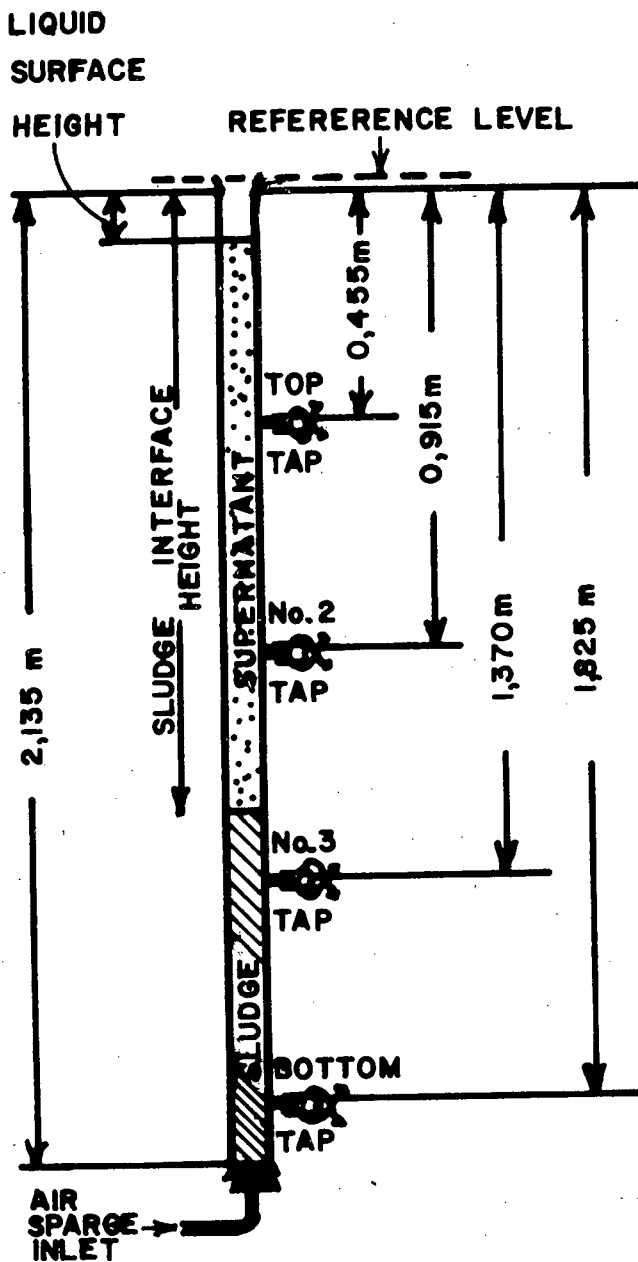
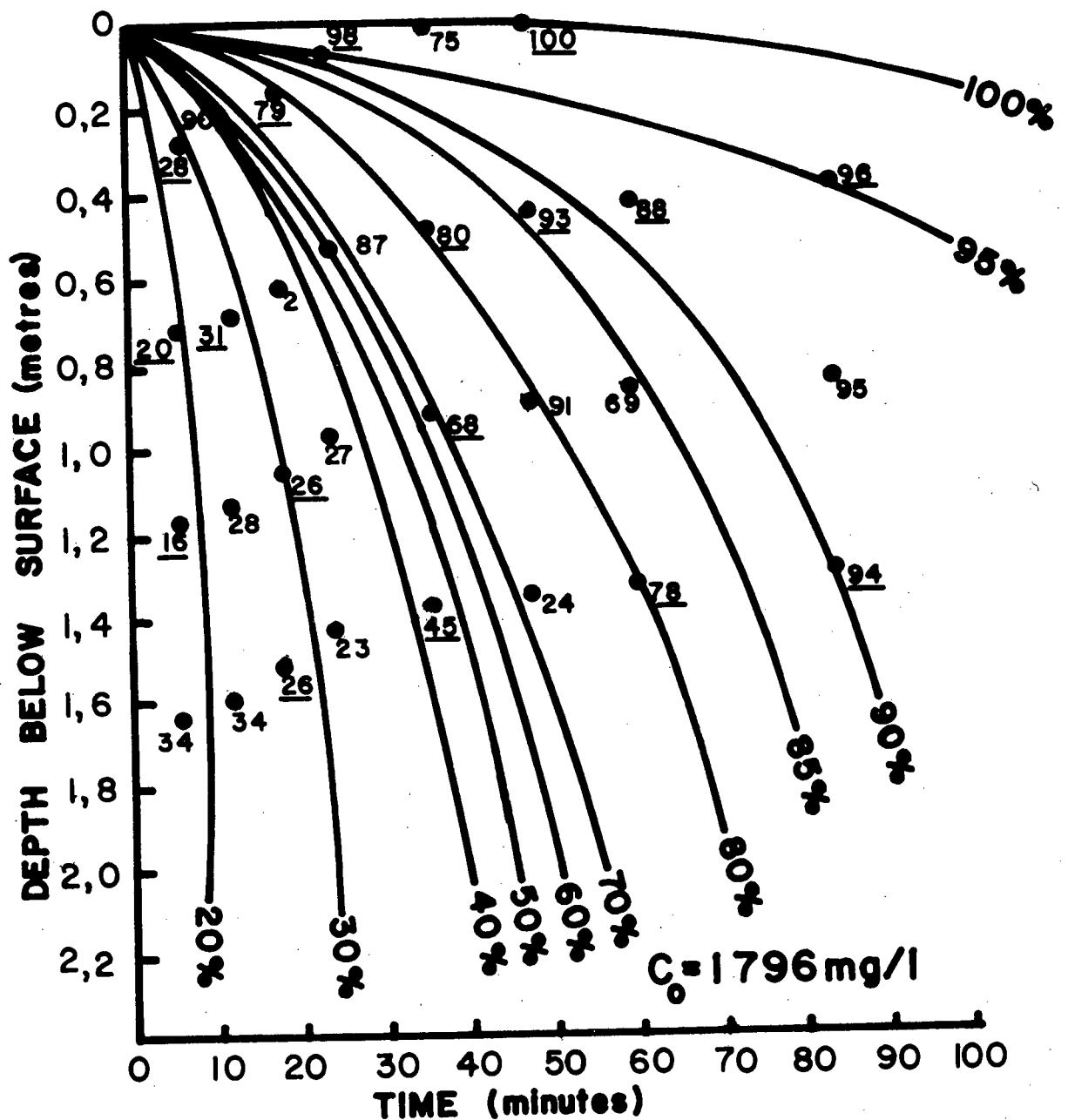


FIG. 5.

SEDIMENTATION
COLUMN 0,055m
DIAMETER.

**FIG. 6. SETTLING PATHS OF EFFLUENT "A"
FLOC: PERCENTAGE SUSPENDED
SOLIDS REMOVAL AT INDICATED
DEPTHS AND TIMES.**

(TABLE 2A)



If $H_2(m)$ represents the height of the interface at time t_2 (hr), and since it has been shown by equation (3) that the upward velocity U m/hr of any specific layer is constant,

$$U_2 = H_2/t_2.$$

Substituting for U_2 in equation (4), and solving for C_2 ,

$$C_2 = \frac{C_o H_o}{H_2 + V_2 t_2} \quad (5).$$

A plot of the height of the interface against time results in a settling curve as shown below in diagram II, and in Figs. 8 to 12 inclusive.

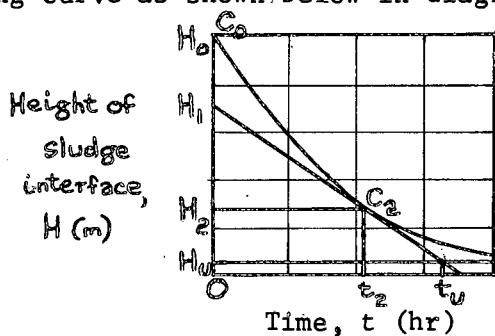


Diagram II. Typical settling curve obtained in a batch settling test.

V_2 is equal to dH/dt at the point on the plot at which the layer having a concentration of C_2 (kg/m^3) comes to the surface of the floc. V_2 is then the slope of the tangent to the curve at the point (H_2, t_2) .

It follows geometrically that the intercept of this tangent on the H axis is

$$H_1 = H_2 + (H_1 - H_2).$$

But
$$V_2 = (H_1 - H_2)/t_2,$$

therefore
$$(H_1 - H_2) = V_2 t_2.$$

Hence,
$$H_1 = H_2 + V_2 t_2.$$

Substituting H_1 for $H_2 + V_2 t_2$ in equation (5),

$$C_2 = C_o H_o / H_1,$$

or $C_2 H_1 = C_o H_o \quad (6).$

From equation (6) it follows that H_1 , the intercept height, is the height the floc would occupy if all the solids present were at the same concentration as the layer at the settling floc/water interface. Thus, for any arbitrarily selected value of C_2 , the corresponding value of H_1 may be calculated from (6). V_2 can then be determined as the slope of the line drawn through point H_1 on the H-axis and tangential to the settling curve. A complete set of data showing V as $f(C)$ can therefore be developed from one batch settling test.

2.2.1. Unit Area and Mass Loading.

To specify the area requirement of a thickener, the concentration layer requiring the maximum area to pass a unit mass of solids must be determined. This may be done by calculating the unit area required for a series of concentrations, using the data showing V as a function of C , $f(C)$, developed in the previous section (2.2.). Whichever concentration layer gives the largest value of unit area is then used as a design basis. Talmage and Fitch⁽⁷⁾ used a simple geometrical construction on the settling curve (Diagram II) to obtain the unit areas directly.

At time t_2 (hr) the solids in the layer existing at the surface of the column of floc are settling at a linear rate of

$$V_2 = (H_1 - H_2)/t_2 \text{ (m/hr)}$$

(i.e. the slope of the H vs. t curve).

If the solids of this layer are assumed as a datum, water is passing the solids at a bulk rate of

$$A(H_1 - H_2)/t_2 \text{ m}^3/\text{hr}.$$

In a continuous thickener, the solids in any zone do not have to settle past all of the water in the zone, since part of this water will accompany the solids to the underflow. The amount of water they must settle past is equal rather to the amount which would be released in bringing solids from layer concentration to underflow concentration. The corresponding quantity of water for the solids present in the batch test would be

$$A(H_1 - H_u) m^3,$$

since H_1 is the height the floc would occupy if all solids present in the batch test were at layer concentration, and H_u is the height the floc would occupy if all solids present in the batch test were at the thickener underflow concentration. The time that would be required to release $A(H_1 - H_u) m^3$ of water through a layer concentration $C_2 \text{ kg/m}^3$ would then be

$$t' = \frac{\text{amount of water to eliminate}}{\text{rate of eliminating water}} = \frac{A(H_1 - H_u)}{A(H_1 - H_2)/t_2} \quad (\text{hr}).$$

Rearranging,

$$t' / (H_1 - H_u) = t_2 / (H_1 - H_2).$$

Referring to the settling curve in Diag. II, by similar triangles,

$$t_2 / (H_1 - H_2) = t_u / (H_1 - H_u);$$

by equating the relationship in the latter equation to that in the previous one,

$$t' = t_u.$$

$$\text{Therefore, } t_u = (H_1 - H_u)t_2 / (H_1 - H_2) \quad (7).$$

The total quantity of solids in the batch test is $C_0 H_0 A \text{ kg}$ and it would take time $t_u \text{ hr}$ for this mass of solids to subside past a layer of

concentration C_2 kg/m³ in a continuous thickener. Therefore, the mass of solids that could be brought through layer concentration C_2 per unit time is

$$C_2 H_o A / t_u \text{ kg/hr,}$$

$$\text{or per unit area, ML} = C_2 H_o / t_u \text{ kg/hr/m}^2 \quad (8).$$

"Unit Area" of a layer is, by definition, the area required to allow unit mass of solids to subside through the layer concentration in unit time. Thus, by (8),

$$\text{Unit Area, UA} = t_u / C_2 H_o \quad (9).$$

The "Mass Loading" is the reciprocal of unit area, and is defined as the mass of solids passed per unit area per unit time, as shown in equation (8).

The method for determining the unit area corresponding to any floc concentration C_2 (the value of C_2 is not known, but a series of values may be selected) in the free settling range is therefore as follows:-

1. Determine H_1 and H_u from the following material balances:

point H_1 corresponding to an arbitrarily selected concentration,

C_2 ; from equation (6),

$$H_1 = C_2 H_o / C_2 \quad (6a)$$

$$\text{and } C_2 H_o = C_2 H_1 = C_u H_u,$$

$$\text{whence } H_u = C_2 H_o / C_u \quad (10),$$

where C_u is the underflow concentration (kg/m³) at time t_u .

2. Draw an "underflow" line parallel to the time axis at $H = H_u$ on the settling curve (diagram II, and Figs. 8 - 12 inclusive).

3. Draw a tangent to the settling curve through point H_1 on the H axis.

4. Read t_u at the intersection of the tangent and the underflow line.

5. Calculate unit area from equation (9),

$$UA = t_u / C_u H_u$$

When the underflow line, H_u , intersects the settling curve above the point where the layer at the surface of the floc goes into compression, the time t_u corresponding to maximum unit area will be the co-ordinate of the intersection, since any other tangent will intersect the underflow line at a lesser value of t_u . When the underflow line intersects the settling curve below the point where the layer at the surface of the floc goes into compression, the tangent giving maximum unit area will be drawn through this compression point, since this tangent gives the highest value of t_u in the free settling range and only free settling zones govern the unit area.

It should be noted that Talmage and Fitch assumed the surface layer of the compression zone was the limiting layer for water removal from the thickening sludge below it. ⁽¹⁷⁾ The fallacy occurred in their further assumption that this limiting concentration had some particular value obtained by the geometric construction described above, and which was independent of the desired underflow concentration. However, for the purposes of calculation these assumptions have been adopted in the present investigation. Wills (1967) ⁽⁴⁾ outlined the Talmage and Fitch method without comment on the possibility of there being a fallacy in the assumptions.

As the underflow concentration C_u increases, H_u decreases [from equation (10)] and t_u increases [by equation (7), and reference to the settling curve]. Therefore, from equation (9) the unit area UA increases. As C_u approaches its maximum concentration C_∞ , the interfacial velocity approaches zero.

Equation (9) shows that the unit area UA will vary with the initial floc solids concentration, C_o , so that the area required for the entire range of expected influent solids concentrations needs to be calculated. For each value of C_u in each of the influent concentrations in this range, the interfacial settling velocity on the settling curve given by

$$V_2 = (H_1 - H_2) / t_2 = U_1,$$

is computed. The corresponding values of the unit areas UA are plotted against the interfacial settling velocities (U_1), as shown in Figs. 13 and 14, respectively. At each underflow concentration, C_u , the maximum value of UA may be read from the plots. The maximum unit area UA existing for each value of C_u is observed to be related to a particular or "critical" interfacial settling velocity.

The time required for the settling sludge to initially go into compression, t_c (hr) (i.e. the time for commencement of consolidation), is required so that the minimum average retention time may be calculated for a continuous thickener.

2.2.2. Estimation of the Time for Initial Sludge Compression

Eckenfelder and Melbinger (1957)⁸ suggest that the compression point can be estimated from the settling curve by projecting tangents from the hindered settling zone and the compression zone, and bisecting the angle formed by the two tangents. The point at which this bisection intersects the settling curve is the approximate compression point. Roberts (1949)⁹ has shown that the rate of sludge thickening with time can be expressed by the relationship,

$$\log \frac{H - H_\infty}{H_c - H_\infty} = K (t - t_c) \quad (11),$$

where H_{∞} is the minimum height the sludge reaches at a maximum concentration C_{∞} , and can be determined from a trial-and-error plot of $\log (H - H_{\infty})$ against time. The correct value of H_{∞} will yield a linear plot from the time t_c that the sludge goes into compression until a later time t when the sludge height is at an intermediate value H . The slope of this plot is K . When t_c is greater than t , the sludge is not initially in compression.

To assist in the estimation of H_{∞} , it has been found in the present investigation that H_{∞} is approximately 0,4 - 0,5 of the sludge height obtained by settling sludge samples for 18 hours. This approximate factor was obtained by centrifugation of the 18-hour settled sludge, the compacted sludge volume thus obtained being assumed to be an estimate of H_{∞} in relation to the 18-hour volume. These values are shown in the Appendix (contained in Tables 2a, 2b and 2c respectively). Values of $(H - H_{\infty})$ so obtained are given in Table 6 in the Appendix, and the plots of $(H - H_{\infty})$ against time are shown in Fig. 7. Time t_c occurs at the beginning of the linear portion of the curves, and these values are indicated on the sedimentation curves (Figs. 8 - 12 inclusive). Figs. 8, 9 and 10, respectively, additionally show the initial compression point determined by the bisection of the tangents from the hindered settling zone and the compression zone respectively. Only in the case of Fig. 8 were these latter points (obtained by bisection of tangents) coincident with those obtained by the plot of equation (11).

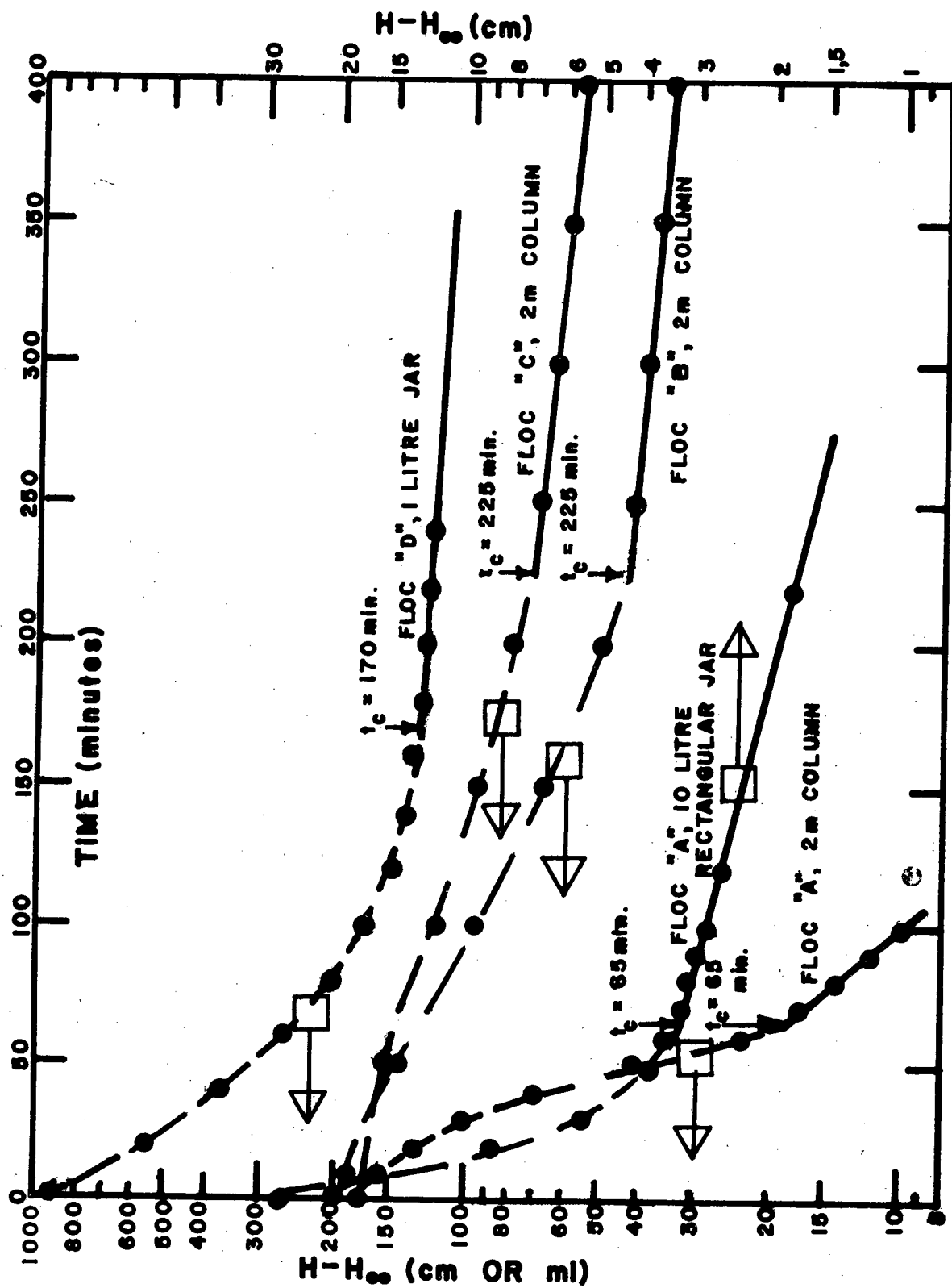


FIG. 7. TIME FOR INITIAL SLUDGE COMPRESSION, t_c .

(TABLE 6)

FIG. 8: SEDIMENTATION CURVE FOR
EFFLUENT "A" FLOC IN 2m
COLUMN.

(TABLE 1A)

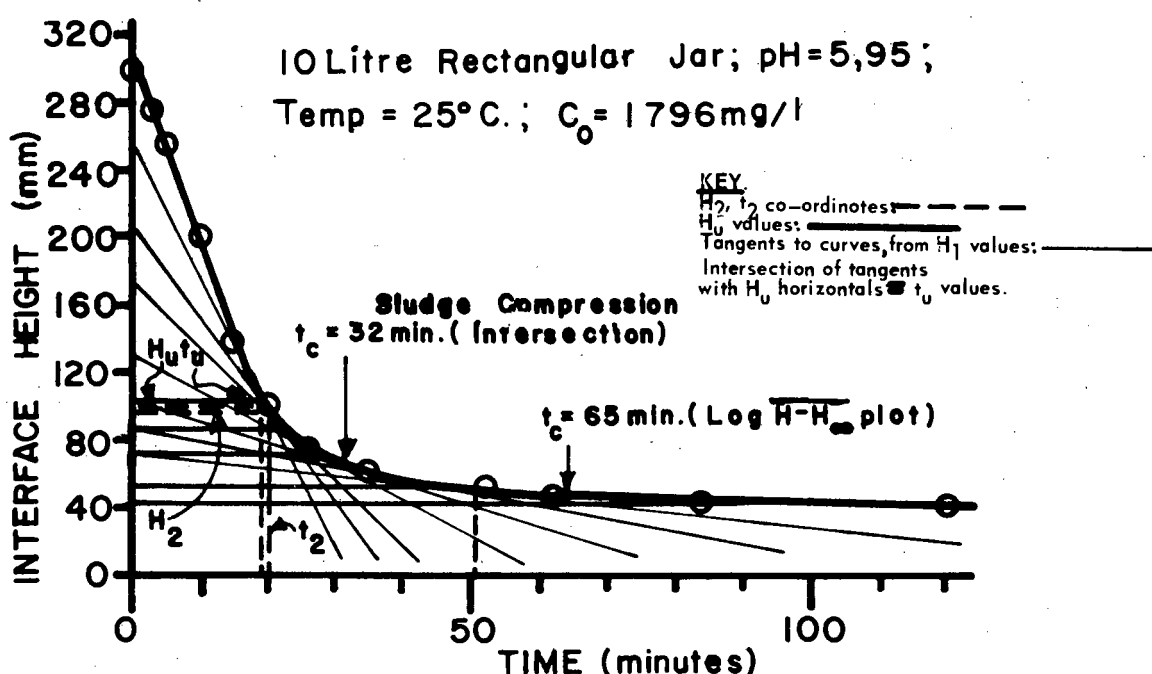
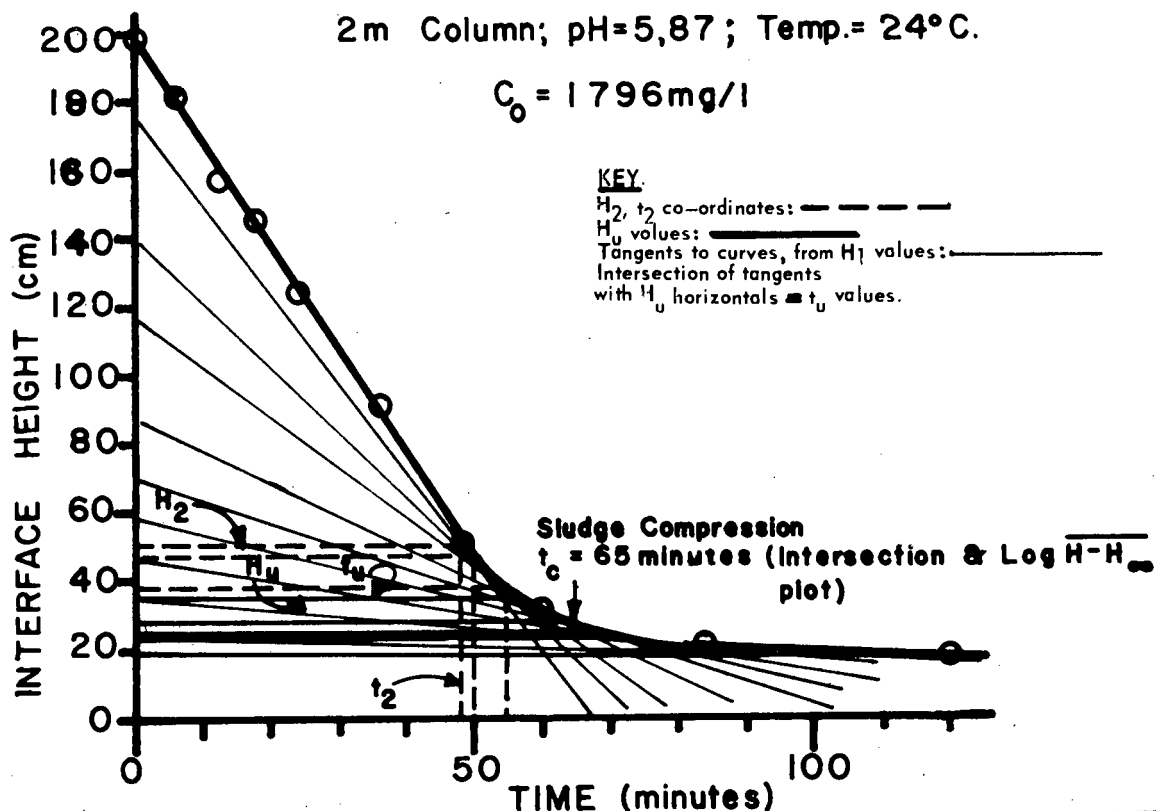


FIG. 9: SEDIMENTATION CURVE FOR
EFFLUENT "A" FLOC IN 10 litre
JAR. (TABLE 2d)

FIG. 10: SEDIMENTATION CURVE FOR EFFLUENT "B" FLOC IN 2m COLUMN.

(TABLE 1B)

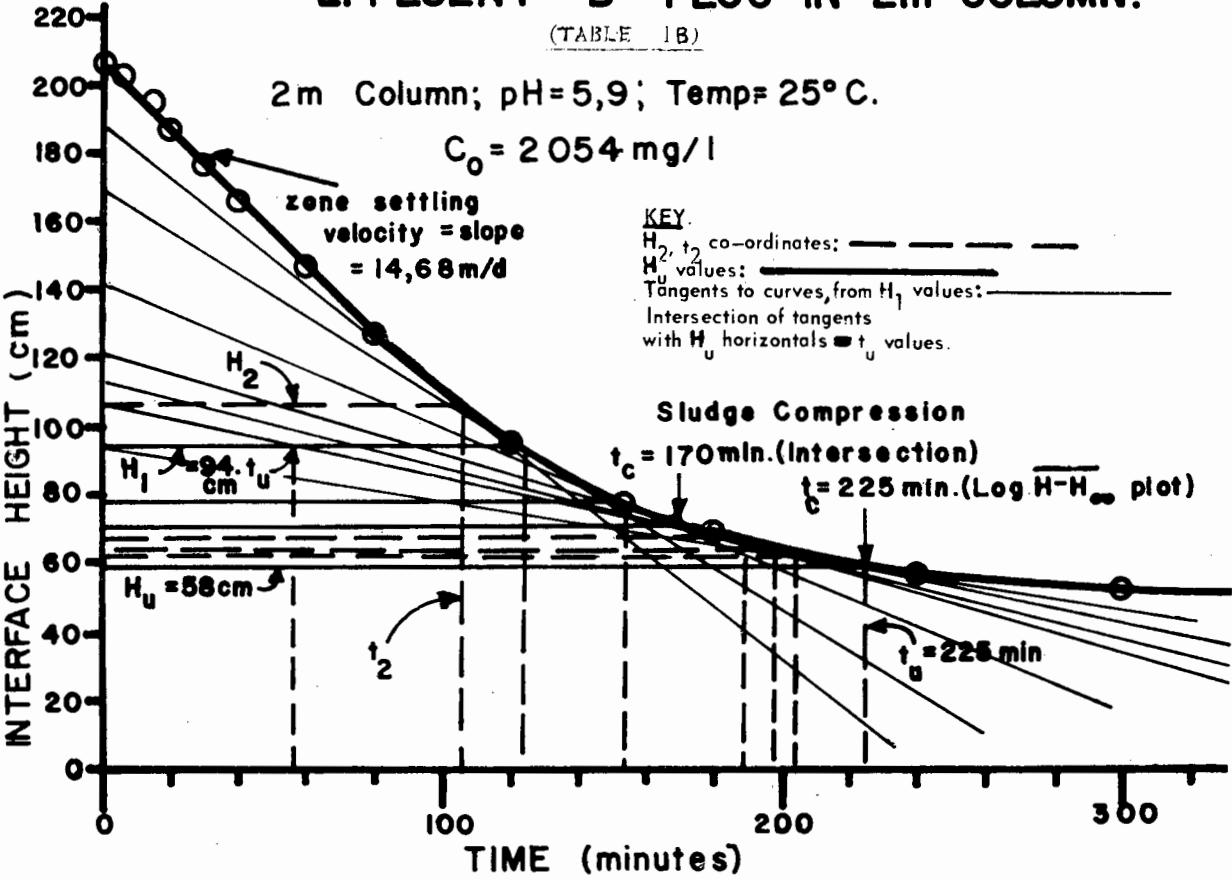


FIG. 11: SEDIMENTATION CURVE FOR EFFLUENT "C" FLOC IN 2m COLUMN. (TABLE 1C)

2m Column; pH= 5,9 ; Temp.= 25° C.

$C_0 = 2018 \text{ mg/l}$

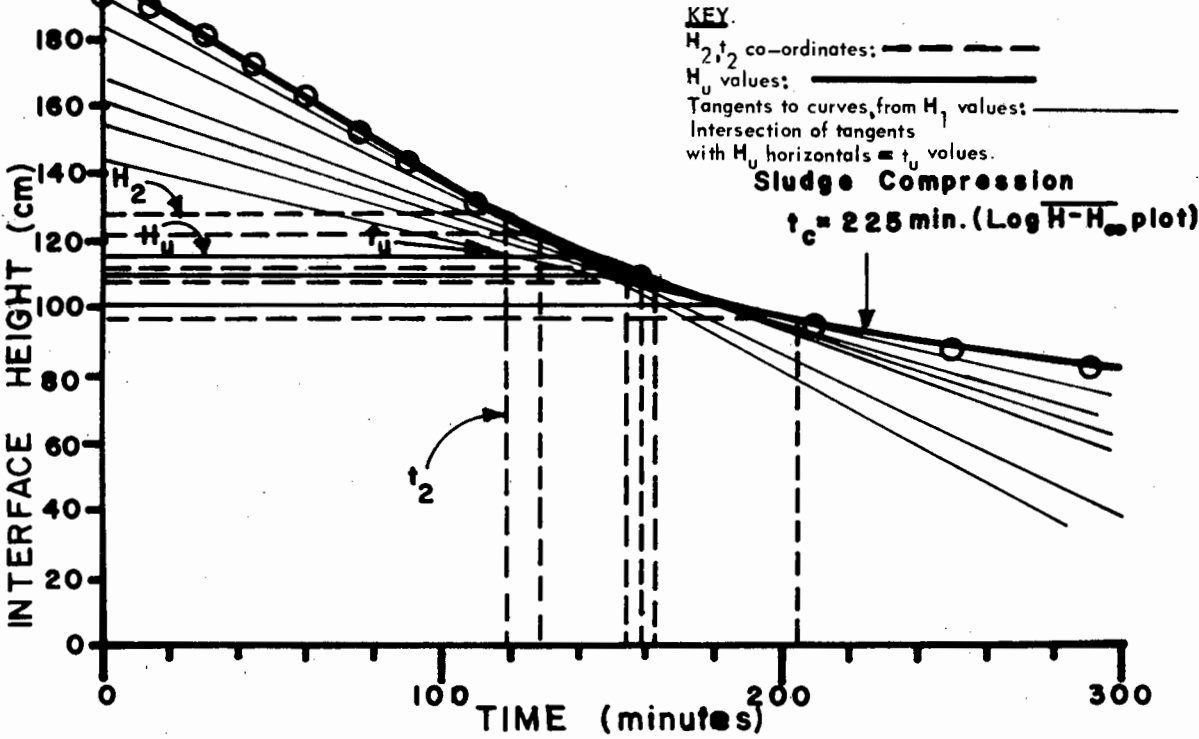
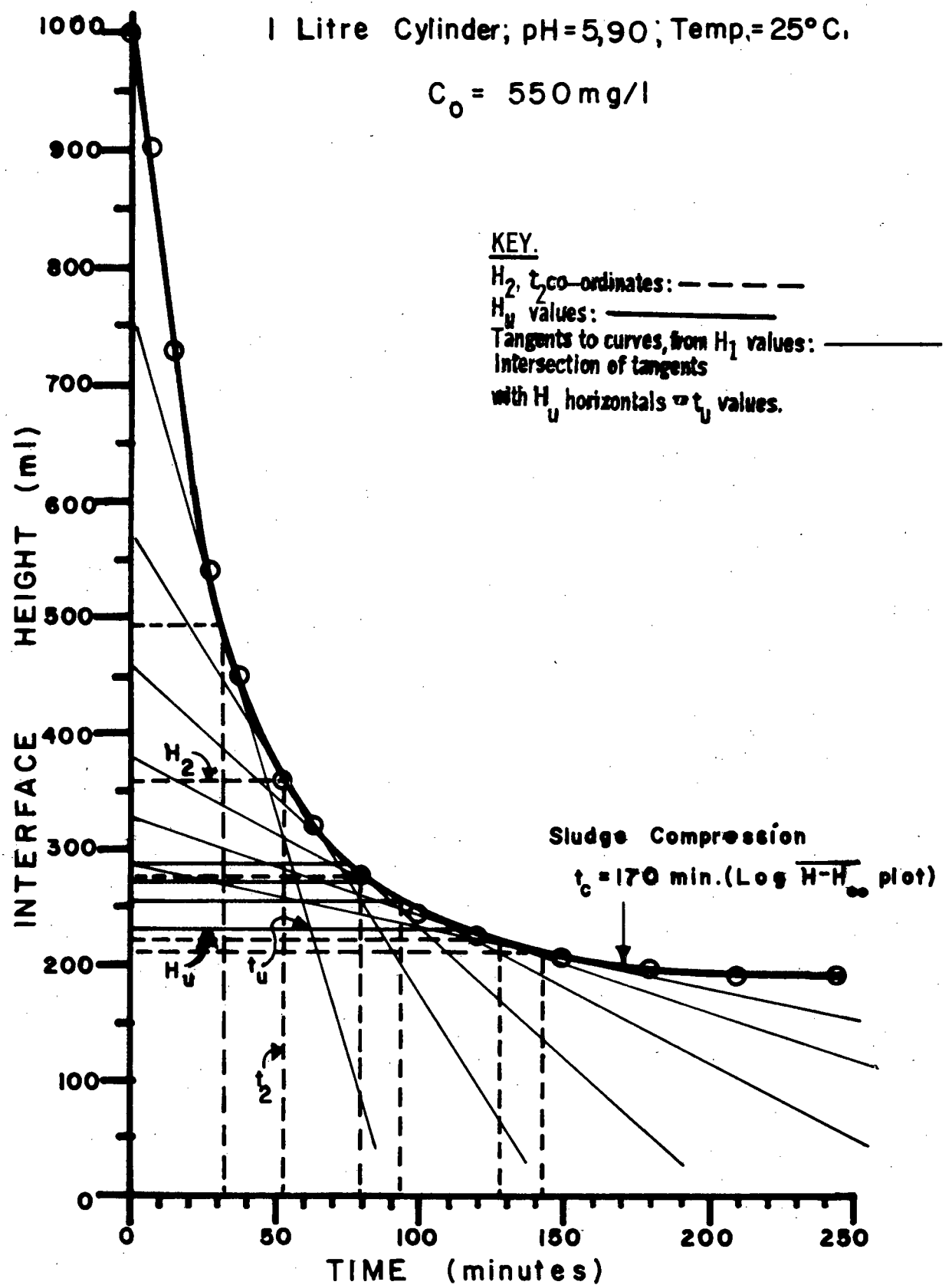


**FIG. 12: SEDIMENTATION CURVE FOR
EFFLUENT "D" FLOC IN 1-litre
MEASURING CYLINDER.**

(TABLE 2e)

1 Litre Cylinder; pH=5.90; Temp.=25°C.

$C_0 = 550 \text{ mg/l}$



2.2.3. Correlation between Mass Loading and Underflow Concentration in Thickening.

Eckenfelder ^(2a) has given a correlation between the mass loading, $ML = 1/UA$, and underflow sludge concentration,

$$(C_u/C_o - 1) = K_b / (ML)^n \quad (12)$$

(the mass loading value employed in this equation being the reciprocal of the UA_{max} values obtained from the curves of the plots of UA against interfacial settling velocity in Figs. 13 and 14),

A plot of $\log (C_u/C_o - 1)$ vs $\log (ML)$ yields a set of straight lines, one for each value of the initial concentration C_o , and all having the same slope, n . The limits of equation (12) are at $C_u = C_\infty$ and $C_u = C_c$, where C_c is the concentration at the start of compression. K_b is a constant for a given initial sludge concentration, C_o , and is the intercept on the plot at $(ML) = 1$. For the sludge investigated in this work, it was found that

$$(C_u/C_o - 1) = K_b / (ML)^{1.28}$$

The intercepts of each of these plots on a vertical axis where $(ML) = 1$ [i.e. $\log (ML) = 0$] gives the value of the constant, K_b (Fig. 15). A further plot of $\log K_b$ vs C_o yields another straight line which may be employed for design purposes for any selected or actual value of C_o (Fig. 16). K_b is seen to be an exponential function of the initial sludge solids concentration, C_o , and according to Eckenfelder ^(2a), also a function of the height of the sludge layer.

Thus,

$$K_b = k' e^{mC_o} \quad (12a),$$

where k' and m are experimentally determined constants, k' being the intercept of the semi-log plot at $C_o = 0$, and m being the slope of the line.

FIG. 13: DETERMINATION OF MAXIMUM UNIT AREA, UA_{max} , FOR FLOC 'A'.

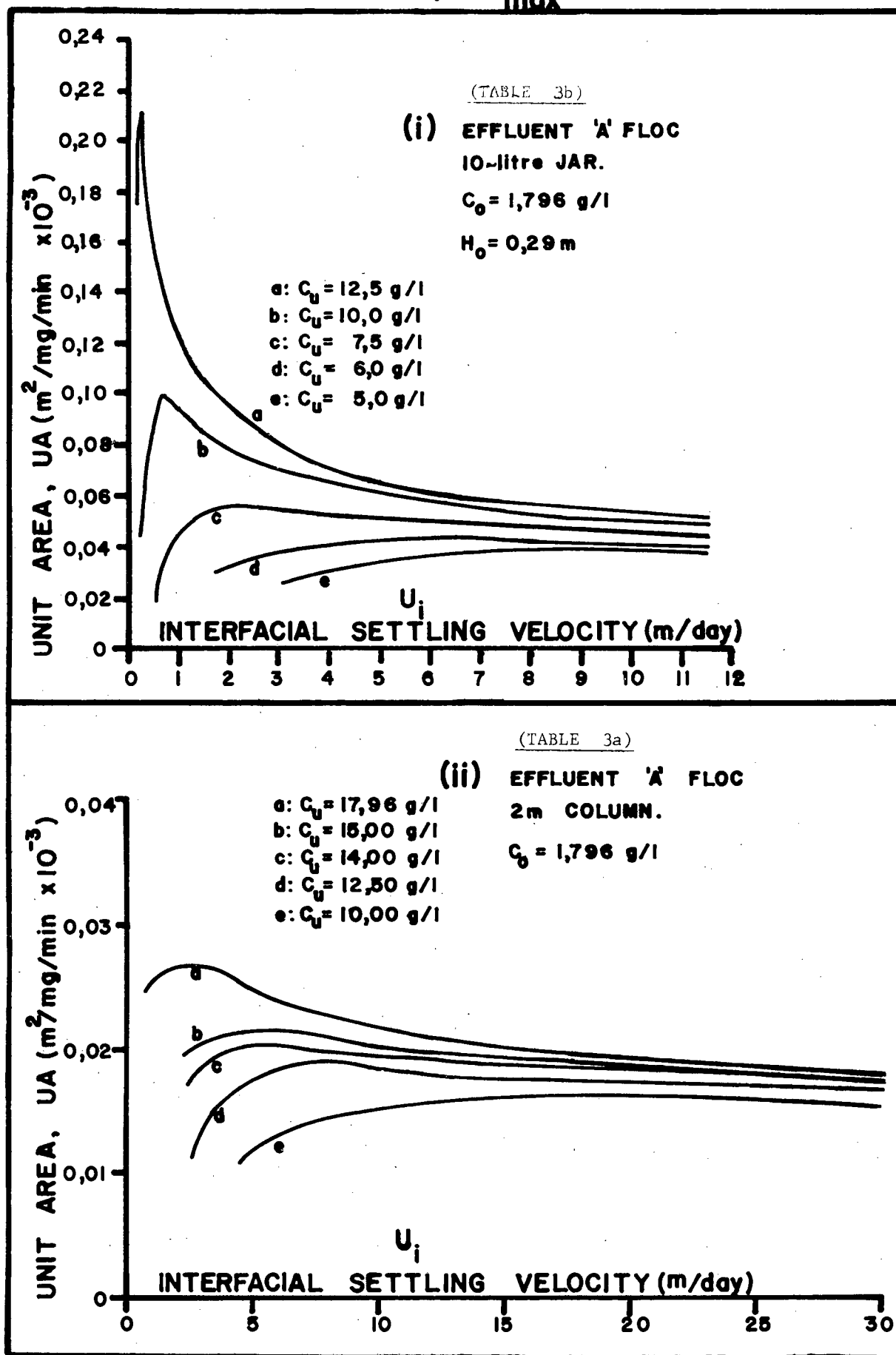


FIG. 14: DETERMINATION OF MAXIMUM UNIT AREA, UA_{max} , FOR FLOCS 'B', 'C', & 'D'.

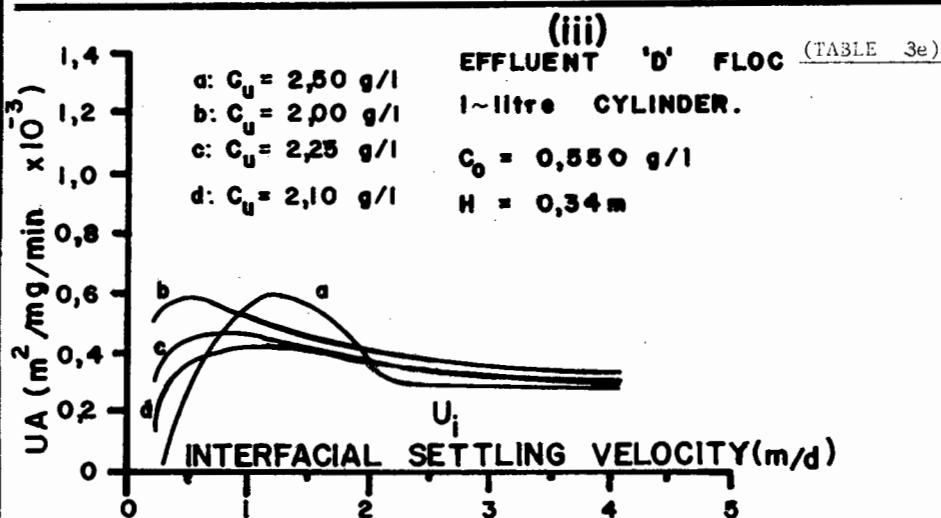
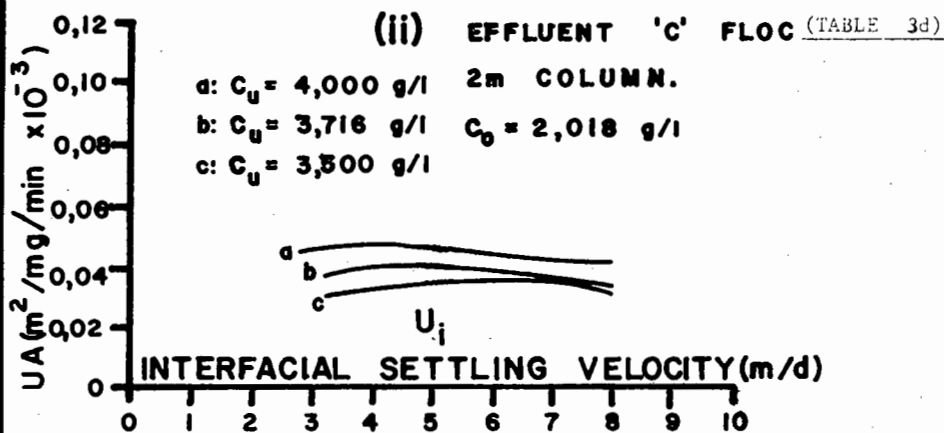
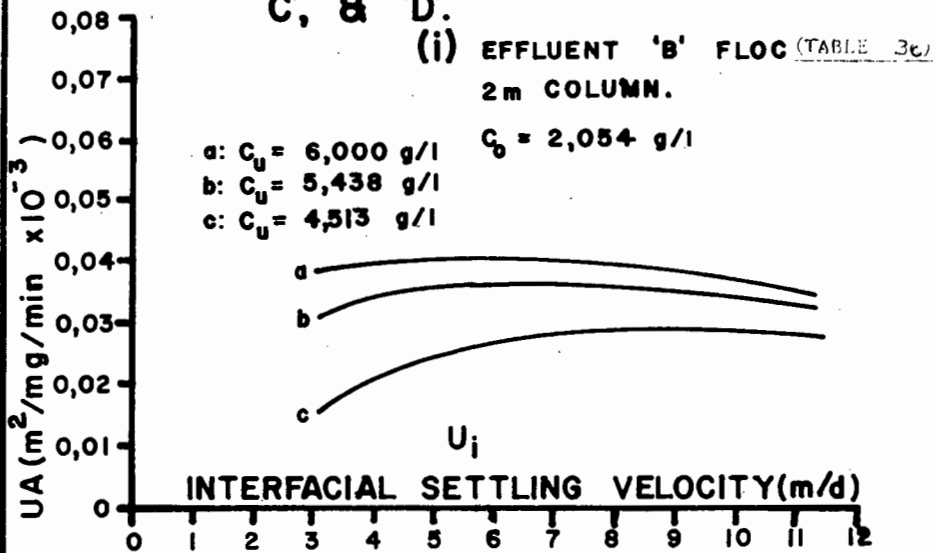
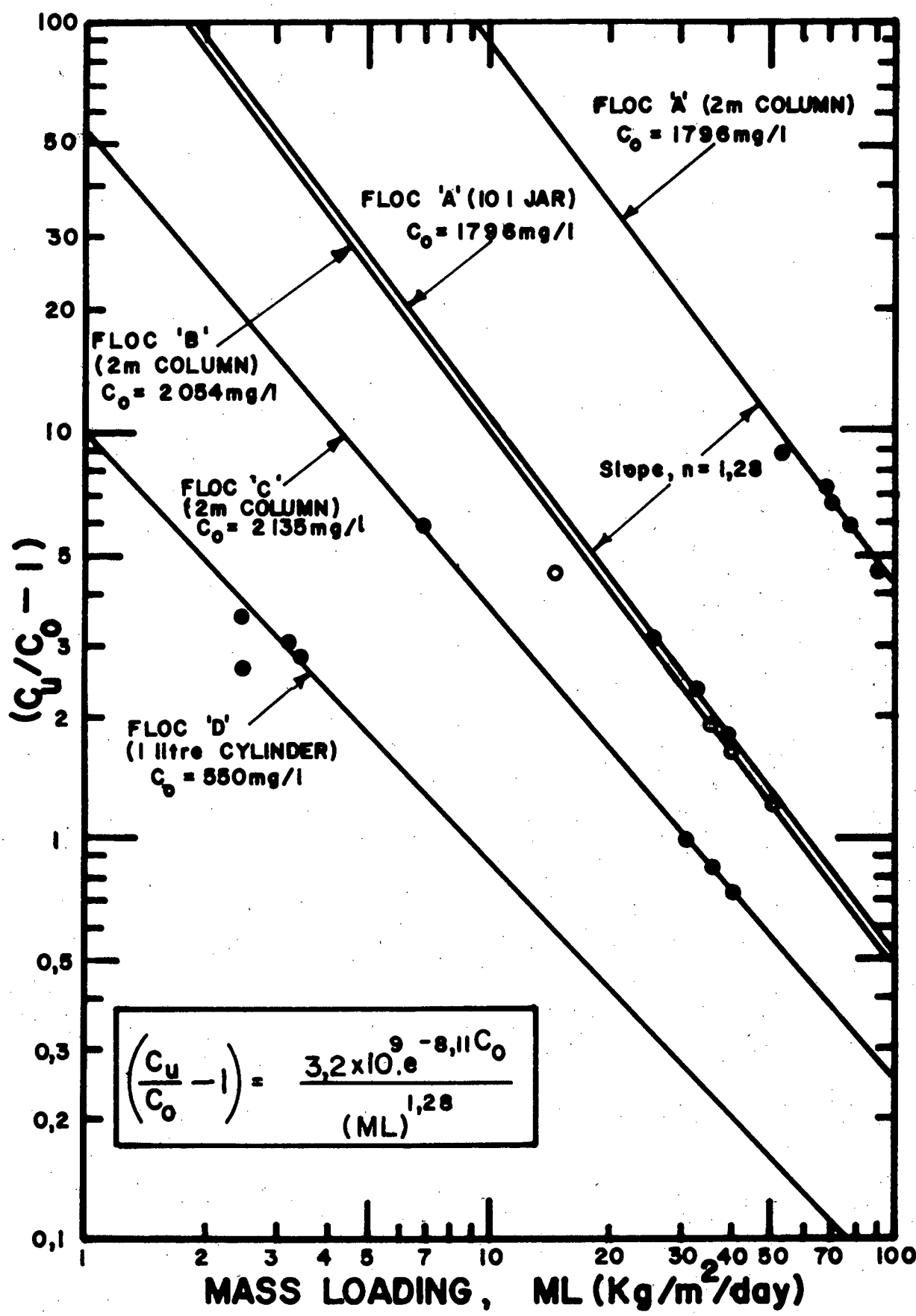


FIG. 15: RELATIONSHIP BETWEEN MASS
LOADING & SOLIDS UNDERFLOW
CONCENTRATION.

(TABLE 4)



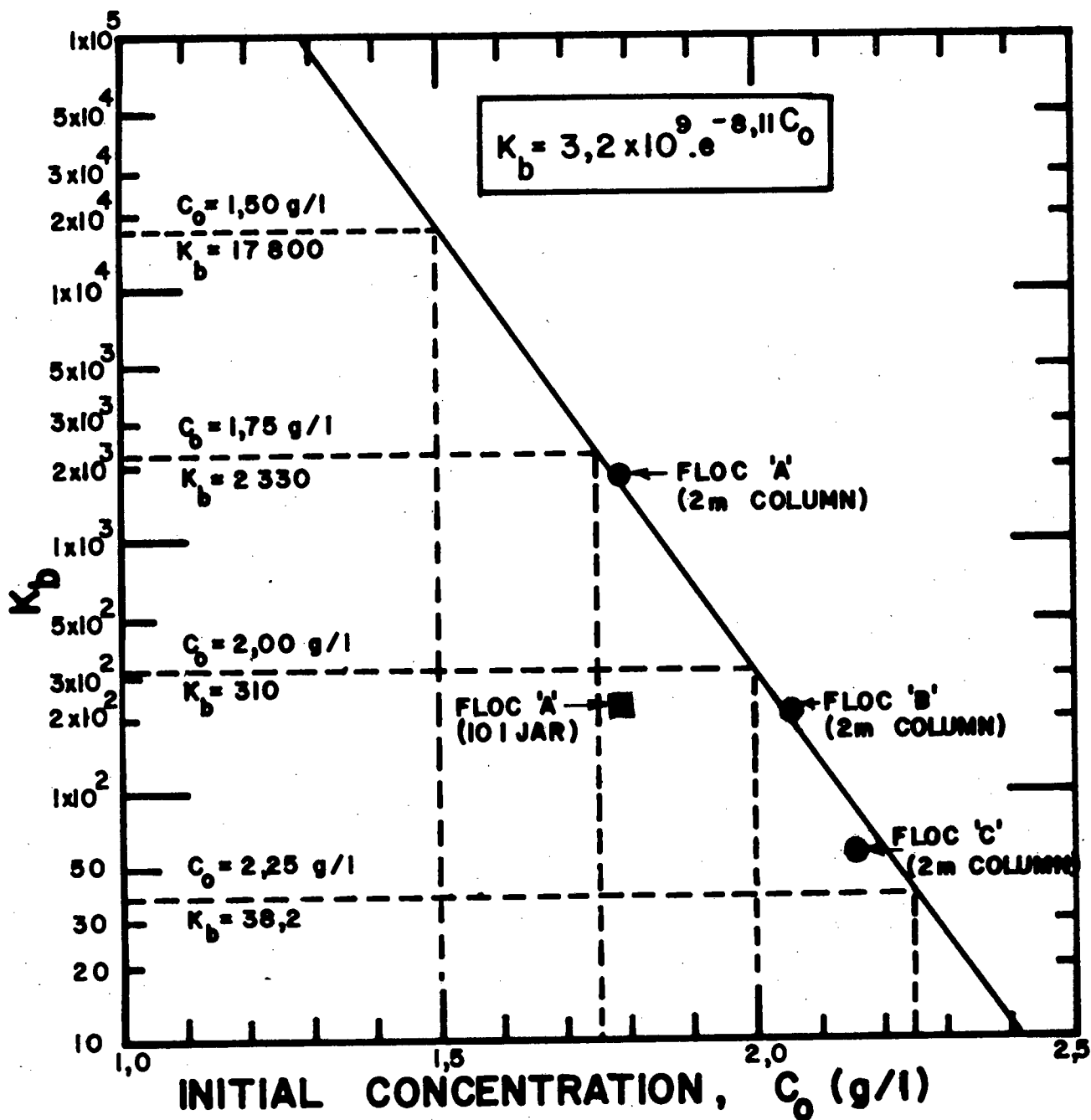


FIG. 16: RELATIONSHIP BETWEEN CONSTANT K_b & INITIAL SLUDGE CONCENTRATION.

(TABLE 5)

In the present investigation, it was found that

$$K_b = 3,2 \times 10^9 e^{-8,11C_o}$$

In the scale-up of a laboratory sedimentation test to actual continuous operation in a plant, the curves relating a tank constant K_T to the laboratory apparatus constant K_b must be known in order to apply a scaling-up correction factor. Usually a plot of K_T vs sludge blanket depth yields a curve such that the correction factors K_T/K_b are less than unity. This correction factor accounts for the lesser effects of tank walls on the sedimentation process in larger diameter vessels, and also any tendency for sludge scouring action in a continuous thickener.

In a continuous thickening operation, the sludge blanket height for a desired underflow concentration must be maintained.

The scaling-up factor K_T may not be significant if the laboratory test apparatus is similar in height to the intended tank and of sufficiently large diameter. In many instances it is not possible to determine K_T values unless a similar tank has operated with the particular floc in question. In these cases, the unit area is increased by an average design factor of 1,25 - 2,0.

2.2.4. Overflow Rate.

In thickening operations, the required unit area is obtained from equation (9),

$$UA = \frac{t_u}{C_o H_o} \quad m^2/kg/day.$$

Rearranging the terms,

$$\frac{H_o}{t_u} = \frac{1}{C_o (UA)},$$

where H_o/t_u has the dimensions of a velocity, m/d.

When C_o is expressed in kg/m^3 , the equivalent overflow rate is thus

$$1/C_o \text{ (UA)} \text{ m}^3/\text{m}^2/\text{d}.$$

This overflow rate is based on the solids loading ($1/\text{UA}$) and is not the same as the overflow rate for settling based on the settling velocities of the particles. In the latter case, the overflow rate in zone settling ($\text{m}^3/\text{m}^2/\text{d}$) is numerically equal to the settling velocity (m/d) of the smallest particle to be removed.

It is convenient to express the overflow rate based on the liquid throughput, given by

$$(\text{OR}) = \frac{1}{C_o \text{ (UA)}} \times \frac{(C_u - C_o)}{C_u} \quad (13).$$

Because $1 \text{ g} = \frac{1}{1000} \text{ kg}$, and $1 \text{ L} = \frac{1}{1000} \text{ m}^3$, expressing concentrations C_o and C_u in g/l, the units for (OR) in equation (13) remain as cubic metres per day/square metre.

For clarification, the initial zone settling velocity (m/day) is numerically equal to the (OR), ($\text{m}^3/\text{day}/\text{m}^2$) (calculated from equation (13)), and whichever overflow rate is the smaller value, will determine whether clarification or thickening controls the surface area design of the settling tank. For clarification, the average rise velocity of the overflowing supernatant must be less than the zone settling velocity of the floc.

For thickening, the tank bottom surface area for a desired underflow concentration is related to the solids loading to the unit:

In the process proposed for the waste investigated in this thesis, it is essential to obtain thickening of the flocculated sludge. The overflowing supernatant is, however, to be clear.

2.2.5. Sedimentation/Thickening Tank Design.

For any selected initial sludge concentration, C_o g/l, the mass of sludge obtained (kg/d) is given by..

$$W_s = [\text{Flow rate}] (m^3/d) \times C_o (g/l) \quad (14).$$

Dividing the mass of sludge obtained per day, W_s , [from equation (14)], by the calculated mass loading, (ML) ($kg/m^2/d$) [from equation (12)], the required surface area, $A m^2$, is obtained. In the case of cylindrical tanks, the diameter D (m) is given by

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{1,275 W_s}{(ML)}} \quad (15).$$

Equation (15) enables the calculation of corresponding tank diameters to provide designed unit areas necessary for different values of the underflow concentration C_u [from equations (12) or (13) respectively]. At a selected constant flow rate, $45 m^3/day$, a plot of $\log C_u$ against $\log D$ for different values of C_o yields a family of parallel straight lines (Fig. 17) such that for each plot

$$C_u = K_D D^n,$$

where K_D is the intercept at $D = 1$ and n is the slope. Plotting the intercept K_D of each parallel line of slope n with the vertical axis at $D = 1$ ($\log D = 0$) against the corresponding C_o value on a semi-log plot, a further straight line resulted (Fig. 18). The equation of this line was of the form

$$K_D = c' e^{mC_o},$$

where c' is the intercept at $C_o = 0$, and m is the slope. The constant K_D is thus an exponential function of C_o .

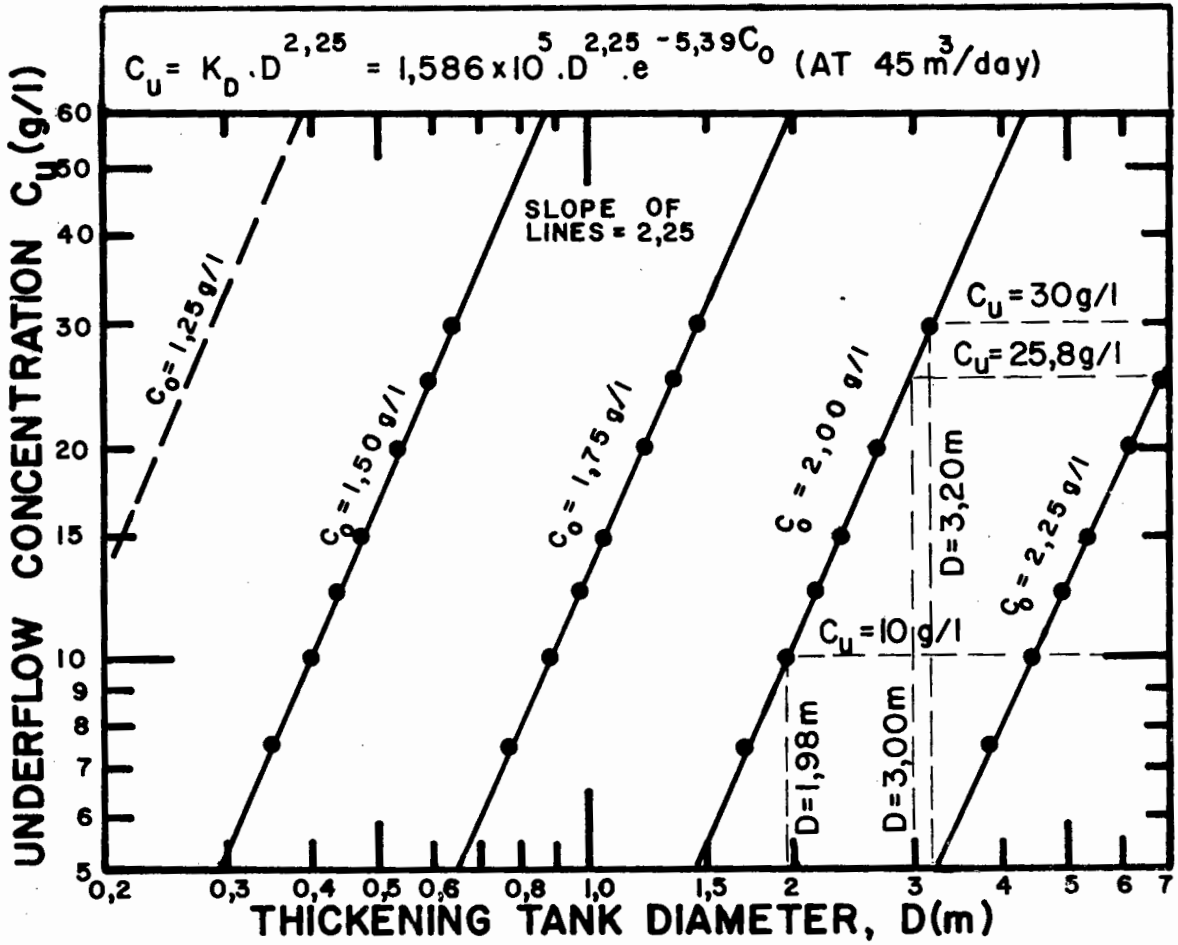


FIG. 17: SLUDGE CONCENTRATION – TANK DIAMETER CRITERIA FOR FLOW RATE OF $45 \text{ m}^3/\text{day}$.

(TABLE 7)

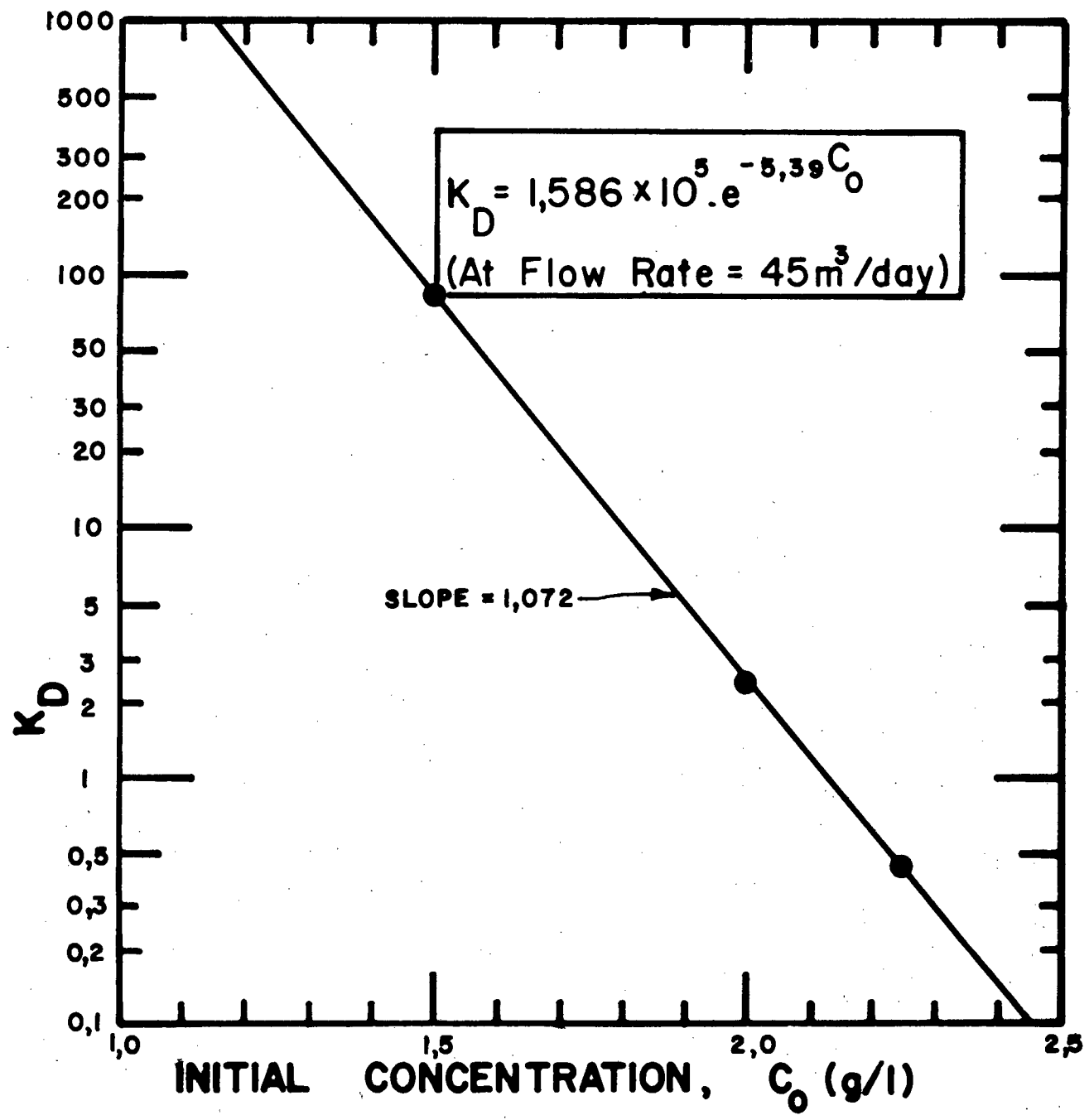


FIG. 18: RELATIONSHIP BETWEEN CONSTANT K_D & INITIAL SLUDGE CONCENTRATION.

FIG. 19a: SETTLING SLUDGE CONCENTRATION GRADIENTS WITH RESPECT TO THE INTERFACE AT DIFFERENT TIMES.

(TABLE 8a)

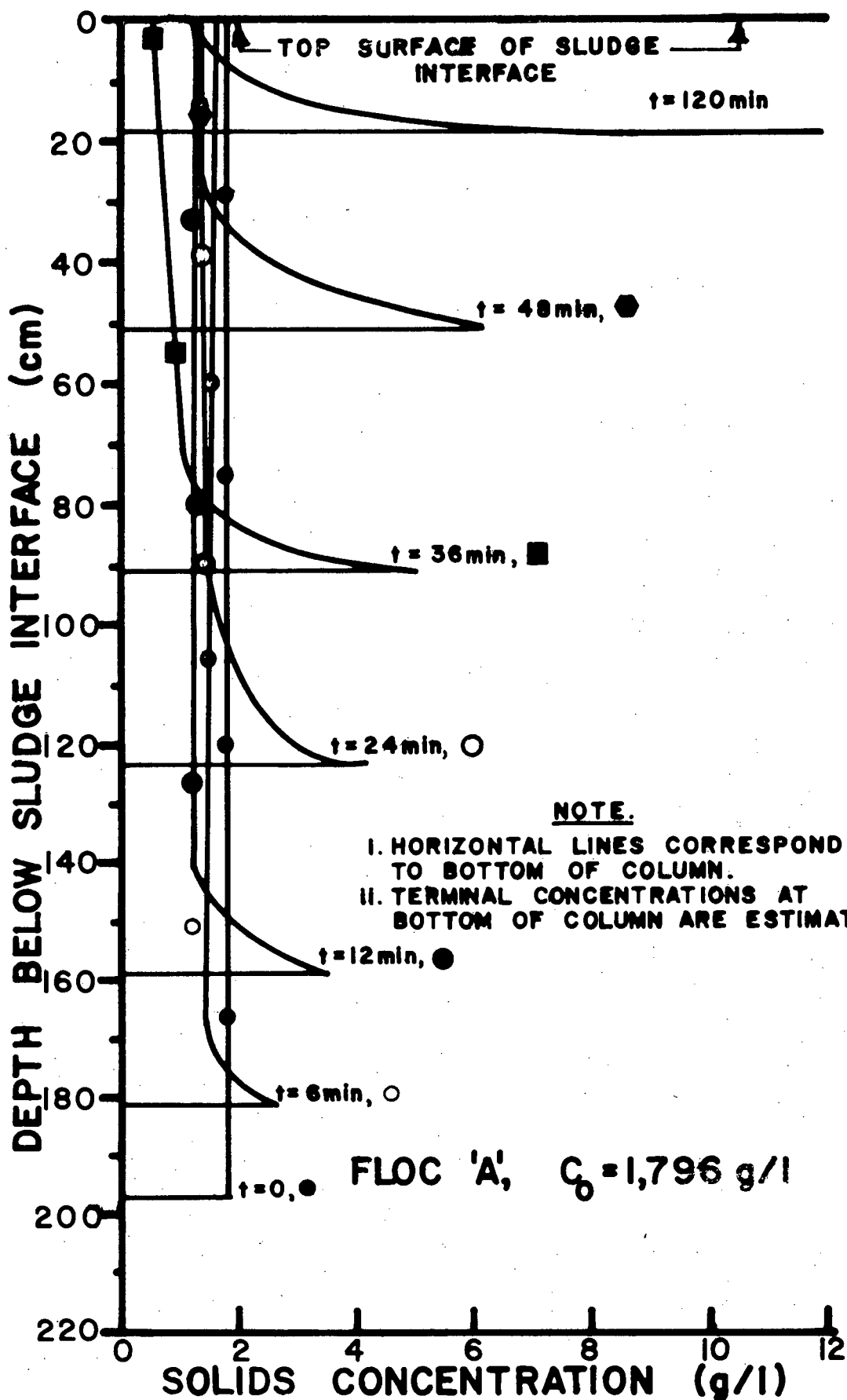
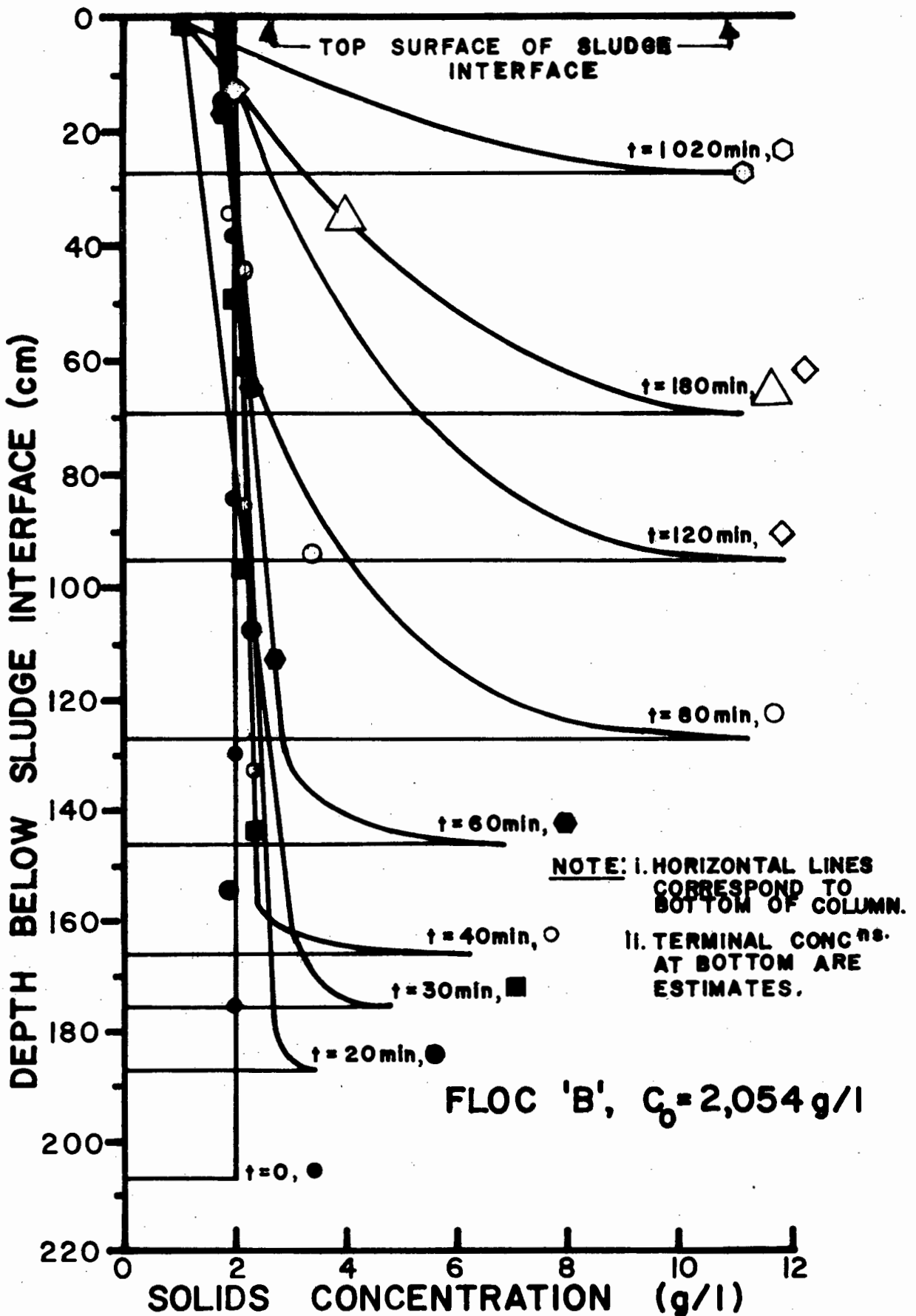


FIG. 19b: SETTLING SLUDGE CONCENTRATION GRADIENTS WITH RESPECT TO THE INTERFACE AT DIFFERENT TIMES.

(TABLE 8b)



In the present investigation, it was found that at a design flow rate of $45 \text{ m}^3/\text{d}$,

$$C_u = K_D D^n = 1,586 \times 10^5 D^{2,25} e^{-5,39 C_o} \quad (16).$$

$$\text{Hence, } D = \sqrt{\frac{C_u}{1,586 \times 10^5 e^{-5,39 C_o}}} \quad (17).$$

Equation (16) or (17) may be used in the design of thickening/clarification tanks with the flocculated sludge in question. They enable the calculation of a tank diameter, involving terms of the initial and underflow sludge concentrations at a fixed flow rate ($45 \text{ m}^3/\text{d}$). However, it follows that when the flow is less than $45 \text{ m}^3/\text{d}$, the reduced overflow rate allows a longer retention time in the sedimentation tank, with a corresponding decrease in sludge interface height and an increased underflow concentration. Correspondingly, at peak flow rates greater than $45 \text{ m}^3/\text{d}$ the converse will hold, and it would be useful to predict if the sludge height might be excessively high, and thus liable to cause scouring into the overflowing supernatant effluent.

2.2.6. The Effect of Flow Rate on Sedimentation/Thickening Tank Design.

Varying the flow rate Q (m^3/day), and computing the data necessary for plotting families of curves of $\log C_u$ against $\log D$ at different values of C_o (Figs. 20 and 21), the slopes of all the straight lines were found to be constant and independent of Q . [The slopes are normally distributed with a mean value of 2,165 and standard deviation 0,05 (Fig. 22)].

Plotting the respective intercepts, K_D , of the lines at $D = 1 \text{ m}$ against the initial concentration C_o on a semi-log plot, a family of parallel straight lines resulted for each value of Q (Q ranging from $15 \text{ m}^3/\text{d}$ to $75 \text{ m}^3/\text{d}$), as shown in Fig. 23.

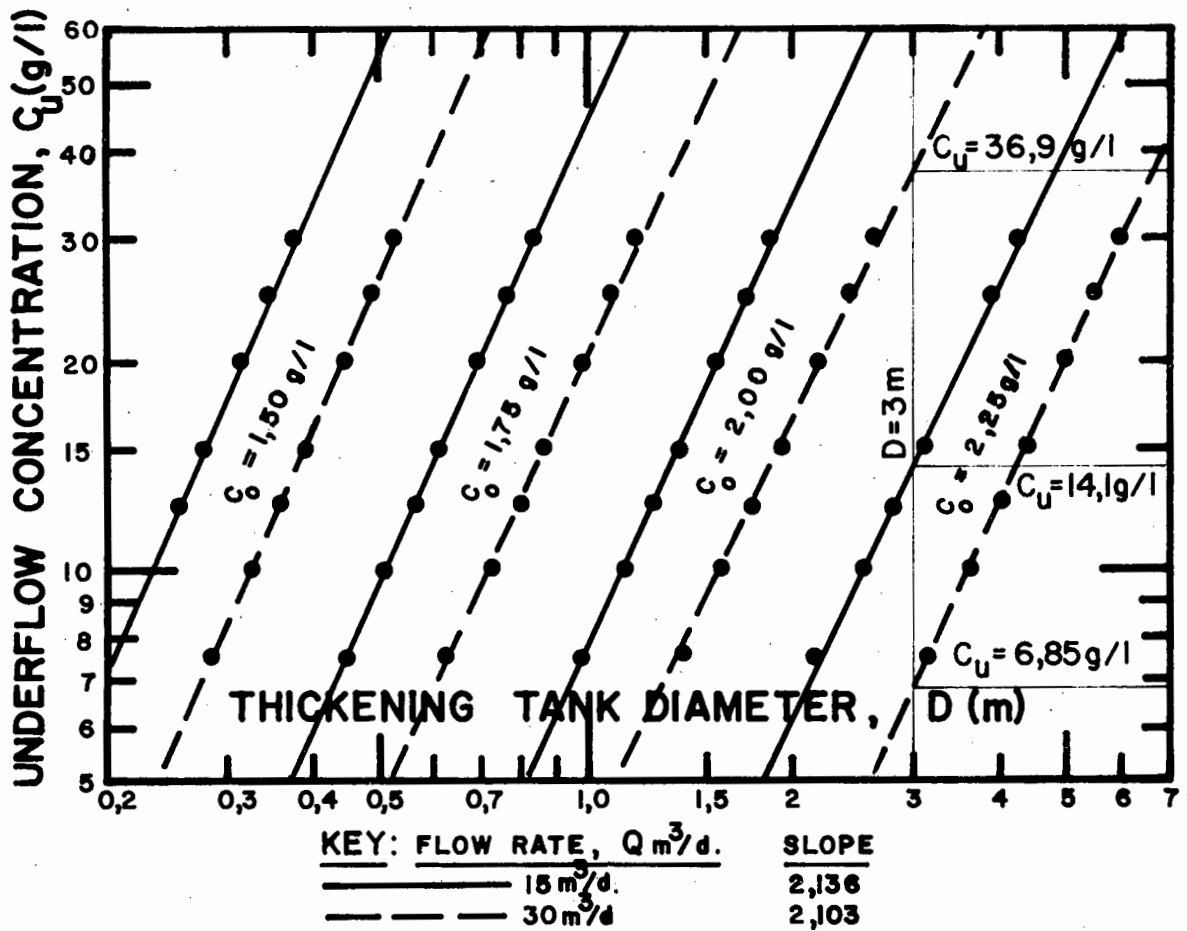


FIG. 20: SLUDGE CONCENTRATION—TANK DIAMETER RELATIONSHIPS FOR $Q=15$ & $30 \text{ m}^3/\text{d}$.

(TABLE 9)

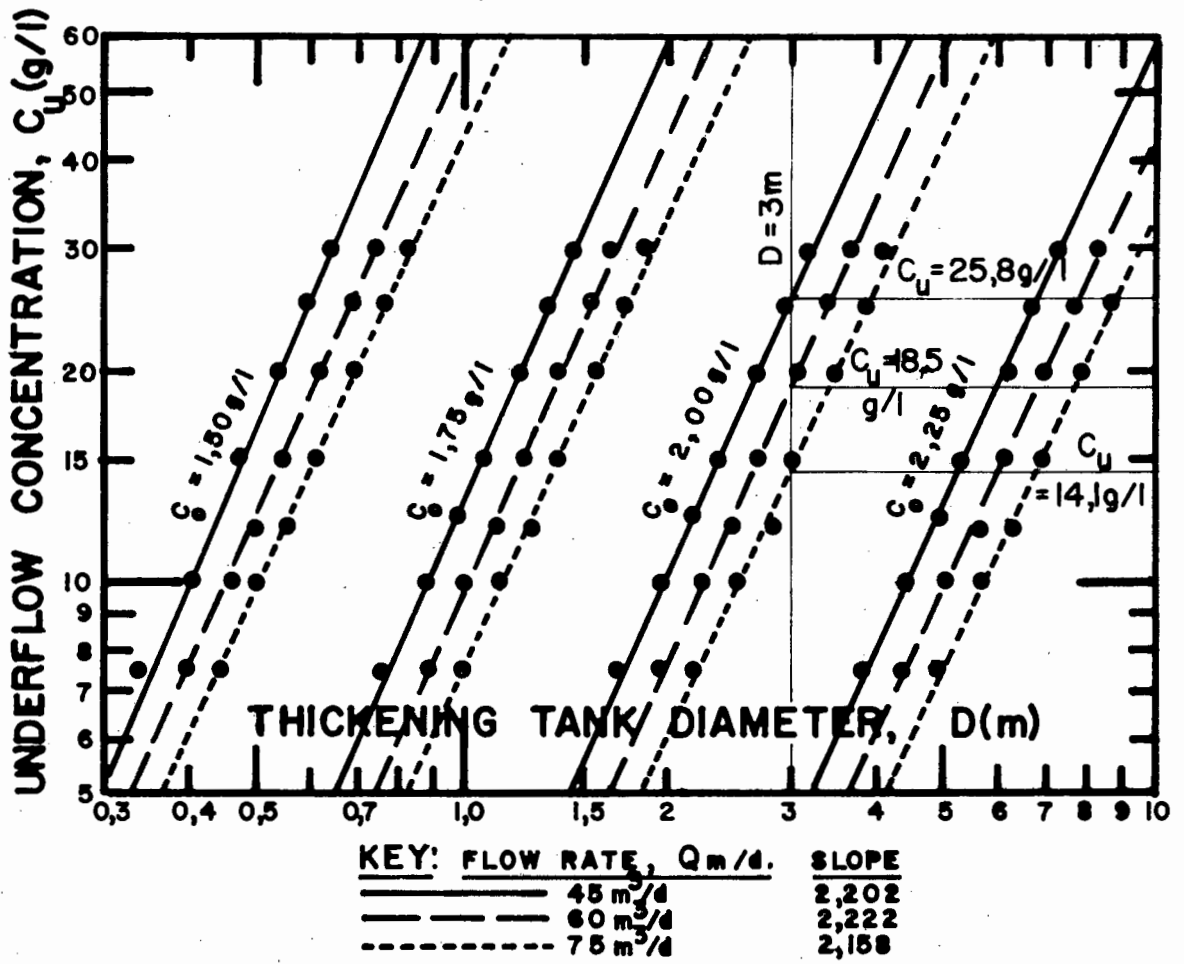


FIG. 21: SLUDGE CONCENTRATION-TANK DIAMETER RELATIONSHIPS FOR $Q = 45, 60, \text{ \& } 75 \text{ m}^3/\text{d}$.

(TABLE 9)

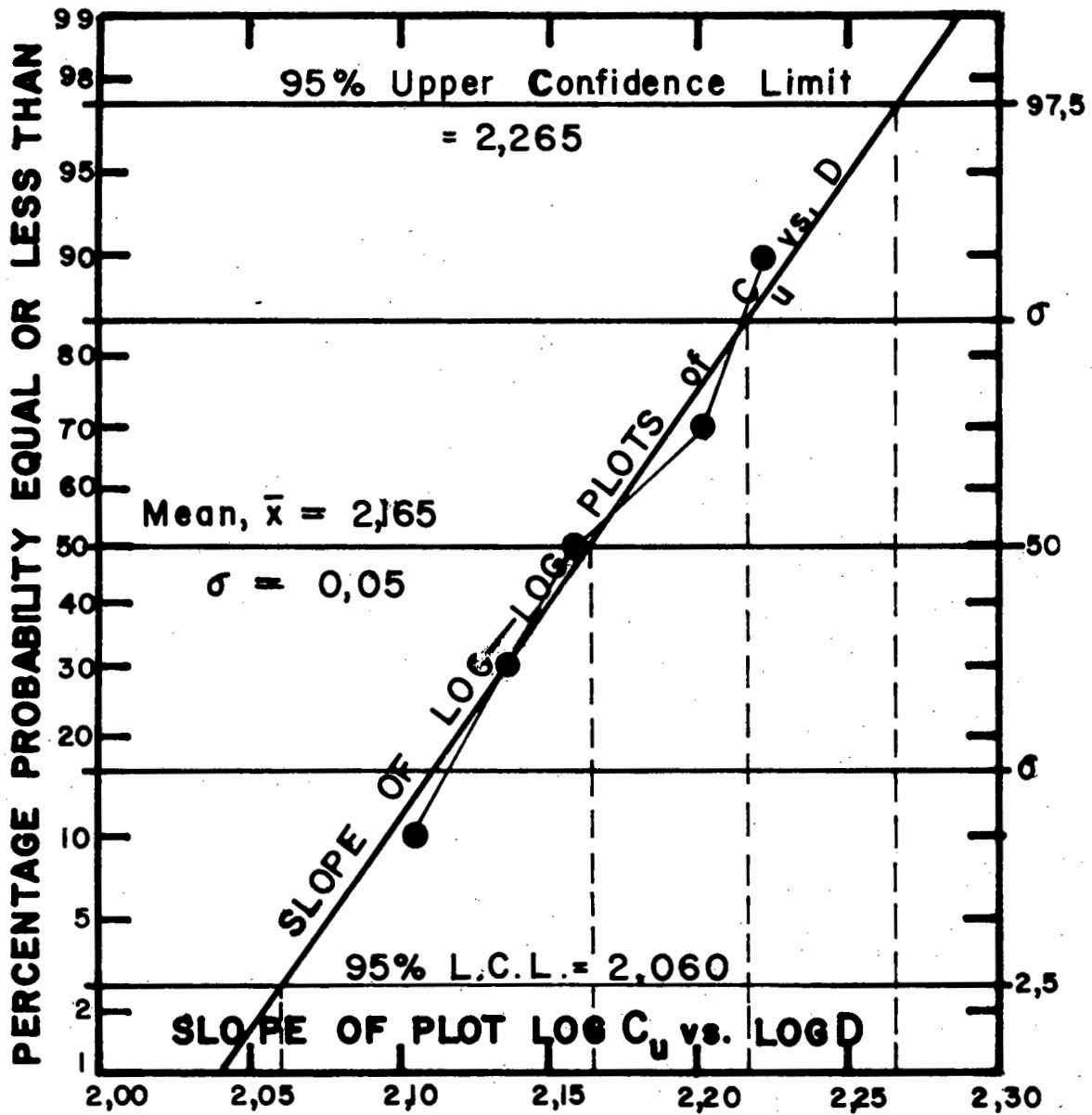


FIG. 22: MEAN EXPONENT OF TANK DIAMETER D , WITH VARIABLE FLOW-RATE, Q .

(ex FIGS. 20 & 21)

The equation of each line in Fig. 23 is of the form

$$\log K_D = n C_o + \text{constant} \quad (18)$$

(n = slope of each line, and is constant).

Equation (18) may be written as

$$\log K_D = n C_o + \log c' \quad (19),$$

where $\log c' = \log K_D$ at $C_o = 0$.

Evaluating $\log c'$ (and thus c') from equation (19) at any selected convenient value of C_o (e.g. $C_o = 2.00$ g/l), and for each value of Q , yet a further plot of $\log c'$ vs. $\log Q$ yielded a straight line, as shown in Fig. 24. The equation of this line is given by

$$c' = \text{constant}/Q^P,$$

where the constant is the intercept at $Q = 1$, and the slope is given by $-P$.

For the sludge under discussion in this work, it was found that

$$c' = \frac{\text{constant}}{Q^{1.0914}} = 33760/Q^{1.0914} \quad (20).$$

Substitution of equation (20) in equation (19) yields

$$\log K_D = \log [33760/Q^{1.0914}] - 1.179 C_o,$$

from which

$$K_D = 33760/Q^{1.0914} e^{-2.7142 C_o} \quad (21).$$

The equations to the lines resulting from the plots in Figs. 20 and 21 (i.e. $\log C_u$ versus $\log D$) have the general form,

$$\log C_u = \text{constant} \times \log D + \log K_D \quad (22),$$

where the constant is the value of the slope.

Substituting the value of K_D from equation (21) in (22), the latter equation becomes

$$\log C_u = \text{constant} \times \log D + \log [33760/Q^{1.0914} e^{-2.7142 C_o}].$$

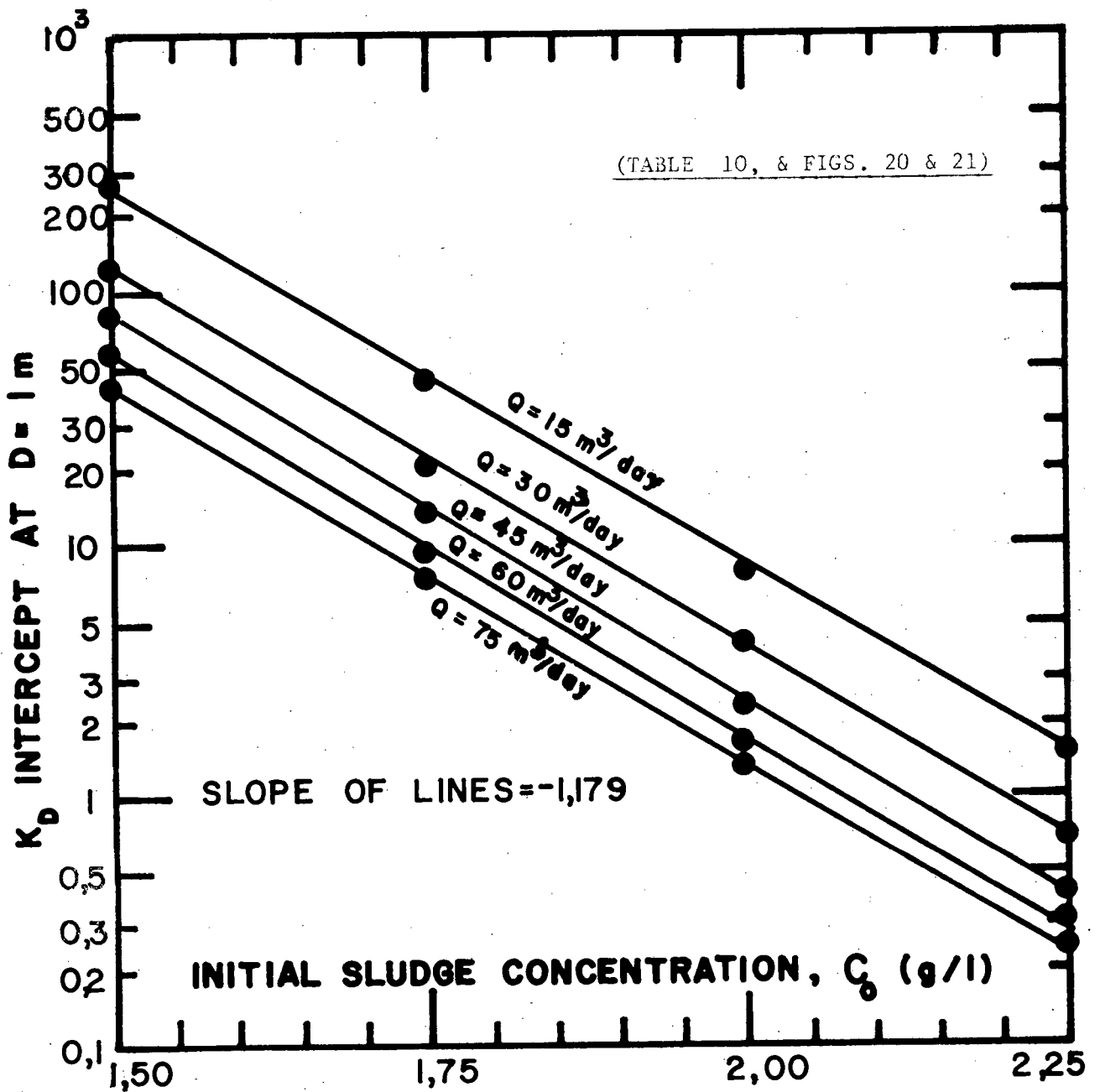


FIG. 23: RELATIONSHIP BETWEEN CONSTANT K_D INTERCEPT & C_0 FOR VARIABLE FLOW-RATE, Q .

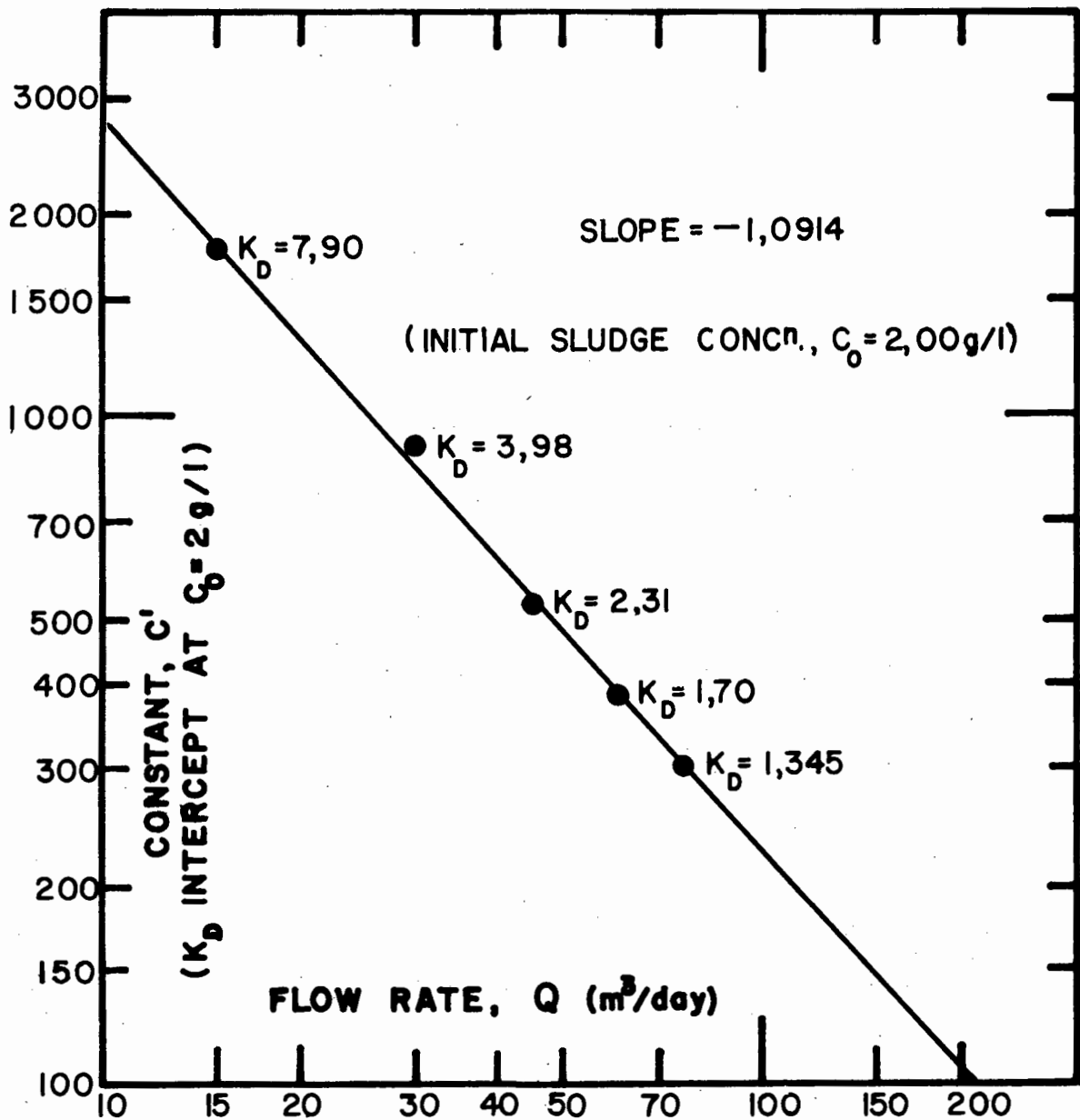


FIG. 24: RELATIONSHIP BETWEEN CONSTANT C' (K_D INTERCEPT AT $C_0 = 2g/l$) & FLOW RATE FOR DETERMINATION OF THE EXPONENT OF Q . (TABLE 11)

Solving for C_u ,

$$C_u = 33760/Q^{1,0914} e^{-2,7142 C_o D} \text{ constant} \quad (23).$$

Evaluation of the exponent of D (Figs. 20, 21 and 22) yielded a value of 2,165 for this constant. Substituting this value for the constant in equation (23), and rearranging,

$$D = 8,0975 \times 10^{-3} \times \sqrt[2,165]{C_u Q^{1,0914} e^{2,7142 C_o}} \quad (24).$$

Equation (24) yields the optimum diameter of the thickening tank required, involving the parameters of flow rate; initial concentration of influent floc; and concentration of underflow sludge. Because the concentration and flow parameters may vary, the equation is proposed as a suitable one for simulation purposes.

When a thickening tank diameter has been selected, a log-log plot of Q against C_u for different values of C_o produces a family of parallel straight lines. Each value of C_u may be computed from equations similar to (23) or (24) respectively. The general form of equation (24) is seen to be

$$D = k \sqrt[n]{C_u Q^p e^{mC_o}} \quad (25),$$

where k , m , n and p are empirical constants or exponents determined from batch sedimentation tests on any given floc.

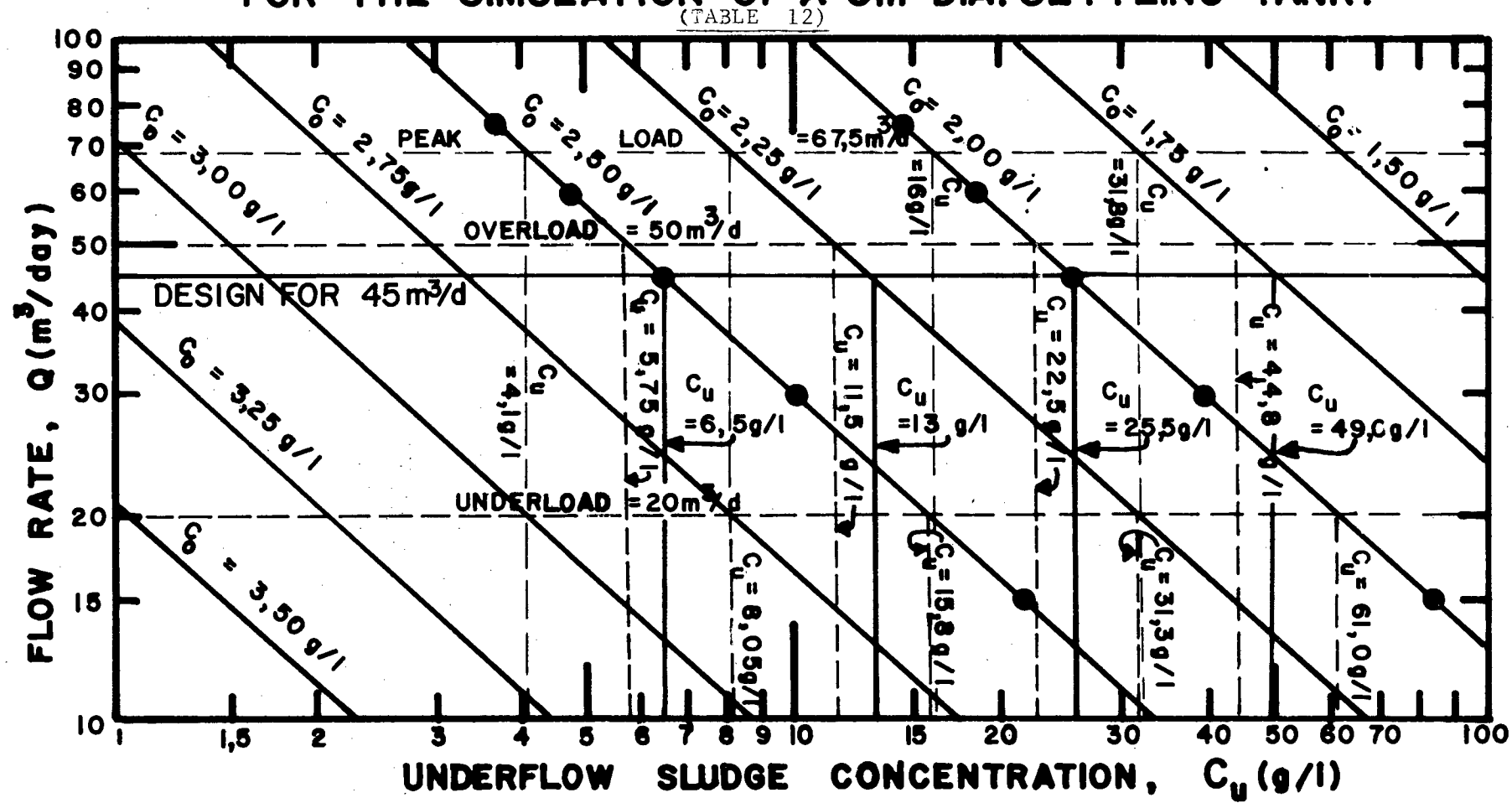
Inspection of equation (24) shows that the desired thickening tank diameter is closely a function of the square root of the flow rate,

$$\text{i.e. } D = k \sqrt[n]{C_u e^{mC_o}} \times \sqrt{Q} \quad (25A),$$

and this latter equation may equally well be employed in calculations.

Fig. 25 is a plot of a simulation chart based on equation (24), and in which the thickening tank diameter is $D = 3$ m. The use of this chart in predicting sedimentation/thickening tank performance will be demonstrated at a later stage (section 3.2.5.).

FIG. 25: THE EFFECT OF FLOW RATE ON UNDERFLOW SLUDGE CONCENTRATION AT VARIABLE INITIAL CONCENTRATIONS, FOR THE SIMULATION OF A 3m DIA. SETTLING TANK.



2.3 Disposal of the Settled Flocculated Sludge by Sand Filtration.

The disposal of a chemical waste sludge requires its dewatering in the most economical manner as a prime objective. Continuous centrifugation will readily accomplish this objective, but mitigating against this procedure is the capital cost of plant and the additional power costs in operation. The cheapest procedure is drainage on a sand filter, where land availability permits. Further drying of dewatered sludge may be achieved at the expense of fuel or thermal energy costs. However, natural air-drying may readily dry the dewatered sludge to a sufficiently low moisture content, which will enable the residue to be dug out and carted away for final dumping.

Eckenfelder ^(2c) described the two mechanisms by means of which sludges are dewatered on sand beds, viz. (a) filtration through the sand bed, and (b) evaporation as a result of radiation and convection. In the case of filtration, there are two methods of operation to consider, as outlined in section 1.4.(4). Either the sludge may be continuously fed and spread onto the sand bed, until some predetermined thickness of sludge has built-up; or the bed may be filled intermittently, allowing sufficient drainage time between applications. The rate of filtration, which, among other parameters, depends on the specific resistance of the sludge cake, determines which operational procedure should be followed. Because the resistance of the cake increases with the depth of the cake formed on the bed, the intermittent application of feed to a given filtration bed will tend to be limited. Higher loading rates can be expected with smaller masses and thicknesses of the cake. For this reason, it was decided to investigate a continuous feed process to a sand filter. At some optimum manageable level, the feed is to be diverted to a

second filter while the first one drains to completion. After a bed has drained, air-drying would continue until the mass is of a spadeable consistency.

Evaporation occurs during the entire filtration cycle, and is strongly dependent on ambient conditions of temperature, humidity and wind velocity, as well as the physico-chemical properties of the sludge itself. Thus, in practice evaporation rates can be expected to vary widely. In the municipal area of Cape Town, it is a regulation that drainage beds leading directly to the sewer must be covered or roofed, in order to minimize rainwater infiltration. This factor reduces some of the expected variations such as when the bed is exposed to rainfall. It would appear sufficient to examine the effects of some of the variables under several constant ambient conditions.

In the filtration period, the effects of drying can essentially be neglected, and hence each process may be discussed separately.

2.4. Sludge Filtration.

Carman ⁽¹⁰⁾ and Coakley and Jones ⁽¹¹⁾ formulated the rate of filtration of sludges based on Poiseuille's and D'Arcy's Laws. The rate equation is given by ⁽¹⁰⁾

$$\frac{dV}{dt} = \frac{PA^2}{\mu (rcV + R_m A)} \quad (26)$$

where V = volume of filtrate (ml),

t = form time for the cake (sec or min),

P = vacuum (g/cm^2),

A = filtration area (cm^2),

μ = filtrate viscosity (poise),

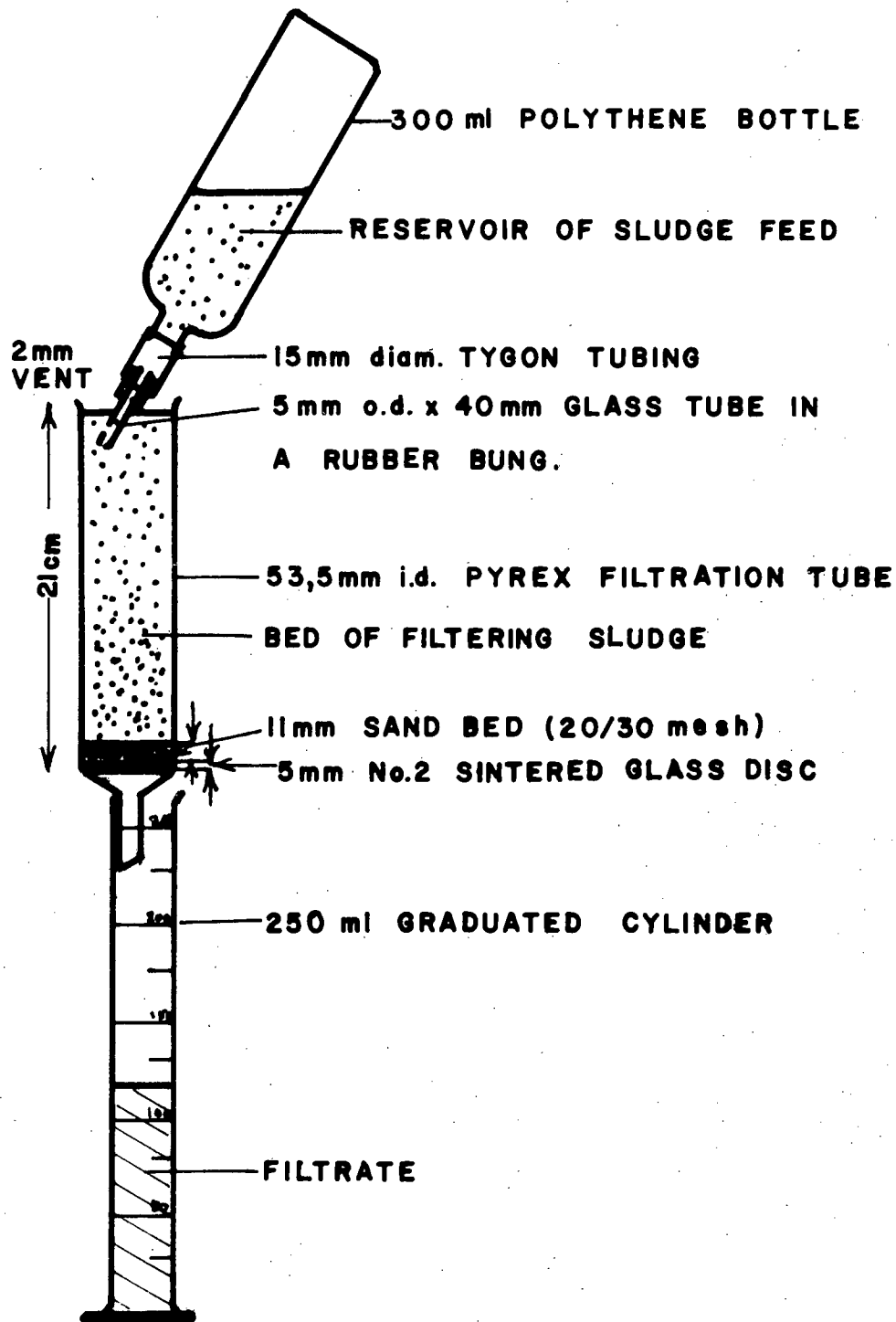


FIG. 26: DIAGRAM OF APPARATUS FOR THE CONSTANT HEAD FILTRATION OF SLUDGES ON A SAND BED.

r = specific resistance of the cake [sec^2/g , or $(\text{sec})(\text{min})/\text{g}$],

c = mass of solids per unit volume of filtrate (g/ml),

R_m = initial resistance of the filter media
[sec^2/cm^2 or $(\text{sec})(\text{min})/\text{cm}^2$].

Integration and re-arrangement of equation (26) yields

$$\frac{t}{V} = \frac{\mu r c}{2PA^2} V + \frac{\mu R_m}{PA} \quad (27)$$

According to equation (27), a plot of the function t/V against V is linear (as shown in Figs. 28, 29 and 30 respectively). The slope b is equal to $(\mu rc/2PA^2)$, and re-arranging,

$$r = \frac{2bPA^2}{\mu c} \quad (28)$$

The specific resistance, r , is a measure of the filterability of the sludge. From equation (28) it is evident that it is numerically equal to the pressure difference required to produce a unit rate of filtrate flow of unit viscosity through unit mass of cake. However, r will only remain constant in the case of an incompressible sludge. For actual sludges r is dependent on pressure because of the reduced permeability as pressure increases, due to a reduction in the voids (Ruth et al., 1933⁽¹²⁾). The specific resistance is related to the pressure by an empirical relationship (Eckenfelder ^(2b)),

$$r = r_0 P^S \quad (29)$$

where S = coefficient of compressibility,

and r_0 = specific resistance at unit pressure.

Thus the filtration rate equation (27) is more precisely described by substituting equation (29) for r in equation (27).

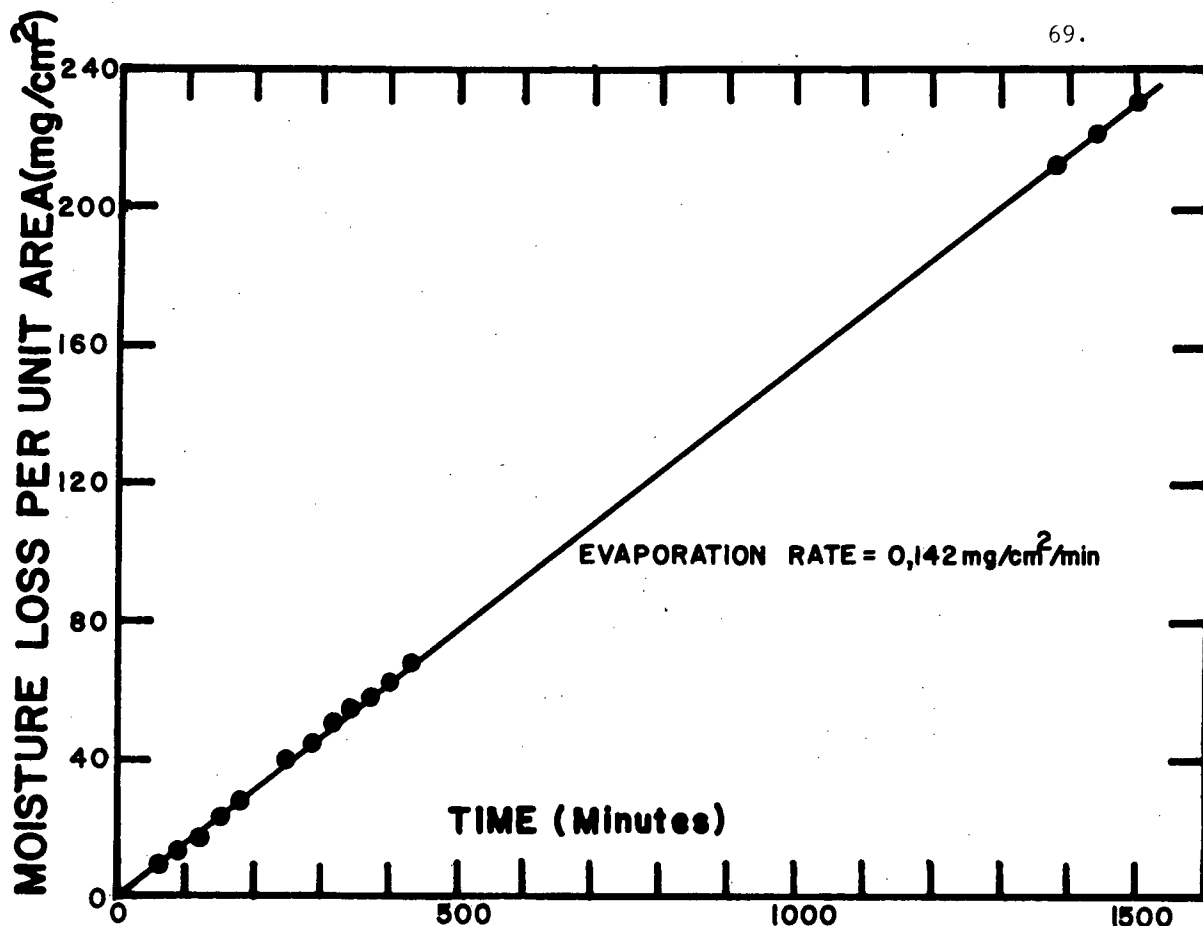


FIG. 27: RATE OF EVAPORATION OF FILTRATE AT 20°C. & 65% RELATIVE HUMIDITY.

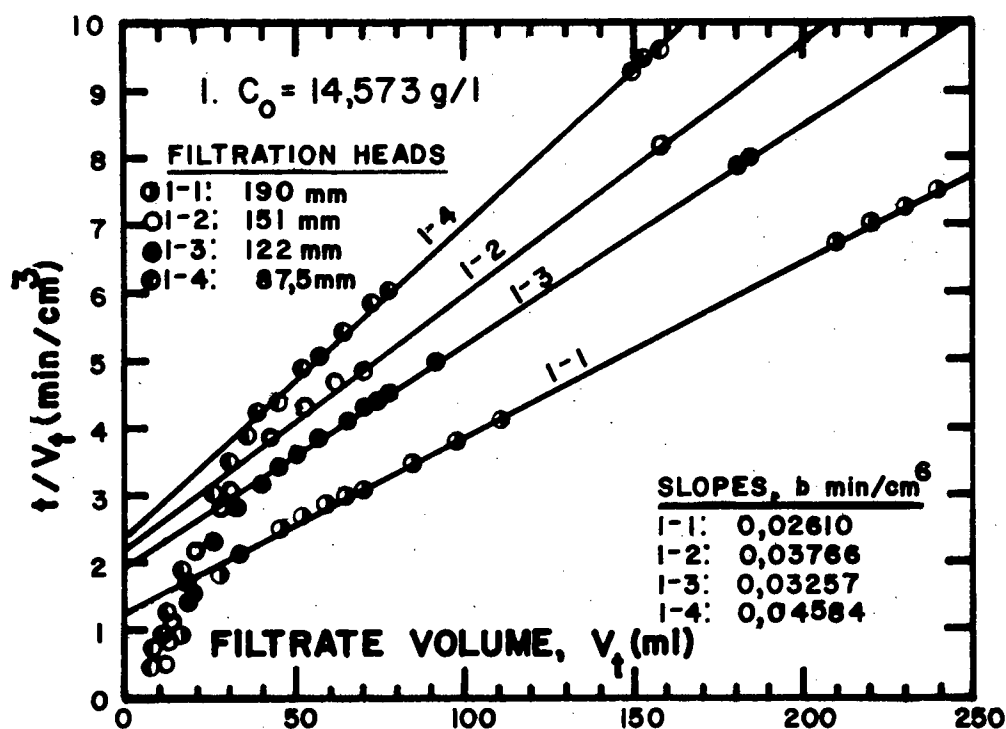


FIG. 28: CONSTANT HEAD SAND FILTRATION AT 20°C. (TABLE 13a)

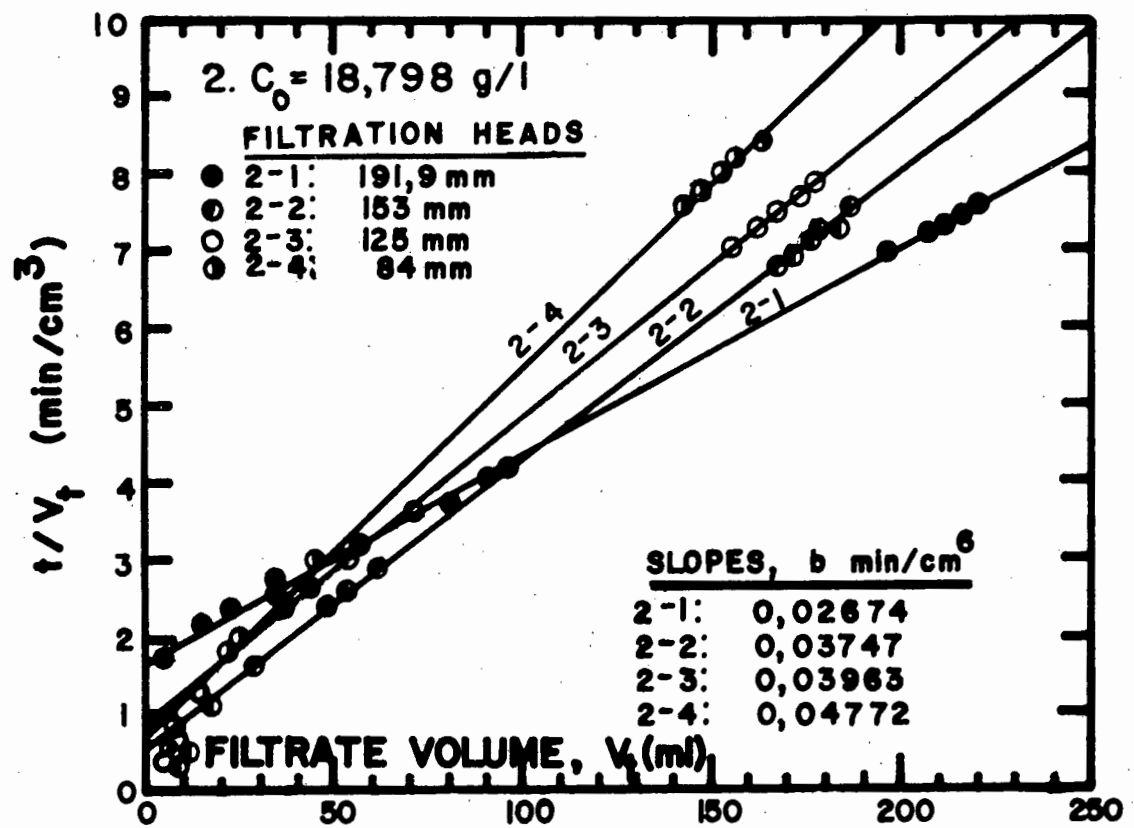


FIG. 29: CONSTANT HEAD SAND FILTRATION
AT 20° C. (TABLE 13b)

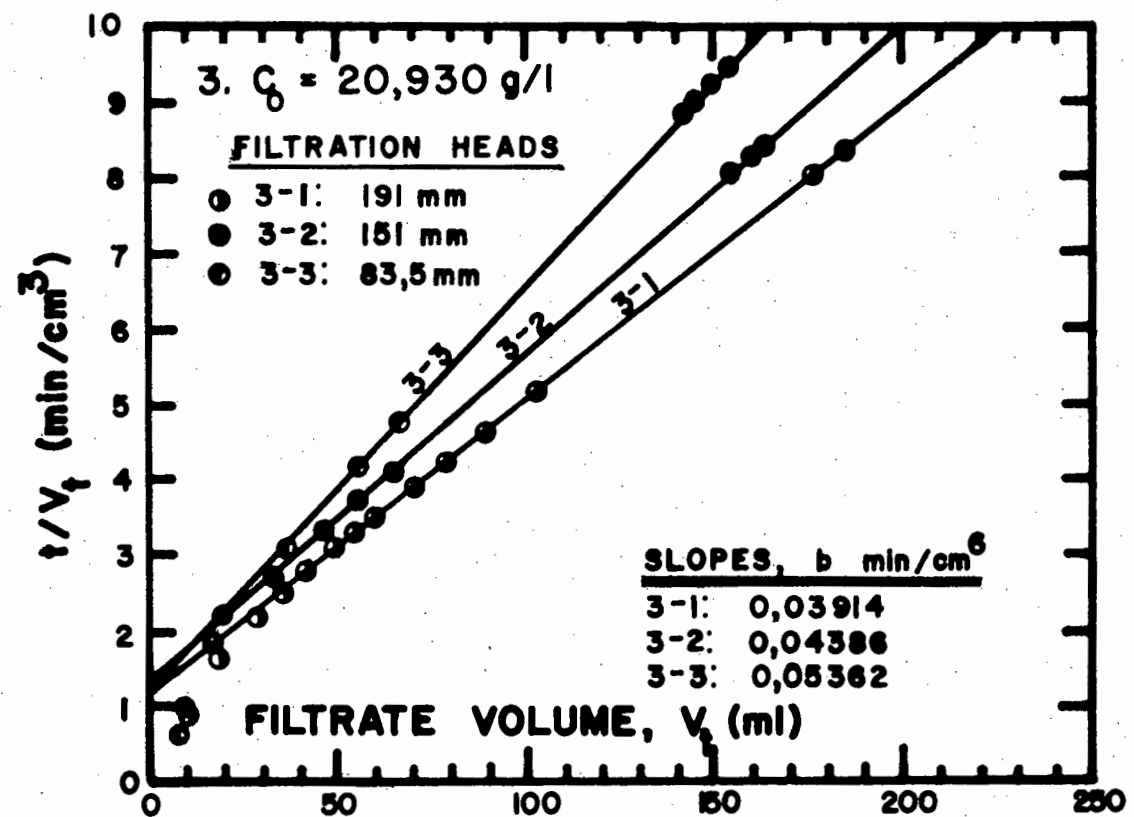


FIG. 30: CONSTANT HEAD SAND FILTRATION
AT 20° C. (TABLE 13c)

$$\text{Thus, } \frac{t}{V} = \frac{\mu r_o c}{2P(1-s)_A^2} \cdot V + \frac{\mu R_m}{PA} \quad (30),$$

$$\text{whence } r_o = \frac{2bP(1-s)_A^2}{\mu c} \quad (31).$$

In most filtration operations, it was proposed by Eckenfelder^(2b) that R_m , the initial filter media resistance, is small compared with the resistance developed by the filter cake, and suggested that it can be neglected. However, it will be shown that this simplification holds only approximately for low values of the pressure head, P , across the filter concerned. Equation (26) was developed by Carman essentially for vacuum filtration, and in applying it to sand bed drainage, Eckenfelder^(2b) defined the pressure term P as the average hydrostatic head of sludge in the bed during drainage, all other terms being defined as in the original Carman equation. In vacuum filtration, the time to form a cake from the beginning of the application of vacuum until the end when vacuum is released and the cake solids can be discharged, is the cycle time, t . In sand filtration, it is interpreted as the time of filtration from the beginning to the time t to produce a filtrate volume V .

It is essential for the proposed dewatering operation that the hydrostatic head be maintained at relatively low values, in order that the cake thickness for drying should be minimal, and consequently the value of P will be relatively low.

Equation (26) is primarily an equation for constant pressure filtration. In a similar form, Ruth et al⁽¹²⁾ evolved a simplified equation,

$$\frac{dV}{dt} = \frac{AP_c}{\mu R},$$

where P_c is the pressure drop across the cake alone, and its resistance is R . The pressure head across the combined cake and filter medium is more readily measured. However, the resistance to the flow of the filtrate consists of the sum of the resistances offered by the cake and the filter medium. The latter resistance (filter medium) is accounted for in terms of an equivalent resistance offered by a mass of cake, which would have been deposited by a corresponding volume of filtrate.

In the present work it is confirmed that operating a filter at constant head, if the pressure head across the sludge and filter medium is the value of P accepted for use in equations (30) and (31) respectively, inclusion of the filter media resistance R_m results in equation (30) being closely satisfied. Consequently, the term $\mu R_m / PA$ in equation (30) may not always be insignificant. When the head of sludge varies (i.e. the liquid level falls during drainage), the rate equation could best be described by an empirical power function ($\log t/V$ plotted against $\log V$). In this instance, by simply assuming a mean value for the head or depth of sludge in the filter, it is an approximation to employ Carman's equation (Figs. 42 and 43 respectively).

Values of the specific resistance, r , may be determined from equation (28) for different experimental values of the pressure P . A plot of $\log r$ vs. $\log P$, which is approximately linear (Fig. 31), will resolve the constants r and S in equation (29). These are required in the calculation of loading rates for design purposes (section 2.4.1. and 2.4.2.).

The concentration term, c , is defined by the mass of solids per unit volume of filtrate. It is therefore a measure of the mass of

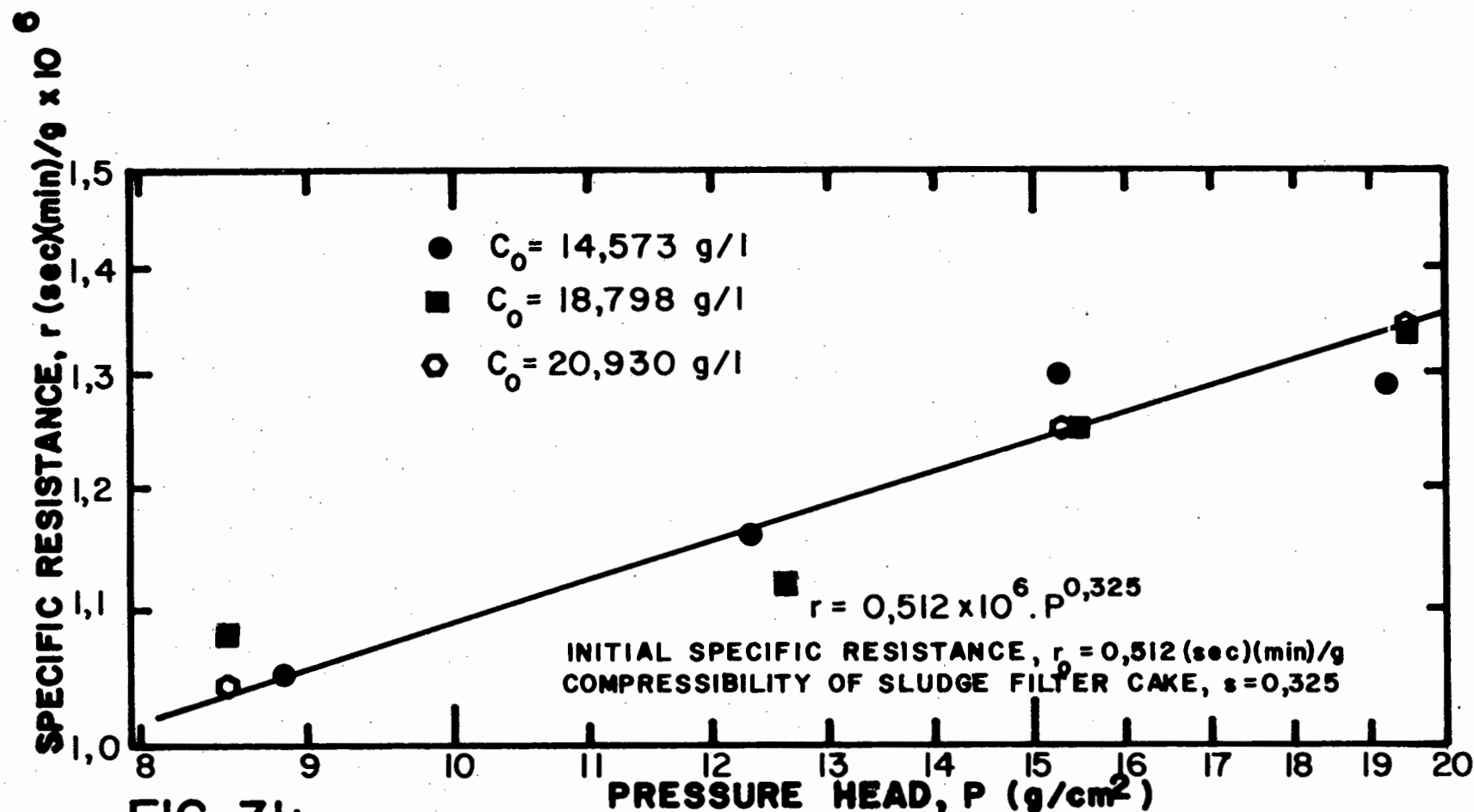


FIG. 31:
RELATIONSHIP BETWEEN SPECIFIC RESISTANCE OF SLUDGE
FILTER-CAKE & PRESSURE DURING CONSTANT HEAD SAND
FILTRATION AT 20° C. (TABLE 14)

solids removed, regardless of their initial concentration. In terms of the initial and final moisture contents of the sludge, c_i and c_f (g water/100g of sludge) respectively, c is derived as follows:-

Initially, 100 g of sludge contains c_i grams of water and $(100-c_i)$ grams of solids.

After filtration, 100 g of sludge contains c_f grams of water and $(100 - c_f)$ g of solids.

Thus, the final sludge contains $c_f/(100-c_f)$ g water per 1 g of dry solids.

But the mass of dry solids removed in the filtration = $(100 - c_i)$ g.

Therefore the mass of water associated with $(100-c_i)$ g of solids in the final sludge =

$$\frac{c_f}{(100-c_f)} \times (100 - c_i) \text{ g}.$$

Mass of water removed in the filtration as filtrate

$$= [c_i - \frac{c_f (100 - c_i)}{(100 - c_f)}] \text{ g}.$$

Assume the density of water is unity (g/ml); then the mass of solids removed per 1 ml of filtrate = c =

$$(100 - c_i) / [c_i - \frac{c_f (100 - c_i)}{100 - c_f}] \text{ g/ml},$$

$$\text{whence } c = \frac{(100 - c_i) (100 - c_f)}{c_i (100 - c_f) - c_f (100 - c_i)} \text{ g/ml},$$

$$\text{or } c = \frac{1}{c_i/(100 - c_i) - c_f/(100 - c_f)} \quad (32).$$

This is the relationship given by Eckenfelder (2b).

However, if C_o = initial average solids content of the feed (units, g/l) and C_f = final average solids content of the drained sludge (g/l); ρ_o and ρ_f being the initial and final densities of the sludge respectively (g/ml); then per 100 g mass of initial sludge,

$$c_i = 100 - \frac{C_o}{10 \cdot \rho_o}, \text{ and per 100 g mass of final sludge,}$$

$$c_f = 100 - \frac{C_f}{10 \cdot \rho_f}$$

Substituting in equation (32) and simplifying,

$$c = \frac{C_o}{1000} \times \frac{C_f}{(C_f \rho_o - C_o \rho_f)} \quad (33)$$

With most sludges during drainage, due to their high degree of hydration, ρ_o is not very much less than ρ_f .

$$\rho_o \approx \rho_f = \rho,$$

where ρ = mean density of initial and final sludges on the bed. Hence, equation (33) may be approximated to.

$$c = \frac{C_o}{1000} \frac{C_f}{\rho (C_f - C_o)} \quad (34)$$

Equation (34) enables the solids removal term c (g/ml) to be calculated from the more conveniently available values of initial sludge slurry concentration C_o (g/l); the final sludge concentration after completion of filtration, C_f (g/l); and the mean sludge density during the filtration period, ρ (g/ml).

2.4.1. Loading Rate Equations Neglecting Filter Media Resistance.

If the filter medium resistance R_m in equation (30) is neglected, solving for the filtrate volume,

$$V = \sqrt{\frac{2P(1-s) A^2 t}{\mu r_o c}} \quad (35)$$

Multiplying both sides of the latter equation by the term c/tA , the filtration rate, or loading rate L ^(2b) is obtained. Thus,

$$L = \frac{Vc}{tA} = \sqrt{\frac{2P(1-s) c}{\mu r_o t}} \quad (36),$$

where L = mass of solids removed per unit area per unit time ($\text{kg/m}^2/\text{d}$).

To account for deviations resulting from variations in the permeability of the cake as more solids are deposited, and also as a result of variations in the feed concentration, equation (36) needs to be modified. Eckenfelder ^(2b) suggested that the terms c and t each be raised to some exponent m and n respectively.

Hence equation (36) becomes,

$$L = \sqrt{\frac{2P(1-s)}{\mu r_o}} \cdot \frac{c^m}{t^n} \quad (37).$$

The values of m and n can be determined experimentally by selecting a series of constant filtration times t and calculating values of L and c . The slope of the log-log plot of L versus c evaluates m . Similarly, when a series of constant solids removal values c is selected, values of L and t may be calculated. The slope of the log-log plots of L against t evaluates n . These plots are shown in Fig. 37 which give values for m and n in c^m/t^n , as $c^{0.52}/t^{0.78}$.

The data for calculating L at constant time or constant solids removal, may be obtained from the original experimental observations of the

filtrate volume v_t at each time t , these being used in equations (27) and (30). By plotting v_t against t for the different filtrations (as shown in Fig. 36), the curves are read at different selected times, and the calculations are described in the Appendix (section 4.3.5.).

When equation (37) has been used to evaluate the loading rate L , the results have been found to be very low compared with the actual loading rates possible. The relationship is therefore still unsatisfactory for design. However, in this investigation it was found that comparing experimentally observed values of L with those obtained from equation (37), an empirical correlation may be derived as follows:-

The ratio between observed loading rates and those calculated from (37) was found to be approximately constant.

i.e. $L_{\text{Observed}} \doteq \text{constant} \times L_{\text{calculated}}$

Substituting for $L_{\text{calculated}}$ from equation (37),

$$L_{\text{Observed}} = k \sqrt{\frac{2P(1-s)}{\mu r_o}} \cdot \frac{c^m}{t^n} \quad (37A).$$

Rearranging,

$$L_{\text{Observed}} \cdot t^n \times \sqrt{\frac{\mu r_o}{2P(1-s)}} = kc^m \quad (37B).$$

Plotting the function on the left-hand side of this equation against the term c on a log-log plot, a straight line is now obtained as shown in Fig. 39, k thus being an experimentally derived constant. A least-squares fit of the data in Fig. 39 (correlation coefficient = 0,83) yields, after re-arranging, the following empirical equation:-

$$L = 8,55 \sqrt{\frac{2P(1-s)}{\mu r_o}} \cdot \frac{c^{0,56}}{t^{0,78}} \quad (37C).$$

Equation (37c) now replaces equation (37), and was found to be suitable at higher sludge concentrations (Table 2.III (c); $C_o > 18\text{g/l}$) when the resistance of the filter media is relatively small in comparison with that of the sludge, and R_m may be ignored.

2.4.2. Theoretical Loading Rate Equations Including Filter Media Resistance.

When the sludge concentration is at lower values ($C_o < 18\text{g/l}$) and the pressure heads are also low, the resistance of the filtering sludge is reduced, and the filter medium resistance R_m is significant. In this investigation the following sequence of equations was derived:-

For a given constant pressure head P and a constant filtration time t , re-arrangement of the terms of the theoretical equation (30) yields the equation of a parabola;

$$\text{thus, } \left(\frac{\mu r_o c}{2P(1-s)_A^2} \right) \times V^2 + \left(\frac{\mu R_m}{PA} \right) \times V - t = 0 \quad (38)$$

Selecting a given fixed value of the filtration time t , the quadratic equation (38) may be solved for V . Thus,

$$V = \frac{\sqrt{\left(\frac{\mu R_m}{PA} \right)^2 + \frac{2\mu r_o c t}{P(1-s)_A^2}} - \frac{\mu R_m}{PA}}{\frac{\mu r_o c}{P(1-s)_A^2}} \quad (39)$$

The loading rate, L , defined by Vc/tA [equation (36)] may be calculated in a similar manner to equation (36) by multiplying both sides of equation (39) by the term c/tA , and simplifying. Thus,

$$L' = \frac{\sqrt{(\mu R_m)^2 + 2 \mu r_o c P (1+s)} \cdot t}{\mu r_o P^s t} - \frac{R_m}{r_o P^s t} \quad (40)$$

Equation (40) is a theoretical loading rate equation, which indicates that the filter media resistance R_m can be of significance in the calculation of the loading rate, depending on its value relative to the sludge cake resistance. In addition, this equation indicates that the physical properties of the sludge (initial specific resistance r_o , and compressibility s) and their variation with respect to the pressure head P and time t , also affect the significance of R_m . As the pressure head and/or time increase, the term $R_m / r_o P^s t$ decreases in value, and R_m then becomes of lesser importance. It is therefore necessary to determine the value of R_m .

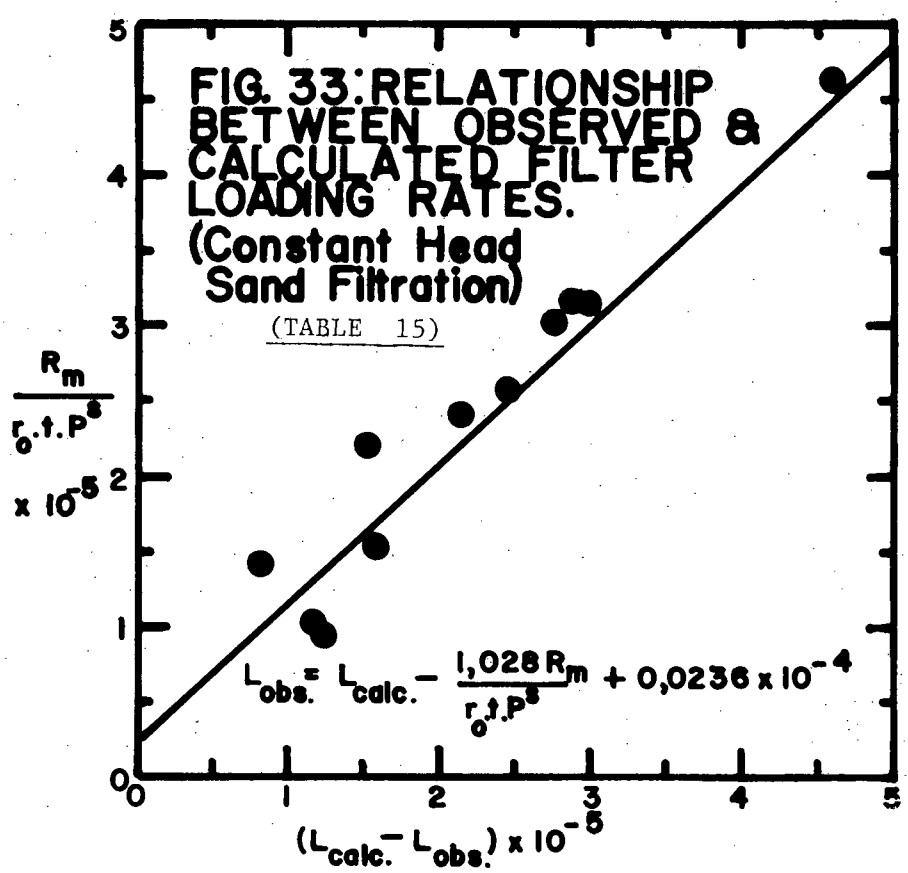
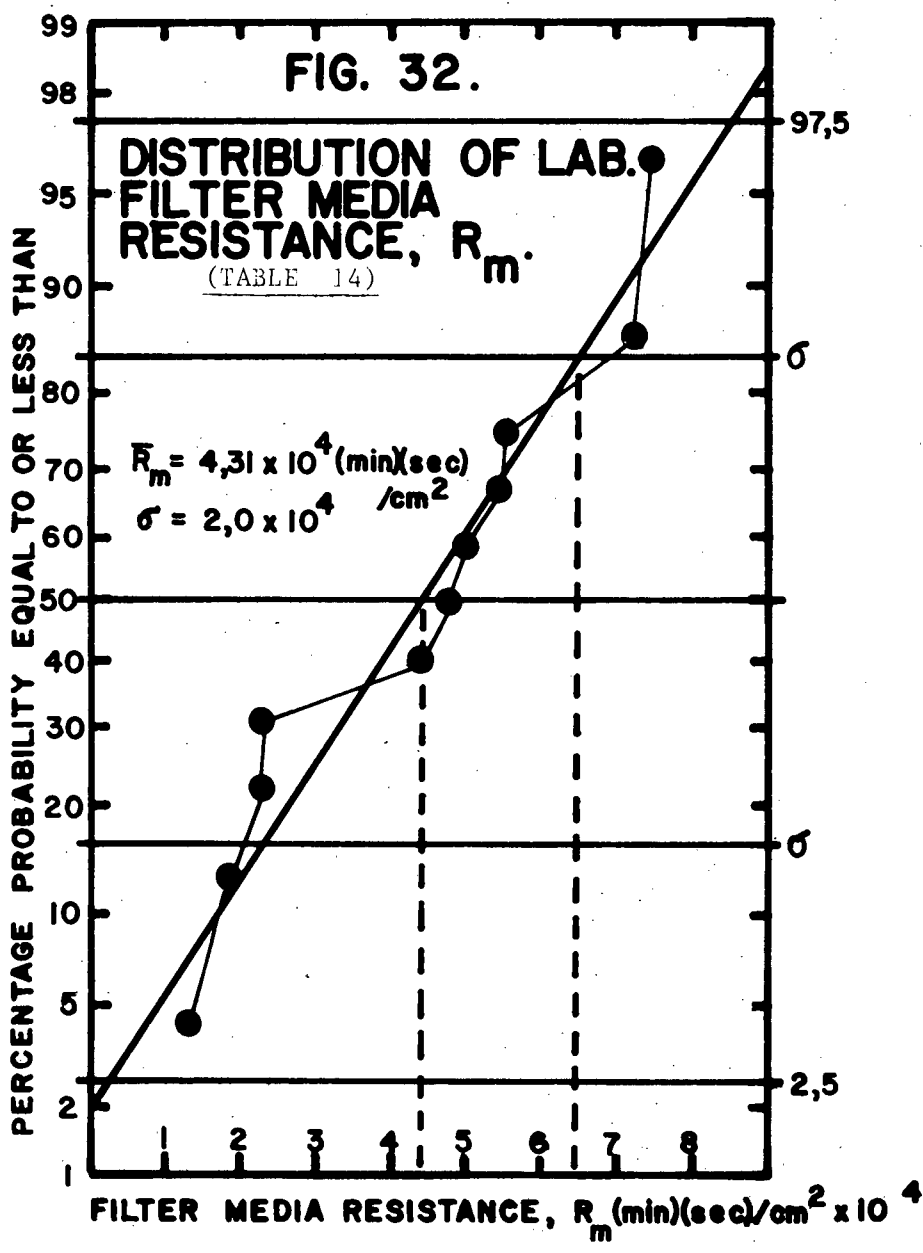
The value of R_m may be obtained as follows:

Reference to the linear equation (30) shows that the intercept, I (min/cm^3), of the plot of t/V versus V on the t/V axis, yields

$$I = \frac{\mu R_m}{PA}$$

which enables the calculation of an estimate of the value of R_m .

Theoretically, for a given sand filter and a sludge having a constant filtrate viscosity, it is expected that the filter media resistance should remain constant, and thus the intercept I should also be a constant. However, reference to Figs. 28, 29 and 30 respectively, showed that this was not always true. Only at a high sludge concentration, was this approximately so (Fig. 30). In the other experiments at lower sludge concentrations, there appeared to be a variation in the permeability of the sand layer which may account for variations in I , and thus in R_m . Nevertheless, values of R_m calculated for eleven constant head filtrations showed that the set of individual values was normally distributed (Fig. 32), and it is



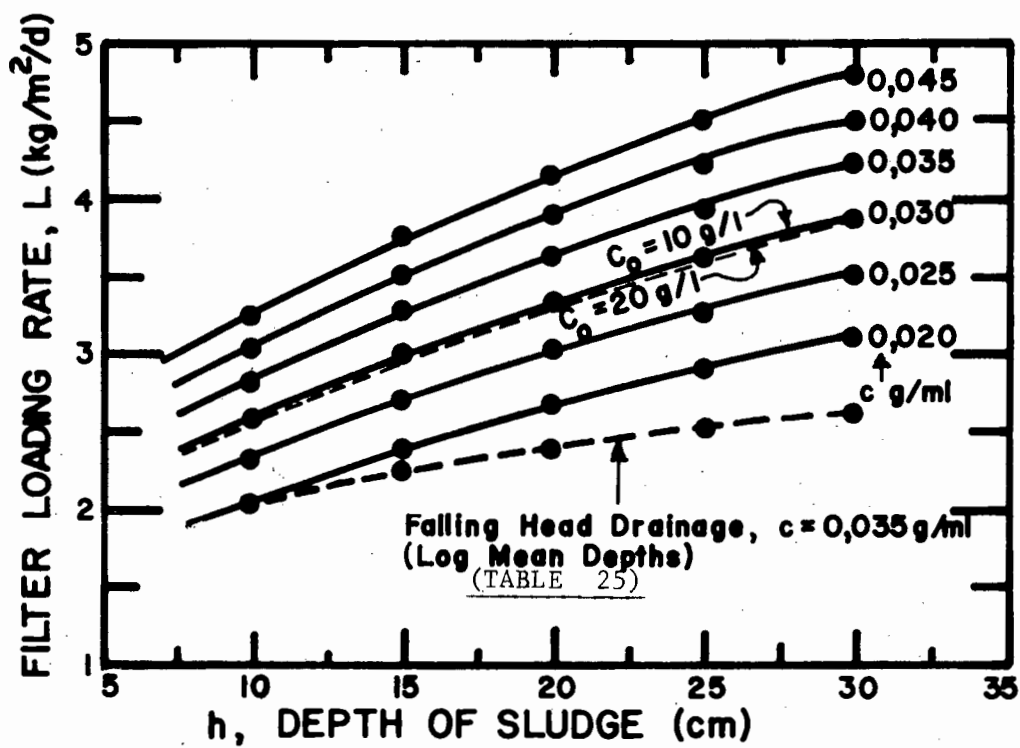


FIG. 34: SAND FILTER LOADING RATE FOR
CONSTANT HEAD FILTRATION AT
20°C. & $C_0 = 14$ g/l. (TABLE 16)

appropriate to use the mean value of R_m in evaluating equation (40).

Comparing the calculated theoretical loading rates from equation (40) with those actually observed (Table 2.III (d)), agreement was not obtained in all the filtration experiments and even this equation requires empirical modification, as follows:-

Equation (36) is based on the theoretical equation (30) in which the term $\mu R_m / PA$ is ignored because R_m is assumed to be negligible. It seems that the calculated loading rates obtained from equation (36) need to be corrected by a function involving R_m and the properties of the filtering sludge. These properties concern the specific resistance of the sludge; its variation with time and pressure head; and the sludge compressibility (as described above). From the theoretical loading rate equation (40), a term involving these parameters is $R_m / r_o P^s t$. Attempting the use of the latter as a correction function to the simplified equation (36), the correction function was plotted against the difference between observed loading rates and those calculated from equation (36). This plot is shown in Fig. 33, and results in a straight line. The best fitting straight line to this plot obtained by least squares, gives an empirical equation of the form,

$$L_{\text{observed}} - L_{\text{calculated}} = k' - k'' R_m / r_o P^s t, \text{ where } k' \text{ and } k'' \text{ are experimentally determined constants.}$$

In this investigation it was found that:

$$L_{\text{observed}} - L_{\text{calculated}} = 0,0236 \times 10^{-4} - \frac{1,028 R_m}{r_o P^s t}$$

$$\text{Rearranging, } L_{\text{observed}} = L_{\text{calculated}} - \frac{1,028 R_m}{r_o P^s t} + 0,0236 \times 10^{-4} \quad (41),$$

Exp. No.	1 - 1	1 - 2	1 - 3	1 - 4	2 - 1	2 - 2	2 - 3	2 - 4	3 - 1	3 - 2	3 - 3
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TABLE 2. III (CONTINUED)

Note:- (a) $L_{\text{CALC}} = \sqrt{\frac{2 P (1-S)_e}{\mu \cdot r_o t}}$ (equation (36), neglecting R_m)

(b) $L_{\text{EMP}} = L_{\text{CALC}} [\text{ex (a)}] - \frac{1,028 R_m}{r_o \cdot P^S t} + 0,0236 \times 10^{-4}$ (equation (41); $\bar{R}_m = 4,31 \times 10^4$) (FIG. 33).

(c) $L_{\text{EMP}} = 8,55 \sqrt{\frac{2P (1-S)}{\mu r_o}} \cdot \frac{0,56}{0,78}$ (equation (37c)) (FIG. 39).

(d) $L^I = \frac{\sqrt{(\mu R_m)^2 + 2 \mu r_o c P (1+S)_e t}}{\mu \cdot r_o \cdot P^S t} - \frac{R_m}{r_o \cdot P^S t}$ (equation (40); $\bar{R}_m = 4,31 \times 10^4$)

(where $L_{\text{calculated}} = \sqrt{\frac{2P(1-s).c}{\mu r_o t}}$, equation (36); R_m = mean value),

Loading rates calculated from the modified empirical equation (41) yielded very good agreement with those observed [Table 2.III (b)] over the entire range of sludge concentrations and pressure heads to be expected. Thus for equation (41) in which R_m is taken into consideration, $L_{\text{Empirical}}$ is approximately equal to L_{Observed} .

2.4.3. Selection of a Design Loading Rate Equation.

When the filter bed loading rates are calculated from both modified empirical equations (37c) and (41) respectively, it is now possible to ascertain whether the filter medium resistance R_m may be neglected. It has been found that at higher initial sludge concentrations, and at higher pressure heads or depths of sludge, the resistance R_m is relatively small compared to that of the sludge, and may be neglected (section 2.4.1.). Equation 37(c) may then be used as a design equation. For lower initial sludge concentrations and low pressure heads or depths of sludge, the resistance R_m becomes increasingly significant and needs to be taken into consideration to ensure that the required drainage area is not under-designed. In this case, equation (41) is then suitable for the design of sand filters (section 2.4.2.).

Thus a method is now available which enables one to select the appropriate design equation. By comparing the experimentally observed loading rates obtained over the range of initial sludge concentrations and pressure heads expected, with those calculated from each of the equations in turn, the equation giving the best agreement is the appropriate one. In Table 2.III, the ratio $L_{\text{observed}}/L_{\text{calculated}}$ is shown for each of the equations in columns (a), (b), (c) and (d) respectively.

The closeness of approach of this ratio to unity is a measure of the suitability of the equation concerned. For the sludge and filtration experiments in this work, equation (41) was found to be the most suitable to use.

2.5. The Parameters in Constant Head Drainage.

Inspection of equation (36)

$$L = \sqrt{\frac{2P(1-s)c}{\mu r_o t}},$$

shows that the filter loading rate varies as the square root of a power of the head or depth of sludge, and also directly as the square root of the concentration term. For a given sludge (r_o constant) the filter loading rate varies inversely as the square root of the viscosity of the filtrate and the cycle time, (or time of application of sludge).

Thus, higher loading rates will be expected by increasing the depth of sludge and operating at high values of the concentration term, c .

However, excessively high values of the pressure head P , tend to create problems with the subsequent drying of the filter cake (longer drying times will be expected, and thus larger areas required). The operating head would then be dependent essentially on the drying characteristics of the cake, and on the availability of land.

The concentration term, c , calculated from equation (34), (which is plotted in Fig. 35) shows that for any given initial sludge concentration C_o , larger values of c were obtained when the final drained sludge concentration C_f is relatively low. However, the object of the drainage was to obtain as high a value of C_f as possible. The family of curves for c tends to have diminishing slopes as C_f increases, and thus during prolonged

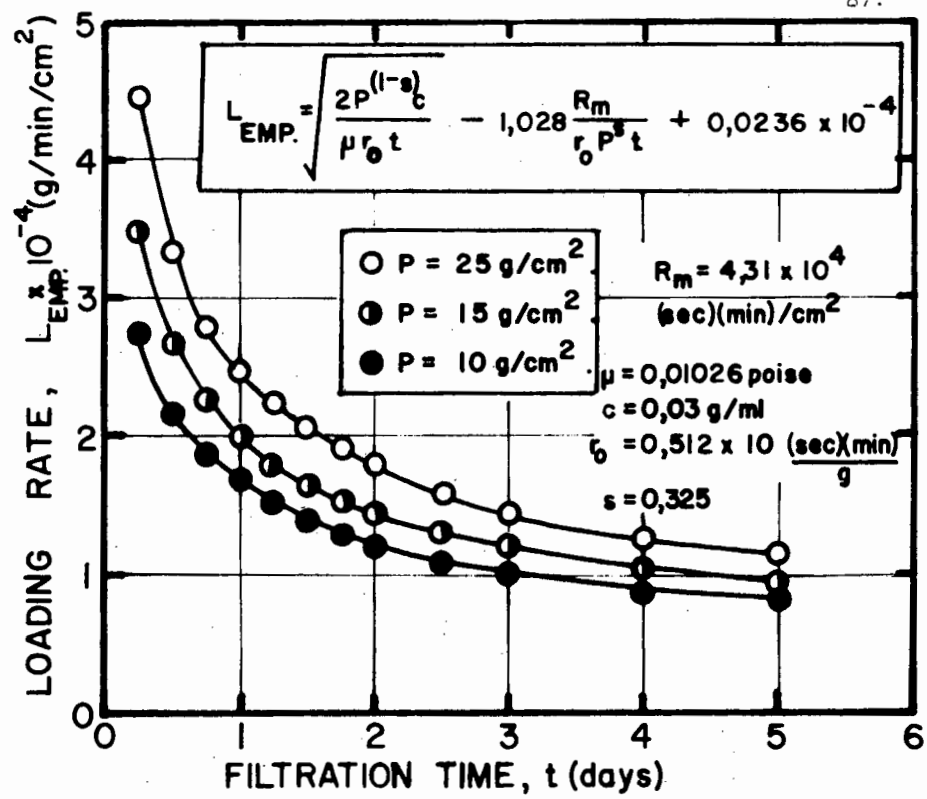


FIG.34a.EFFECT OF TIME OF FILTRATION ON LOADING RATE IN CONSTANT HEAD FILTRATION (20°C).

(TABLE 2.1Va)

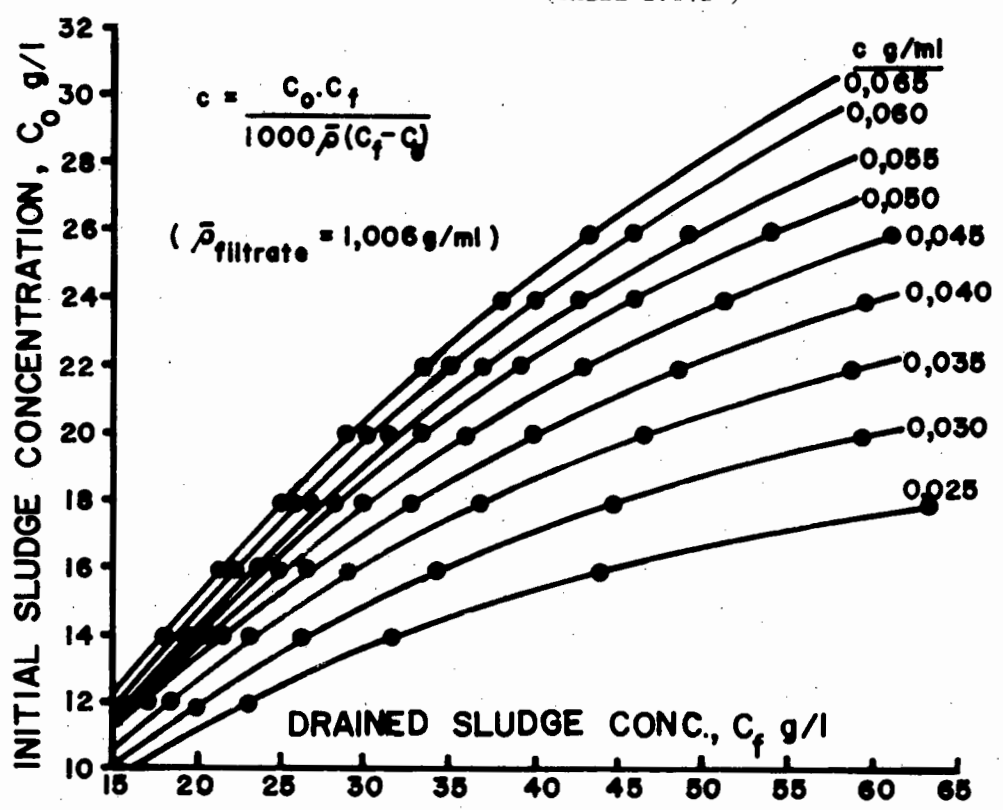


FIG. 35: CALCULATION OF MASS OF SOLIDS PER UNIT VOLUME OF FILTRATE, c.
(TABLE 17)

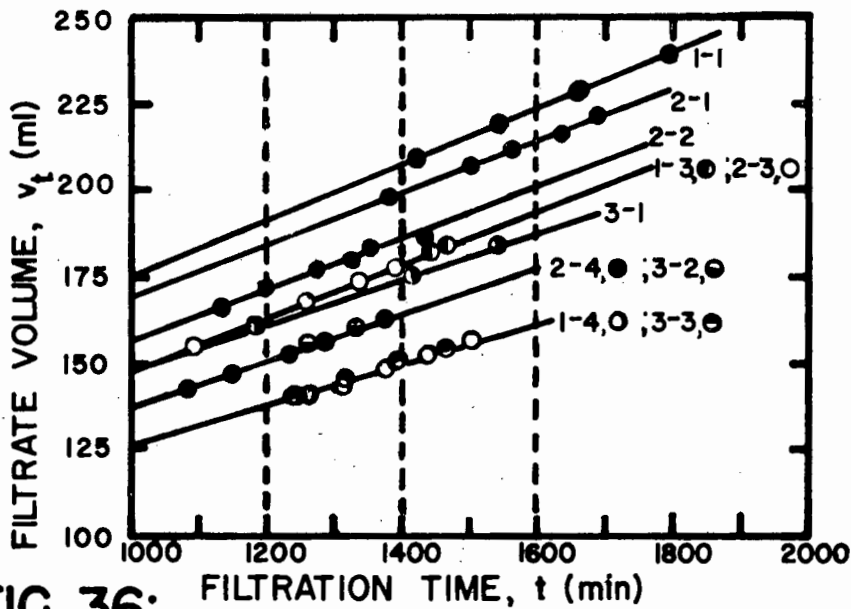


FIG. 36: CONSTANT HEAD FILTRATION. DETERMINATION OF FILTRATE VOLUME AT CONSTANT TIMES.

(TABLES 13a, 13b & 13c)

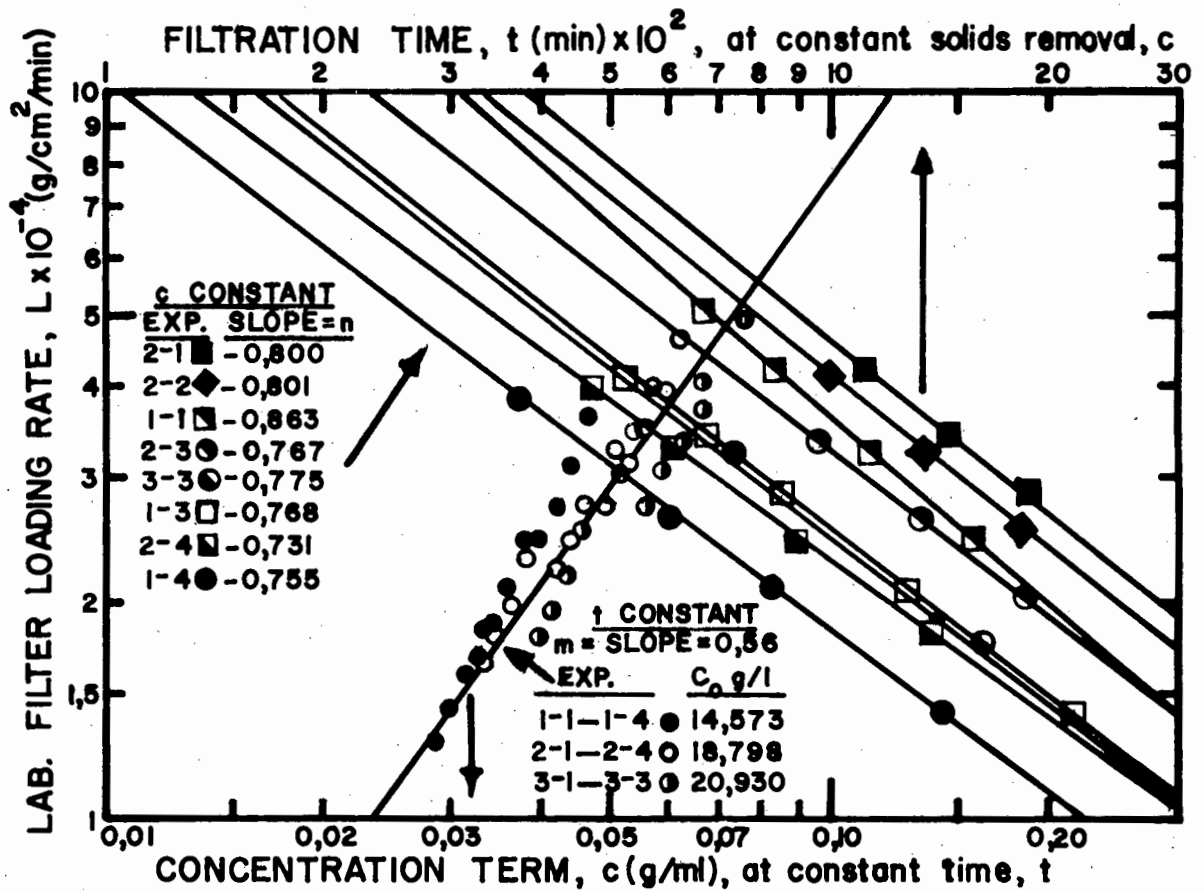


FIG. 37: DETERMINATION OF EXPONENTS OF t & c IN CONSTANT HEAD FILTRATION.

(TABLES 18 & 19)

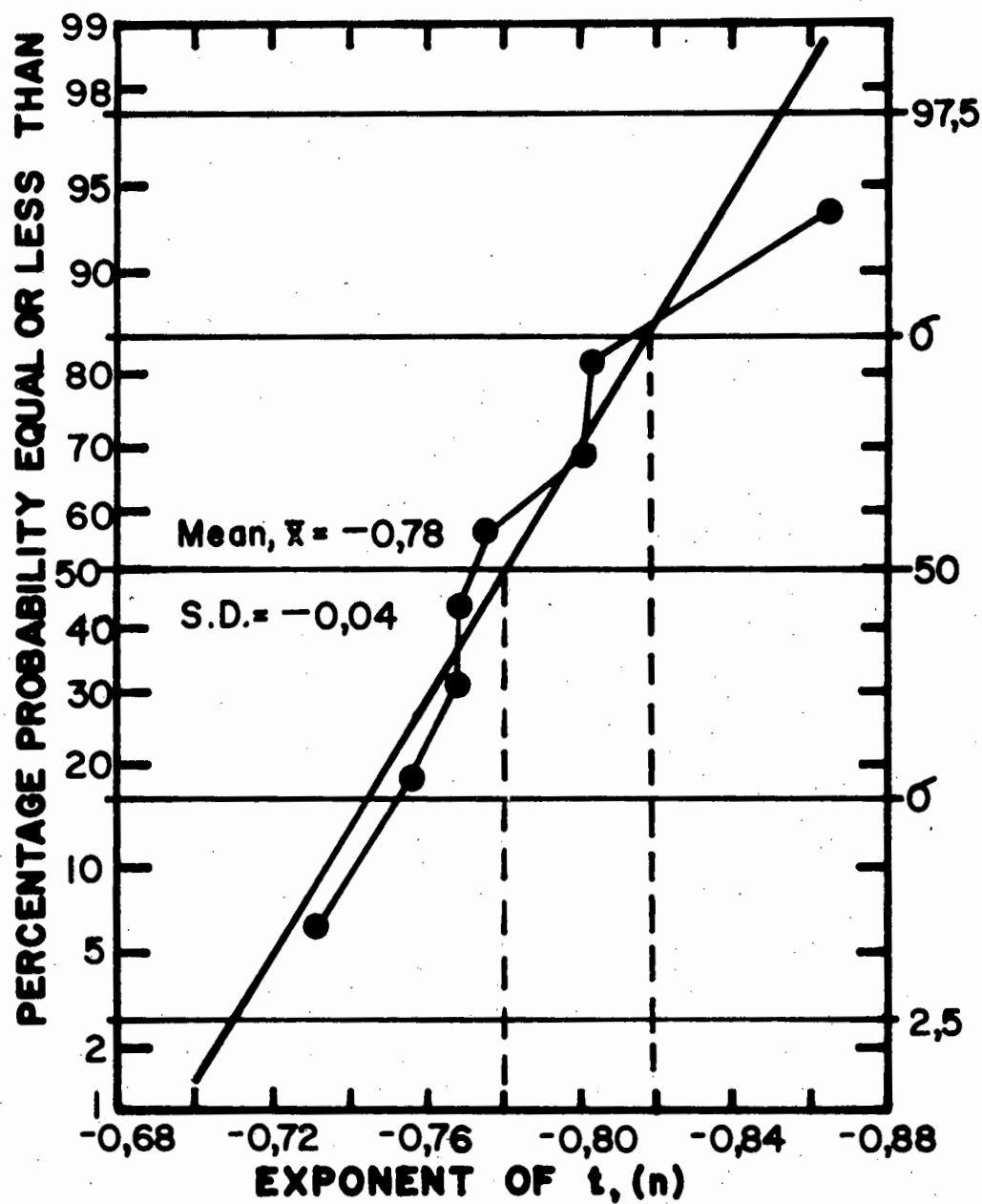


FIG. 38: DISTRIBUTION OF THE
EXPONENT OF TIME, n .

(ex FIG. 37)

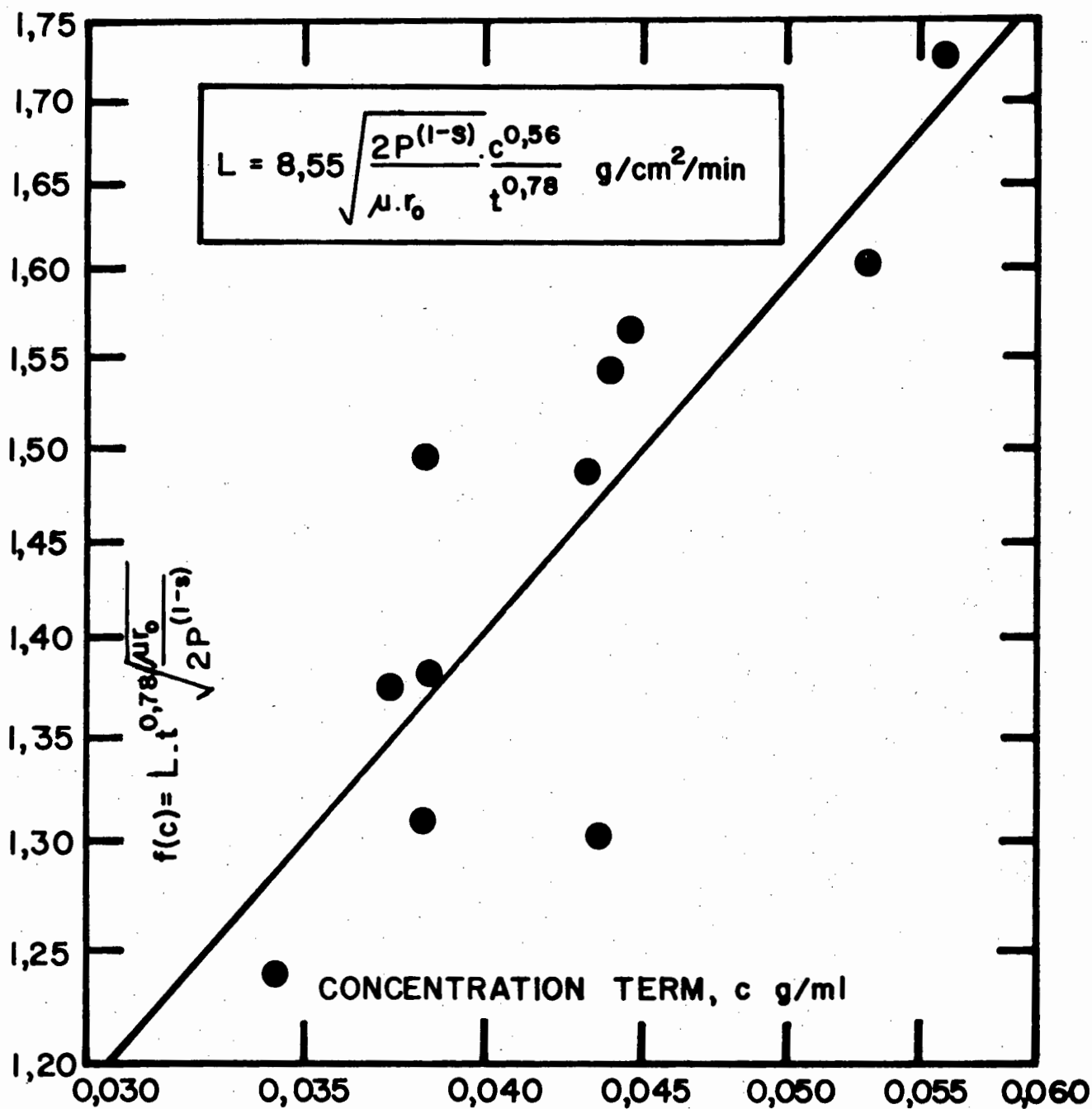


FIG.39: DETERMINATION OF RELATIONSHIP BETWEEN c & f(c) TO ESTABLISH A MODIFIED EMPIRICAL LOADING RATE EQUATION FOR CONSTANT HEAD SAND FILTRATION OF SLUDGE. (TABLES 20 & 2.IIIc)

drainage, relatively small changes in c result in greater increases in C_f . In this work, it was found that the sludge concentration term ranges from $c = 0,03 \text{ g/ml}$ to $c = 0,06 \text{ g/ml}$, and the initial sludge concentration C_o from $10 - 20 \text{ g/l}$. Selecting a fixed value of $c = 0,03 \text{ g/ml}$, the following table shows the negligible effect of the initial concentration C_o on the loading rate:-

TABLE 2.IV

Calculation of Constant Head Filtration Loading Rates at Different Initial Sludge Concentrations and Depths.

$c = 0,03 \text{ g/ml}$				
$C_o \text{ (g/l)}$		$C_f \text{ (g/l)}$	Mean Sludge Density $\bar{\rho} \text{ (g/ml)}$	
10		15,0	1,0095	
20		59,3	1,0165	
$C_o \text{ (g/l)}$	Bed Depth $h \text{ (cm)}$	Pressure Head, $P \text{ (g/cm}^2\text{)}$	[equation(36)] L_{CALC} $\text{(kg/m}^2\text{/d)}$	[equation (41)] L_{EMP} $\text{(kg/m}^2\text{/d)}$
10	30	31,90	4,126	3,879
20	30	32,12	4,136	3,889
10	25	26,85	3,893	3,630
20	25	27,04	3,902	3,640
10	20	21,81	3,629	3,345
20	20	21,96	3,638	3,355
10	15	16,76	3,321	3,008
20	15	16,87	3,328	3,017
10	10	11,71	2,942	2,587
20	10	11,79	2,949	2,595

TABLE 2.IVa (Fig. 34a)

The Effect of Filtration Time on Loading Rate in Constant
Head Filtration.

Pressure Head, $P(\text{g/cm}^2)$	10	15	25
Filtration Time, $t(\text{days})$	$L_{\text{EMP}} \times 10^{-4*}$ [g/(min)(cm ²)]	$L_{\text{EMP}} \times 10^{-4*}$ [g/(min)(cm ²)]	$L_{\text{EMP}} \times 10^{-4*}$ [g/(min)(cm ²)]
0,25	2,76	3,47	4,46
0,50	2,19	2,67	3,33
0,75	1,88	2,26	2,79
1,00	1,68	2,00	2,45
1,25	1,53	1,81	2,22
1,50	1,42	1,67	2,04
1,75	1,33	1,56	1,90
2,00	1,25	1,47	1,78
2,50	1,14	1,33	1,61
3,00	1,05	1,22	1,48
4,00	0,92	1,07	1,29
5,00	0,83	0,97	1,16

* NOTE:-

$$L_{\text{EMP}} = \sqrt{\frac{2P(1-s)c}{\mu r_o t}} - 1,028 \frac{R_m}{r_o P^s t} + 0,0236 \times 10^{-4},$$

where $r_o = 0,512 \times 10^6$ (sec)(min)/g; $\mu = 0,01026$ poise;

$s = 0,325$; $c = 0,03$ g/ml; $R_m = 4,31 \times 10^4$ (min)(sec)/cm²,

Figure 34 is a plot of the filter loading rate against sludge depth at constant head, for a range of values of c calculated at the average expected value of $C_o = 14$ g/l. The curve for $c = 0,03$ g/ml is plotted for both $C_o = 10$ g/l and $C_o = 20$ g/l. This shows the negligible influence of C_o . The cycle time has been kept constant at $t = 1440$ minutes (1 day). This set of curves may be employed at a later stage for design purposes, once the depth of sludge has been selected.

Calculating the loading rate from equation (41),

$$L_{EMP} = \sqrt{\frac{2P(1-s)c}{\mu r_o t}} - 1,028 \frac{R_m}{r_o P^s t} + 0,0236 \times 10^{-4},$$

in which the solids removal term $c = 0,03$ g/ml and the mean value of $R_m = 4,31 \times 10^4$ (min)(sec)/cm², the effect of constant head filtration for different filtration times and pressure heads is apparent in Table 2.IVa. The data are plotted in Fig. 34a. It is seen that as the filtration time increases, the rate of drainage decreases, and the rate at which slurry may be fed to the filter to maintain a constant head, will thus also decrease.

After a period of approximately one day's constant head filtration, the loading rate is considerably reduced. This was confirmed in the experimental filtrations of this particular sludge, when regardless of the initial concentration, C_o , or operating depth, h , a filtration time in excess of one day produced an insignificant amount of filtrate. Although a reduction of the filtration time increases the loading rate for a given sludge dewatering, too short a filtration time achieves insignificant sludge dewatering. It was thus apparent that for this sludge, the constant head filtration time should not exceed approximately one day.

The loading rate is also temperature dependent, by virtue of L varying inversely as the square root of the viscosity [Equations (36), (37c) and (41)]. Thus it would be appropriate to operate at as high a temperature as possible, in order to reduce the liquor viscosity, and thereby increase the loading rate. Hence, for a given filtration, $L \propto \sqrt{1/\mu}$. If L_1 and L_2 are respective loading rates at temperatures T_1 and T_2 at which the viscosities are μ_1 and μ_2 , respectively, then

$$\frac{L_1}{L_2} = \sqrt{\frac{\mu_2}{\mu_1}} .$$

The effect of temperature is indicated in the following examples:-

$$L_{10^{\circ}\text{C}} = 0,87 \times L_{20^{\circ}\text{C}} ,$$

$$L_{30^{\circ}\text{C}} = 1,12 \times L_{20^{\circ}\text{C}} .$$

2.6. Falling Head Drainage.

As an alternative procedure to constant head filtration, a sand filter may be filled and then allowed to drain [section 1.4. (4)]. In this case, filtration is under a falling hydrostatic head of sludge. Although the head of sludge reduces with time, the resistance of the filter progressively increases as a result of compaction of the mass. Application of the Carman equation (27) produces a plot which is approximately linear, as shown in Fig. 40. The linearity is not as close as that observed in the case of constant head operation (Figs. 28 - 30 inclusive).

In calculating the specific resistance according to equation (28), the pressure term suggested by Eckenfelder is defined by the average hydrostatic head, H , of sludge during drainage. However, a typical plot on semi-log paper of the head of sludge against time, shows approximate

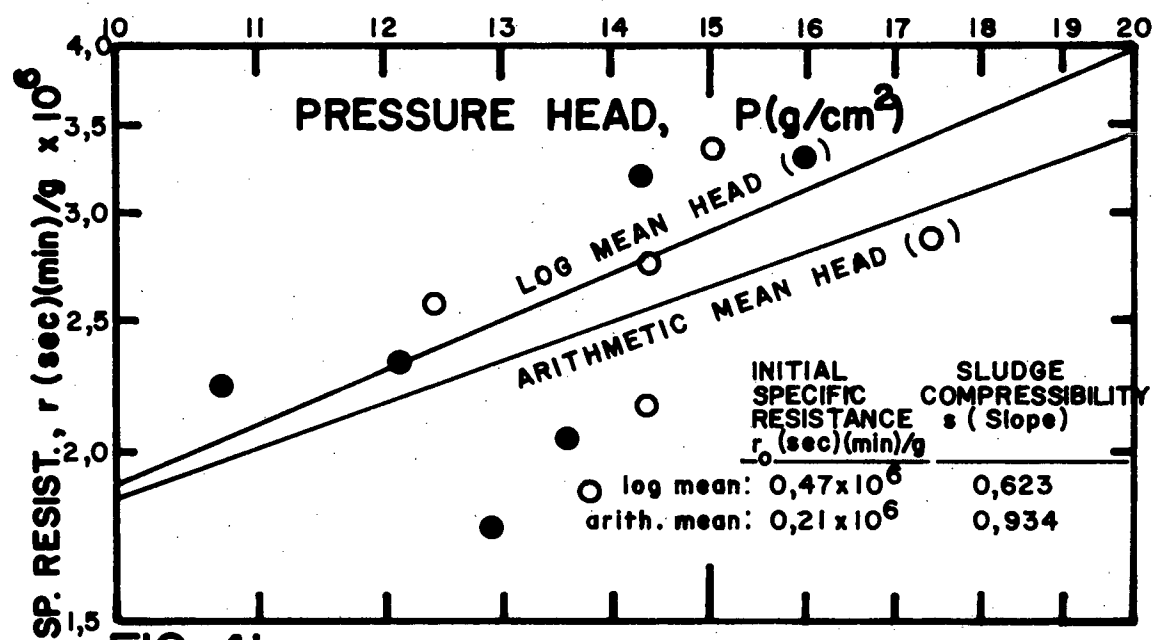
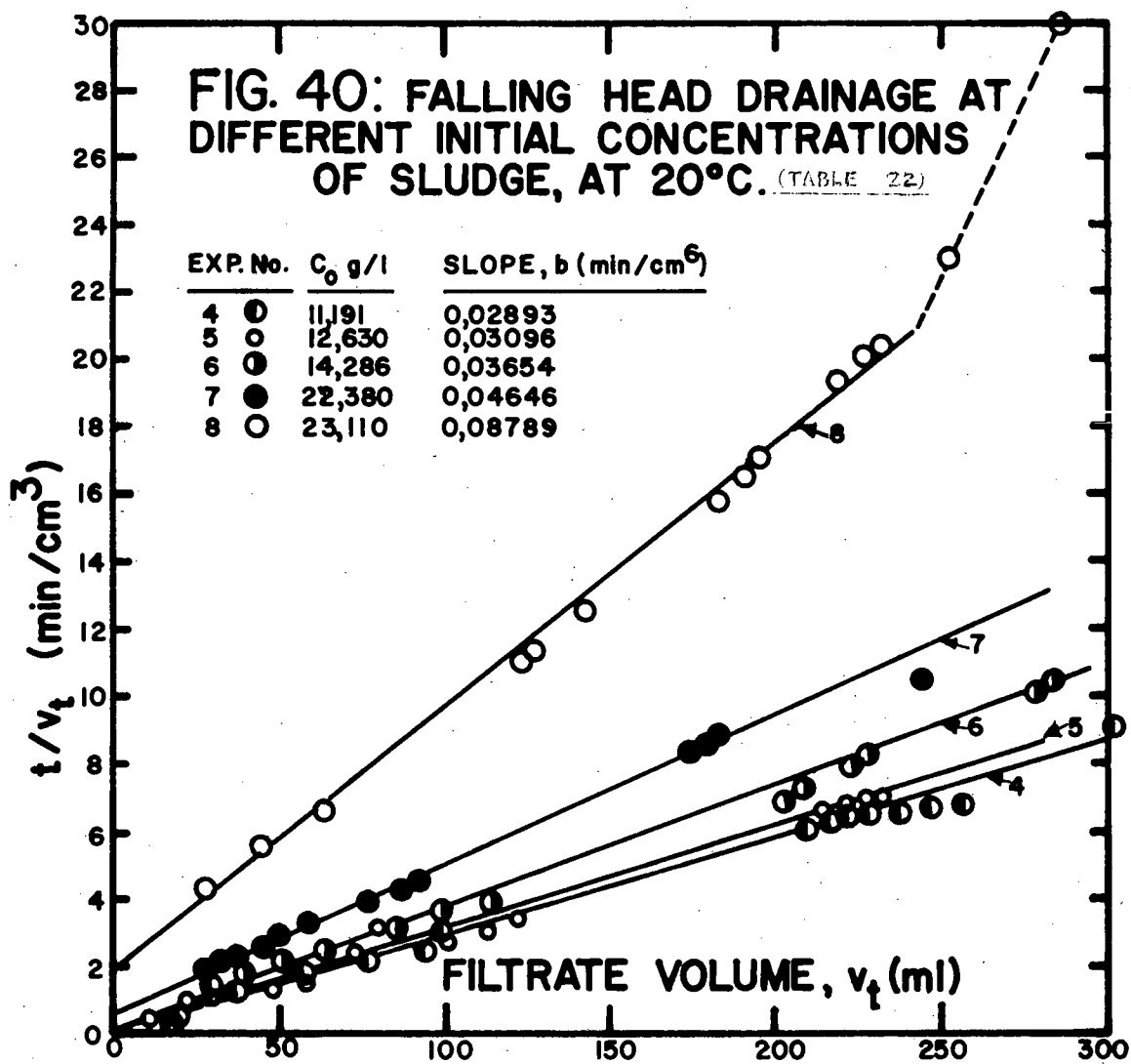


FIG. 41: RELATIONSHIP BETWEEN SPECIFIC RESISTANCE OF SLUDGE & PRESSURE.(FALLING HEAD).
(TABLE 23)

linearity (Fig. 44), which rather suggests the use of a log-mean head, defined by

$$\bar{H}_{(L.M.)} = \frac{H_o - H_f}{2,303 \log \frac{H_o}{H_f}} \quad (42)$$

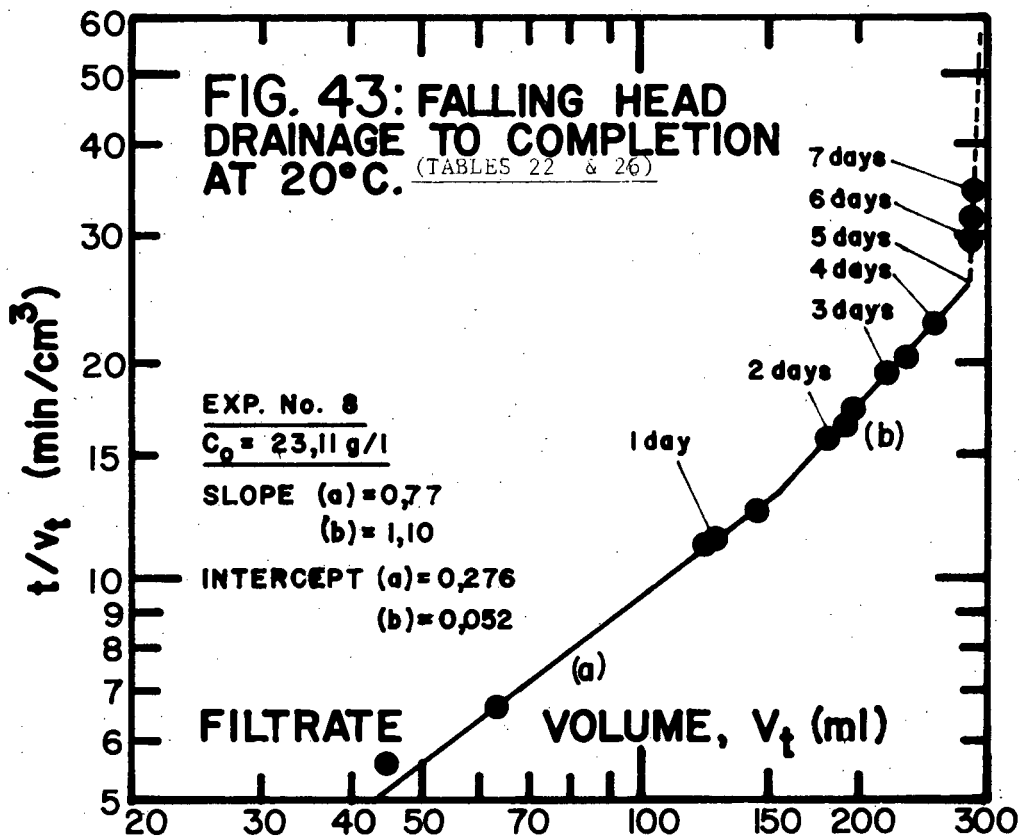
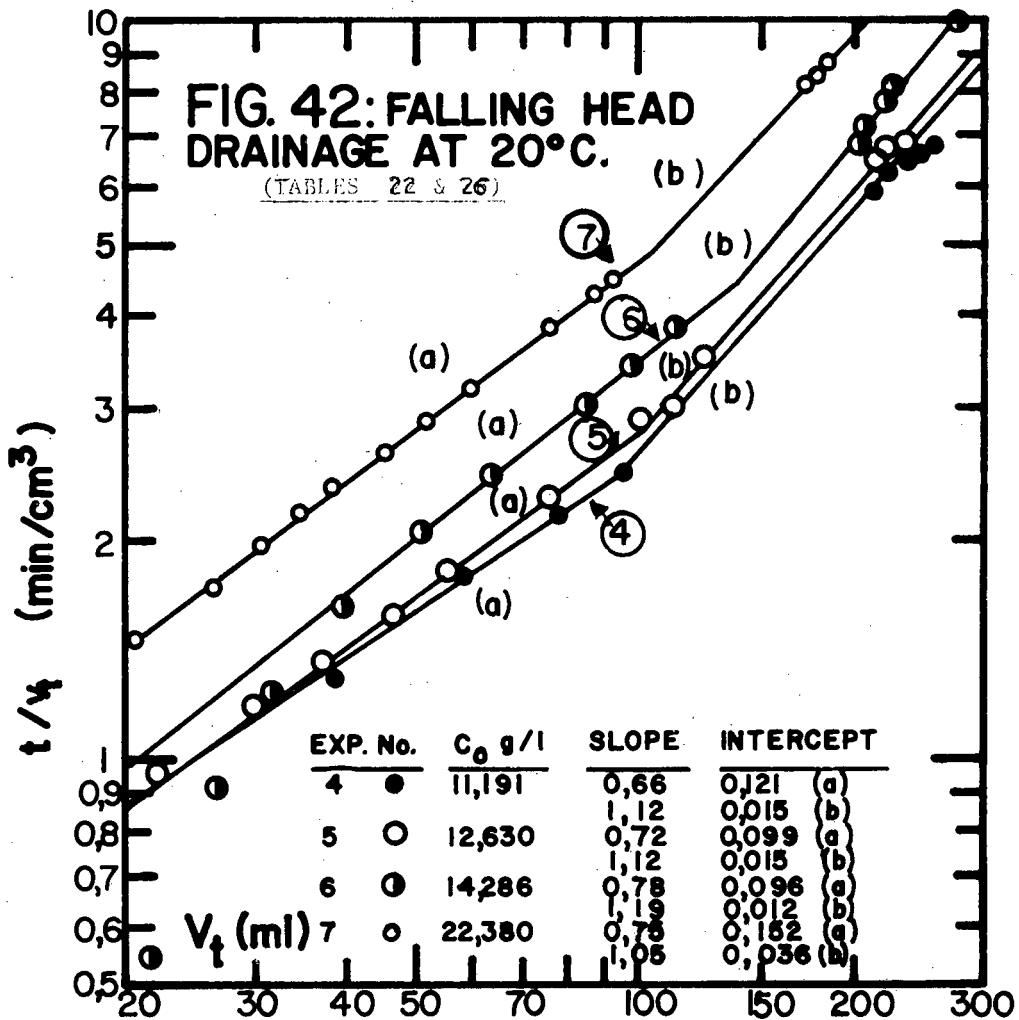
The log-log plot of specific resistance of sludge against pressure, where the latter is calculated both at the arithmetic-mean and logarithmic-mean heads respectively, shows considerable scattering of the data, as shown in Fig. 41. This indicates the variable permeability of the draining sludge. Fitting straight-lines by least-squares, there is virtually no correlation for the data plotted against the arithmetic-mean head (correlation coefficient = 0,03). The log-mean head plot, however, shows a slightly better correlation (correlation coefficient = 0,33). The initial specific resistance, r_o , is lower than that obtained during constant head filtration, and the compressibility, s , is higher.

A log-log plot of t/v_t against v_t (Figs. 42 and 43), shows that for each initial sludge concentration, two straight lines may be obtained. The first line approximately describes the filtration during the first day's drainage, and thereafter the characteristics of the sludge change, and a second line with increased slope (reduced rate of filtration) applies during the following 2 to 5 days. This change may result from the compaction of the sludge. Each of the log-log plots has the form

$$t/v_t = k \times v_t^n \quad (43)$$

where k and n are empirical constants. (In the present investigation, it was found that during the first day of drainage, $n \approx 0,7 - 0,8$; during the following 5 days, $n \approx 1,1 - 1,2$).

Table 2.V. shows the extent of divergence between observed loading rates, and those calculated from the Carman equation. Table 2.VI compares the loading rates when operating under falling head drainage, with those



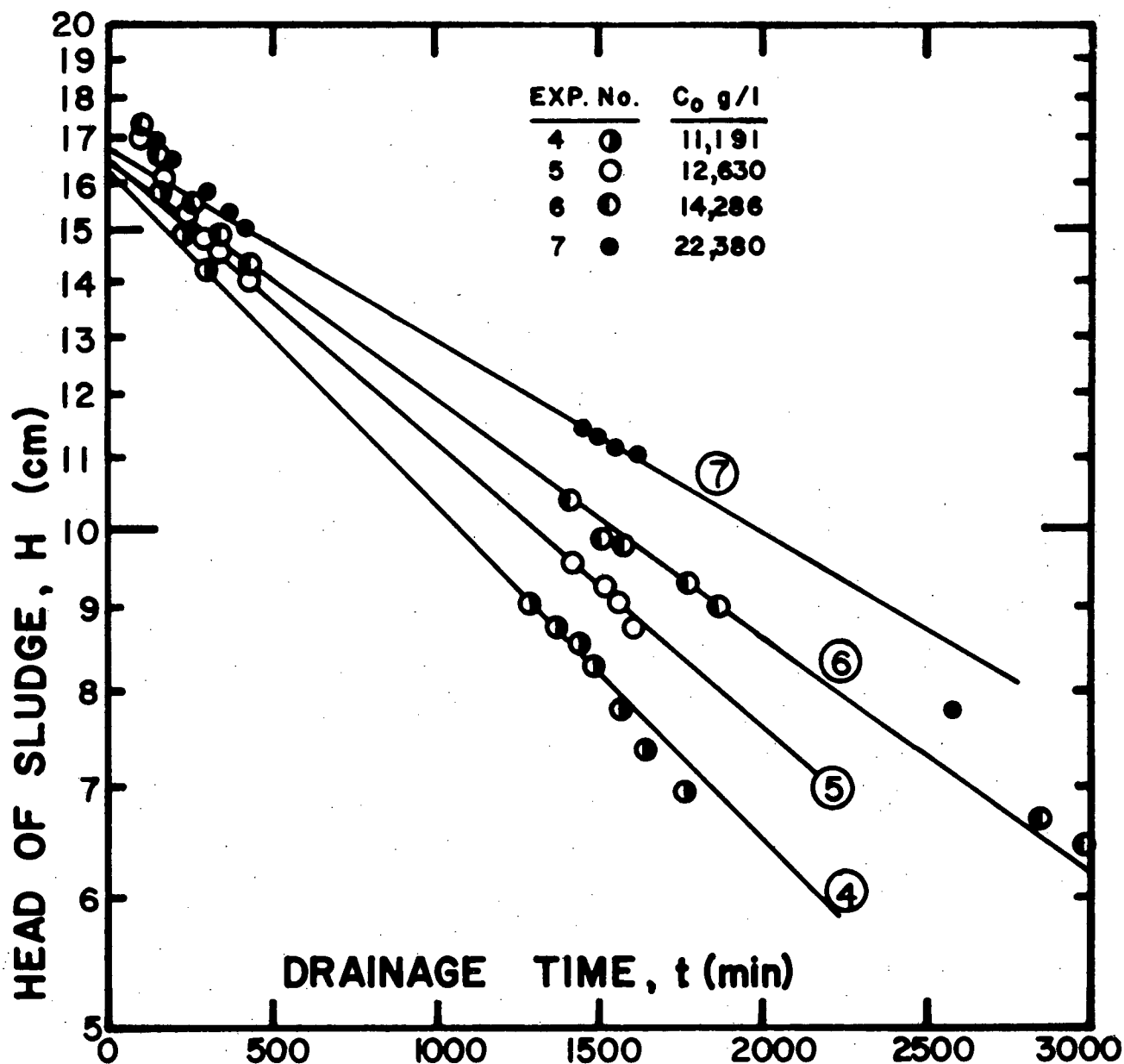


FIG. 44: FALLING HEAD DRAINAGE AT 20°C., SHOWING EXPONENTIAL RELATIONSHIP BETWEEN SLUDGE HEAD & DRAINAGE TIME.

(TABLE 22)

TABLE 2. V

99.

FALLING HEAD DRAINAGE ON SAND FILTER

(Temp. = 20°C.; $r_o = 0,466 \times 10^{-6}$ (sec)(min)/g; $s = 0,623$)

Exp. No.	C _g (g/l)	H _{L.M.} (cm)	P _{L.M.} (g/cm ²)	t (min)	c (g/ml)	L _{OBS} × 10 ⁴ [g/(min) (cm ²)]	L _{CALC} × 10 ⁴ [g/(min) (cm ²)]	$\frac{L_{OBS}}{L_{CALC}}$
4	11,191	10,60	10,76	2730	0,01372	0,67	0,72	0,94
5	12,630	13,46	13,62	1610	0,02032	1,31	1,19	1,10
6	14,286	11,95	12,14	2985	0,01860	0,80	0,82	0,97
7	22,380	12,70	12,93	2582	0,03362	1,42	1,20	1,18
8	23,110	14,08	14,32	4740	0,03885	0,85	0,97	0,87

NOTE:- $L_{OBS} = \frac{V_c}{tA}$; $L_{CALC} = \sqrt{\frac{2 \cdot P(1-s) \cdot c}{\mu r_o \cdot t}}$

TABLE 2. VI

COMPARISON OF FALLING HEAD & CONSTANT HEAD DRAINAGE AT
EQUIVALENT HYDROSTATIC HEADS

(c = 0,035 g/ml; t = 1440 min.)

Pressure Head P (g/cm ²)	L _{CALC} (Falling Head) × 10 ⁻⁴ (g/(min)(cm ²))	L _{CALC} (Constant Head) × 10 ⁻⁴ (g/(min)(cm ²))	$\frac{L_{CONSTANT HEAD}}{L_{FALLING HEAD}}$
10,76	1,58	2,09	1,32
12,14	1,61	2,23	1,38
12,93	1,64	2,28	1,39
13,62	1,65	2,32	1,41
14,32	1,67	2,36	1,42

NOTE:- (i) $L_{FALLING HEAD} = \sqrt{\frac{2 \cdot P(1-0,623) \cdot c}{\mu \cdot t \times 0,466 \times 10^{-6}}}$ (ii) $L_{CONST. HEAD} = \sqrt{\frac{2P(1-0,325) \cdot c}{\mu t \cdot 0,512 \times 10^{-6}}}$

obtained at constant heads equal to the log-mean hydrostatic head during falling head drainage. In both cases, the simplified Carman equation (36) has been used, ignoring filter media resistance. It is seen that for a given sludge draining on a given area of sand bed, the loading rate in the pressure head range 10 - 14 g/cm² is 30 - 40% higher in the case of constant head filtration. Table 2.VII compares the two methods of operation at different depths of sludge. These calculations are plotted in Fig. 34. It is thus apparent that a process which is initially operated as a constant head filtration and then followed by drainage, requires less bed area to handle a given sludge than one which is operated only as a falling head drainage process.

It is therefore proposed to operate the sand filter at constant head for one day, followed by drainage and drying. Hence, only the loading rate derived from the constant head equation (41) need be considered. During the subsequent falling head drainage, the filter bed has no further solids handling capacity. Equation (43) has no useful significance under these conditions.

TABLE 2. VII

COMPARISON OF FALLING HEAD & CONSTANT HEAD DRAINAGE AT INCREMENTAL
SLUDGE DEPTHS.

(Mean sludge density assumed, $\rho = 1,014 \text{ g/ml}$).

Sludge Depth h (cm)	Hydrostatic Head H (cm)	Pressure Head P (g/cm^2)	$L_{\text{FALLING HEAD}}$ ($\text{Kg/m}^2/\text{d}$)	$L_{\text{CONSTANT HEAD}}$ ($\text{kg/m}^2/\text{d}$)	$L_{\text{CONSTANT HEAD}}$ $L_{\text{FALLING HEAD}}$
10	11,6	11,76	2,05	2,82	1,38
15	16,6	16,83	2,26	3,28	1,45
20	21,6	21,90	2,41	3,64	1,51
25	26,6	26,97	2,53	3,95	1,56
30	31,6	32,04	2,64	4,22	1,60
Note:- (i) In Falling Head Drainage, h, H and P are log-mean values. (ii) In Constant Head Filtration, values of P are the same as in Falling Head Drainage at corresponding sludge depths.					

2.7. Atmospheric Sludge Drying.

Evaporation of moisture from the sludge in the sand filter will proceed throughout the filtration period. At some stage, the rate at which filtrate drains from the filter becomes negligible, the rate of drying becoming significant. Drying is a diffusion process, the rate being dependent on the temperature, humidity of the air, and air velocity.

Drying theory (2c, 6c) shows that evaporation proceeds at a constant rate, until a critical moisture content is reached. Thereafter, evaporation proceeds at a falling rate, until the equilibrium moisture content is reached. During the constant rate period, the plane of vaporisation is near to the surface, and the rate will not be greatly affected by the nature of the sludge. At, and beyond the critical moisture content, the rate of diffusion of water to the surface decreases, and this results in a falling rate of drying period. During the latter period, the physical and chemical properties; thickness; and atmospheric conditions; will be dependent variables. Eventually, a sub-surface drying phase occurs, until the equilibrium moisture content is reached. The latter is dependent on the humidity and temperature of the air, as well as on the characteristics of the dried sludge itself. However, although the equilibrium moisture content under specific conditions is the ultimate dried state resulting from air drying, it is not necessary to dry to this degree, if the ultimate requirement is only the ability to be lifted from a sand-bed.

During the constant drying rate period, physical changes occur on the surface of the sludge. These can affect the permeability of the surface layer. If the surface evaporation rate is higher than the mass transfer rate

from within the mass to the surface, "case-hardening" of the latter may occur, and a surface skin is formed which reduces permeability. This results in shrinkage of the surface and an overall fall in the evaporation rate. If the shrinkage is sufficiently large, surface cracking and fissure formation occurs. The latter increases the exposed surface area, tending to produce a temporary increase in the evaporation rate.

Thus, the physical characteristics of the hydrated sludge are responsible for complex variations in the rate of drying, during the normally so-called constant drying rate period.

2.7.1. Ideal Drying Theory.

Under ideal conditions, when the drying sludge is "inert", the rate of drying may be described by the following equation (Badger and McCabe^(6c)):-

$$\frac{M}{t} = k'_G A (H_1 - H_a) \quad (44),$$

where M = mass of moisture (g) evaporated in time t (hr),

k'_G = film coefficient for the diffusion process $[g/(hr)(m^2)]$,

A = area of surface/air interface (m^2),

H_1 = humidity of the air at the interface (g water/g dry air)

(equivalent to saturation humidity at the interface temperature,
g water/g dry air),

H_a = humidity of the drying air (g water/g dry air).

The diffusion coefficient k'_G is a function of the mass velocity of the air ^(6c), i.e.

$$k'_G = \alpha (V\rho)^n \quad (44A),$$

where V = velocity of the air (m/hr),

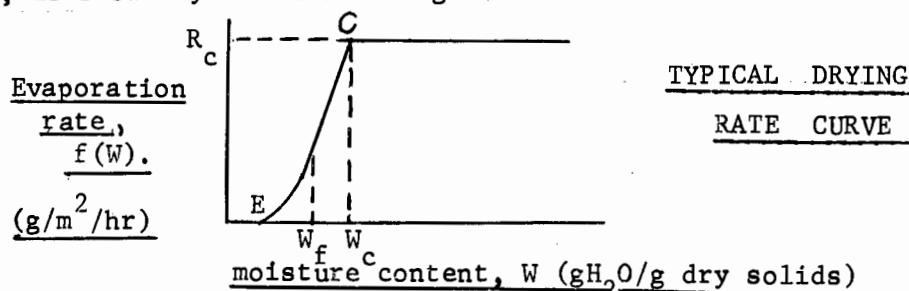
ρ = density of the air (g/ml),

α = a constant, and

n = experimentally determined exponent ≈ 0.8 .

Equations (44) and (44A) respectively, indicate that the drying rate is proportional to the humidity, as well as being a function of the velocity of the drying air. It is assumed that the temperature of the diffusion interface is equal to that of the surrounding air. In practice, surface evaporation causes the temperature of the interface to be slightly lower than the air temperature, and is equivalent to the interface wet-bulb temperature.

When the critical moisture content is reached, a falling rate period follows, as shown by CE in the diagram below:-



In the falling rate period, the evaporation rate is a function of the moisture content. Commencing at the critical moisture content W_c , (g water/g dry solids), the mass transfer rate of moisture from within the mass begins to be less than the surface evaporation rate. The rate reduces progressively, until the equilibrium moisture content E (g water/g dry solids) is reached, and no further evaporation occurs.

At any instant during the drying phases, the moisture is present as both bound and unbound (or free) water. The free moisture content is equal to the difference between the total moisture content and the equilibrium moisture content.

It is assumed that the rate of drying during the falling rate period CE is a function of the free moisture content $(6c)$. The mass of water evaporated in a short time interval is equal to the product of the mass of dry solids, and the change in free moisture content per unit mass of dry solids.

If m = mass of free moisture (g) at time t (hr),

D = mass of dry solids (g),

A = drying surface area (m^2),

W = total moisture content of the mass (g water/g dry solids),

E = equilibrium moisture content (g water/g dry solids), and

$f(W-E)$ = ordinate of the rate of drying curve ($g/m^2/hr$),

where the free moisture content is $(W-E)$ g/g, then

$$m = D(W-E) \quad (45).$$

Differentiating (45) with respect to the free moisture content, $(W-E)$,

$$dm = D \times d(W-E) \quad (46).$$

But by the assumption, that the rate of drying during the falling rate period is a function of the free moisture content ^(6c),

$$-\frac{dm}{dt \times A} = f(W-E) \quad (47).$$

Substituting (46) in (47)

$$-\frac{D}{A} \times \frac{d(W-E)}{dt} = f(W-E),$$

$$\text{or} \quad dt = -\frac{D}{A} \times \frac{d(W-E)}{f(W-E)} \quad (48).$$

Integrating (48) over the limits from the beginning of the falling rate period, to a time t_f in this period; and from the critical moisture content W_c , to the value W_f ($W_c > W_f > E$),

$$\int_0^{t_f} dt = t_f = \frac{D}{A} \int_{W_f}^{W_c} \frac{d(W-E)}{f(W-E)} \quad (49) \quad (6c).$$

Equation (49) may be solved graphically by plotting $1/f(W-E)$ against $(W-E)$, the latter being obtained from an experimental drying-rate curve, and the former from the area under this curve between the limits W_f and W_c .

An approximation ⁽¹³⁾ may be made by assuming that the drying rate in

the falling-rate period is directly proportional to the free moisture content. The approximation was found to be valid at lower loadings, as shown in plot 1 of Fig. 46. Equation (47) then becomes

$$-\frac{dm}{A dt} = b(W-E) = \frac{R_c (W-E)}{(W_c-E)} \quad (50).$$

(R_c is the ordinate of the rate at the critical moisture content, and b is the slope of the falling rate curve, CE in above diagram).

Substituting (46) in (50), transposing and integrating,

$$\int_0^{t_f} dt = \frac{D}{A} \cdot \frac{(W_c-E)}{R_c} \int_{W_f}^{W_c} \frac{d(W-E)}{(W-E)} \quad (51).$$

Equation (51) evaluates to give

$$t_f = \frac{D}{A} \cdot \frac{(W_c-E)}{R_c} \ln \frac{(W_c-E)}{(W_f-E)} \quad (52),$$

$$(W_f \neq E),$$

This is the equation given in Badger and McCabe (6c).

Under given constant conditions, W_c and E are constant. A plot of the log of the free moisture content (W_f-E) against time, results in a straight line. This is shown in Fig. 49 during the earlier stages of the falling rate period. At later stages the plot deviates from linearity. If the slope of the semi-log plot is K_f , equation (52) may be written as

$$\ln \frac{(W_c-E)}{(W_f-E)} = K_f t_f \quad (52A),$$

(W_c-E) is the free moisture content at the "critical" moisture content, and is the intercept of the plot at $t_f = 0$.

2.7.2. Modification of Drying Theory.

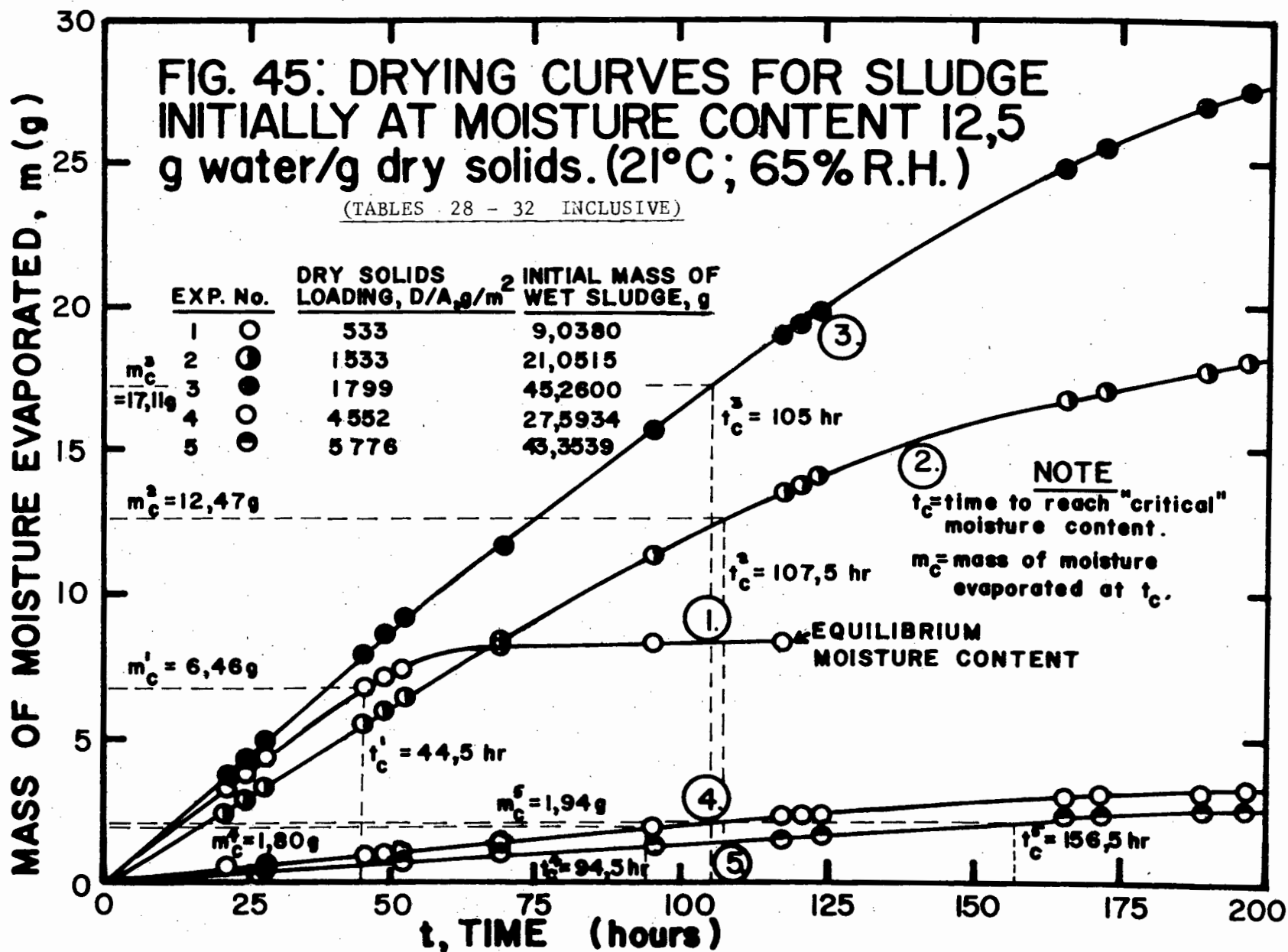
The problem with the use of the theoretical equations (section 2.7.1.) for sludge drying in the present work, is that the values of R_c and W_c are not known unless they are determined experimentally.

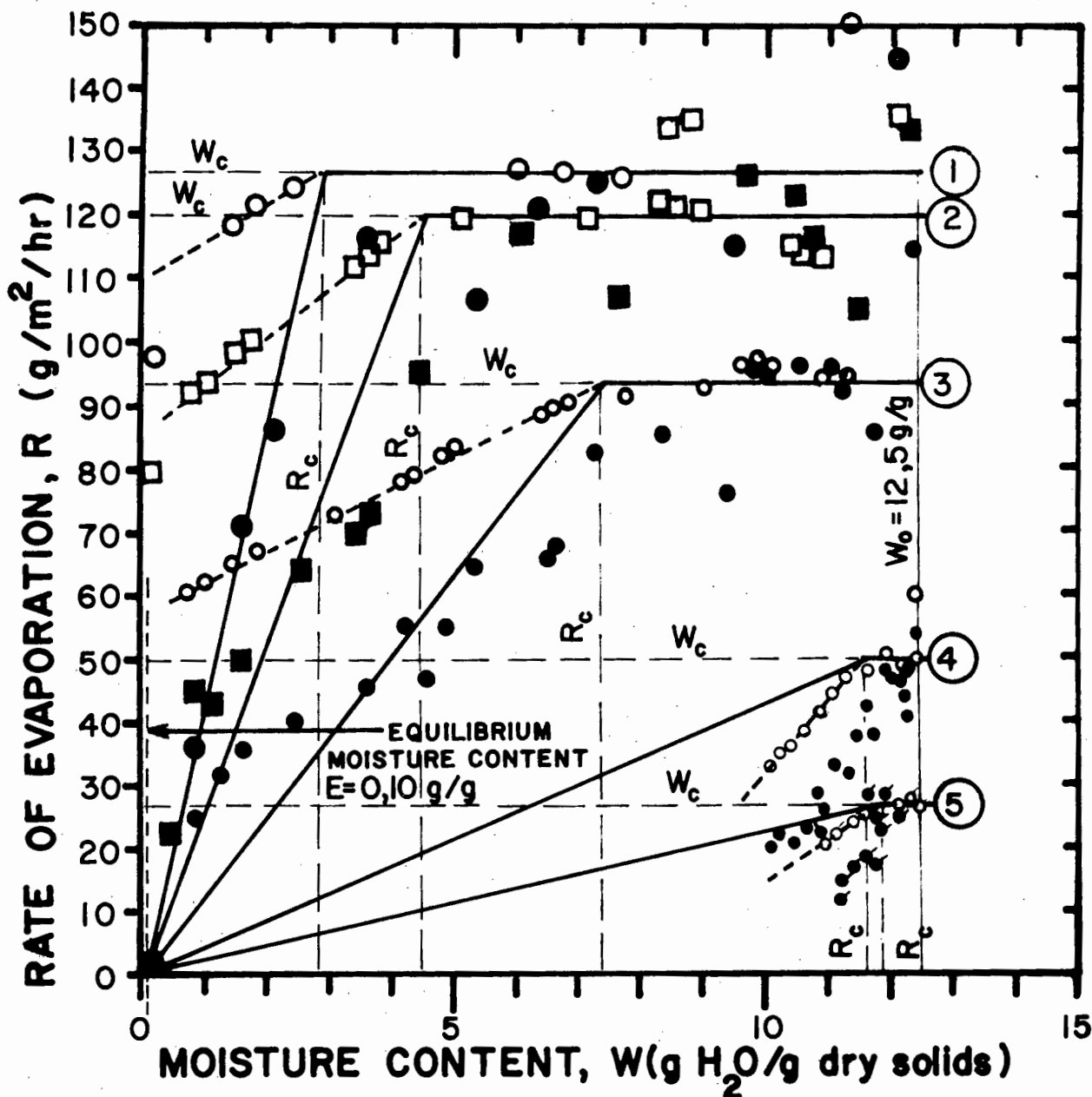
The varying characteristics of the sludge and its interface during drying, results in the constant drying rate and critical moisture contents being ill-defined, as shown in Fig. 46. Maintaining constant air temperature, humidity and air mass velocity, the diffusion constant varies in a complex manner. Varying the loading, different values of W_c and R_c were obtained. Experimental variation of the exposed surface area and the depth of the drying sludge, varies the mass loading term, D/A (defined as "the mass of dry solids per unit area"). The deeper the sludge bed, the greater the mass of free moisture beneath the interface. Because the change in free moisture content is measured on the entire mass (the moisture content at any time thus being an average value), the physical dimensions of the surface, and the depth of the sludge, will be expected to contribute towards deviations from the theoretical analysis.

The writer proposes the following sequence of plots to obtain values of R_c and W_c , which may then be used in the theoretical drying equations for design purposes:-

With aliquots of the same sludge drying under variable loading conditions, the relationships between the deviations of the theoretical constants may be estimated as follows:-

Calculating the average rate of evaporation per unit area at each time t from $t = 0$ to $t = t$, and plotting these values against the moisture content at t , Fig. 46 results. It was observed that an artificial falling-rate line is obtained, which is linear. The intersection of this line with the horizontal constant - rate values, yields co-ordinates for R_c and W_c .





EXP. No.	LOADING (kg/m ²)	'CRITICAL' MOISTURE CONTENT, W_c (g/g)	'CONSTANT' DRYING RATE, R_c (g/m ² /hr)
1 ● ○	0,533	2,85	126,8
2 ■ □	1,533	4,50	120,0
3 ● ○	1,799	7,40	93,7
4 ● ○	4,552	11,62	50,0
5 ● ○	5,776	11,90	26,8

NOTE: SOLID DATA POINTS CORRESPOND TO AVERAGE VALUES OF R & W. OPEN POINTS ARE PLOTS OF $R = M/\Delta t$ vs. W.

FIG. 46: RATE OF DRYING CURVES AT DIFFERENT BED LOADINGS(21°C., 65% R.H., 0,8m/min Air Velocity).

This appears to be reasonable. Joining the intersection point with the value of the equilibrium moisture content E at zero rate of evaporation, the course of the falling rate period may be approximated. This method seems to give good agreement at low loadings (Plot 1), but progressively deviates at higher loadings (Plots 2 - 5 inclusive).

From the plots at different loadings D/A in Fig. 46, it seemed that the parameters R_c and W_c in the theoretical equation (52) are functions of the mass loading D/A . Inspection of equation (52) and (52A) shows that because the falling rate constant, K_f , is given by

$$K_f = \frac{\frac{R_c}{D(W_c - E)}}{\frac{A}{A}} \quad (\text{units of } K_f = \text{hr}),$$

K_f is already a function of the mass loading. However, since K_f includes the parameters R_c and W_c , the theoretical value of K_f should be modified to a value given by

$$K_f = \frac{R_c}{(W_c - E)} \times f(D/A),$$

This modification is shown to hold experimentally, in the following manner:

Plotting the "critical" moisture content W_c against the bed loading D/A on log-log paper, a straight line is obtained, as shown in Fig. 47. The equation of this line is of the form

$$W_c = c' (D/A)^n \quad (53),$$

where c' is the intercept on the W_c axis at $D/A = 1 \text{ g/m}^2$, and n is the slope of the line. In Fig. 47, it was found that

$$W_c = 0,045 \left(\frac{D}{A} \right)^{0,655}.$$

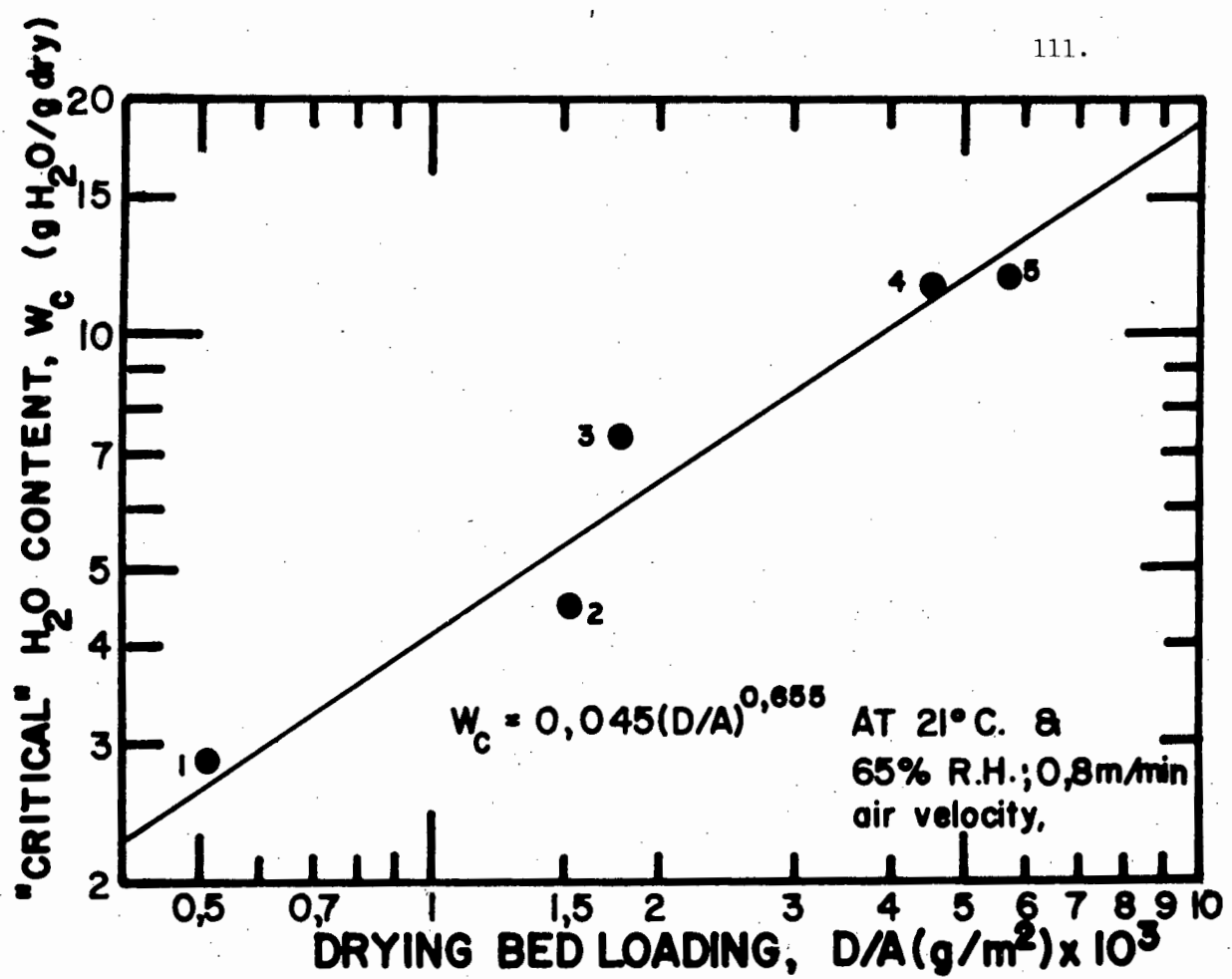


FIG. 47: RELATIONSHIP BETWEEN "CRITICAL" MOISTURE CONTENT & DRY SOLIDS LOADING, D/A.
(TABLE 34)

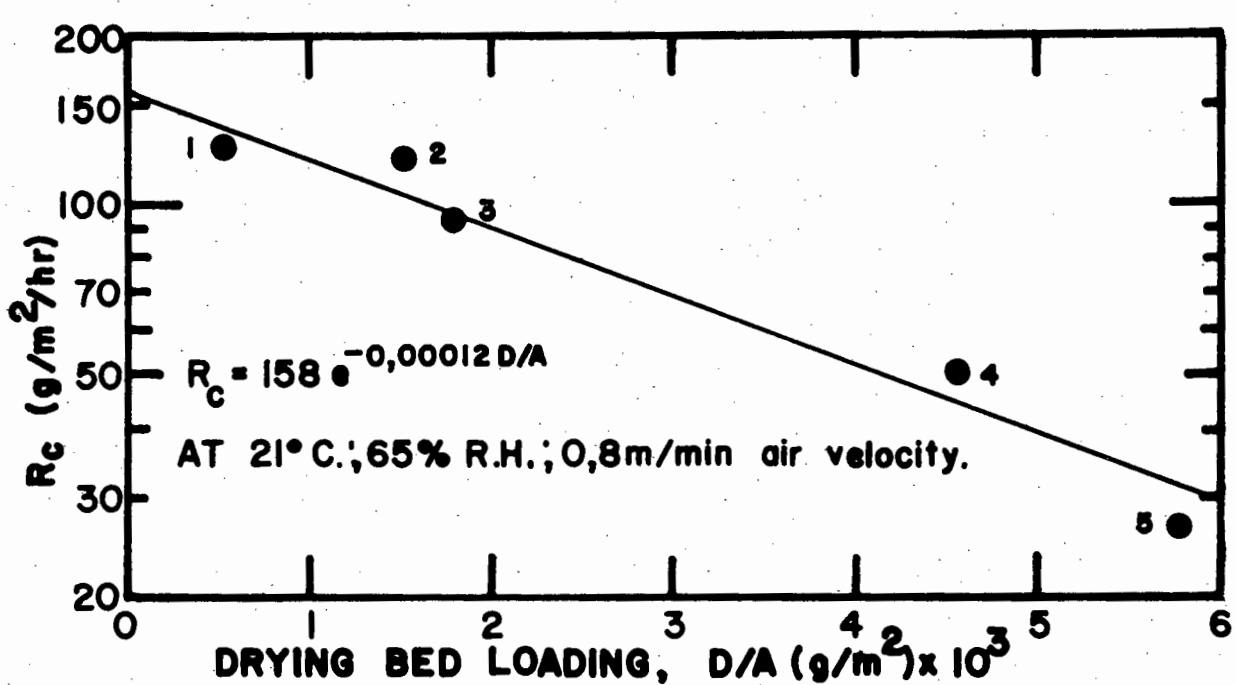


FIG. 48: RELATIONSHIP BETWEEN "CONSTANT" RATE OF DRYING, R_c , & DRY SOLIDS LOADING, D/A.
(TABLE 34)

Plotting the "constant" drying rate R_c against the bed loading on semi-log paper, a further straight line is obtained (Fig. 48). The equation of this line is given by

$$R_c = R_o e^{c'' D/A} \quad (54),$$

where R_o is the intercept at zero loading, and c'' is the slope of the plot. In this work (Fig. 48), it was found that

$$R_c = 158 e^{-0,00012 D/A}.$$

Plotting the free moisture contents ($W_f - E$) against time t_f in the falling rate period on a semi-log plot (Fig. 49), the slopes of the straight lines enable the evaluation of the values of constant K_f [equation (52A)]. Plotting the values of K_f against the bed loading D/A on log-log paper a straight line is obtained, as shown in Fig. 50. From the equation to this line, the modification to K_f is given by

$$K_f = c''' (D/A)^m \quad (55),$$

where c''' is the intercept at $D/A = 1$, and m is the slope of the line. In this work (Fig. 50), it was found that the following equation was obtained:

$$K_f = 2,78 \times 10^6 (D/A)^{-2,67}.$$

Experimental equations (53) - (55) inclusive show the parameters W_c , R_c and K_f respectively, as functions of the mass loading, D/A .

The terms may now be used in the theoretical equations (52) or (52A), thus enabling an estimation of drying times in the falling rate period. These equations are also useful in scaling-up constant temperature and humidity drying experiments to actual bed loadings.

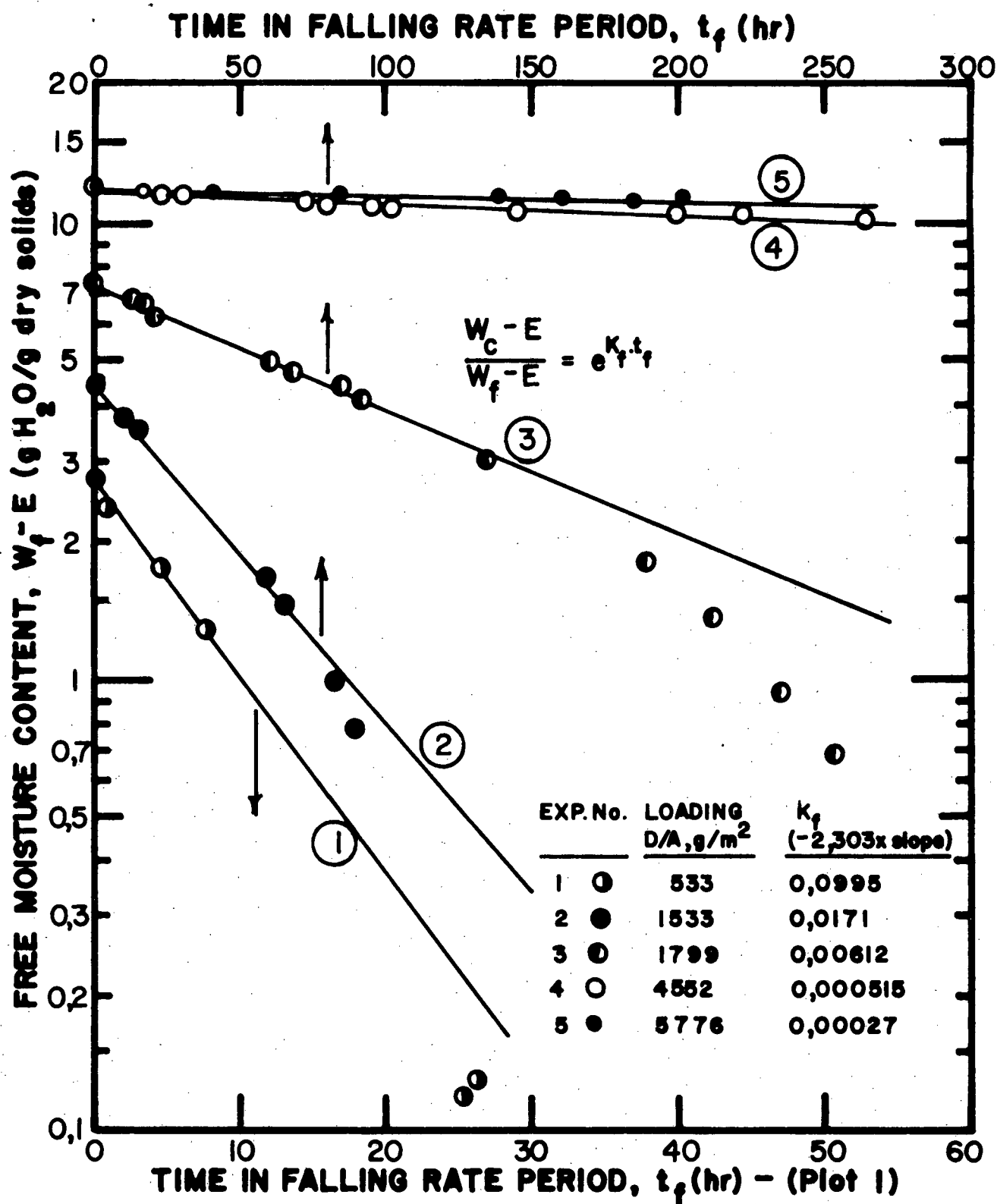


FIG. 49: RELATIONSHIP BETWEEN FREE MOISTURE CONTENT & TIME IN THE FALLING RATE PERIOD OF DRYING AT 21°C. & 65% R.H.

(TABLE 33)

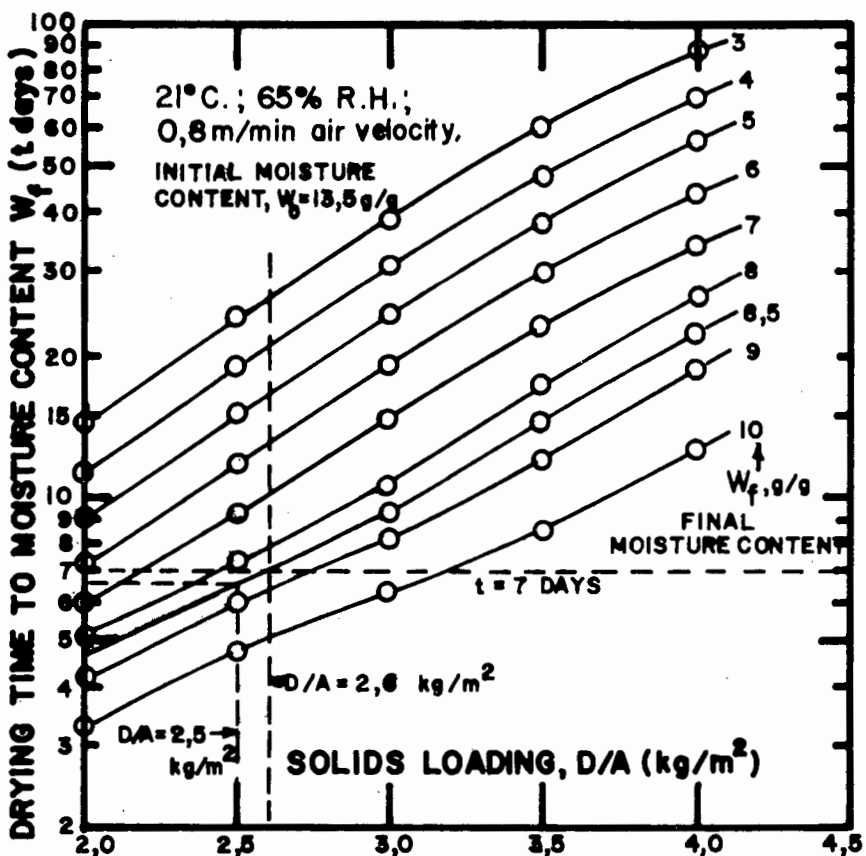
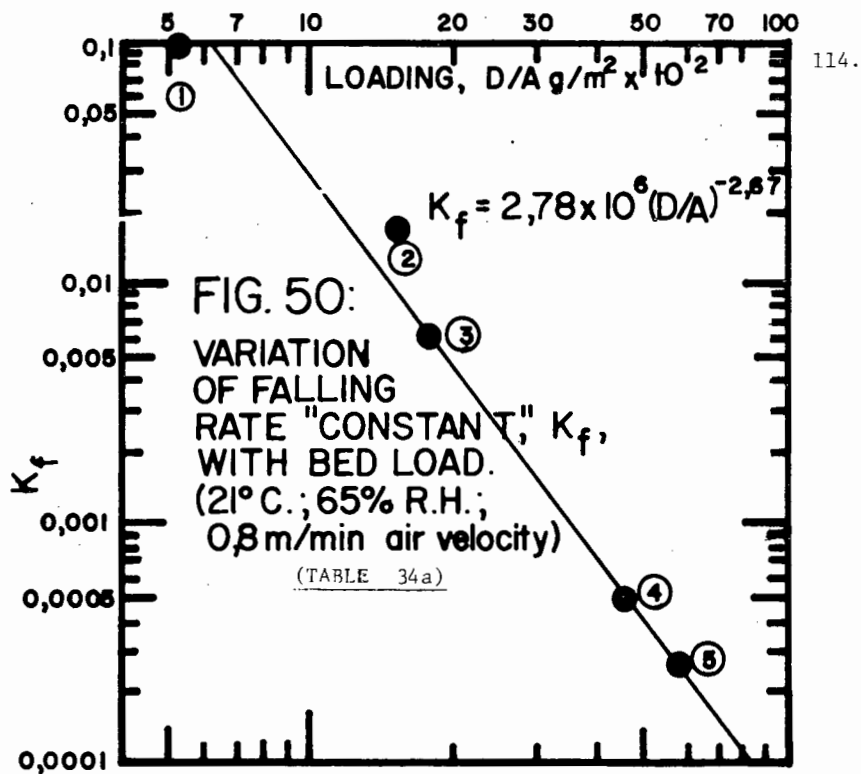


FIG. 51: DRYING TIME CURVES, $W_0 = 13.5 \text{ g/g}$.

(TABLE 36)

2.7.3. Drying Time Equations

Having determined W_c , R_c and K_f , the drying times under identical constant ambient air conditions may be predicted. If the maximum final moisture content enabling the sludge to be spadeable is W , and the moisture content at the commencement of drying is W_o , then if W is greater than or equal to the "critical" moisture content W_c , the drying time in the "constant" rate period, t_c , is given by

$$t_c = \frac{(W_o - W)}{R_c} \cdot \frac{D}{A} \quad (56).$$

In this case, all the drying will occur in the "constant" drying rate period.

However, if the desired moisture content is less than the "critical" moisture content, the drying time will be the sum of the time t_c in the "constant" rate period, up to the "critical" moisture content, and the time t_f in the falling rate period.

$$\text{i.e. } t = t_c + t_f$$

$$\text{where } t_f = \frac{\ln\left(\frac{W_c - E}{W - E}\right)}{K_f} = \frac{2,303 \log\left(\frac{W_c - E}{W - E}\right)}{K_f} \quad (\text{from equation 52A}).$$

$$\text{Thus, } t = \frac{(W_o - W_c)}{R_c} \cdot \frac{D}{A} + \frac{2,303 \log\left(\frac{W_c - E}{W - E}\right)}{K_f} \quad (57).$$

Employing these equations, Table 36 (Section 4.4.5.) has been calculated for the range of bed loadings resulting from the sand filtration (Table 2. IV section 2.5).

These results are shown graphically on the semi-log plot in Fig. 51. The total drying time (in days) to reach a desired final moisture content, W_f (g/g), is plotted against the solids loading to the drying bed, D/A (kg/m^2).

A separate curve is plotted for each final moisture content in the range expected. When the maximum moisture content for spadeability has been determined by experiment, the effect of the solids loading on the total drying time can be seen from this set of plots.

In the present investigation, a maximum moisture content of 8,5 g/g dry solids was found to be satisfactory for spadeability, and at a bed loading of $2,6 \text{ kg/m}^2$, drying was achieved in 7 days (see Appendix, section 4.4.6.).

3. DESIGN CALCULATIONS.

The calculations in connection with the proposed treatment process are limited essentially to the criteria necessary for the design of the sedimentation/thickening tank, and the sand filtration and drying beds.

It is intended that dosing of flocculation chemicals occurs en route to the sedimentation tank, immediately after the balancing tank. Flocculation, coagulation, and sedimentation take place in the sedimentation tank.

The plant is to be designed to process an expected effluent flow of $45 \text{ m}^3/\text{d}$. From Fig. 2, the ratio of the upper 95% confidence limit of the distribution of the flow rate, to the mean value, is given by $12,96 \text{ m}^3/\text{d}$ to $9,00 \text{ m}^3/\text{d} = 1,44$.

Therefore, the plant is to be designed to handle $1,5 \times 45 \text{ m}^3/\text{d} = 67,5 \text{ m}^3/\text{day}$, so that peak flows may be accounted for.

3.1. Neutralisation and Flocculation.

Two balancing and neutralisation tanks are required, each having a minimum capacity of $67,5 \text{ m}^3$ (1 day's storage with peak flow).

A suitable size for each tank is $5 \text{ m} \times 5 \text{ m}$ square, and $2,75 \text{ m}$ deep (volumetric capacity = $68,75 \text{ m}^3$). This size allows a freeboard of

$$\frac{(68,75 - 45)}{5 \times 5} = 0,95 \text{ m} ,$$

when filled with the normal 45 m^3 of raw effluent.

The tanks may be constructed in concrete, the internal surfaces being treated with a chemically resistant coating. The reason for this is the relatively long periods of contact with solutions at pH values varying from $\text{pH} = 11$ to $\text{pH} = 3$. Each tank requires a simple low-speed

mechanical stirrer, equipped with chemically resistant paddle blades.

With 2 balancing tanks, the effluent flow feeds into each tank on alternate days. While the one tank collects the flowing waste, the other tank (which has been filled with the previous day's flow) is ready for treatment.

The treatment consists of mixing the collected effluent, and then neutralising it with sulphuric acid to a pH = 6,5. A further dose of acid is added, equal to half the quantity required in the first neutralisation to pH = 6,5.

Slow stirring of the slightly acidic contents of the treated tank is maintained with advantage throughout the continuous dosing for flocculation, which follows. This procedure will ensure uniformity of the composition of the feed to the continuous flocculation, in the event of a tendency for partial separation of components in the balancing/neutralisation tank.

Aluminium sulphate and sodium aluminate solutions are injected into the feed line connecting the balancing tanks with the sedimentation/thickening tank. The two chemical feeders are flow-operated, and the dosing rate for each is preferably predetermined experimentally. However, a constant dose of 500 ppm of aluminium sulphate has been found suitable (section 4.1, Appendix), and if the pH in the balancing tank is controlled, the sodium aluminate dosage will also remain fairly constant from batch to batch. For the relatively small volume of effluent to be treated, rapid mixing after each chemical addition is simply effected by over-and-under baffled pipe sections. The neutralised effluent is normally pumped through the chemical dosers to the sedimentation tank at a rate of $45 \text{ m}^3/\text{d}$. This is equivalent to $45/24 \times 3600 \text{ m}^3/\text{sec}$. The

desired velocity for baffled pipe/rapid mixing should be in the range 0,3 - 1 m/sec, and head losses 0,3 - 1 m⁽¹⁴⁾.

Using 40 mm i.d. piping, the velocity in the baffled pipe is approximately

$$\frac{45 \times 4 \times 10^6}{24 \times 3600 \times \pi \times 40^2} = 0,42 \text{ m/sec,}$$

which should be satisfactory.

3.2.1. Sedimentation Tank Design.

The typical mixed effluent after neutralization and flocculation, has been found to have a suspended solids concentration of $C_o = 2,0 \text{ g/l}$. This is similar to effluent "B" (Table 2B, Appendix; and Fig. 10).

From Fig. 16, when $C_o = 2,00 \text{ g/l}$, constant $K_b = 310$.

However, because data are not available for the tank constant, K_T , the value of K_b found experimentally for floc "B" ($C_o = 2,054 \text{ g/l}$), $K_b = 210$, is acceptable. Thus, the value of K_T assumed is $210/310 = 0,68$. The lower value for K_b will compensate inter alia for wall effects, and lead to a conservative estimate for the mass loading [equation (12)].

3.2.2. Thickening.

Selecting extreme values for the underflow concentration, $C_u = 10 \text{ g/l}$ and $C_u = 30 \text{ g/l}$ respectively, Fig. 17 shows that at $C_o = 2,00 \text{ g/l}$ and flow rate $Q = 45 \text{ m}^3/\text{d}$, the tank diameters required to provide sufficient unit area for the thickener, are respectively $D = 1,98 \text{ m}$ and $D = 3,2 \text{ m}$ for the minimum and maximum expected values of C_u .

To reduce the dewatering load on the sand drainage beds, it is advantageous to obtain an underflow concentration as high as possible.

Therefore selecting a nominal tank diameter $D = 3,0$ m, Fig. 17 shows that at $C_o = 2,00$ g/l, $C_u = 25,8$ g/l. (This latter underflow concentration is close to the maximum experienced in the experimental tests for floc "A", where $C_u = 27,38$ g/l; Table 2A, p A.13).

When $C_u = 25,8$ g/l, $(C_u/C_o - 1) = 11,9$ and the mass loading calculated from equation (12),

$$(ML)^{1,28} = K_b / (C_u/C_o - 1) = 210/11,9 \quad ,$$

yields,

$$\underline{(ML) = 9,42 \text{ kg/m}^2/\text{d}.}$$

At lower values of C_u , say $C_u = 10$ g/l, $(C_u/C_o - 1) = 4,0$ and thus

$$\underline{(ML) = 22,08 \text{ kg/m}^2/\text{d}.}$$

The overflow rate, based on liquid throughput and calculated from equation (13) (Section 2.2.4.),

$$(OR) = \frac{1}{C_o \text{ (UA)}} \times \frac{(C_u - C_o)}{C_u} = \frac{(C_u - C_o)(ML)}{C_u C_o} \quad ,$$

yields for maximum and minimum values of the underflow concentration as follows:-

Overflow rate: (Based on liquid throughput)

$$\text{At } C_u = 25,8 \text{ g/l, } (OR)_c = 4,35 \text{ m}^3/\text{d/m}^2 \text{ ;}$$

$$\text{and at } C_u = 10,0 \text{ g/l, } (OR)_c = 8,83 \text{ m}^3/\text{d/m}^2 .$$

The overflow rate based on the solids loading to the thickener

is given by

$$(OR)_T = \frac{1}{C_o(UA)} = \frac{(ML)}{C_o},$$

(Section 2.2.4.). Thus,

Overflow rate: (Based on solids loading)

$$(OR)_T = \frac{9,42}{2,00} = 4,71 \text{ m}^3/\text{d}/\text{m}^2, \quad (C_u = 25,8 \text{ g/l});$$

$$(OR)_T = \frac{22,08}{2,00} = 11,04 \text{ m}^3/\text{d}/\text{m}^2, \quad (C_u = 10,0 \text{ g/l}).$$

From section 2,2,4., because the overflow rates based on liquid throughput, $(OR)_C$, are lower than those based on the solids loading to the thickener $(OR)_T$, the former (i.e. clarification) controls the surface area required.

Hence the average rise velocities of the overflowing supernatant will vary from 4,35 to 8,83 m/d as the sludge underflow concentration is varied from $C_u = 25,8 \text{ g/l}$ to $C_u = 10,0 \text{ g/l}$.

Using the sedimentation curve for floc "B" (Fig.10), the slope of the initial linear portion of the curve gives the initial constant velocity of the settling floc during zone settling,
 $V = 14,68 \text{ m/d}$.

Because $14,68 \text{ m/d} > 8,83 \text{ m/d}$ or $4,35 \text{ m/d}$, the overflow rise velocity is less than the zone settling velocity, and clarification of the supernatant will be ensured.

The tangent to the settling curve (Fig. 10) at the initial sludge compression point ($t_c = 225$ minutes; $H = 0,58$ m), intersects the vertical axis at $H_1 = 0,94$ m. This is the height the interface would have reached if all the sludge were at the same uniform concentration, C_2 g/l, as that at the interface.

$$\text{For this curve, } C_0 H_0 = 2,054 \times 2,065 = 4,242 \text{ g/10}^{-3} \text{ m}^2.$$

$$\text{But by equation (6), } C_0 H_0 = C_2 H_1,$$

$$\therefore C_2 = C_0 H_0 / H_1 = 4,242 / 0,94 \text{ g/l} = \underline{4,50 \text{ g/l}}.$$

This is the interfacial layer sludge concentration at the estimated commencement of sludge compression.

However, it is desirable to have sufficient retention time, to ensure that the floc is well into the compression zone. Operating at a sludge interface height equal to 15% of the tank depth, i.e. $H_u = 0,30$ m, the underflow concentration is given by

$$C_u = C_0 H_0 / H_u = 4,242 / 0,30 = \underline{14,14 \text{ g/l}}.$$

At the latter expected underflow sludge concentration,

$$(\text{ML})^{1,28} = 210 / (14,14 / 2,00 - 1),$$

$$\text{whence } (\text{ML}) = 14,09 \text{ kg/m}^2/\text{d} \text{ [equation (12)]}.$$

The overflow rate [by equation (13)]

$$= (\text{OR})_c = (14,14 - 2,00) 14,09 / 14,14 \times 2 = \underline{6,05 \text{ m}^3/\text{d/m}^2}.$$

The average overflow rise velocity = $6,05 \text{ m/d} < 14,68 \text{ m/d}$. Thus, clarification of the supernatant will occur in the continuous thickener when the sludge depth is $0,30$ m. This height of sludge at an underflow concentration $C_u = 14,14 \text{ g/l}$ occurs at a time t greater than the value of t_c (section 3.2.3).

For a 3m diameter tank, cross-sectional area =

$$\frac{\pi}{4} (3)^2 = 7,07 \text{ m}^2.$$

$$\therefore \text{overflow rate (OR)} = 6,05 \times 7,07 \text{ m}^3/\text{d} = 42,77 \text{ m}^3/\text{d}.$$

This is the expected overflow rate from the continuous thickener when the influent flow rate is $45 \text{ m}^3/\text{d}$, and the underflow concentration is $14,14 \text{ g/l}$.

3.2.3. Retention Time.

The settling curve for floc "B" (Fig. 10) gives the time for the sludge to initially go into compression as

$$t_c = 225 \text{ min} = \underline{3,75 \text{ hours.}}$$

This is the minimum retention time.

Volume of cylindrical portion of 3m diameter settling tank at a depth of 2m = $\pi/4 \times 3^2 \times 2 = 14,14 \text{ m}^3$.

$\therefore$ average retention time at flow rate $Q = 45 \text{ m}^3/\text{d}$,

$$= 14,14/45 \times 24 = \underline{7,54 \text{ hours.}}$$

When $Q = 67,5 \text{ m}^3/\text{d}$ (i.e. peak flow rate), retention time

$$= 14,14/67,5 \times 24 = \underline{5,03 \text{ hours.}}$$

For both normal and peak flow rates, the tank retention times are greater than that estimated for the sludge to initially go into the consolidated condition ($t_c = 3,75 \text{ hours}$).

Therefore the tank size is acceptable, because the underflow sludge will always be in the compression zone, and thus consolidated.

3.2.4. Mass of Sludge Solids Produced.

The average dry mass of sludge produced is given by equation (14).

$$W_S = \text{Flow Rate} \times C_O = 45 \times 2,00 = 90 \text{ kg/d.}$$

At peak flow rate, $W_S = 67,5 \times 2,00 = 135 \text{ kg/d.}$

This is the dry solids loading of wet sludge to be dewatered on the filtration beds when the initial influent solids concentration to the thickener is 2,00 g/l.

3.2.5. Simulation of Sedimentation Tank Performance in Overload and Underload Conditions.

Using Fig. 25 for rapid computation, the proposed 3 m diameter tank may be tested under conditions of variable flow rates, and variable initial floc concentrations.

It has been calculated (Section 3.2.2) that the sludge concentration must not be less than 4,50 g/l, for the sludge to be in the compression zone at an interface height of 0,58 m; and that the underflow concentration is 14,14 g/l for an underflow height of 0,30 m. The underflow concentrations computed from equation (23) may be read from the chart for a 3 m diameter tank (Fig. 25). Using the sedimentation data for floc "B" Fig. 10, the interface height is estimated from

$$H_u = C_O H_O / C_u = C_O \times 2,065 / C_u.$$

Flow Rate Q	C _o (g/l)	C _u (Fig. 25) (g/l)	Sludge Condition	Underflow Interface Height, H _u (m)
(a) 45 m ³ /d (normal)	1,75	49,0	In Initial Compression	0,074
	2,00	25,5	" " "	0,162
	2,25	13,0	" " "	0,358
	2,50	6,5	" " "	0,794
	2,75	3,3	Not in initial "	1,721
(b) 50 m ³ /d (Overload)	1,75	44,8	In Initial Compression	0,081
	2,00	22,5	" " "	0,184
	2,25	11,5	" " "	0,404
	2,50	5,75	" " "	0,898
	2,75	2,95	Not in initial " and likely to overflow	1,925
(c) 67,5 m ³ /d (Peak Load)	1,75	31,3	In Initial Compression	0,115
	2,00	16,0	" " "	0,258
	2,25	8,05	" " "	0,577
	2,50	4,1	Not In Initial "	1,259
	2,75	2,1	" " " " Will overflow.	2,705
(d) 20 m ³ /d (Underload)	1,75	120,0	In Initial Compression	0,030
	2,00	61,0	" " "	0,068
	2,25	31,3	" " "	0,149
	2,50	15,8	" " "	0,327
	2,75	8,05	" " "	0,706

The above calculations show that the tank will function at all loadings up to peak flow rate ($Q = 67,5 \text{ m}^3/\text{d}$) at initial floc solids concentrations less than $C_o = 2,25 \text{ g/l}$.

At $C_o = 2,50 \text{ g/l}$, the sludge is not in initial compression at peak flow rate ($67,5 \text{ m}^3/\text{d}$). The flow rate is limited to a maximum of

50 m³/d at this influent concentration.

At higher values of C_o ($> 2,25$ g/l), there is a tendency for the underflow sludge interface height to be too high (i.e. $>$ the proposed 0,30 m). However, at $C_o = 2,00$ g/l, the underflow sludge interface height will always be less than the design value of 0,30 m.

The final tank is now designed to have a diameter of 3 m, and a cylindrical height of 3 m (2m, as designed for thickening and clarification; plus 1 m free-board and safety factor, for flow-rates in excess of peak flow-rate = 67,5 m³/d). The tank is fitted with a conical bottom having a vertical height = 0,25 m. This provides a slope of 0,25: 1,5 = 1:6, so that sludge can slide down the bottom and concentrate in the conical bottom for withdrawal.

Volume of vertical cylindrical portion, including freeboard = $\pi \times 3^2 \times 3/4 = 21,21 \text{ m}^3$.

Volume of conical bottom = $\pi r^2 h/3 = \pi \times 1,5^2 \times 0,25/3 = 0,59 \text{ m}^3$.
Therefore retention time ratio (section 3.2.3.) = $21,80/14,14 = 1,54$.

The increased volume of the tank thus increases the retention times by approximately 50%. This will be an additional safety factor to ensure that the underflow sludge is at least in initial compression.

It is proposed that the feed of floc will be introduced into the middle of the tank at a height of 1 m above the bottom of the tank. This will then allow an upward flow through 2,25 m of liquid to a peripheral V-notched weir at the top of the tank. The weir overflow would feed into a peripheral collection channel which drains to the sewer. The inlet velocity should not exceed 0,15 m/sec ⁽¹⁴⁾ (a 75 mm i.d. inlet, without allowing for head losses, will yield an inlet velocity of 0,12 m/sec @ $Q = 45 \text{ m}^3/\text{d}$; and 0,18 m/sec @ $Q = 67,5 \text{ m}^3/\text{d}$. This would appear to be a

satisfactory size for the inlet nozzle).

A slow speed paddle stirrer (4-6 revs/min) supported such that the bottom of the vertical paddle is about 0,25 m above the inlet nozzle, will promote floc growth, sedimentation, and thickening.

Sludge is drawn off from the bottom of the tank and pumped to the sand filtration beds.

The maximum volume of sludge occurs when the underflow concentration is at a minimum. At a peak estimated sludge production of 135 kg/d, when the underflow sludge concentration is $C_u = 10$ g/l, the maximum rate of sludge withdrawal = $135 \times 1000/10 = 13\,500$ l/d, i.e. $13500/24 \times 60 = 9,375$ l/min ($13,5$ m³/d).

A sludge pump selected to handle 10 l/min, may be operated intermittently when the sludge withdrawal rate is less than $13,5$ m³/d (which will occur at underflow sludge production values of less than 135 kg/d).

The velocity of the underflow sludge at the outlet should be minimal in order to avoid disturbing the sludge blanket and its underflow level. Thus it is proposed that the outlet will be covered by a baffle hood supported above the outlet, and the orifice should therefore be as large as is convenient, and preferably 150 mm diameter.

3.3. Sand Filtration.

The underflow effluent from the sedimentation tank (which is now the feed to the sand filters), will normally have a solids concentration in the range of 10 to 20 g/l. Table 2.IV (Section 2.5) shows the negligible effect of the usual range of variation in initial concentration on the filter loading rate in obtaining equal solids removals by constant head filtration. This applies at all sludge depths, from

$h = 10\text{ cm}$, to $h = 30\text{ cm}$. (Fig. 34, where $C_o = 14\text{ g/l}$).

3.3.1. Sand Filtration/Drying Bed Design.

Fig. 34 shows how the filter loading rate may be increased by increasing the depth of the sludge on the sand bed. However, although a higher filtration rate indicates that a smaller area of filter is required, Fig. 51 shows how the drying time of the sludge to spadeability increases exponentially with the sludge solids loading. During drying, the sand filter cannot be used to drain further quantities of wet sludge. Additional drainage area is required for the additional flocculated sludge until the first bed is dried. Thus, the drying time to spadeability is the limiting parameter.

As a guide to the estimation of a suitable filter loading, referring to Fig. 51 (at 65% R.H. and 21°C), the drying time to spadeability at moisture content $8.5\text{ gH}_2\text{O/g dry solids}$ is 7 days at a bed loading of 2.6 kg/m^2 . The moisture content at the beginning of drying was assumed to be 13.5 g/g for the purposes of calculating the data for the curves in Fig. 51.

The feed to the filter from sedimentation has a concentration range $C_o = 10\text{ to }20\text{ g/l}$ (typically, 14 g/l).

The experimental work showed that operating the filter at constant head for 1 day, the average sludge concentration would be $C_f = 25\text{ to }30\text{ g/l}$, (typically, 14 g/l).

From Figs. 35 and 34 respectively, for constant head filtration only, the following data were obtained:-

C_o (g/l)	C_f (g/l)	c (g/ml)	Sludge Depth (cm), at $L = 2,6 \text{ kg/m}^2/\text{d}$.
10	17	0,025	13,3
10	25	0,017	25,5
14	20	0,045	<5
14	25	0,032	9,3
14	30	0,026	12,8
20	29	0,065	<5
20	35	0,046	<5
20	46	0,035	7,8

Thus to obtain a filter sludge with a concentration of about 25 g/l after 1 day's operation, a head of up to 25 cm of sludge is required. This depth of sludge will account for all normal variations in the sand filter feed solids concentration. However, for the typical sludge ($C_o = 14 \text{ g/l}$; $C_f = 25 - 30 \text{ g/l}$) the range of head required is seen to be 9,3 - 12,8 cm from the above data.

The experimental work showed that after stopping constant head feed to a sand filter, drainage is essentially complete after an additional 3 days of falling head drainage (Fig. 43). The average sludge concentration at the end of drainage is approximately in the range 60 - 70 g/l.

Neglecting the evaporation during the first day's constant head feed, dewatering is achieved by both drainage and evaporation during the falling head drainage. Thereafter, drying by evaporation alone dewateres the residual sludge.

Considering the average sludge concentration at the beginning of falling head drainage to be 25 to 30 g/l (approximately equivalent to moisture contents of 39,0 to 32,3 $\text{gH}_2\text{O/g}$ dry solids, respectively); and that at the end of drainage, the moisture content $W_0 = 13,5 \text{ g/g}$ (sludge

concentration = 69 g/l; 93,1% moisture content by mass);

water removed by drainage = (39,0 - 13,5) or (32,3 - 13,5) =

25,5 g H₂O/g dry or 18,8 gH₂O/g dry.

At a bed loading of 2,6 kg/m², water removed by drainage

$$= 25,5 \times 2,6 \text{ or } 18,8 \times 2,6 \text{ kg/m}^2$$

$$= 66,30 \text{ kg/m}^2 \text{ or } 48,88 \text{ kg/m}^2.$$

Additionally, during the 3 day falling head drainage, the evaporation rate of moisture will be entirely in the constant rate period.

By equation (54), the constant rate of evaporation will be at least

$$R_c = 158 e^{-0,00012 \times 2600} = \underline{115,7 \text{ g/m}^2/\text{hr.}}$$

The corresponding "critical" moisture content,

$$W_c = 0,045 (2600)^{0,655} = \underline{7,76 \text{ g/g}} \text{ [equation (53)]}.$$

∴ water lost by evaporation during a 3-day falling head drainage

$$= R_c \times 3 \times 24 = 115,7 \times 72 = 8,33 \text{ kg/m}^2.$$

Hence, total dewatering during falling head drainage over the initial

concentration range 25 to 30 g/l = (66,30 + 8,33) to (48,88 + 8,33) kg/m²

$$= 74,63 \text{ kg/m}^2 \text{ to } 57,21 \text{ kg/m}^2.$$

At a bed loading = 2,6 kg/m², the corresponding dewatering is

$$74,63/2,6 \text{ to } 57,21/2,6 = 28,7 \text{ gH}_2\text{O/g dry solids to } 22,0 \text{ gH}_2\text{O/g dry solids}.$$

The average sludge moisture content at the end of falling head drainage, allowing for simultaneous draining and evaporation, is

$$W_o = (39,0 - 28,7) \text{ or } (32,3 - 22,0) \text{ gH}_2\text{O/g dry solids}$$

$$= 10,3 \text{ g/g or } 10,3 \text{ g/g (i.e. identical, regardless of the}$$

sludge concentration at the beginning of falling head drainage being 25 or 30 g/l).

From this stage onwards, dewatering is accomplished solely by evaporation.

Additional time required to dry to "critical" moisture content

$$W_c = 7,76 \text{ g/g,}$$

$$t_c = \frac{(W_o - W)}{R} \cdot \frac{D}{A} \quad [\text{equation (56)}].$$

$$\text{i.e. } t_c = \frac{(10,30 - 7,76) 2600}{115,7} = 57 \text{ hours (2,4 days).}$$

$W_c = 7,76 < 8,5$; therefore the sludge is spadeable in a total time from commencement of sand filtration = (time for constant head filtration) + (time in falling head drainage) + (time to dry to critical moisture content) = $1 + 3 + 2,4 = 6,4$ days.

After a nominal convenient period of 7 days altogether on the sand bed, drying will continue in the falling drying rate period for a further 0,6 day (= 15 hours).

The final expected moisture content W_f is calculated from equation(52A):

$$2,303 \log \frac{(W_c - E)}{(W_f - E)} = K_f t_f.$$

The falling rate constant, $K_f = 2,78 \times 10^6 (2600)^{-2,67}$ [Equation (55)]

$$\text{i.e. } K_f = 0,0027.$$

Equilibrium moisture content, $E = 0,10 \text{ g/g.}$

∴ by equation 52 (a),

$$2,303 \log \frac{(7,76 - 0,10)}{(W_f - 0,10)} = 0,0027 \times 15,$$

whence $W_f = 5,21 \text{ gH}_2\text{O/g dry solids}$, which is less than $W_f = 8,5 \text{ g/g.}$

Thus, at a bed loading of $2,6 \text{ kg solids/m}^2$, the sludge will be liftable after a total residence of 7 days on the sand filter.

During constant head filtration feed, the head of sludge can be maintained in the range 9 - 13 cm. However, this is not very critical, from Fig. 34, and operation at a nominal 10-15 cm will be satisfactory.

3.3.2. Calculation of Filtration/Drying Bed Areas.

The average mass of sludge solids produced daily is 90 kg; and at peak loading, 135 kg. (Section 3.2.4.).

At a bed solids loading = $2,6 \text{ kg/m}^2$, area of drying beds = $90/2,6 = 34,62 \text{ m}^2/\text{d}$ (average flow rate),

which is approximately = $35 \text{ m}^2/\text{d}$;

or at peak flow rate, $135/2,6 = 51,92 \text{ m}^2/\text{d}$,

which is approximately = $52 \text{ m}^2/\text{d}$.

Each bed is designed to be in operation for 7 days to accomplish drying (section 3.3.1.),

therefore, total bed area required for average flow of effluent,

$Q = 45 \text{ m}^3/\text{d}$,

= $7 \times 35 = 245 \text{ m}^2$;

or at peak effluent flow

$Q = 67,5 \text{ m}^3/\text{d}$,

bed area = $7 \times 52 = 364 \text{ m}^2$.

Allowing for a lower mean temperature of operation such as 10°C , the filter loading being reduced by the increased viscosity, from section 2.5,

$$L_{10^\circ\text{C}} = 0,87 \times L_{20^\circ\text{C}}$$

and the total bed areas required are

$$90/2,6 \times 7/0,87 \text{ m}^2 \text{ and } 135/2,6 \times 7/0,87 \text{ m}^2,$$

for average and peak flow rates respectively,

i.e. 280 m^2 and 420 m^2 , respectively.

At lower mean temperatures, and under wintry conditions, sludge was actually found to yield a moisture content $W = 5,43 \text{ g/g}$ in 6 days. (Appendix, section 4.4.6.). This is a similar moisture content to that which was calculated for drying in 7 days at 65% R.H. and 21°C . (section 3.3.1.). Therefore the reduced rate of evaporation to be expected at the lower temperature of 10°C ., need not be considered in

these calculations.

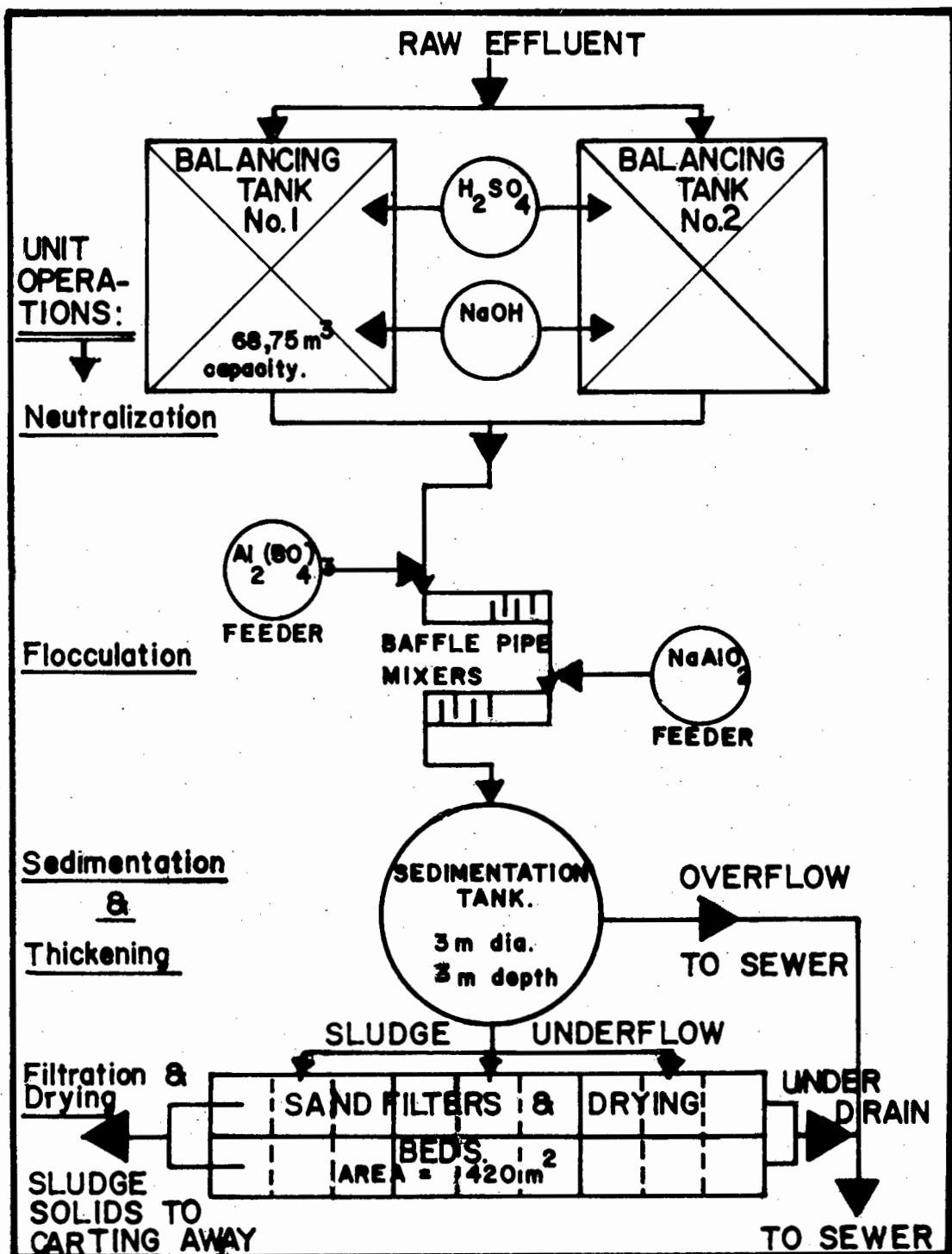
If each filtration/drainage bed is 6 m x 10 m, 7 such beds provide a surface area = $7 \times 60 = 420 \text{ m}^2$. This is the area required to cope with peak flow (as above). For average flow conditions, 5 beds of this size will have an area of $5 \times 6 \times 10 = 300 \text{ m}^2$, which is more than the 280 m^2 required above, and will thus be sufficient.

If each bed is subdivided into 3 by means of wooden baffles, such that the partitioned beds are each 6 m x 3.3 m, constant head loading feeding is more readily obtained. After the first subdivision is filled to a depth of 10 - 15 cm, the flow is diverted to the second, and then the third, subdivision. Thereafter, sludge feed is diverted among the 3 subdivisions to "top up" and thus maintain an approximately constant head filtration throughout the day. The sludge may be distributed in wooden channels traversing the length of the sand beds, and distributed into each subdivision in turn by means of sliding baffles operated manually.

Each bed is proposed to consist of approximately a 5 cm layer of 20 - 30 mesh graded sand supported on approximately 15 - 20 cm of graded stone. The stone rests on open glazed drain pipes, draining to the sewer. A simple supported roof, with open sides, covers the drainage beds. A transparent plastic material is suitable for the roofing. This would promote increased evaporation by radiation. The open sides would allow wind access, and from equation (44a) (Section 2.7.1.), the increased wind velocity would promote the drying rate.

The waste treatment process plant layout, is shown in the flow diagram of Fig. 52.

FIG. 52: WASTE TREATMENT PROCESS FLOWSHEET; TO PROCESS 45 m^3 per day.



A P P E N D I X

4. EXPERIMENTAL PROCEDURES, RESULTS AND CALCULATIONS.

4.1 Flocculation

20 litres of alkaline composite effluent were neutralised with 10% sulphuric acid solution to a pH value = 7,0. 400 ml aliquots of the neutralised solution were placed in a set of six identical 600 ml beakers. Individual pH adjustments were made by the dropwise addition of 10% sulphuric acid solution (to reduce the pH), or the dropwise addition of 10% sodium hydroxide (to increase the pH).

10% m/v solutions of aluminium sulphate hydrate and sodium aluminate were used to dose the effluent aliquots. The aluminium sulphate hydrate was analysed and found to have the composition $Al_2(SO_4)_3 \cdot 14H_2O$. The solid sodium aluminate contained 80% $NaAlO_2$. In each 400 ml aliquot of effluent, the addition of 0,1 ml of the flocculating reagent solution is equivalent to 25 ppm.

In the initial experiments, 2 ml of 10% aluminium sulphate (i.e. 500 ppm) was added to each aliquot, and the test solution rapid-mixed for 1 minute. The pH was adjusted to the value indicated with 10% sodium hydroxide, and slow-mixed (60 rpm) for exactly 20 minutes. Stirring was stopped, and the time noted for the flocculating mass to first commence separating and show a clear supernatant layer below the liquid surface.

The following results were obtained:-

(a) Constant Alum Dose = 500 ppm (20°C.)

pH:	7	6	5	4	3
Time to separate (min):	>120	20	35	90	>120

(b) Constant Alum Dose = 500 ppm (20°C.)

pH:	6,1	6,0	5,9	5,8	5,7
Time to separate (min):	25	20	15	18	27

The tests in (b) are at fractional pH values. The best time for separation of the floc occurred at pH = 5,9. In the next series of tests, incremental dosages of alum were given to aliquots of the neutralised effluent. The pH was adjusted to 5,9 in each case with sodium hydroxide solution, and the time for the floc to separate again noted.

(c) Constant pH = 5,9 (20°C.)

Alum Dose (ppm):	100	200	300	350	400	450	500	550	600	700	800
Time to separate (min):	>120	96	75	72	66	45	32	23	20	15	14

The best floc appeared to form with a minimum of 6 - 700 ppm of alum. However, the addition of a smaller dose of alum to effluent initially acidified to approximately pH = 5,0 was found by a trial and error procedure, to give a better floc when the pH was finally increased with increments of sodium aluminate solution, as follows:-

(d) Constant Alum Dosage = 500 ppm (Effluent at pH = 5,0 at 20°C.)

Sodium Aluminate Dosage (ppm):	300	400	500	600	700	800
Final pH value:	5,52	5,70	5,85	5,95	6,10	6,21
Time to separate (min):	25	20	10	10	14	17

The optimum sodium aluminate dosage for this effluent appeared to be 5 - 600 ppm; the final pH value being 5,85 - 5,95.

Following this procedure, aliquots of 4 different composite bulk effluents were collected and flocculated in a similar manner. In each preparation, the pH was first adjusted to 6,5 with sulphuric acid; an additional one-half of the volume of acid used added to pH $\pm$ 5,0; 500 ppm of aluminium sulphate added, and the solution rapid-mixed; sodium aluminate added to pH 5,85 - 5,95, and again rapid mixed. Finally, the solution was slow-mixed for 20 minutes, and allowed to settle.

The comparative analyses of influent and effluent was as follows:-

Laboratory Batch Flocculation Data

	Effluent "A" (red)	Effluent "B" (white)	Effluent "C" (Brown)	Effluent "D" (Dark Red)
<u>Flocculation</u>				
1. Total Sulphuric Acid added: (equiv. 98%) (ppm)	607	1083	587*	1868
2. Aluminium Sulphate: (Commercial $Al_2(SO_4)_3 \cdot 14H_2O$) (ppm)	475	477	500	483
3. Sodium Aluminate: (Commercial $NaAlO_2$, 80%) (ppm)	300	631	786	888
<u>Analyses</u>				
4. Initial COD (mg/l):	3193	1129	8013	2930
5. Supernatant COD (mg/l):	1600	641	5054	1400
6. % Reduction in COD:	50%	43%	37%	52%
7. Initial pH value:	10,32	10,90	4,20	11,30
8. Supernatant pH value:	5,87	5,90	5,90	5,90

Laboratory Batch Flocculation Data (continued)

	Effluent "A" (red)	Effluent "B"(white)	Effluent "C"(Brown)	Effluent "D" (Dark Red)
9. Initial S.S. C_o (g/l):	1,796	2,054	2,018	0,550
10. Final Sludge S.S. : (18 hr) C_u (g/l)	27,380	11,130	11,038	8,350
11. Concentration ratio: (C_u/C_o)	15,24	5,43	5,47	15,18
12. Initial Chloride (as Cl): (mg/l)	888	1298	659	1280
13. 4 hour OA on Supernatant:(mg/l)	298	336	1185	259
14. Ammoniacal-N on Supernatant: (mg/l)	18,2	11,2	15,4	23,1
15. Total-N on Supernatant: (mg/l)	48,3	30,5	31,3	59,1

*Note:

This effluent had been acidified at the factory. For consistency, 688 ppm of NaOH was added to bring it to pH = 7,0, and then 587 ppm of sulphuric acid added. This was equivalent to treating the original effluent with 209 ppm of sodium hydroxide, from its original pH of 4,20 to a pH value of 5,0.

4.2.1 Procedure for Determination of the Sedimentation Characteristics and Tank Design Parameters.

Batches of floc suspension were prepared from effluents A to D inclusive (section 4.1). Immediately after ceasing the slow stirring (after completion of flocculation), the contents of the 5 l (or 10 l) beakers, in which flocculation was effected, were introduced into the

sedimentation column (Fig. 5). The column was clamped in a vertical position, adjusting the alignment with spirit-levels. An air sparge was introduced at the bottom for a few minutes, in order to obtain an initially uniform suspended solids concentration; and then it was closed. Known volumes of samples of the original suspension were retained for analysis.

Space in the diameter of the column was insufficient to employ a slow-speed stirrer operating at about 4 - 6 revolutions per hour, as recommended by Eckenfelder (2a). The floc was therefore allowed to settle quiescently.

For convenience, all measurements of liquid surface levels and sludge levels were measured from the top of the column itself as a reference point. The sludge level was observed by shining a low-wattage torch behind the column, in order not to heat the liquid, and thus avoid production of convection currents. This procedure was successful, and provided that the column was not sited in a draughty position, the temperature remained essentially constant throughout the test. (Flocculation and sedimentation were effected in the same laboratory, and at the same temperature).

At fixed time intervals, after recording both the height of the sludge interface and liquid surface below the reference point (i.e. from the top of the column), each of the 4 taps was bled to rinse out residual suspension, and 10 ml samples collected in weighed; numbered; and graduated centrifuge tubes. This process was repeated over a period of 120 minutes and longer where necessary, and also after an

overnight settling (18 hours nominally).

Samples of the bottom sludge after 18 hours settling, were collected in 40 ml tared graduated centrifuge tubes. The initial slurry was also collected in a 40 ml tared graduated centrifuge tube. Each of the centrifuge tubes was allowed to stand 18 hours, and the sludge volume recorded. After centrifugation, the sludge volume was again recorded, and in the case of the completely settled 18-hour sludge, these results were used to assist in the estimation of H_{∞} and C_{∞} (Section 2.2.2.).

The supernatant was drained out of each tube, and the contents dried for several hours at 120°C . The concentration of sludge at each depth and at each time interval, was thus estimated.

These results are tabulated in Tables 1A, 1B and 1C respectively, and in the case of the 10ℓ rectangular jar and 1ℓ measuring cylinders, Tables 2 (d) and 2 (e) respectively. (Only initial and final samples were taken in the latter two cases, and no intermediate samples).

For computation of Tables 1A, 1B and 1C, the following procedure was employed:

Each time samples were taken, the liquid surface height and the sludge interface height in the column decreased. In order to correct the sludge height and obtain the correct depths of sampling taps 1, 2, 3, and 4 respectively (Fig. 5) below the liquid surface, the samples at time t minutes are assumed to have been drawn from a depth below the surface, which was the mean value immediately before sampling and immediately after sampling at time t .

TABLE I A

EFFLUENT "A" FLOC SETTLING RATE

(Data For FIG. 8)

Time Min.	Distance of liquid surface from top A cm	Sludge height from top of column B cm	Liquid depth in column C cm	Sludge inter- face from liquid surface D cm	Incremental fall in liquid depth E cm	Top tap to liquid surface F cm	No. 2 tap to liquid surface G cm	No. 3 tap to liquid surface H cm	No. 4 tap to liquid surface J cm	Sludge height from bottom K cm	Sludge volume L %	Corrected sludge height M cm
0	17,00	17,00	196,50	0	0	28,50	74,50	120,00	165,50	196,50	100,00	196,50
6	17,00	32,00	196,50	15,00	4,70	26,15	72,15	117,65	163,15	181,50	92,37	181,50
12	21,70	59,70	191,80	38,00	2,65	22,48	68,48	113,98	159,48	153,80	80,19	157,57
18	24,35	73,85	189,15	49,50	12,40	14,95	60,95	106,45	151,95	139,65	73,83	145,08
24	36,75	102,75	176,75	66,00	5,95	5,78	51,78	97,28	142,78	110,75	62,66	123,13
36	42,70	134,70	170,80	92,00	3,65	0,98	46,98	92,48	137,98	78,80	46,14	90,66
48	46,35	170,60	167,15	124,25	2,65	-	43,83	89,33	134,83	42,90	25,67	50,43
60	49,00	186,90	164,50	137,90	2,50	-	41,75	86,75	132,25	26,60	16,17	31,77
84	51,50	195,50	162,00	144,00	3,10	-	38,45	83,95	129,45	18,00	11,11	21,83
120	54,60	198,60	158,90	144,00	0	-	36,90	82,40	127,90	14,90	9,38	18,43

TABLE I B

EFFLUENT "B" FLOC SETTLING RATE

(Data For FIG. 10)

Time Min	Distance of liquid surface from top A cm	Sludge height from top of column B cm	Liquid depth in column C cm	Sludge interface from liquid surface D cm	Incremental fall in liquid depth E cm	Top tap to liquid surface F cm	No. 2 tap to liquid surface G cm	No. 3 tap to liquid surface H cm	No. 4 tap to liquid surface J cm	Sludge height from bottom K cm	Sludge volume L %	Corrected sludge height M cm
0	7,20	7,20	206,30	0	0	38,30	84,30	129,80	175,30	206,30	100,00	206,30
6	7,20	10,70	206,30	3,50	2,30	37,15	83,15	128,65	174,15	202,80	98,30	202,80
15	9,50	20,30	204,00	10,80	2,50	34,75	80,75	126,25	171,75	193,20	94,71	195,37
20	12,00	31,50	201,50	19,50	2,85	32,07	78,07	123,57	169,07	182,00	90,32	186,34
30	14,85	43,60	198,65	28,75	3,10	29,10	75,10	120,60	166,10	169,90	85,53	176,44
40	17,95	56,20	195,55	38,25	2,20	26,45	72,45	117,95	163,45	157,30	80,44	165,95
60	20,15	76,20	193,35	56,05	2,70	24,00	70,00	115,50	161,00	137,30	71,01	146,50
80	22,85	95,80	190,65	72,95	1,75	21,77	67,77	113,27	158,77	117,70	61,74	127,36
120	24,60	126,00	188,90	101,40	2,05	19,87	65,87	111,37	156,87	87,50	46,32	95,56
180	26,65	151,00	186,85	124,35	0,65	18,52	64,52	110,02	155,52	62,50	33,45	69,01
1020	27,30	188,91	186,20	161,61	0	18,52	64,52	110,02	155,52	24,59	13,21	27,24

TABLE I C

EFFLUENT "C" FLOC SETTLING RATE

(Data For FIG. 11)

Time Min	Distance of liquid surface from top cm	Sludge height from top of column cm	Liquid depth in column cm	Sludge interface from liquid surface cm	Incremental fall in liquid depth cm	Top tap to liquid surface cm	No. 2 tap to liquid surface cm	No. 3 tap to liquid surface cm	No. 4 tap to liquid surface cm	Sludge height from bottom cm	Sludge volume %	Corrected sludge height cm
0	21,00	21,00	192,5	0	0	24,50	70,50	116,00	161,50	192,50	100,00	192,5
15	21,00	23,60	192,5	2,60	3,20	22,90	68,90	114,40	159,90	189,90	98,65	190,0
30	24,20	34,80	189,3	10,60	3,35	19,63	65,63	111,13	156,63	178,70	94,40	181,72
45	27,55	46,70	185,95	19,15	4,10	15,90	61,90	107,40	152,90	166,80	89,70	172,68
60	31,65	59,60	181,85	27,95	3,65	12,03	58,03	103,53	149,03	153,90	84,63	162,91
75	35,30	72,10	178,20	36,80	3,10	8,65	54,65	100,15	145,65	141,40	79,35	152,75
90	38,40	82,50	175,10	44,10	2,50	5,85	51,85	97,35	142,85	131,00	74,81	144,02
110	40,90	96,10	172,60	55,20	3,10	3,05	49,05	94,55	140,05	117,40	68,02	130,94
291	44,00	141,00	169,50	97,00	4,10	-	45,45	90,95	136,45	72,50	42,77	82,34
914	48,10	182,60	165,4	134,50	0	-	43,40	88,90	134,40	30,90	18,60	35,96

Let A = distance measured from liquid surface to the top of the column reference point. (A_1 = initial distance)

B = distance measured from the top of the sludge interface to the top of the column reference point.

The distance from the reference point at the column top to the top tap is 45,5 cm; the inter-tap distances down the column are 46,0; 45,5; and 45,5 cm respectively; the total length of the apparatus = 213,5 cm.

Let C = liquid depth in the column (C_1 = initial depth)

D = sludge interface height in the column

E = incremental fall in level of the surface as a result of sampling at each interval.

F = mean depth of top tap below liquid level

G = mean depth of No. 2 tap below liquid level

H = mean depth of No. 3 tap below liquid level

J = mean depth of bottom tap below liquid level

K = sludge height calculated from the bottom of the column

L = sludge height at each interval as a fraction of total liquid height.

M = corrected sludge height at each interval.

(Units for all the above are centimetres, excepting L, which is dimensionless).

Then at period t , (from $t = 0 \dots \dots \dots t = t_n$),

$$C_t = 213,5 - A_t$$

$$D_t = B_t - A_t$$

$$E_t = A_{t+1} - A_t$$

$$F_t = 45,5 - (A_t + \frac{1}{2} E_t) = 45,5 - \frac{1}{2} (A_t + A_{t+1})$$

$$G_t = F_t + 46,0 = 91,5 - \frac{1}{2} (A_t + A_{t+1})$$

$$H_t = G_t + 45,5 = 137,0 - \frac{1}{2} (A_t + A_{t+1})$$

$$J_t = H_t + 45,5 = 182,5 - \frac{1}{2} (A_t + A_{t+1})$$

$$K_t = C_t - D_t = 213,5 - B_t$$

$$L_t = (K_t / C_t) \times 100 = \frac{213,5 - B_t}{213,5 - A_t}$$

$$M_t = \frac{L_t \times C_t}{100} = \frac{(213,5 - B_t)(213,5 - A_t)}{(213,5 - A_t)}$$

In the form above, calculations were readily effected in a computer. The letters A, B, etc., refer to column headings in Tables 1A, 1B, and 1C respectively.

For each corrected sludge height at time t (column M), the settling curves in Figures 8, 10 and 11 have been plotted. Tables 2 (D) and 2 (E) (batch sedimentation in a 10 l jar and 1 l measuring cylinder respectively) required no correction to sludge height, and the settling curves are plotted directly to give Figures 9 and 12 respectively.

Tables 2A, 2B and 2C were also computed on a computer, as follows:

TABLE 2A

EFFLUENT "A" SUSPENDED SOLIDS
(Data For FIG. 6)

Tap position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge volume (ml) V_C	Mass of sludge (g) W_S	Sludge concentration g/l	18 hr. % sludge volume %	Compacted % sludge vol. %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original % sludge solids
Top	6	1	26,15	9,6	0,60	0,27	0,0124	1,292	6,3	2,8	0,046	0,021	28
2		2	72,15	9,5	0,65	0,34	0,0137	1,442	6,8	3,6	0,040	0,021	20
3		3	117,65	9,6	0,70	0,30	0,0144	1,500	7,3	3,1	0,048	0,021	16
Bottom		4	163,15	9,8	0,65	0,31	0,0117	1,194	6,6	3,2	0,038	0,018	34
Top	12	5	22,48	10,0	0,20	0,10	0,0017	0,170	2,0	1,0	0,017	0,009	91
2		6	68,48	10,0	0,60	0,25	0,0124	1,240	6,0	2,5	0,050	0,021	31
3		7	113,98	9,6	0,65	0,29	0,0125	1,302	6,8	3,0	0,043	0,019	28
Bottom		8	159,48	10,0	0,70	0,34	0,0119	1,190	7,0	3,4	0,035	0,017	34
Top	18	9	14,95	10,0	0,10	0,10	0,0037	0,370	1,0	1,0	0,037	0,037	79
2		10	60,95	9,5	0,55	0,25	0,0167	1,758	5,8	2,6	0,067	0,030	2
3		11	106,45	9,7	0,60	0,28	0,0129	1,330	6,2	2,9	0,046	0,022	26
Bottom		12	151,95	9,4	0,60	0,30	0,0125	1,330	6,4	3,2	0,042	0,021	26
Top	24	13	5,78	9,7	0,05	0,02	0,0004	0,041	0,5	0,2	0,020	0,008	98
2		14	51,78	9,9	0,15	0,10	0,0023	0,232	1,5	1,0	0,023	0,015	87
3		15	97,28	9,6	0,65	0,29	0,0126	1,313	6,8	3,0	0,043	0,019	27
Bottom		16	142,78	9,5	0,60	0,29	0,0132	1,389	6,3	3,1	0,046	0,022	23
Top	36	17	0,98	10,0	0,01	0,01	0,0044	0,440	0,1	0,1	0,440	0,440	76
2		18	46,98	9,4	0,05	0,03	0,0034	0,362	0,5	0,3	0,113	0,068	80
3		19	92,48	9,5	0,40	0,15	0,0054	0,568	4,2	1,6	0,036	0,014	68
Bottom		20	137,98	10,0	0,60	0,26	0,0099	0,990	6,0	2,6	0,038	0,017	45
2	48	21	43,83	10,0	0,05	0,05	0,0013	0,130	0,5	0,5	0,026	0,026	93
3		22	89,33	9,5	0,01	0,01	0,0016	0,168	0,1	0,1	0,160	0,160	91
Bottom		23	134,83	9,5	0,65	0,31	0,0129	1,358	6,8	3,3	0,042	0,019	24

(CONTINUED)

TABLE 2A (CONT.)

Tap Position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge volume (ml) V_C	Mass of sludge (g) W_S	Sludge concentration g/l	18 hr. % sludge volume %	Compacted sludge volume %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original sludge solids. %
2	60	24	41,25	10,0	0,01	0,01	0,0021	0,210	0,1	0,1	0,210	0,210	88
3		25	86,75	10,0	0,01	0,01	0,0055	0,550	0,1	0,1	0,550	0,550	69
Bottom		26	132,25	10,0	0,05	0,03	0,0040	0,400	0,5	0,3	0,133	0,080	78
2	84	27	38,45	8,0	0,01	0,01	0,0006	0,075	0,1	0,1	0,060	0,060	96
3		28	83,95	7,6	0,02	0,02	0,0007	0,092	0,3	0,3	0,035	0,035	95
Bottom		29	129,45	8,6	0,01	0,01	0,0010	0,116	0,1	0,1	0,100	0,100	94

Initial suspended solids concentration = 1,796 g/l

Final sludge suspended solids concentration = 27,380 g/l

TABLE 2 B

EFFLUENT "B" SUSPENDED SOLIDS

Tap position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge vol. (ml) V_C	Mass of sludge (g) W_S	Sludge concentration g/l	18 hr. % sludge volume %	Compacted sludge volume. %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original sludge solids. %
Top	6	1	37,15	10,0	2,0	0,70	0,0188	1,88	20,0	7,00	0,027	0,0094	8,5
2		2	83,15	10,2	2,0	0,70	0,0203	1,99	19,6	6,86	0,029	0,0102	3,1
3		3	128,65	9,9	2,3	0,70	0,0216	2,18	23,2	7,07	0,031	0,0094	-6,2
Bottom		4	174,15	9,9	2,4	0,60	0,0138	1,39	24,2	6,06	0,023	0,0058	32,2
Top	15	5	34,75	10,2	1,9	0,75	0,0213	2,09	18,6	7,35	0,028	0,0112	-1,6
2		6	80,75	10,3	1,9	0,70	0,0183	1,78	18,4	6,80	0,026	0,0096	13,5
3		7	126,25	10,2	2,0	0,70	0,0200	1,96	19,6	6,86	0,029	0,0100	4,6
Bottom		8	171,75	10,3	2,1	0,70	0,0221	2,15	20,4	6,80	0,032	0,0105	-4,5
Top	20	9	32,07	9,8	2,0	0,75	0,0177	1,81	20,4	7,65	0,024	0,0089	12,1
2		10	78,07	10,6	2,0	0,90	0,0236	2,23	18,9	8,49	0,026	0,0118	-8,4
3		11	123,57	10,0	1,8	1,00	0,0238	2,38	18,0	10,00	0,024	0,0132	-15,9
Bottom		12	169,07	10,6	1,9	1,10	0,0198	1,87	17,9	10,38	0,018	0,0104	9,1
Top	30	13	29,10	10,0	1,7	0,70	0,0110	1,10	17,0	7,00	0,016	0,0065	46,5
2		14	75,10	10,0	1,9	1,00	0,0212	2,12	19,0	10,00	0,021	0,0112	-3,2
3		15	120,60	10,0	1,8	0,90	0,0218	2,18	18,0	9,00	0,024	0,0121	-6,1
Bottom		16	166,10	11,0	2,3	1,20	0,0246	2,24	20,9	10,91	0,021	0,0107	-8,8

(CONTINUED)

TABLE 2 B (CONTINUED)

Tap position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge volume (ml) V_C	Mass of sludge (g) M_S	Sludge concentration g/l	18 hr. % sludge volume %	Compacted sludge volume %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original sludge solids. %
2	40	17	72,45	9,8	1,9	0,90	0,0212	2,16	19,4	9,18	0,024	0,0112	-5,3
3		18	117,95	10,0	2,0	1,20	0,0218	2,18	20,0	12,00	0,018	0,0109	-6,1
Bottom		19	163,45	10,9	2,7	1,10	0,0251	2,30	24,8	10,10	0,023	0,0093	-12,1
2	60	20	70,00	10,0	1,8	1,00	0,0186	1,86	18,0	10,00	0,018	0,0103	9,5
3		21	115,50	10,3	2,0	0,90	0,0232	2,25	19,4	8,74	0,026	0,0116	-9,6
Bottom		22	161,00	10,5	2,7	1,20	0,0291	2,77	25,7	11,43	0,024	0,0108	-34,9
3	80	23	113,27	10,0	2,0	1,00	0,0220	2,20	20,0	10,00	0,022	0,0110	-7,1
Bottom		24	158,77	11,8	3,4	1,60	0,0402	3,41	28,8	13,56	0,025	0,0118	-65,8
3	120	25	111,37	10,0	2,2	0,80	0,0216	2,16	22,0	8,00	0,027	0,0098	-5,1
Bottom		26	156,87	11,3	4,1	1,10	0,1384	12,25	36,3	9,73	0,126	0,0340	-496,3
Bottom	180	27	155,20	10,8	4,2	1,70	0,0441	4,08	38,9	15,74	0,026	0,0105	-98,7

Initial suspended solids concentration = 2,054 g/l

Final sludge suspended solids concentration = 11,130 g/l

TABLE 2 C

EFFLUENT "C" SUSPENDED SOLIDS

Tap position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge volume (ml) V_C	Mass of sludge (g) W_S	Sludge concentration g/l	18 hr. % sludge volume %	Compacted sludge volume %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original sludge solids, %
Top	15	1	22,90	9,8	1,8	0,40	0,0231	2,36	18,4	4,10	0,058	0,0128	-17,0
2		2	68,90	9,7	1,9	0,60	0,0228	2,35	19,6	6,20	0,038	0,0120	-17,0
3		3	114,40	9,7	1,8	0,70	0,0259	2,67	18,6	7,20	0,037	0,0144	-32,0
Bottom		4	159,90	10,0	1,7	0,50	0,0157	1,57	17,0	5,00	0,031	0,0092	22,2
Top	30	5	19,63	9,9	1,8	0,65	0,0208	2,10	18,2	6,60	0,032	0,0116	-4,0
2		6	65,63	9,0	1,7	0,65	0,0223	2,48	18,9	7,20	0,034	0,0131	-22,9
3		7	111,13	10,0	2,0	0,65	0,0210	2,10	20,0	6,50	0,032	0,0105	-4,1
Bottom		8	156,63	9,9	2,0	0,70	0,0238	2,40	20,2	7,10	0,034	0,0119	-18,9
Top	45	9	15,90	10,0	0,5	0,20	0,0102	1,02	5,0	2,00	0,051	0,0204	49,5
2		10	61,90	9,8	1,9	0,50	0,0173	1,77	19,4	5,10	0,034	0,0091	12,3
3		11	107,40	9,3	1,9	0,60	0,0174	1,87	20,4	6,50	0,029	0,0092	7,3
Bottom		12	152,90	9,6	2,0	0,60	0,0206	2,15	20,8	6,25	0,034	0,0103	-6,5
Top	60	13	12,03	9,8	0,3	0,06	0,0025	0,26	2,6	0,61	0,042	0,0100	87,1
2		14	58,03	9,8	1,9	0,60	0,0217	2,21	19,4	6,12	0,036	0,0114	-9,6
3		15	103,53	9,8	1,9	0,70	0,0237	2,42	19,4	7,14	0,034	0,0125	-19,9
Bottom		16	149,03	9,8	2,3	0,85	0,0275	2,81	23,5	8,67	0,032	0,0120	-39,2

(CONTINUED)

TABLE 2 C (CONTINUED)

Tap position	Time Min.	Sample No.	Mean tap depth from surface (cm)	Total volume of sample (ml) V_T	18 hr. sludge volume (ml) V_S	Centrifuged sludge volume (ml) V_C	Mass of sludge (g) W_S	Sludge concentration g/l	18 hr. % sludge volume, %	Compacted sludge Vol., %	Concentration of compacted sludge solids g/ml	Concentration of 18 hr. sludge solids g/ml	Sludge solids removal on original sludge solids, %
Top	75	17	8,65	9,8	0,2	0,10	0,0028	0,29	2,0	1,02	0,028	0,0140	85,6
2		18	54,65	9,8	1,8	0,50	0,0109	1,11	18,4	5,10	0,022	0,0061	45,0
3		19	100,15	10,0	2,2	0,70	0,0201	2,01	22,0	7,00	0,029	0,0091	0,4
Bottom		20	145,65	9,7	2,5	0,85	0,0270	2,78	25,8	8,80	0,032	0,0108	-37,8
Top	90	21	5,85	10,0	0,1	0,05	0,0024	0,24	1,0	0,50	0,048	0,0240	88,1
2		22	51,85	9,8	1,8	0,70	0,0229	2,34	18,4	7,10	0,033	0,0127	-16,0
3		23	97,35	9,8	2,2	0,75	0,0287	2,93	22,4	7,70	0,038	0,0130	-44,7
Bottom		24	142,85	9,9	2,6	0,75	0,0254	2,57	26,3	7,60	0,034	0,0098	-27,4
Top	110	25	3,05	9,8	0,2	0,10	0,0024	0,24	2,04	1,00	0,024	0,0120	88,1
2		26	49,05	10,2	0,2	0,10	0,0036	0,35	1,96	0,98	0,036	0,0180	82,7
3		27	94,55	10,8	1,3	0,80	0,0177	1,64	12,04	7,41	0,022	0,0136	18,7
Bottom		28	140,05	10,0	1,6	1,00	0,0264	2,64	16,00	10,00	0,026	0,0165	-30,8
Top	291	29	-0,55	9,8	0,01	0,01	0,0014	0,14	0,10	0,10	0,140	0,1400	92,9
2		30	45,45	8,8	0,01	0,01	0,0017	0,19	0,11	0,11	0,170	0,1700	90,4
3		31	90,95	10,0	0,1	0,05	0,0015	0,15	1,00	0,50	0,030	0,0150	92,6
Bottom		32	136,45	9,7	3,4	0,90	0,0406	4,19	35,10	9,30	0,045	0,0120	-107,1

Initial suspended solids concentration

= 2,018 g/l

Final sludge suspended solids concentration

= 11,038 g/l

TABLE 2 (D) (Data For FIG. 9)

Effluent "A" Floc sedimentation

Curve in 10 l Rectangular Jar

Liquid Depth = 0,29 metres

Time (minutes)	Sludge Interface Height (cm)	Sludge Volume % V/V
0	290,0	100,0
3	275,0	94,9
5	255,0	88,0
10	200,0	69,0
15	137,5	47,4
20	100,0	34,5
26	74,0	25,5
35	63,0	21,8
52	55,0	19,0
62	47,5	16,4
85	45,0	15,5
120	40,0	13,8
220	32,5	11,2
1050	30,0	10,4

TABLE 2 (E) (Data For FIG. 12)

Effluent "D" Floc Sedimentation

Curve in 1 l Cylindrical Jar

Liquid Depth = 1 litre = 0,34 metres

Time (minutes)	Sludge Interface Height (ml)	Sludge Volume % V/V
0	1000	100,0
7	902	90,2
15	730	73,0
28	542	54,2
38	452	45,2
53	360	36,0
64	321	32,1
80	279	27,9
100	245	24,5
120	226	22,6
150	207	20,7
180	199	19,9
210	193	19,3
244	192	19,2
1036	142	14,2

Let A_1 = Initial distance from liquid surface to the
column-top reference point (cm)

A_t = Liquid surface distance to the reference point at
period t (cm)

A_{t+1} = Liquid surface distance to the reference point at
($t+1$) (cm)

B_t = sludge interface distance to the reference point at
period t (cm)

V_t = volume of sample tapped at period t (ml)

V_s = volume of sludge (ml) in the sample after 18 hours
settling; tube in a vertical position.

V_c = volume of sludge in the sample after centrifuging (ml)

W_s = mass of the sludge in the sample of volume V_t after
drying (g)

C_o = initial s.s. concentration (g/l)

(i) Then sludge concentration at period t ,

$$C_t = \frac{W_s}{V_t} \times 1000 \text{ g/l}$$

$$(ii) \text{ 18 hour \% sludge volume} = \frac{V_s}{V_t} \times 100\%$$

$$(iii) \text{ Compacted \% sludge volume} = \frac{V_c}{V_t} \times 100\%$$

$$(iv) \text{ Concentration of compacted sludge} = \frac{W_s}{V_c} \text{ g/l}$$

$$(v) \text{ Concentration of 18 hour sludge} = \frac{W_s}{V_s}$$

(vi) Percentage sludge solids removal on original settled solids concⁿ.

$$= \frac{(C_o - C_t)}{C_o} \times 100\%$$

Only in the case of effluent "A" (Table 2A) was it possible to obtain some resemblance of flocculent settling, and a corresponding set of suspended solids removal curves at different times and depths. However, the resemblance was extremely poor, and was not supported in any of the other experiments, and was thus abandoned. Fig. 6 shows the approximate curves, but in the other experiments no similar curves could be obtained. (The results of (vi) above were essentially negative in the cases of effluents "B" and "C" respectively (Tables 2B and 2C), and clearly indicated thickening). Results (ii), (iii), (iv) and (v) above were employed in the estimation of H_{∞} . (Table 6; Fig 7; and section 2.2.2.).

4.2.2. Calculation of Unit Area And Mass Loading.

Table 3 summarises the initial and final conditions for calculation of the product $C_o H_o$ in each of the different settling apparatus, and for different flocs. The expected variation in the range of concentrations in the effluent is thus covered.

Table 6 lists the plotting points for the plot of $\log (H - H_{\infty})$ vs. time, required for the estimation of the initial sludge compression point at time t_c (equation (11), Fig. 7). The intersection of t_c with the settling curves served as a guide in selecting values of C_u and

TABLE 3

UNIT AREA CALCULATIONS

1.

2.

3.

4.

5.

EFFLUENT FLOC

EFFLUENT:-	"A"	"B"	"C"	"D"	"A" 10 L
Settling Apparatus:-	Column	Column	Column	I-L Jar	Rect. Jar
1. 18-hour settling, S.S. concentration, C_u (mg/l)	27380	11130	11038	8350	27380
2. Initial S.S. Concentration, C_o (mg/l)	1796	2054	2018	550	1796
3. Initial Height of Interface, H_o (metres)	1965	2065	2000	1045 L	0,29
4. $C_o \cdot H_o$ (mg/10 cm ²)	3527	4242	4036	575 mg.	521

+ Sludge height in Litres (1 - Litre = 0,34 metres)

TABLE 3 (a) (Data For FIG. 13(ii))
Effluent "A" - Column, CoHo = 3527 mg./10 cm²

Cu (mg./l) $H_u = \frac{C_2 H_1}{C_u}$ (metres)

10000	0,353
12500	0,282
14000	0,252
15000	0,235
17960	0,197

C ₂ (mg/l)	$H_1 = \frac{CoHo}{C_2}$ (m)	H ₂ (m)	t ₂ (min)	H ₁ - H ₂ (m)	$V_2 = \frac{H_1 - H_2}{t_2}$ (m/min)	V ₂ (m/day)	Hu = 0,353m		Hu = 0,282 m		Hu = 0,252 m		Hu = 0,235 m		Hu = 0,197 m	
							t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+
2000	1,764	0,510	48,0	1,254	0,0262	37,64	54,0	0,01531	56,7	0,01608	57,5	0,01630	58,5	0,01658	60,0	0,01701
2500	1,412	0,460	50,5	0,952	0,0189	27,18	56,0	0,01588	59,8	0,01694	61,0	0,01729	62,2	0,01764	64,5	0,01829
3000	1,175	0,390	55,0	0,785	0,0143	20,53	57,0	0,01617	61,9	0,01754	64,0	0,01813	65,2	0,0185	68,0	0,01928
4000	0,882	0,310	60,5	0,572	0,00946	13,62	56,0	0,01588	63,2	0,01791	67,0	0,01900	68,5	0,01942	73,0	0,02078
5000	0,705	0,290	63,0	0,415	0,00659	9,48	54,0	0,01531	64,8	0,01837	69,0	0,01955	72,0	0,02041	78,5	0,02225
6000	0,589	0,260	69,0	0,328	0,00476	6,85	49,5	0,01403	64,8	0,01837	71,0	0,02010	74,7	0,02116	83,0	0,02353
7500	0,470	0,240	74,0	0,230	0,00311	4,48	38,5	0,01092	60,5	0,01714	71,0	0,02010	75,0	0,02126	89,0	0,02521
10000	0,353	0,200	90,0	0,153	0,00170	2,45	0	0	41,5	0,01176	60,0	0,01701	71,0	0,02010	95,0	0,02692
15000	0,235	0,192	104,0	0,043	0,00041	0,59	-	-	-	-	-	-	-	-	86,5	0,02450

* Consistent Units for UA = m²/(mg/min) × 10⁻³

TABLE 3 (b) (Data For FIG. 13(i))
 Effluent "A" = 10² Rectangular Jar, CoHo = 521 mg/10 cm²

Cu (mg/l)	Hu = $\frac{C_2 H_1}{Cu}$ (metres)
5000	0,104
6000	0,0868
7500	0,0695
10000	0,0521
125000	0,0417

C ₂ (mg/l)	H ₁ = $\frac{CoHo}{C_2}$ (M)	H ₂ (m)	t ₂ (min)	H ₁ -H ₂ (m)	$V_2 = \frac{H_1-H_2}{t_2^2}$ (m/min) ²	V ₂ (m/day)	Hu = 0,104 m		Hu = 0,0868 m		Hu = 0,0695 m		Hu = 0,0521 m		Hu = 0,0417 m	
							t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+	t _u (min)	UA+
2000	0,2604	0,100	20,0	0,1604	0,00803	11,57	19,2	0,0369	21,3	0,0409	23,3	0,0447	25,5	0,0490	26,8	0,0515
2500	0,2084	0,094	20,8	0,1144	0,00551	7,93	19,0	0,0365	22,0	0,0422	25,2	0,0483	28,2	0,0542	30,0	0,0576
3000	0,1736	0,087	22,6	0,0866	0,00383	5,52	18,0	0,0346	22,2	0,0427	26,8	0,0515	31,2	0,0599	34,0	0,0652
4000	0,1302	0,074	26,0	0,0562	0,00216	3,11	12,3	0,0236	20,0	0,0384	28,3	0,0543	35,8	0,0688	40,3	0,0775
5000	0,1042	0,067	32,0	0,0377	0,00118	1,70	0	0	15,5	0,0298	28,5	0,0547	42,6	0,0817	51,0	0,0979
6000	0,0868	0,056	42,0	0,0308	0,000733	1,06	-	-	0	0	24,7	0,0474	47,4	0,0910	60,4	0,1160
7030	0,0740	0,052	51,5	0,0220	0,000427	0,62	-	-	-	-	10,0	0,0192	52,0	0,0998	71,5	0,1370
7500	0,0695	0,050	54,0	0,0195	0,000361	0,57	-	-	-	-	0	0	48,0	0,0922	76,0	0,1460
9300	0,0560	0,048	62,0	0,0080	0,000129	0,19	-	-	-	-	-	-	23,8	0,0467	110,5	0,2120
10000	0,0521	0,038	132,0	0,0141	0,000107	0,15	-	-	-	-	-	-	0	0	91,0	0,1750

+ CONSISTENT UNITS FOR UA = m²(mg/min) x 10⁻³

TABLE 3 (c) (Data For FIG. 14(i))

Effluent "B" - Column, CoHo = 4242 mg/10 cm²

Cu (mg/l)	$H_u = \frac{C_2 H_1}{C_u}$ (metres)
4513	0,940
5438	0,780
6000	0,707

C ₂ (mg/l)	$H_1 = \frac{CoHo}{C_2}$ (m)	H ₂ (m)	t ₂ (min)	H ₁ -H ₂ (m)	$V_2^2 = \frac{H_1 - H_2}{t_2}$ (m/min)	V ₂ (m/day)	H _u = 0,940 m		H _u = 0,780 m		H _u = 0,707 m	
							tu (min)	UA+	tu (min)	UA+	tu (min)	UA+
2250	1885	1061	105,5	0,824	0,00781	11,24	119,0	0,0281	140,5	0,0331	150	0,0354
2500	1697	0,940	124,0	0,757	0,00610	8,79	123,0	0,0290	148,0	0,0348	161	0,0379
3000	1414	0,780	154,0	0,634	0,00412	5,93	115,0	0,0271	154,0	0,0363	173	0,0407
3500	1212	0,667	188,5	0,545	0,00289	4,16	92,0	0,0217	147,5	0,0347	172	0,0404
3750	1131	0,640	198,0	0,491	0,00248	3,57	76,5	0,0180	141,0	0,0332	169	0,0398
4000	1061	0,621	204,3	0,440	0,00215	3,10	66,0	0,0155	131,0	0,0309	165	0,0388

+ CONSISTENT UNITS FOR UA = M²/(mg/min) x 10⁻³

TABLE 3 (d) (Data For FIG. 14(ii))

Effluent "C" - Column, CoHo = 4036 mg/10 cm²

C_u (mg/l)	$H_u = \frac{C_2 H_1}{C_u}$ (metres)
3500	1,153
3716	1,086
4000	1,009

C_2 (mg/l)	H_1 (m)	H_2 (m)	t_2 (min.)	$H_1 - H_2$ (m)	$V_2 = \frac{H_1 - H_2}{t_2}$ (m/min)	V_2 (m/day)	$H_u = 1,153$ m		$H_u = 1,086$ m		$H_u = 1,009$ m	
							t_u (min)	UA+	t_u (min)	UA+	t_u (min)	UA+
2100	1,922	1,280	119,0	0,642	0,00539	7,76	137	0,0339	149,0	0,0349	164,0	0,0407
2200	1,835	1,220	128,0	0,615	0,00480	6,91	140	0,0347	153,0	0,0379	171,0	0,0423
2400	1,682	1,120	154,0	0,562	0,00366	5,27	143	0,0354	160,0	0,0396	181,5	0,0449
2500	1,614	1,100	158,5	0,514	0,00326	4,69	140	0,0347	160,0	0,0396	187,0	0,0463
2600	1,552	1,080	162,0	0,472	0,00291	4,19	137	0,0339	158,0	0,0391	188,0	0,0467
2800	1,441	0,970	204,3	0,471	0,00230	3,31	123	0,0305	153,0	0,0379	187,0	0,0463

+ CONSISTENT UNITS FOR UA = m²/(mg/min) × 10⁻³

TABLE 3 (e) (Data For FIG. 14(iii))

Effluent "D" - 1 Litre Cylindrical Jar, CoHo = 575 mg (= 195,5 mg/10 cm²)

C_u (mg./l)	$H_u = \frac{C_u H_2}{C_2}$ (Litres)
2000	0,288
2250	0,256
2500	0,230

N.B. 1 Litre = 0,34 metres							$H_u = 0,288 \ell$		$H_u = 0,274 \ell$		$H_u = 0,256 \ell$		$H_u = 0,230 \ell$	
C_2	H_1	H_2	t_2	$H_1 - H_2$	$V_2 = \frac{H_1 - H_2}{t_2}$	V_2	t_u	UA+	t_u	UA+	t_u	UA+	t_u	UA+
(mg/l)	(l)	(l)	(min)	(l)	(l/min)	(m/day)	(min)		(min)		(min)		(min)	
750	0,765	0,495	32,5	0,270	0,00831	4,06	57	0,292	58	0,295	61	0,312	64	0,327
1000	0,575	0,360	53,0	0,215	0,00406	1,99	72	0,368	74	0,379	80	0,408	85	0,434
1250	0,460	0,278	79,0	0,182	0,00230	1,13	76	0,585	82	0,419	88	0,450	99	0,505
1500	0,383	0,256	93,0	0,127	0,00137	0,67	70	0,414	79	0,404	89	0,455	110	0,562
1750	0,329	0,221	127,0	0,108	0,000850	0,42	50	0,256	66	0,337	86	0,438	114	0,582
2000	0,288	0,211	142,0	0,077	0,000542	0,27	0	0	27	0,138	60	0,306	106	0,541

+ CONSISTENT UNITS FOR UA ABOVE :- $m^2 / (mg/min) \times 10^{-3}$

calculating H_u , as listed in the unit area calculations of Table 3 (a) - 3 (e) inclusive, respectively. The point of intersection of the tangents to the curve drawn from the H_1 values of Table 3, yielded the co-ordinates H_2 , t_2 , from which the interfacial settling velocities, given by $V = \frac{H_1 - H_2}{t_2}$ were calculated.

Figures 13 and 14 are plots of the unit areas vs. interfacial settling velocities for different values of C_u , the underflow concentration.

Table 4, records the maximum values of UA for each value of C_u from these plots, and the mass loading values at these UA maxima.

Fig. 15 is a log-log plot of the correlation,

$$\left(\frac{C_u}{C_o} - 1\right) = \frac{K_b}{(ML)^n} \quad (\text{equation (12)}).$$

Figure 16 is a plot of $\log K_b$ vs. initial concentration C_o obtained from Fig. 15 and tabulated in Table 5. The slope and intercept gave the relationship:

$$K_b = 3,2 \times 10^9 e^{-8,11 C_o}$$

Thus, from the data of Figs. 15 and 16, Eckenfelder's correlation equation was found to be:

$$\left(\frac{C_u}{C_o} - 1\right) = \frac{3,2 \times 10^9 e^{-8,11 C_o}}{(ML)^{1,28}}$$

Table 7 summarises calculations of tank diameters for different values of C_o and C_u and ML, calculated from Eckenfelder's equation, and at a fixed flow rate of $45m^3/d$.

TABLE 4

A.28

Maximum Unit Areas & Minimum Mass Loading

(Data For FIG. 15, ex Figs. 13 & 14)

Cu (g/l)	(a) FLOC "A" - COLUMN. Co = 1,796 g/l			
	UA ¹ Max (m ² /mg/min) x 10 ⁻³	UA = $\frac{1000}{1440}$ UA ¹ Max. (m ² /Kg/day)	ML = 1/UA (Kg/m ² /day)	$\frac{Cu}{Co} - 1$
17,96	0,0272	0,0189	53,0	9,00
15,00	0,0212	0,0147	68,0	7,35
14,00	0,0205	0,0142	70,5	6,80
12,50	0,0186	0,0129	77,5	5,97
10,00	0,0160	0,0111	90,2	4,57
12,50 10,00 7,50 6,00 5,00	(b) FLOC "A" - 10 l Rectangular Jar. Co = 1,796 g/l			
	0,2120	0,1472	6,8	5,97
	0,0997	0,0692	14,6	4,57
	0,0560	0,0389	25,7	3,17
	0,0440	0,0306	32,7	2,34
	0,0372	0,0258	38,8	1,79
6,00 5,44 4,51	(c) FLOC "B" - COLUMN. Co = 2,054 g/l			
	0,0407	0,0283	35,4	1,92
	0,0363	0,0252	39,7	1,65
	0,0290	0,0201	49,7	1,20
3,50 3,72 4,00	(d) FLOC "C" - COLUMN. Co = 2,018 g/l			
	0,0354	0,0246	40,7	0,74
	0,0396	0,0275	36,4	0,84
	0,0467	0,0324	30,9	0,98
2,00 2,25 2,50 2,10	(e) FLOC "D" - 1 Litre Cylindrical Jar. Co = 0,550 g/l			
	0,582	0,404	2,48	2,64
	0,455	0,316	3,17	3,09
	0,585	0,406	2,47	3,54
	0,419	0,291	3,44	2,82

TABLE 5 (Data For FIG. 16)

INTERCEPT (K_b) AND SLOPE (n) FROM PLOT $\text{LOG} \left(\frac{C_u}{C_o} - 1 \right)$ vs. LOG (ML)
(Ex FIG. 15)

	Co. (g/l)	Slope "n"	Intercept K_b
1. Floc "A" (Column)	1,796	1,32	1900
2. Floc "B" (column)	2,054	1,33	210
3. Floc "C" (column)	2,135	1,16	55
4. Floc "A" (Rect. 10 L jar)	1,796	1,32	225
5. Floc "D" (Meas. 1 L Cyl.)	0,550	1,05	10

Mean Slope "n", (except "D") = 1,28

TABLE 6

A. 30

Estimation of Time of Initial Sludge Compression
(Data For FIG. 7)

Time (min.)	Floc "A" (column) $H_{\infty} = 9 \text{ cm}$		Floc "A" (Rect. Jar) $H_{\infty} = 1,5 \text{ cm}$		Floc "D" (1 $\frac{1}{2}$ Cyl.) $H_{\infty} = 70 \text{ ml.}$	
	Sludge Height H (cm)	H- H_{∞} (cm)	Sludge Height H (cm)	H- H_{∞} (cm)	Sludge Height H (ml)	H- H_{∞} (ml)
0	196,5	187,5	29,0	27,5	1000	930
10	163,0	154,0	20,0	18,5		
20	139,0	130,0	10,0	8,5	625	555
30	108,0	99,0	6,7	5,2		
40	77,0	68,0	5,8	4,3	440	370
50	47,0	38,0	5,6	4,1		
60	32,0	23,0	5,0	3,5	335	265
70	26,0	17,0	4,7	3,2		
80	23,0	14,0	4,6	3,1	275	205
90	20,5	11,5	4,5	3,0		
100	19,0	10,0	4,3	2,8	245	175
120	18,5	9,5	4,1	2,6	220	150
140					210	140
160					204	134
180					199	129
200					195	125
220			3,3	1,8	192	122
240					191	121

(CONTINUED)

TABLE 6 (CONTINUED)

Time (minutes)	Floc "B" (Column) $H_{\infty} = 13 \text{ cm}$		Floc "C" (column) $H_{\infty} = 18 \text{ cm}$	
	Sludge Height H (cm)	$H - H_{\infty}$ (cm)	Sludge Height H (cm)	$H - H_{\infty}$ (cm)
0	206,3	193,3	192,5	174,5
50	156,0	143,0	168,5	150,5
100	110,0	97,0	137,0	119,0
150	80,0	67,0	113,5	95,5
200	63,0	50,0	98,0	80,0
250	55,5	42,5	87,0	69,0
300	52,0	39,0	81,5	63,5
350	50,0	37,0	77,5	59,5
400	48,0	35,0	73,5	55,5

TABLE 7 (FIG. 17)

Design Criteria for Thickener/Clarifier

A. 32

For Flocculated Sludge

(Maximum Flow: $45 \text{ m}^3/\text{day}$)

$C_o = 1,5 \text{ g/l}$ $K_b = 17800$			$C_o = 1,75 \text{ g/l}$ $K_b = 2330$		$C_o = 2,00 \text{ g/l}$ $K_b = 310$		$C_o = 2,25 \text{ g/l}$ $K_b = 38,2$	
C_u g/l	Mass Loading ML (Kg/m ² /day)	Tank Diameter D (Metres)	Mass Loading ML (Kg/m ² / day)	Tank Diameter D (Metres)	Mass Loading ML (Kg/m ² / day)	Tank Diameter D (Metres)	Mass Loading ML (Kg/m ² / day)	Tank Diameter D (Metres)
7,5	708	0,348	169,8	0,768	40,1	1,688	8,91	3,810
10,0	537	0,400	128,8	0,882	29,9	1,956	6,57	4,425
12,5	447	0,439	103,5	0,983	24,2	2,175	5,27	4,950
15,0	380	0,475	88,1	1,070	20,6	2,367	4,44	5,380
20,0	295	0,538	68,3	1,210	15,9	2,690	3,43	6,125
25,0	246	0,593	56,7	1,330	13,1	2,960	2,89	6,675
30,0	208	0,643	48,7	1,434	11,3	3,190	2,42	7,290
Sludge day	67,5 Kg.		78,8 Kg		90,0 Kg		101,3 Kg	

$$\left[\left(\frac{C_u}{C_o} - 1 \right) = \frac{K_b}{(ML)^n} \right]$$

Fig. 17 is a set of plots at a flow rate of $45m^3/d$ (design flow-rate) of $\log C_u$ vs. $\log D$, where D = cylindrical tank diameter.

The slope of the linear plots for different C_o values is found to be $m = 2,25$. The intercepts provided a "tank diameter constant", K_D . Figure 18 is a plot of $\log K_D$ vs. C_o .

From the slope and intercept of this plot,

$$K_D = 1,586 \times 10^5 e^{-5,39 C_o}.$$

The final equation developed, relating all thickener calculations for this particular floc sedimentation, and a flow of $45m^3/d$, is

$$C_u = K_D D^{2,25} = 1,586 \times 10^5 D^{2,25} e^{-5,39 C_o} \text{ [equation (16)]}$$

4.2.3. Concentration Profiles in the Settling Sludge

Plots of the height of the thickening column vs. concentration (as determined by analysis) at separate time intervals, did not provide distinct profile curves. (The concentrations were more or less constant above the compression zone. Data was extracted from Tables 1 and 2). This may be due to the lowest sampling point not being situated at the column bottom and hence the bottom layer concentration could not be obtained analytically. However, calculation of the suspended solids concentrations at particular intervals, and at known depths below the sludge interface level, provided a profile of the concentration gradient in the sludge at different intervals. Tables 8 (a) - (c) inclusive, list these sludge concentration gradients. Figures 19a and 19b are plots of the concentration gradients at various times for flocs 'A' and 'B' respectively.

CONCENTRATION GRADIENTS DURING SLUDGE THICKENING

A.34

TABLE 8 (A) (FIG. 19(a))

FLOC A

Time (minutes)	Corrected Sludge height (cm)	Sampling Depth below corrected interface (cm)	Sludge concentration (g/l)
0	196,50	28,50	1,796
		74,50	1,796
		120,00	1,796
		165,50	1,796
6	181,50	13,50	1,292
		59,50	1,442
		105,00	1,500
		150,50	1,194
12	157,57	32,58	1,240
		79,19	1,302
		125,81	1,190
24	123,13	38,08	1,313
		88,66	1,389
36	90,66	2,65	0,568
		54,99	0,990
48	50,43	13,99	1,358
120	18,43	18,43	27,380

CONCENTRATION GRADIENTS DURING SLUDGE THICKENING

TABLE 8 (B) (FIG. 19(b))

A.35

FLOC B

Time (minutes)	Corrected Sludge height (cm)	Sampling Depth below corrected interface (cm)	Sludge concentration (g/l)
0	206,30	38,30	2,054
		84,30	2,054
		129,80	2,054
		175,30	2,054
6	202,80	34,80	1,880
		80,80	1,990
		126,30	2,180
		171,80	1,390
20	186,34	14,33	1,810
		61,43	2,230
		108,02	2,380
		154,60	1,870
30	176,44	1,97	1,100
		49,74	2,120
		97,00	2,180
		144,25	2,240
40	165,95	37,24	2,160
		85,24	2,180
		133,25	2,300
60	146,50	16,33	1,860
		64,87	2,250
		113,42	2,770
80	127,36	44,58	2,200
		93,82	3,410
120	95,56	12,01	2,160
		61,70	12,250
180	69,01	34,78	4,080
1020	27,24	27,24	11,130

TABLE 8 (C)

FLOC C.

Time (minutes)	Corrected Sludge height (cm)	Sampling Depth below corrected interface (cm)	Sludge concentration (g/l)
0	192,50	24,50	2,018
		70,50	2,018
		116,00	2,018
		161,50	2,018
15	190,00	21,91	2,360
		67,94	2,350
		113,46	2,670
		158,98	1,570
30	181,72	10,88	2,100
		57,66	2,480
		103,93	2,100
		150,20	2,400
45	172,68	46,38	1,770
		93,48	1,870
		140,59	2,150
60	162,91	33,77	2,210
		81,93	2,420
		130,10	2,810
75	152,75	20,96	1,110
		70,11	2,010
		119,26	2,780
90	144,02	9,89	2,340
		59,92	2,930
		109,94	2,570
110	130,94	45,62	1,640
		96,36	2,640
291	82,34	47,13	4,190
914	35,96	35,96	11,038

TABLE 9

DESIGN CRITERIA FOR THICKENERS/CLEARIFIERS FOR FLOCCULATED SLUDGE; VARIABLE MAXIMUM FLOW-RATES

(FIGS. 20 & 21)

 $C_o = 1,50 \text{ g/l}$ $K_b = 17800$ $C_o = 1,75 \text{ g/l}$ $K_b = 2330$

C_u g/l	Mass loading ML ($\text{Kg/m}^2/\text{day}$)	FLOW RATE, m^3/Day					Mass loading ML ($\text{Kg/m}^2/\text{day}$)	FLOW RATE, m^3/Day				
		15	30	45	60	75		15	30	45	60	75
		Tank Diameter; D metres						Tank Diameter, D metres				
7,5	708	0,201	0,285	0,349	0,402	0,450	169,8	0,444	0,628	0,769	0,888	0,992
10,0	537	0,231	0,327	0,400	0,462	0,517	128,8	0,510	0,721	0,883	1,019	1,139
12,5	447	0,253	0,358	0,439	0,506	0,566	103,5	0,568	0,804	0,985	1,137	1,271
15,0	380	0,275	0,388	0,476	0,549	0,614	88,1	0,616	0,871	1,067	1,232	1,378
20,0	295	0,312	0,441	0,540	0,623	0,697	68,3	0,700	0,990	1,212	1,399	1,565
25,0	246	0,341	0,483	0,591	0,683	0,763	56,7	0,768	1,086	1,330	1,536	1,717
30,0	208	0,371	0,525	0,643	0,742	0,830	48,7	0,829	1,172	1,435	1,657	1,853
Sludge per day, Kg:-		22,5	45,0	67,5	90,0	112,5	Kg/d	26,25	52,5	78,75	105,0	131,25

(CONTINUED)

TABLE 9 (CONTINUED)
(FIGS. 20 & 21)

DESIGN CRITERIA FOR THICKENERS/CLARIFIERS FOR FLOCCULATED SLUDGE; VARIABLE MAXIMUM FLOW-RATES

$C_0 = 2,00 \text{ g/l}$ $K_b = 310$

$C_0 = 2,25 \text{ g/l}$ $K_b = 38,2$

C_u g/l	Mass loading ML ($\text{Kg/m}^2/\text{d}$)	FLOW RATE, m^3/Day					Mass loading ML ($\text{kg/m}^2/\text{d}$)	FLOW RATE, m^3/Day				
		15	30	45	60	75		15	30	45	60	75
		Tank Diameter; D metres						Tank Diameter, D metres				
7,5	40,1	0,976	1,381	1,691	1,952	2,183	8,91	2,197	3,107	3,805	4,394	4,912
10,0	29,9	1,131	1,599	1,958	2,261	2,528	6,57	2,634	3,618	4,431	5,117	5,720
12,5	24,2	1,257	1,777	2,177	2,513	2,810	5,27	2,856	4,040	4,947	5,713	6,387
15,0	20,6	1,362	1,926	2,359	2,724	3,046	4,44	3,112	4,401	5,390	6,224	6,958
20,0	15,9	1,550	2,193	2,685	3,101	3,467	3,43	3,541	5,007	6,132	7,081	7,917
25,0	13,1	1,708	2,416	2,958	3,416	3,819	2,89	3,857	5,455	6,681	7,714	8,625
30,0	11,3	1,843	2,607	3,192	3,686	4,121	2,42	4,215	5,961	7,301	8,430	9,425
Sludge per day, Kg:-		30,0	60,0	90,0	120,0	150,0	Kg/d	33,75	67,5	101,25	135,0	168,75

TABLE 10
(FIG. 23)

INTERCEPTS, K_D , AT DIA. = 1 metre (LOG 1 = 0)

(EX FIGURES 20 and 21)

Flow Rate (Q_m /day)	Mean Slope, n.	K_D , INTERCEPT			
		$C_0 = 1,50$ g/l	$C_0 = 1,75$ g/l	$C_0 = 2,00$ g/l	$C_0 = 2,25$ g/l
15	2,136	268	44,2	7,82	1,505
30	2,103	125	20,5	4,20	0,695
45	2,202	84	13,15	2,31	0,411
60	2,222	57,3	9,46	1,65	0,335
75	2,158	43,1	7,55	1,425	0,252

4.2.4. Effect of Flow Rate on Tank Design Parameters

For maximum flow rates, $Q \text{ m}^3/\text{d}$ (other than $45 \text{ m}^3/\text{d}$), the following calculations have been made:-

Table 9 was calculated on a computer (in the same manner as Table 7), and the plots of $\log C_u$ vs $\log D$ drawn for each value of $Q = 15, 30, 45, 60$, and $75 \text{ m}^3/\text{d}$ respectively, as in Figures 20 and 21 respectively. The slopes are constant and independent of Q . Graphical statistical analysis shows these to be normally distributed with mean slope $\bar{x} = 2,165$ and S.D. = $0,05$ (Fig. 22). The slopes of these lines, and their intercepts, are tabulated in Table 10. The plots of the log of the intercepts K_D versus initial sludge concentration, C_o , are shown in Figure 23.

At each value of Q , the slopes were constant and equal to $-1,179$. From the intercept co-ordinates (read at $C_o = 2,00 \text{ g/l}$) the intercepts at $C_o = 0$ were calculated as follows:-

$$\log K_D = -1,179 C_o + \log c' \quad [\text{equation (19)}].$$

$$\text{When } C_o = 2,00, \log c' = \log K_D + 2,3571.$$

Thus the constant c' is found, and these calculations are summarised in Table 11 :-

TABLE 11 Calculation of Intercept c' at different Flow Rates for a 3m Diameter Tank. (Data for Fig. 24).

$Q \text{ (m}^3/\text{d)}$	K_D intercept ($C_o = 2,00 \text{ g/l}$)	Constant c'
15	7,90	1797,6
30	3,98	905,7
45	2,31	525,7
60	1,70	387,0
75	1,345	306,1

Plotting $\log c'$ vs. $\log Q$ (Fig. 24), gave a linear plot with a slope of -1,0914; and intercept of $c' = 2735$, at $Q = 10 \text{ m}^3/\text{day}$.

The equation to this line is

$$\log 2735 = -1,0914 \log 10 + \log c''.$$

$$\text{When } \log c'' = 4,5284, c'' = 33760.$$

$$\text{Thus, } \log c' = \log 33760 - 1,0914 \log Q$$

$$\therefore c' = 33760/Q^{1,0914} \quad [\text{equation (20)}]$$

Substituting the value of c' in equation (20) into equation (19), the latter now evaluates as

$$\log K_D = \log \frac{33760}{Q^{1,0914}} - 1,179 C_o$$

$$\text{i.e. } 2,303 \log K_D = 2,303 \log \frac{33760}{Q^{1,0914}} - 2,303 \times 1,179 C_o$$

$$\text{or } \ln K_D = \ln \frac{33760}{Q^{1,0914}} - 2,7142 C_o$$

$$\therefore K_D = \frac{33760}{Q^{1,0914}} e^{-2,7142 C_o} \quad [\text{equation (21)}].$$

From Figures 20 and 21,

$$\log C_u = 2,165 \log D + \log K_D \quad [\text{equation (22)}].$$

Substituting equation (21) in equation (22) gives

$$\log C_u = 2,165 \log D + \log \left[\frac{33760}{Q^{1,0914}} e^{-2,7142 C_o} \right]$$

$$\therefore C_u = \frac{33760}{Q^{1,0914}} e^{-2,7142 C_o} D^{2,165} \quad [\text{equation (23)}]$$

$$\text{Rearranging, } D = \sqrt[2,165]{\frac{C_u Q^{1,0914} e^{2,7142 C_o}}{33760}}$$

$$\text{or, } D = 8,0975 \times 10^{-3} \sqrt[2,165]{C_u Q^{1,0914} e^{2,7142 C_o}} \quad [\text{equation (24)}]$$

4.2.5. Construction of a Design Chart for Simulation.

With the selected tank diameter of $D = 3,0 \text{ m}$, Table 12 shows the concentration of the underflow sludge for various flow rates, $Q \text{ m}^3/\text{d}$, and for the expected range of initial flow sludge concentrations, $C_o = 2,00 \text{ g/l}$ and $C_o = 2,50 \text{ g/l}$ respectively:-

TABLE 12 Calculation of C_u at Different Flow Rates For a 3m

Diameter Tank (Data for Fig. 25).			
Flow Rate	$Q^{1,0914}$	$C_o = 2,00 \text{ g/l}$ $C_u = \frac{1600,6}{Q^{1,0914}} \text{ g/l}$	$C_o = 2,50 \text{ g/l}$ $C_u = \frac{412,1}{Q^{1,0914}} \text{ g/l}$
15	19,213	83,30	21,45
30	40,935	39,10	10,07
45	63,724	25,12	6,47
60	87,138	18,37	4,73
75	111,30	14,38	3,70

A plot of $\log Q$ vs. $\log C_u$ is shown in Fig. 25. The set of parallel lines led to a family of lines, ranging in initial concentration $C_o = 1,50 \text{ g/l}$ to $C_o = 3,50 \text{ g/l}$.

The effect of varying the flow rate and initial concentration of the floc sludge on the underflow sludge concentration, may now be rapidly evaluated.

4.3. Sand Filtration of Sludge.

Sludges of different concentrations were prepared from the same flocculated composite sample. By settling in the sedimentation column for different times, different average sludge concentrations were obtained in the settled sludge.

To minimise the effect of temperature, experiments were carried out in a constant temperature and constant humidity laboratory (20°C; 65% R.H.). Because of the extended duration of the filtration experiments, all filtrate volumes have been corrected for evaporation loss. The rate of evaporation of the filtrate at 20°C. and 65% Relative Humidity was found to be 0,142 mg/cm²/min. (Fig. 27). This is equivalent to an evaporation loss of 0,14 ml/cm²/10³ min.

The apparatus employed is shown diagrammatically in Fig. 26. It consists of a 5,35 cm i.d. glass filtration tube with 21 cm wall height. A 5 mm No. 2 sintered glass disc is fused in at the bottom. The lower end is tapered to fit into the neck of a tall 250 ml graduated cylinder. The sintered glass disc acts as a support for a shallow sand bed (and also provides a source of resistance to filtration). An 11 mm bed of graded 20/30 mesh washed sand rests on top of the sintered disc, representing the top layer of a sand filter. The filtration tube is clamped in a vertical position above the measuring cylinder.

For constant head filtration, a 300 ml polythene bottle was clamped inverted over the filtration tube. A 5 mm glass tube, in which a 2 mm vent was blown approximately 5 - 10 mm from the tapered outlet end, was fitted through a bung into a length of 15 mm Tygon tubing. The latter fits over the neck of the bottle. In operation, when the bottle was one-half to two-thirds filled with feed sludge, and clamped in an

inclined position as shown in the diagram, sludge fed out of the bottle until the surface of the liquid in the filtration tube just covered the small vent. (The latter was positioned facing upwards). When the liquid level in the filter fell to the lower edge of the vent, sludge fed into the filter once again, until the upper edge of the vent was again covered. During feeding, air bubbles rose into the neck of the bottle and into the reservoir above. In this latter operation, the sludge feed in the reservoir was gently mixed, thus promoting the feeding of a constant concentration of slurry. This apparatus provided a simple automatic feeder which maintained a fairly constant head in the filter. The depth of sludge in the bed was varied by adjusting the height of clamping of the bottle, and the length of the feeder tube.

At the commencement of each filtration, the sand bed and supporting sintered disc were wetted by filtering a portion of filtrate and allowing to drain. At the end of each filtration, after removal of the sludge, the sand bed was back-washed with mains water. It was then rinsed with filtrate before commencing further filtrations.

4.3.1. Constant Head Filtration. Determination of Specific Resistance, r .

The volume v_t of filtrate collected at each time interval t was recorded for different constant heads of sludge. Three initial sludge concentrations were used, viz. $C_0 = 14,573$ g/l; $C_0 = 18,798$ g/l; and $C_0 = 20,930$ g/l. The results are shown in Tables 13 (a) to 13 (c) inclusive. Figures 28, 29 and 30, are the respective plots of the function t/v_t against v_t for each initial concentration.

TABLE 13 (A) (FIG. 28)

CONSTANT HEAD FILTRATION

	Exp. 1 - 1	Exp. 1 - 2	Exp. 1 - 3	Exp. 1 - 4
Initial Solids Concentration, C_0 (g/l)	14,573	14,573	14,573	14,573
Final Solids Concentration, C_f (g/l)	23,365	21,763	25,158	23,395
Mean Sludge Density, $\bar{\rho}$ (g/ml)	1,010	1,010	1,010	1,010
Concentration Term, c (g/ml)	0,03834	0,04367	0,03429	0,03826
Initial Compacted Volume of Sludge, %	19,00	19,00	19,00	19,00
Final Compacted Volume of Sludge, %	25,25	25,00	25,00	27,50

Exp. 1-1 Head = 19,00cm			Exp. 1-2: Head = 15,10 cm.			Exp. 1-3: Head = 12,20 cm			Exp. 1-4: Head = 8,75 cm.		
TIME t(min)	FILTRATE VOL. V_t (ml)	t/ V_t (min/ml)	TIME t(min)	FILTRATE VOL. V_t (ml.)	t/ V_t (min/ml)	TIME t(min)	FILTRATE VOL. V_t (ml)	t/ V_t (min/ml)	TIME t/(min)	FILTRATE VOL. V_t (ml)	t/ V_t (min/ml)
0	0	0	0	0	0	0	0	0	0	0	0
3	9,8	0,306	5	9,4	0,532	3	9,7	0,309	3	6,8	0,441
6	11,8	0,508	10	11,4	0,877	6	11,1	0,541	6	8,0	0,750
9	13,2	0,682	15	13,1	1,145	25	17,2	1,453	9	9,8	0,918
15	16,0	0,938	30	17,1	1,760	30	19,1	1,575	15	11,8	1,271
50	27,5	1,818	45	20,7	2,174	60	25,6	2,344	30	16,1	1,869
70	33,1	2,115	75	26,9	2,788	90	32,2	2,799	75	24,4	3,074
115	45,2	2,544	90	29,4	3,066	125	39,4	3,173	105	30,0	3,506
140	51,3	2,729	165	43,0	3,842	150	44,1	3,405	137	35,2	3,892
170	58,5	2,906	226	52,4	4,317	180	49,6	3,629	165	39,3	4,204
190	63,3	3,002	285	61,5	4,638	220	56,4	3,904	195	44,1	4,422
217	68,4	3,175	347	70,4	4,932	270	65,3	4,135	255	52,0	4,904
295	84,2	3,506	1285	158,5	8,107	310	71,4	4,342	285	56,5	5,049
375	97,9	3,830				330	74,3	4,441	345	63,8	5,412
450	109,7	4,102				355	77,8	4,566	420	72,1	5,829
1425	210,1	6,784				455	91,5	4,973	465	76,9	6,047
1545	219,6	7,037				1450	182,9	7,928	1385	149,2	9,286
1668	229,4	7,271				1475	184,3	8,003	1445	153,3	9,429
1800	238,9	7,536							1505	157,0	9,589

TABLE 13 (B) (FIG. 29)

CONSTANT HEAD FILTRATION											
			Exp. 2 - 1			Exp. 2 - 2			Exp. 2 - 3		
Initial Solids Concentration, C_o (g/l)			18,798			18,798			18,798		
Final Solids Concentration, C_f (g/l)			36,307			31,608			32,485		
Mean Sludge Density, $\bar{\rho}$ (g/ml)			1,014			1,014			1,014		
Concentration Term, c (g/ml)			0,03844			0,04574			0,04400		
Initial Compacted Volume of Sludge, %			25,00			25,00			25,00		
Final Compacted Volume of Sludge, %			37,50			37,50			42,50		
EXP. 2 - 1: Head = 19,19 cm			EXP. 2 - 2: Head = 15,25 cm			EXP. 2 - 3: Head = 12,45 cm			EXP. 2 - 4: Head = 8,40 cm		
TIME	FILTRATE	t/V_t	TIME	FILTRATE	t/V_t	TIME	FILTRATE	t/V_t	TIME	FILTRATE	t/V_t
$t(\text{min})$	VOLUME V_t (ml)	(min/ml)	$t(\text{min})$	VOLUME V_t (ml)	(min/ml)	$t(\text{min})$	VOLUME V_t (ml)	(min/ml)	$t(\text{min})$	VOLUME V_t (ml)	(min/ml)
0	0	0	0	0	0	0	0	0	0	0	0
3	1,8	1,667	3	10,0	0,300	3	6,5	0,462	3	6	0,500
6	3,6	1,667	6	11,8	0,508	6	8,4	0,714	6	8	0,750
9	5,2	1,731	20	18,2	1,102	20	15,0	1,333	40	22,1	1,814
35	15,9	2,208	47	29,0	1,621	45	24,9	1,811	51	25,1	2,032
56	23,1	2,424	116	48,2	2,407	92	38,2	2,412	90	35,4	2,546
96	35,1	2,739	141	54,0	2,611	115	44,2	2,602	130	44,1	2,948
126	43,2	2,917	181	62,3	2,905	165	54,8	3,014	1085	142,9	7,593
188	57,2	3,287	1132	167,4	6,762	1097	155,7	7,046	1151	147,7	7,793
257	71,6	3,589	1202	172,4	6,972	1180	162,1	7,282	1232	153,5	8,029
309	81,5	3,791	1272	177,9	7,150	1265	168,2	7,521	1290	157,5	8,190
379	93,5	4,053	1327	180,1	7,368	1340	174,1	7,697	1375	163,2	8,428
411	98,5	4,175	1353	184,1	7,349	1395	178,3	7,824			
1382	198,2	6,973	1432	187,5	7,639						
1504	208,0	7,233									
1566	212,3	7,378									
1634	217,4	7,516									
1688	221,3	7,629									

TABLE 13 (c) (Fig. 30)

CONSTANT HEAD FILTRATION

			EXP. 3 - 1			EXP. 3 - 2			EXP. 3 - 3		
Initial Solids Concentration, C_o (g/l)			20,930			20,930			20,930		
Final Solids Concentration, C_f (g/l)			32,995			34,643			39,830		
Mean Sludge Density, $\bar{\rho}$ (g/ml)			1,016			1,016			1,016		
Concentration Term, c (g/ml)			0,05634			0,05326			0,04329		
Initial Compacted Volume of Sludge, %			30,00			30,00			30,00		
Final Compacted Volume of Sludge, %			42,50			46,90			45,00		
EXP. 3 - 1: Head = 19,10 cm			EXP. 3 - 2: Head = 15,10 cm			EXP. 3 - 3: Head = 8,35 cm					
TIME t (min)	FILTRATE VOLUME V_t (ml)	t/V_t (min/ml)	TIME t (min)	FILTRATE VOLUME V_t (ml)	t/V_t (min/ml)	TIME t (min)	FILTRATE VOLUME V_t (ml)	t/V_t (min/ml)			
0	0	0	0	0	0	0	0	0			
5	8,0	0,625	5	7,6	0,658	5	8,0	0,625			
10	10,3	0,971	10	10,3	0,971	10	10,0	1,000			
30	18,1	1,662	30	17,1	1,760	45	20,6	2,190			
60	27,7	2,166	95	34,1	2,786	120	37,3	3,217			
90	35,8	2,517	155	46,5	3,337	180	49,5	3,636			
120	42,2	2,844	215	56,6	3,802	240	56,2	4,270			
150	48,5	3,096	275	65,6	4,192	320	66,2	4,834			
180	54,4	3,309	1262	155,2	8,134	1260	141,8	8,889			
210	59,2	3,547	1337	160,7	8,320	1320	145,9	9,050			
275	70,2	3,917	1380	163,0	8,464	1395	150,1	9,294			
330	78,2	4,220				1470	154,5	9,515			
420	89,9	4,674									
532	102,6	5,185									
420	175,9	8,075									
545	184,4	8,381									

From the slope of the lines, $b \text{ min/cm}^6$, the specific resistance was calculated, $r = \frac{2bPA^2}{\mu c}$, [equation (28)] . The filter medium resistance, $R_m = \frac{IPA}{\mu} (\text{min}) (\text{sec})/\text{cm}^2$ (section 2.4.2.), was calculated from the intercept, $I \text{ min/cm}^3$. These results are given in Table 14.

The viscosity of the filtrate, μ , was determined by timing the flow between two fixed marks in a U-tube capillary viscometer t_1 , and comparing with the time required for distilled water, t_2 , at the same constant temperature. The densities of the filtrate, d_1 , and distilled water, d_2 , were determined at the same temperature in an S.G. bottle. The following results were obtained:-

Mean time for flow of filtrate, $t_1 = 86,3 \text{ sec}$.

Mean time for flow of water, $t_2 = 85,1 \text{ sec}$.

Density of filtrate at 20°C ., $d_1 = 1,0044 \text{ g/ml}$.

Density of water at 20°C ., $d_2 = 0,9982 \text{ g/ml}$.

Viscosity of water at 20°C , $\mu' = 0,01005 \text{ poise}$.

∴ Viscosity of filtrate at 20°C ., $\mu = \mu' \times d_1 t_1 / d_2 t_2$

$$\mu = 0,01005 \times \frac{1,0044 \times 86,3}{0,9982 \times 85,1} = 0,01026 \text{ poise}$$

The mean pressure head P was evaluated from the sum of the depth of sludge $h(\text{cm})$ plus the height of the filter media (16 mm), and the average of the sludge density at the beginning and at the end of each filtration respectively. The densities were determined in an S.G. bottle.

$$\text{Then } P = (h + 1,6) \bar{\rho} \text{ g/cm}^2$$

TABLE 14 (FIGS. 31 & 32)

CALCULATION OF SPECIFIC RESISTANCE OF SLUDGE, r , & FILTER MEDIUM RESISTANCE, R_m FROM PLOTS OF t/V_t vs. V_t

Filtration Area A = 22,5 cm ² ; Temperature = 20°C ; Filtrate Viscosity, = 0,01026 cp									
Initial Sludge Concentration C_o (g/l)	Exp. No.	Fig. No.	Head H (cm)	Pressure, $P = \frac{\rho}{2} H$ (g/cm ²)	Concentration Term c(g/ml of filtrate)	Slope. b (min/cm ⁶)	Specific Resistance $r = \frac{2bPA^2}{\mu c}$ (min)(sec)/g	Intercept $l = (\text{min}/\text{cm}^3)$	Filter Medium Resistance $R_m = \frac{lPA}{\mu}$ (min)(sec) cm ²
14,573	1 - 1	28	19,00	19,19	0,03834	0,02610	$1,29 \times 10^6$	1,303	$5,48 \times 10^4$
	1 - 2		15,10	15,25	0,04367	0,03766	$1,30 \times 10^6$	2,212	$7,40 \times 10^4$
	1 - 3		12,20	12,32	0,03429	0,03257	$1,16 \times 10^6$	1,998	$5,40 \times 10^4$
	1 - 4		8,75	8,84	0,03826	0,04584	$1,05 \times 10^6$	2,441	$4,73 \times 10^4$
18,798	2 - 1	29	19,19	19,52	0,03844	0,02674	$1,34 \times 10^6$	1,678	$7,18 \times 10^4$
	2 - 2		15,25	15,46	0,04574	0,03747	$1,25 \times 10^6$	0,535	$1,81 \times 10^4$
	2 - 3		12,45	12,62	0,04400	0,03963	$1,12 \times 10^6$	0,831	$2,30 \times 10^4$
	2 - 4		8,40	8,52	0,03737	0,04772	$1,08 \times 10^6$	0,721	$1,35 \times 10^4$
20,930	3 - 1	30	19,10	19,41	0,05634	0,03915	$1,33 \times 10^6$	1,175	$5,00 \times 10^4$
	3 - 2		15,10	15,34	0,05326	0,04386	$1,25 \times 10^6$	1,306	$4,39 \times 10^4$
	3 - 3		8,35	8,48	0,04329	0,05362	$1,04 \times 10^6$	1,250	$2,33 \times 10^4$

4. 3.2. Determination of Initial Specific Resistance, r_0 , and Sludge Compressibility, s .

A log-log plot of the specific resistance, r , against the pressure head P , calculated in Table 14, is shown in Fig. 31. The equation of this line is given by

$$r = 0,512 \times 10^6 P^{0,325}$$

The intercept at unit pressure yielded the initial specific resistance, $r_0 = 0,512 \times 10^6$ (sec)(min)/g

The slope of the line gave the sludge compressibility,

$$s = 0,325$$

(The units of r_0 are consistent with those of the measured parameters, and are convenient in the subsequent calculations. However, converting minutes to seconds,

$$r_0 = 3,072 \times 10^7 \text{ sec}^2/\text{g}).$$

4.3.3. Estimation of Filter Media Resistance, R_m .

Table 14 shows that there was considerable variation in the filter media resistance calculated from the experimental results, although the order of the resistance was similar in each case. With the same apparatus and filtrate, the filter media resistance is expected to remain constant. However, variations in permeability and compaction of the sludge in the filter will influence the calculated values of R_m .

Fig. 32 is a cumulative probability plot of the experimental results, and showed a normal distribution, with mean resistance

$$\bar{R}_m = 4,31 \times 10^4 \text{ (min)(sec)/cm}^2;$$

and standard deviation = $2,0 \times 10^4 \text{ (min)(sec)/cm}^2$.

4.3.4. Calculation of Rate of Loading in Constant Head Filtration.

Using the experimental data obtained at the end of each filtration from Tables 13(a) to 13(c) inclusive and the calculated values of the filter media resistance R_m in Table 14, Table 15 shows the observed loading rates $\left(L_{OBS} = \frac{V_t^c}{tA}\right)^m$, compared with those calculated on the assumption that R_m is negligible

$$\left(L_{calc} = \sqrt{\frac{2P^{(1-S)}_c}{mr_o t}} \right), \quad \text{equation (36)}$$

and those calculated to include R_m

$$\left(L_{EMP} = L_{calc.} - \frac{1,028 R_m}{r_o t P^s} + 0,0236 \times 10^{-4}, \quad \text{equation (41)}\right)$$

Fig. 33 is a plot of the function $\frac{R_m}{r_o t P^s}$ against the difference

between the loading rates $(L_{calc} - L_{OBS})$.

This plot showed the relationship between the two values, leading to the equation of the straight line:-

$$L_{OBS} = L_{CALC} - \frac{1,028 R_m}{r_o t P^s} + 0,0236 \times 10^{-4} \quad [\text{equation (41)}]$$

Table 16 shows the calculation of the loading rate based on the latter equation, and at a time $t = 1$ day (1440 minutes), and initial concentration $C_o = 14$ g/l. The sludge depth has been varied from 10 to 30 cm. These results were plotted in Fig. 34, which also shows the curves for different

TABLE 15 (FIG. 33)

CALCULATION OF CONSTANT HEAD FILTRATION LOADING RATES

Exp. No.	Time t (min)	Filtrate Volume V _t (ml)	Pressure P (g/cm ²)	Concentration c (g/ml)	Media Resistance R × 10 ⁴ (min)(sec) ² cm ²	Observed Loading Rate L _{OBS} V _t × 10 ⁻⁴ tA g/(min)(cm ²)	Calculated Loading Rate + L _{CALC} × 10 ⁻⁴ g/(min)(cm ²)	Empirical Loading Rate ++ L _{EMP} × 10 ⁻⁴ g/(min)(cm ²)	(L _{CALC} - L _{OBS}) × 10 ⁻⁵ g/(min)(cm ²)	$\frac{R_m}{r_o \cdot t \cdot P^3} \times 10^{-5}$
1 - 1	1800	238,9	19,19	0,03834	5,48	2,26	2,41	2,21	1,52	2,23
1 - 2	1285	158,5	15,25	0,04367	7,40	2,39	2,85	2,40	4,59	4,64
1 - 3	1475	184,3	12,32	0,03429	5,40	1,90	2,20	1,89	2,91	3,16
1 - 4	1505	157,0	8,84	0,03826	4,73	1,77	2,05	1,77	2,79	3,02
2 - 1	1688	221,3	19,52	0,03844	7,18	2,39	2,54	2,24	2,99	3,16
2 - 2	1432	187,5	15,46	0,04574	1,81	2,66	2,78	2,70	1,17	1,02
2 - 3	1395	178,3	12,62	0,04400	2,30	2,50	2,58	2,46	0,79	1,41
2 - 4	1375	163,2	8,52	0,03737	1,35	1,97	2,10	2,02	1,25	0,95
3 - 1	1545	184,4	19,41	0,05634	5,00	2,99	3,20	2,98	2,18	2,41
3 - 2	1380	163,1	15,34	0,05326	4,39	2,80	3,05	2,81	2,49	2,56
3 - 3	1470	154,5	8,48	0,04329	2,33	2,02	2,18	2,04	1,56	1,54

Note: (i) $L_{CALC} = \sqrt{\frac{2P(1-S)}{\mu \cdot r_o \cdot t}} \cdot c$ (equation (36))

(ii) $++ L_{EMP} = L_{CALC} - 1,028 \frac{R_m}{r_o \cdot t \cdot P^3} + 0,0236 \times 10^{-4}$ (equation (41))

TABLE 16 (FIG. 34)

LOADING RATE IN CONSTANT HEAD DRAINAGE OF SLUDGE AT INITIAL SOLIDS CONCENTRATION, $C_0 = 14 \text{ g/l}$ (20°C.)

	CONSTANT DEPTH OF SLUDGE ON BED H (cm)				
	30	25	20	15	10
Loading Rate at $c = 0,020 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	3,12	2,92	2,68	2,40	2,05
Loading Rate at $c = 0,025 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	3,52	3,29	3,03	2,72	2,33
Loading Rate at $c = 0,030 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	3,88	3,63	3,35	3,01	2,59
Loading Rate at $c = 0,035 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	4,22	3,95	3,64	3,28	2,83
Loading Rate at $c = 0,040 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	4,52	4,24	3,91	3,52	3,04
Loading Rate at $c = 0,045 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	4,81	4,51	4,16	3,76	3,25
Loading Rate at $c = 0,050 \text{ g/ml}$, $L \text{ Kg/m}^2$ (day)	5,08	4,77	4,41	3,98	3,46

TABLE 17
CALCULATION OF FINAL SLUDGE SOLIDS CONCENTRATION
AFTER CONSTANT HEAD FILTRATION

(Data For Fig. 35)

	c = 0,065 g/ml	c = 0,060 g/ml	c = 0,055 g/ml	c = 0,050 g/ml	c = 0,045 g/ml	c = 0,040 g/ml	c = 0,035 g/ml	c = 0,030 g/ml	c = 0,025 g/ml
C _o (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)	C _f (g/l)
10	11,81	11,99	12,21	12,48	12,84	13,31	13,97	14,96	16,60
12	14,70	14,98	15,32	15,76	16,33	17,10	18,20	19,92	22,95
14	17,81	18,27	18,74	19,40	20,27	21,47	23,24	26,11	31,58
16	21,18	21,77	22,51	23,46	24,75	26,56	29,33	34,05	43,98
18	24,84	25,65	26,68	28,03	29,88	32,57	36,83	44,60	63,31
20	28,81	29,91	31,32	33,20	35,83	39,76	46,30	59,29	
22	33,15	34,62	36,52	39,10	42,80	48,54	58,64		
24	37,92	39,84	42,38	45,90	51,08	59,47	75,38		
26	43,16	45,67	49,05	53,82	61,08	73,47			

$$C_f = \frac{1}{\frac{1}{C_o} - \frac{1}{\bar{p}c}}, \text{ where } \bar{p} = 1,006 \text{ g/ml}$$

solids removals, c g/ml. The negligible effect of C_o on the loading rate curves, was shown by the plots for $C_o = 10$ g/l and $C_o = 20$ g/l, at $c = 0,03$ g/ml.

At incremental values of the concentration term c , the final filtered sludge solids concentration C_f has been calculated from equation (34),

$$c = \frac{C_o C_f}{1000 \bar{\rho} (C_f - C_o)}$$

whence
$$C_f = \frac{1}{\frac{1}{C_o} - \frac{1}{\bar{\rho} c}} .$$

$\bar{\rho}$ has been taken as the mean sludge density = 1,006 g/ml; and different values of the initial sludge solids concentration C_o taken from $C_o = 10$ g/l to $C_o = 26$ g/l.

The calculations for C_f are contained in Table 17, and plotted in Fig. 35.

4.3.5. Calculation of Rate of Loading Based on Modified equation

$$L = \sqrt{\frac{2P(1-S)}{\mu r_o}} \cdot \frac{c^m}{t^n} \quad [\text{equation (37)}].$$

A plot of the filtrate volume v_t against time t , for values of t between 1000 and 1800 minutes, is shown in Fig. 36. Experiment 1-2 was omitted because it terminated earlier at $t = 1285$ minutes. The data were extracted from the experimental results in Tables 13 (a) to 13 (c) inclusive. (In this range of times, the parabolic curves are essentially linear). At constant time intervals, $t = 1000$ min; 1200 min; 1400 min; and 1600 min, respectively, the filtrate volume v_t was read from each curve in Fig. 36. The initial solids concentration, C_o , was known by analysis. However, the final average solids concentration

C_f at each constant time t , is calculated as follows:-

If V ml = constant volume of sludge in the filter during constant head filtration,

Initial mass of dry solids on filter = $C_o V/1000$ grams.

Since v_t ml of feed at concentration C_o g/l have been fed to the filter at time t to maintain a constant head, mass of dry solids on filter at t = $C_o V/1000 + C_o v_t/1000$ grams.

Average concentrations of solids on filter at t

$$= C_f = C_o (V/1000 + v_t/1000) \times \frac{1000}{V}$$

$$\text{i.e. } C_f = C_o \left(1 + \frac{v_t}{V}\right) \text{ g/l} \quad \text{----- (a).}$$

$$\text{But } c = \frac{C_f C_o}{1000 \bar{\rho} (C_f - C_o)} \quad \text{----- (b).}$$

Substituting (a) in (b),

$$c = \frac{C_o (V + v_t)}{1000 \bar{\rho} v_t} \quad \text{----- (c).}$$

Equation (c) is used to evaluate the solids removal or concentration term, c .

The loading rate is given by $L = \frac{v_t c}{tA}$.

Substituting the value of c obtained in equation (c), in this loading rate equation

$$L = \frac{C_o (V + v_t)}{1000 \bar{\rho} t A} \quad \text{----- (d).}$$

If h = depth of sludge in the filter above the sand surface in cm, then $V = hA$.

Substituting in (d), the loading rate is calculated,

$$L = \frac{C_o (h + v_t/A)}{1000 \bar{\rho} t} \quad \text{----- (e).}$$

Table 18 shows the calculation of the parameters c and L at various

TABLE 18 (FIG. 37 ex FIG. 36)

CALCULATION OF CONCENTRATION c & LOADING RATE L AT CONSTANT TIMES

Initial Conc. C_0	Exp. No	Sludge depth h (cm)	Mean den- sity $\bar{\rho}$ (g/ml)	$t = 1000$ minutes				$t = 1200$ minutes				$t = 1400$ minutes				$t = 1600$ minutes			
				V_t (ml)	C_f (g/l)	c (g/ml)	$L \times 10^{-4}$ (g/cm ² /min.)	V_t (ml)	C_f (g/l)	c (g/ml)	$L \times 10^{-4}$ (g/cm ² /min.)	V_t (ml)	C_f (g/l)	c (g/ml)	$L \times 10^{-4}$ (g/cm ² /min.)	V_t (ml)	C_f (g/l)	c (g/ml)	$L \times 10^{-4}$ (g/cm ² /min.)
$C_0 =$ 14,573 g/l	1-1	17,40	1,010	175,0	21,087	0,04671	3,63	191,5	21,701	0,0439	3,11	207,2	22,286	0,04169	2,74	223,5	22,892	0,03970	2,46
	1-3	10,60	1,010	146,5	23,525	0,03792	2,46	160,5	24,380	0,03587	2,13	174,8	25,254	0,03412	1,89	188,0	26,060	0,03273	1,70
	1-4	7,15	1,010	125,0	25,896	0,03330	1,83	138,0	27,074	0,03125	1,59	150,5	28,206	0,02985	1,42	162,0	29,248	0,02876	1,29
$C_0 =$ 18,793 g/l	2-1	17,59	1,014	168,5	26,801	0,06208	4,64	183,7	27,523	0,05848	3,97	199,0	28,250	0,05541	3,50	214,0	28,962	0,05282	3,14
	2-2	14,65	1,014	156,0	27,694	0,05771	4,00	171,5	28,578	0,05417	3,44	186,0	29,405	0,05139	3,03	201,3	30,278	0,04889	2,73
	2-3	10,85	1,014	147,0	30,117	0,04933	3,22	162,5	31,311	0,04639	2,79	177,5	32,466	0,04404	2,48	193,7	33,713	0,04190	2,25
	2-4	6,40	1,016	136,5	36,617	0,03802	2,30	150,7	38,471	0,03618	2,01	163,7	40,168	0,03478	1,80	178,0	42,034	0,03347	1,65
$C_0 =$ 20,930 g/l	3-1	17,50	1,016	146,5	28,717	0,07597	4,94	160,3	29,451	0,07120	4,22	174,5	30,206	0,06708	3,71	188,0	30,923	0,06375	3,32
	3-2	13,50	1,016	136,5	30,336	0,06644	4,03	150,7	31,314	0,06212	3,46	163,7	32,210	0,05883	3,05	178,0	33,195	0,05575	2,75
	3-3	6,75	1,016	125,0	38,156	0,04563	2,53	138,0	39,948	0,04327	2,21	151,2	41,767	0,04129	1,98	163,0	43,393	0,03979	1,80

constant times. Fig. 37 is a log-log plot of the loading rate L against the concentration term c .

The best fitting line has a slope $m = 0,56$. This is the exponent of the parameter c .

Transposing equation (c) above,

$$v_t = \frac{hA}{1000 \cdot \bar{\rho} \cdot c \cdot C_o - 1} \quad \text{----- (f)}$$

Thus, at constant values of solids removal, c g/ml, the experimental filtrate volume v_t was calculated. Referring to the curves in Fig. 36, the times t were read corresponding to each value of v_t . The loading rate was again calculated from.

$$L = \frac{v_t \cdot c}{tA}$$

The calculated values of t and L at constant c are given in Table 19. Only results which are within the expected loading range, have been tabulated. A log-log plot of the loading rate L against time t is shown in Fig. 37. A series of nearly parallel straight lines resulted. The slopes of these lines were plotted on cumulative probability paper, as shown in Fig. 38. The mean slope, $n = 0,78$ is the exponent of the parameter t .

To obtain an empirical fit of the modified equation ignoring filter media resistance,

$$L = \frac{\sqrt{2P(1-S)}}{\mu r_o} \cdot \frac{c^m}{t^n} \quad \text{[equation (37)]}$$

the expression $L \frac{\sqrt{\mu r_o}}{\sqrt{2P(1-S)}} t^{0,78}$ has been calculated [equation (37B)]

from the observed values of L , P , and t given in Table 15, and Table 20 was constructed.

The equation of each line in Fig. 23 is of the form

$$\log K_D = n C_o + \text{constant} \quad (18)$$

(n = slope of each line, and is constant).

Equation (18) may be written as

$$\log K_D = n C_o + \log c' \quad (19),$$

where $\log c' = \log K_D$ at $C_o = 0$.

Evaluating $\log c'$ (and thus c') from equation (19) at any selected convenient value of C_o (e.g. $C_o = 2,00$ g/l), and for each value of Q , yet a further plot of $\log c'$ vs. $\log Q$ yielded a straight line, as shown in Fig. 24. The equation of this line is given by

$$c' = \text{constant}/Q^P,$$

where the constant is the intercept at $Q = 1$, and the slope is given by $-P$.

For the sludge under discussion in this work, it was found that

$$c' = \frac{\text{constant}}{Q^{1,0914}} = 33760/Q^{1,0914} \quad (20).$$

Substitution of equation (20) in equation (19) yields

$$\log K_D = \log [33760/Q^{1,0914}] - 1,179 C_o,$$

from which

$$K_D = 33760/Q^{1,0914} e^{-2,7142 C_o} \quad (21).$$

The equations to the lines resulting from the plots in Figs. 20 and 21 (i.e. $\log C_u$ versus $\log D$) have the general form,

$$\log C_u = \text{constant} \times \log D + \log K_D \quad (22),$$

where the constant is the value of the slope.

Substituting the value of K_D from equation (21) in (22), the latter equation becomes

$$\log C_u = \text{constant} \times \log D + \log [33760/Q^{1,0914} e^{-2,7142 C_o}].$$

Table 20: Empirical Loading Rate Equation (R_m negligible)

(Data for Fig. 39)

Exp. No.	Time t (min)	Concentration c (g/ml)	Pressure P (g/cm ²)	Observed Loading Rate L_{OBS} (g/cm ² /min) $\times 10^{-4}$	$L \sqrt{\frac{\mu r_o}{2P(1-S)}} t^{0.78}$
1-1	1800	0,03834	19,19	2,26	1,496
1-2	1285	0,04367	15,25	2,39	1,301
1-3	1475	0,03429	12,32	1,90	1,239
1-4	1505	0,03826	8,84	1,77	1,311
2-1	1688	0,03844	19,52	2,39	1,386
2-2	1432	0,04574	15,46	2,66	1,567
2-3	1395	0,04400	12,62	2,50	1,544
2-4	1375	0,03737	8,52	1,97	1,375
3-1	1545	0,05634	19,41	2,99	1,727
3-2	1380	0,05326	15,34	2,80	1,605
3-3	1476	0,04329	8,48	2,02	1,488

Fig. 39 is a log-log plot of the function

$$f(c) = L \sqrt{\frac{\mu r_o}{2P(1-S)}} t^{0.78}$$

against the concentration term c. Fitting the best line to the scattered results by least squares, the equation of the line was evaluated:-

$$f(c) = 8,55 c^{0.56} \text{ (coefficient of correlation} = 0,83\text{)}.$$

From these calculations, the empirical fit to the data is given by

$$L = 8,55 \sqrt{\frac{2P(1-S)}{\mu r_o}} \times \frac{c^{0,56}}{t^{0,78}} \quad [\text{equation (37c)}]$$

Using this equation, Table 21 shows the loading rates calculated empirically, and compared with the observed loading rates:-

TABLE 21

Comparison of Loading Rates Calculated Empirically, (R_m negligible)
and Observed Values.

Exp. No.	$L_{OBS} \times 10^{-4}$ (g/cm ² /min)	$L_{EMP} \times 10^{-4}$ (g/cm ² /min)
1-1	2,26	2,08
1-2	2,39	2,73
1-3	1,90	1,99
1-4	1,77	1,86
2-1	2,24	2,23
2-2	2,66	2,55
2-3	2,50	2,41
2-4	1,97	1,97
3-1	2,99	2,95
3-2	2,80	2,89
3-3	2,02	2,00

The agreement was not as close as that obtained by using empirical equation (41), and reported in Table 15.

4.3.6. Falling Head Drainage

Measured volumes of sludges at different initial solids concentrations, C_o , were applied to the sand filter. At different time intervals t , the volume of filtrate collected, v_t , and the height of sludge above the sand layer were measured. These results are shown in Table 22. Values of t/v_t plotted against v_t are shown in Fig. 40. It is seen that the data are not as linear as in the case of constant head filtration (Figs. 28, 29, and 30). Drainage continued for approximately 2 days before slowing down to an extremely low rate. However, one experiment (No. 8) was allowed to proceed to its ultimate over a period of 7 days.

Because of the continuous compaction of the sludge during falling head drainage, homogenising of the final sludge for analytical sampling was unsatisfactory. Thus, the final average solids content, C_f , was calculated as follows, rather than determined analytically:-

If V ml = volume of sludge originally applied to the filter at a solids concentration C_o g/l,

Mass of dry solids on filter = $C_o V/1000$ g .

Since no solids have been added to or taken from the filter, concentration of solids at time $t = C_f = \frac{C_o V}{(V-v_t)}$ g/l ----- (a)

Solids "removal" concentration term

$$c = \frac{C_f C_o}{1000 \bar{p} (C_f - C_o)} \text{ g/ml .}$$

Substituting for C_f from equation (a),

$$c = \frac{C_o V}{1000 \bar{p} v_t} \text{ ----- (b)}$$

TABLE 22
FALLING HEAD DRAINAGE AT 20°C.

(FIGURES 40, 42, 43 & 44)

				Exp. 4				Exp. 5				Exp. 6				Exp. 7				Exp. 8							
Initial Solids Concentration, C_0 (g/l)								11,191				12,630				14,286				22,380				23,110			
Final Solids Concentration, C_f (g/l)								56,94				32,740				56,67				64,67				55,67			
Mean Sludge Density, (g/ml), $\bar{\rho}$								1,015				1,012				1,016				1,018				1,017			
Concentration Term, c (g/ml)								0,01372				0,02032				0,01880				0,03362				0,03885			
Initial Compacted Volume of Sludge, %								16,00				16,25				17,70				25,25				25,00			
Final Compacted Volume of Sludge, % (after drainage ceases)								69,20				65,00				57,70				60,00				65,00			
EXPERIMENT 4				EXPERIMENT 5				EXPERIMENT 6				EXPERIMENT 7				EXPERIMENT 8											
TIME	FIL- TRATE	SLUDGE HEAD	t/V _t	TIME	FIL- TRATE	SLUDGE HEAD	t/V _t	TIME	FIL- TRATE	SLUDGE HEAD	t/V _t	TIME	FIL- TRATE	SLUDGE HEAD	t/V _t	TIME	FIL- TRATE	SLUDGE HEAD	t/V _t								
t(min)	VOLUME V _t (ml)	H(cm)	(min/ ml)	t(min)	VOLUME V _t (ml)	H(cm)	(min/ ml)	t(min)	VOLUME V _t (ml)	H(cm)	(min/ ml)	t(min)	VOLUME V _t (ml)	H(cm)	(min/ ml)	t(min)	VOLUME V _t (ml)	H(cm)	(min/ml)								
0	-	19,66	-	0	-	19,68	-	0	-	19,89	-	0	-	19,36	-	0	-	20,20	-								
5	16,8	18,90	0,298	6	12,0	19,08	0,500	6	18,7	18,99	0,321	15	15,5	18,66	0,968												
10	20,0	18,74	0,500	21	22,0	18,61	0,955	9	20,0	18,87	0,450	30	20,3	18,39	1,478	120	28,0		4,286								
50	38,4	17,71	1,304	36	30,0	18,34	1,200	12	21,8	18,78	0,550	45	26,1	18,14	1,724	250	44,6		5,605								
105	58,2	16,25	1,804	51	37,3	17,96	1,367	25	26,8	18,57	0,933	60	30,1	17,97	1,993	420	62,8		6,686								
170	78,1	15,77	2,177	75	46,9	17,41	1,599	40	32,1	18,36	1,248	75	34,3	17,84	2,187	1380	123,5		11,174								
235	94,7	14,99	2,483	100	55,4	17,02	1,807	65	39,8	17,93	1,633	90	38,2	17,60	2,359	1440	126,5		11,388								
300	99,5	14,26	3,017	175	76,4	16,03	2,292	105	51,0	17,38	2,059	120	45,2	17,26	2,655	1800	143,0		12,587								
1287	212,5	9,08	6,056	245	81,0	15,36	3,025	156	63,6	16,69	2,455	150	51,5	16,96	2,915	2893	183,0		15,809								
1368	218,2	8,75	6,271	302	102,2	14,86	2,955	265	85,4	15,60	3,103	195	59,9	16,54	3,255	3180	192,4		16,528								
1440	223,3	8,54	6,450	345	112,8	14,58	3,060	343	99,0	14,94	3,487	300	76,7	15,81	3,911	3360	196,4		17,198								
1490	229,0	8,30	6,507	437	122,8	14,05	3,559	445	114,2	14,27	3,897	377	87,4	15,33	4,314	4260	219,9		19,377								
1568	237,9	7,80	6,591	1420	215,1	9,59	6,602	1407	202,7	10,30	6,941	420	92,7	15,08	4,533	4568	228,1		20,026								
1650	247,1	7,39	6,677	1523	221,6	9,28	6,873	1515	207,9	9,87	7,289	1457	174,6	11,33	8,345	4740	232,2		20,413								
1761	258,0	6,96	6,827	1565	226,6	9,08	6,908	1575	211,0	9,80	7,466	1500	177,1	11,24	8,472	5800	252,7		22,952								
2730	302,1	4,87	9,038	1610	232,8	8,72	6,916	1775	222,3	9,30	7,987	1560	180,4	11,18	8,647	8580	286,1		29,990								
								1870	227,1	9,00	8,234	1622	183,7	11,03	8,830	8640	286,3		30,178								
								2846	279,3	6,70	10,192	2582	245,2	7,79	10,530	9060	287,2		31,546								
								2985	284,2	6,47	10,505					10080	288,8	4,91	34,903								

$$C_f = C_0 V / V - v_t$$

Equations (a) and (b) have been used in Table 22 for the calculation of the variables C_f and c respectively.

From the slopes of the plots in Fig. 40, the apparent specific resistance of each sludge was calculated. Pressure head terms were calculated for both the arithmetic-mean depth of the sludge, and the log-mean depth of sludge.

The latter is given by

$$\bar{H}_{LM} = \frac{(H_o - H_f)}{2,303 \log \frac{H_o}{H_f}} \quad [\text{equation (42)}]$$

The specific resistance r was calculated at both the arithmetic-mean pressure, and the log-mean pressure (Exp. 8 was calculated at $t = 4740$ minutes, where the slope of the plot t/v_t against v_t in Fig. 40, changed).

The results are given in Table 23.

Fig. 41 is a log-log plot of the specific resistance against pressure. The data were seen to be scattered. Least-squares best-fit lines drawn for the mean-head calculations and the log-mean head, showed hardly any correlation in the former case (correlation coefficient = 0,03), and only 0,32 coefficient of correlation in the latter case. The results for the arithmetic-mean head were thus abandoned. The intercept and slope of the log-mean plot yielded the initial specific resistance, $r_o = 0,47 \times 10^6$, and compressibility $s = 0,623$, respectively.

These results are shown in Table 23.

Table 24 compares the loading rate observed ($L = \frac{v_t c}{tA}$), with that calculated at the log-mean pressure (ignoring filter media resistance). The experimentally determined initial sludge resistance and compressibility have been used ($r_o = 0,47 \times 10^6$; $s = 0,623$).

TABLE 23

CALCULATION OF SPECIFIC RESISTANCE IN FALLING HEAD DRAINAGE
(FIG. 41)

Exp. No.	Slope of t/V_t vs V_t Plot, b (min/cm ⁶)	Concentration Term, c (g/ml)	Mean Sludge Density, $\bar{\rho}$ (g/ml)	Initial Sludge Depth, H_0 (cm)	Final Sludge Depth, H_f (cm)	Mean Sludge Depth H (cm)	Log Mean Depth, H_{LM} (cm)	Mean Pressure P (g/cm ²)	Log Mean Pressure, P_{LM} (g/cm ²)	Specific Resist $r(on P) \times 10^6$ (sec)(min)/g	Specific Resist $r(on P_{LM}) \times 10^6$ (sec)(min)/g.
4	0,0289	0,01372	1,015	19,66	4,87	12,27	10,60	12,45	10,76	2,59	2,24
5	0,0310	0,02032	1,012	19,68	8,72	14,23	13,46	14,40	13,62	2,17	2,05
6	0,0365	0,01880	1,016	19,89	6,47	14,18	11,95	14,41	12,14	2,76	2,33
7	0,0465	0,03362	1,018	19,36	7,79	13,58	12,70	13,82	12,93	1,89	1,76
8	0,0879	0,03885	1,017	20,20	9,35	14,78	14,08	15,03	14,32	3,36	3,20

TABLE 24

Comparison of Observed and Calculated Loading Rates

During Falling Head Drainage

Exp. No.	Time t (min)	Concentration c (g/ml)	Log-Mean Pressure (g/cm ²)	$L_{OBS} = \frac{v_t \cdot c}{tA}$ (g/min/cm ² x 10 ⁻⁵)	$L_{CALC} =$	L_{OBS}
					$\frac{2P(1-0,623)c}{\mu \cdot 0,47 \times 10^6 t}$	L_{CALC}
4	2730	0,01372	10,76	6,75	7,18	0,94
5	1610	0,02032	13,62	13,06	11,91	1,10
6	2985	0,01880	12,14	7,96	8,22	0,97
7	2582	0,03362	12,93	14,19	11,98	1,18
8	4740	0,03885	14,32	8,46	9,67	0,87

The agreement between calculated loadings and observed values is not as close as in the case of constant head filtration (Table 15).

4.3.7. Falling Head Drainage Versus Constant Head Filtration.

The effect of feeding the same sludge to constant head or falling head filters, is shown in Table 25. At the same solids removal (c = 0,035 g/ml) and at the same time; mean pressure; and depth of sludge (log-mean values in the case of the falling head filter), the loading rates have been calculated using equation (41) (Section 4.3.4) in the case of constant head filtration, and from

$$L = \sqrt{\frac{2P(1-S)c}{\mu r_o t}}$$

in the case of falling head drainage:-

TABLE 25

Comparison of Falling Head Drainage and Constant Head

Drainage Loading Rates.. (Part data for Fig. 34)

Sludge Depth h(cm)	Head H(cm)	Pressure ($\rho = 1,014$ g/ml) $P(g/cm^2)$	Loading Rate $L_{const.head}$ ($kg/m^2/d$)	Loading Rate, $L_{fall.head}$ ($kg/m^2/d$)	$\frac{L_{constant head}}{L_{falling head}}$
30	31,6	32,04	4,22	2,64	1,60
25	26,6	26,97	3,95	2,53	1,56
20	21,6	21,90	3,64	2,41	1,51
15	16,6	16,83	3,28	2,26	1,45
10	11,6	11,76	2,83	2,05	1,38
(c = 0,035 g/ml; t = 1440 min.)					

The plot of loading rate against sludge depth for falling head drainage at $c = 0,035$ g/ml, is included in Fig. 34. Under similar conditions to produce similar dewatering of sludge, constant head filtration has a greater loading rate on a given filter bed than falling head drainage.

4.3.8 Empirical Relationship between t/v_t and v_t in Falling

Head Drainage

Figs. 42 and 43 are log-log plots of the function t/v_t against v_t for falling head drainage. The data were taken from Table 22. Two distinct lines were obtained in each case, the slope changing within approximately 1 day of the commencement of drainage. In the case of prolonged drainage (Exp. 8, Fig. 43), there was a further distinct change in the slope after 5 days.

Reading the co-ordinates of the intersection (t/v_t , v_t) of the plots, the time at which the change occurred was calculated from $t/v_t \times v_t$. The slopes and intercepts of the two lines are tabulated in Table 26:-

TABLE 26
Falling Head Drainage (Figs. 42 and 43)

Exp. No.	Initial Solids Conc., C_o (g/l)	t/v_t at Inter-section (min/cm ³)	v_t at Inter-section (ml)	Drainage Time at Inter-section t_i (min)	SLOPE		INTERCEPT	
					< t_i	> t_i	< t_i	> t_i
4	11,19	2,5	97	243	0,66	1,12	0,121	0,015
5	12,63	2,9	104	302	0,72	1,12	0,099	0,015
6	14,29	4,4	135	594	0,78	1,19	0,096	0,012
7	22,38	4,9	106	519	0,75	1,05	0,152	0,036
8	23,11	13,0	152	1976	0,77	1,10	0,276	0,052
Average Slope:-					0,75	1,12		

Table 26 shows that the slopes of the plots are similar in both phases of the drainage. However, the time at which the change occurs is a complex function of both the initial solids concentration, and the change in the permeability characteristics of the compacting sludge.

The intercept is similarly a complex function of the initial solids concentration, and permeability of the sludge.

Each equation is in the form of functions given respectively by

$$\begin{aligned}
 & t/v = k_1 v^{0,75}, \text{ whence } t = k_1 v^{1,75} \\
 \text{and } & t/v = k_2 v^{1,12}, \text{ whence } t = k_2 v^{1,12}
 \end{aligned}
 \left. \vphantom{\begin{aligned} t/v = k_1 v^{0,75} \\ t/v = k_2 v^{1,12} \end{aligned}} \right\} \text{[equation (43) section 2.6]}$$

Both of these equations are of little use in calculating the loading rate, because the time for the change in permeability and slope to occur, is not predicted.

4.4. Sludge Drying.

The drying which occurs throughout the constant head and falling head filtrations, has not been considered. A large mass of drained sludge was prepared in several batches, by applying settled sludge to the filtration apparatus (Fig. 26). Drainage was essentially complete in 3 days. The composite sample of drained sludge was analysed and found to contain 12,50 g moisture/g bone dry solids (i.e. 92,59% water; 74,07 g solids/kg).

To establish drying curves and investigate the effect of drying bed loadings, aliquots of the drained sludge were poured into a set of tared flat-bottomed stainless steel tubes of varying diameters (4,00 cm; 3,60 cm; 4,87 cm; 2,39 cm; and 2,66 cm respectively); and to varying depths (1,00 cm; 3,20 cm; 4,10 cm; 6,35 cm; and 7,90 cm respectively). This gave a range of loadings of 0,533 to 5,776 kg/m² (Table 27).

Prior to commencing the drying experiments, the bulk mass of drained sludge was thermally equilibrated in a closed bottle placed overnight in a laboratory with controlled atmosphere (65% Relative Humidity $\pm$ 2%; 21°C. $\pm$ 1°C.)

The 5 drying beds were mounted together in a plastic stand.

The latter consisted of a panel drilled with holes slightly larger in diameter than that of the outside diameter of the tubes, and supported about 1 cm above a solid panel bottom.

Throughout the 15 day experimental work, the drying beds were weighed

at intervals in the controlled atmosphere laboratory.

The air velocity was constant at 0,8 m/min.

4.4.1. Determination of Drying Curves and "Critical" Moisture

Content at 65% R.H. and 21°C.

Results are recorded in Tables 28-32 inclusive.

Instantaneous moisture contents were calculated in the following manner:-

If M = initial mass of wet sludge (g),

M_d = bone-dry mass of solids (g)

m = mass of moisture (g) evaporated up to time t (hours)

A = surface area of drying bed (cm^2)

Initial moisture content, $W_o = (M - M_d) / M_d \text{ gH}_2\text{O} / \text{g dry solids}$.

Instantaneous moisture content,

$$W = (M - M_d - m) / M_d \text{ gH}_2\text{O} / \text{g dry solids}$$

Instantaneous average rate of evaporation,

$$R = m / At \times 10^4 \text{ g/m}^2 / \text{hr.}$$

If m_1 = mass of moisture (g) evaporated up to time t_1 (hours), and

m_2 = mass of moisture (g) evaporated up to time t_2 (hours),

average drying rate during period $(t_2 - t_1) =$

$$\bar{R} = \frac{(m_2 - m_1) \times 10^4}{(t_2 - t_1) A} \text{ g/m}^2 / \text{hr.}$$

If W_1 = moisture content ($\text{gH}_2\text{O} / \text{g dry solids}$) at time t_1 (hr.), and

W_2 = moisture content ($\text{gH}_2\text{O} / \text{g dry solids}$) at time t_2 (hr.),

Average moisture content in period $(t_2 - t_1) =$

$$\bar{W} = \frac{W_1 + W_2}{2} \text{ (g/g)}$$

Fig. 45 is a plot of the drying curves obtained during the first 200 hours (mass of moisture evaporated, m , against time, t). Fig. 46 is a plot

TABLE 27

A.71

ATMOSPHERIC SLUDGE DRYING @ 65% RELATIVE HUMIDITY, 21°C, & AIR VELOCITY = 0,8 m/min

Exp. No.	1	2	3	4	5
Initial Sludge Depth (cm)	1,0	3,2	4,1	6,35	7,9
Initial Surface Area, A (cm ²),	12,56	10,17	18,63	4,49	5,56
Initial Mass of Wet Sludge, M (g)	9,0380	21,0515	45,2600	27,5934	43,3539
Mass of Bone Dry Solids, M _d (g)	0,6694	1,5593	3,3524	2,0438	3,2112
Loading, D/A = $M_d/A \times 10^4$ (g.dry solids/m ²)	533	1533	1799	4552	5776

Initial Moisture Content = 92,59 g/100 g. sludge.

Initial Moisture W₀ = 12,50 g H₂O / g dry solids

Initial Solids Content = 74,07 g / kg sludge.

TABLE 28 (FIGS. 45(I) & 45(II))

SLUDGE DRYING AT CONSTANT AMBIENT CONDITIONS

Exp. I : Loading, D/A = 533 g/m ² ; Area, A = 12,56 cm ²					
Time t (hours)	Mass H ₂ O Evaporated m (g)	Drying Rate, $R = \frac{m}{t} \times 10^4$ (g/m ² /hr)	Moisture Content, W (gH ₂ O/g dry)	Average Drying Rate, $\bar{R}^+$ (g/m ² /hr)	Average Moisture Content, $\bar{W}^{++}$ (gH ₂ O / g dry)
0	0	0	12,50		
4,25	0,7657	150,1	11,36	143,4	12,06
21,25	3,2123	125,9	7,70	114,6	9,53
25,00	3,8002	126,7	6,83	124,8	7,27
28,25	4,3334	127,8	6,03	130,6	6,43
45,25	6,7418	124,1	2,43	117,0	4,23
49,00	7,1457	121,5	1,83	85,8	2,13
52,25	7,4399	118,6	1,39	72,1	1,61
69,75	8,2204	98,2	0,22	35,5	0,81
95,50	8,3006	72,4	0,10	2,5	0,16
117,25	8,2993	58,9	0,10	0	0,10
121,00	8,3012	57,2	0,10	0	0,10
124,25	8,2994	55,6	0,10	0	0,10

$$\bar{R}^+ = \frac{(m_2 - m_1) \times 10^4}{(t_2 - t_1) \cdot A}$$

$$\bar{W}^{++} = \frac{(W_1 + W_2)}{2}$$

TABLE 29

SLUDGE DRYING AT CONSTANT AMBIENT CONDITIONS
(FIGS. 45(2) & 46(2))

Exp. 2: Loading, $D/A = 1533 \text{ g/m}^2$; Area, $A = 10,17 \text{ cm}^2$					
Time t (hours)	Mass H_2O Evaporated m (g)	Drying Rate $R = m/t \times 10^4$ ($\text{g/m}^2/\text{hr.}$)	Moisture Content, W ($\text{gH}_2\text{O/g dry}$)	Average Drying Rate, $\bar{R}$ + ($\text{g/m}^2/\text{hr}$)	Average Moisture Content $\bar{W}$ ++ ($\text{gH}_2\text{O/g dry}$)
0	0	0	12,50		
4,25	0,5788	136,1	12,13	133,9	12,32
21,25	2,4083	113,3	10,96	105,8	11,55
25,00	2,8521	114,0	10,67	116,4	10,82
28,25	3,2597	115,3	10,41	123,3	10,54
45,25	5,4572	120,6	9,00	127,1	9,71
49,00	5,9730	121,8	8,67	135,2	8,84
52,25	6,4161	122,7	8,39	134,1	8,53
69,75	8,3245	119,3	7,16	107,2	7,78
95,50	11,3932	119,3	5,19	117,2	6,18
117,25	13,5017	115,1	3,84	95,3	4,52
121,00	13,7811	113,8	3,66	73,3	3,75
124,25	14,0127	112,7	3,51	70,1	3,59
165,50	16,7217	101,0	1,78	64,6	2,65
172,50	17,0771	98,9	1,55	49,9	1,67
189,25	17,7959	94,0	1,09	42,2	1,32
196,25	18,1170	92,3	0,88	45,1	0,99
239,25	19,1391	79,9	0,23	23,4	0,56
294,00	19,2734	65,5	0,14	2,4	0,19
316,00	19,2867	61,0	0,13	0,6	0,14
340,00	19,2931	56,7	0,13	0,3	0,13
358,00	19,2909	53,8	0,13	0	0,13

$$R = \frac{(m_2 - m_1) \times 10^4}{(t_2 - t_1)A}$$

$$\bar{W} = \frac{(W_1 + W_2)}{2}$$

TABLE 31

A.74

SLUDGE DRYING AT CONSTANT AMBIENT CONDITIONS
(FIGS. 45(4) & 46(4))

Exp. 4: Loading, $D/A = 4552 \text{ g/m}^2$; Area, $A = 4,49 \text{ cm}^2$					
Time t (hours)	Mass H_2O Evaporated $m \text{ (g)}$	Drying Rate $R = m/tA \times 10^4$ ($\text{g/m}^2/\text{hr}$)	Moisture Content, W ($\text{g/H}_2\text{O/gdry}$)	Average Drying Rate, $\bar{R} +$ ($\text{g/m}^2/\text{hr}$)	Average Moisture Content, $\bar{W} ++$ ($\text{gH}_2\text{O/g dry}$)
0	0	0	12,50		
4,25	0,1021	60,0	12,45	53,5	12,48
21,25	0,4150	48,8	12,30	41,0	12,38
25,00	0,4895	48,8	12,26	44,2	12,28
28,25	0,5571	49,2	12,23	46,3	12,25
45,25	0,9223	50,8	12,05	47,8	12,14
49,00	1,0016	51,0	12,01	47,1	12,03
52,25	1,0721	51,3	11,98	48,3	12,00
69,75	1,3682	49,0	11,83	37,7	11,91
95,50	1,8581	48,5	11,74	42,4	11,79
117,25	2,2295	47,5	11,41	38,0	11,58
121,00	2,2834	47,0	11,38	32,0	11,40
124,25	2,3297	46,8	11,36	31,7	11,37
165,50	2,9254	44,0	11,07	32,2	11,22
172,50	3,0073	43,5	11,03	26,1	11,05
189,25	3,1792	41,8	10,95	22,9	10,99
196,25	3,2684	41,5	10,90	28,4	10,93
239,25	3,7245	38,8	10,68	23,6	10,77
294,00	4,2612	36,0	10,42	21,8	10,55
316,00	4,4741	35,3	10,31	21,6	10,37
340,00	4,7034	34,5	10,20	21,3	10,26
358,00	4,8647	33,8	10,12	20,0	10,16

$$+ \bar{R} = \frac{(m_2 - m_1) \times 10^4}{(t_2 - t_1) A}$$

$$++ \bar{W} = \frac{(W_1 + W_2)}{2}$$

TABLE 32

A.75

SLUDGE DRYING AT CONSTANT AMBIENT CONDITIONS
(FIGS. 45(5) & 46(5))

Exp. 5: Loading, $D/A = 5776 \text{ g/m}^2$; Area, $A = 5,56 \text{ cm}^2$					
Time t (hours)	Mass H_2O Evaporated $m(\text{g})$	Drying Rate $R = m/tA \times 10^4$ ($\text{g/m}^2/\text{hr}$)	Moisture Content, W ($\text{gH}_2\text{O/g dry}$)	Average Drying + Rate, $\bar{R}$ ($\text{g/m}^2/\text{hr}$)	Average ++ Moisture Content, $\bar{W}$ ($\text{gH}_2\text{O/g dry}$)
0	0	0	12,50		
4,25	0,0904	42,4	12,47	38,3	12,49
21,25	0,3034	28,4	12,41	22,5	12,44
25,00	0,3528	28,2	12,39	23,7	12,40
28,25	0,3976	28,0	12,39	24,8	12,39
45,25	0,6258	27,6	12,31	24,1	12,35
49,00	0,6894	28,0	12,29	30,5	12,30
52,25	0,7436	28,4	12,27	30,0	12,28
69,75	0,9192	26,2	12,22	18,0	12,25
95,50	1,2583	26,2	12,11	23,7	12,17
117,25	1,5629	26,6	12,01	25,2	12,06
124,25	1,6749	26,8	11,98	28,8	12,00
165,50	2,2075	26,6	11,81	23,2	11,90
172,50	2,3051	26,6	11,78	25,1	11,80
189,25	2,4670	26,0	11,73	17,4	11,76
196,25	2,5769	26,2	11,70	28,2	11,72
239,25	3,0199	25,2	11,56	18,5	11,63
294,00	3,5363	24,0	11,40	17,0	11,48
316,00	3,7374	23,6	11,34	16,4	11,37
340,00	3,9369	23,0	11,28	15,0	11,31
358,00	4,0552	22,6	11,24	11,8	11,26
$+ \bar{R} = \frac{(m_2 - m_1) \times 10^4}{(t_2 - t_1) \cdot A}$ $++ \bar{W} = \frac{(W_1 + W_2)}{2}$					

of the rate of evaporation R against moisture content W . Data for both of these sets of plots have been taken from Tables 28-32 inclusive. In Fig. 46, the intersection of the sloping lines and the horizontal "constant"-rate lines, was accepted to be the points at which the "critical" moisture content W_c had been reached. Straight lines joining the latter points with the equilibrium moisture content E approximate the falling rate-of-drying period. Average drying rates $\bar{R}$ at average moisture contents $\bar{W}$, were also plotted, and show good co-linearity with the falling rate period at low loadings [Plot 1 in Fig. 46]. At higher loadings, there was an increasing divergence [Plots 2-5 in Fig. 46].

4.4.2. Determination of The Falling Rate Drying Curves At 65% R.H. and 21°C.

The "critical" moisture contents W_c read from Fig. 46 enabled the estimation of the drying times required to reach these values, as follows:-

Mass of Residual Moisture at "critical" moisture content = $W_c M_d$.

Mass of Moisture evaporated in the "constant" rate period to the time t_c of the "critical" moisture content = $M - M_d - W_c M_d = m$.

(symbols defined as in section 4.4.1.)

Referring to the drying curves in Fig. 45, the time to evaporate the m grams of moisture was read. These values yielded the t_c times. The t_c times occurred at the end of the "constant" rate period, and were also the zero times for the commencement of the falling rate periods.

Subtracting corresponding t_c values from the total times ($t \geq t_c$) in Tables 28-32 inclusive, the nett times t_f in the falling rate period were obtained. Simultaneously, subtracting the equilibrium moisture content, $E = 0.10$ g/g, from the total moisture contents W at these corresponding times, the free moisture contents W_f were obtained. These calculations are tabulated in Table 33, and plotted on semi-log paper in Fig. 49.

TABLE 33

SLUDGE DRYING IN FALLING RATE PERIOD AT CONSTANT AMBIENT CONDITIONS
(FIG. 49)

Exp. No.	'Critical' Moisture Content, W_c (Fig. 46) (gH ₂ O/g dry)	Mass of Dry Solids, M_d (g)	Mass of Residual Moisture at 'Critical' Moisture Content ($M_d \cdot W_c$) (g)	Initial Mass of Moisture ($M - M_d$) (g)	Mass of Moisture Evaporated at 'Critical' Moisture Content, $M_c = M - M_d - M_d \cdot W_c$ (g)	Time at 'Critical' Moisture Content, t_c (Fig. 45) (hours)
1	2,85	0,6694	1,91	8,37	6,46	44,5
2	4,50	1,5593	7,02	19,49	12,47	107,5
3	7,40	3,3524	24,80	41,91	17,11	105,0
4	11,62	2,0438	23,75	25,55	1,80	94,5
5	11,90	3,2112	38,20	40,14	1,94	156,5

EQUILIBRIUM MOISTURE CONTENT AT 65% R.H. & 21°C., $E = 0,10$ g H₂O/ g dry solids.

1. $D/A = 533$ g/m ²		2. $D/A = 1533$ g/m ²		3. $D/A = 1799$ g/m ²		4. $D/A = 4552$ g/m ²		5. $D/A = 5776$ g/m ²	
Time in Falling Rate Period $t_f = t - t_c$ (hr)	Free Moisture Content $W_f = W - E$ (g/g)	Time in Falling Rate Period $t_f = t - t_c$ (hr)	Free Moisture Content $W_f = W - E$ (g/g)	Time in Falling Rate Period $t_f = t - t_c$ (hr)	Free Moisture Content $W_f = W - E$ (g/g)	Time in Falling Rate Period $t_f = t - t_c$ (hr)	Free Moisture Content $W_f = W - E$ (g/g)	Time in Falling Rate Period $t_f = t - t_c$ (hr)	Free Moisture Content $W_f = W - E$ (g/g)
0	2,75	0	4,40	0	7,30	0	11,52	0	11,80
0,75	2,33	9,75	3,74	12,25	6,73	22,75	11,31	9,00	11,71
4,50	1,73	13,50	3,56	16,00	6,59	26,50	11,28	16,00	11,68
7,75	1,29	16,75	3,41	19,25	6,47	29,75	11,26	32,75	11,63
25,25	0,12	58,00	1,68	60,50	4,99	71,00	10,97	39,75	11,60
40,50	0	65,00	1,45	67,50	4,77	78,00	10,93	82,75	11,46
		81,75	0,99	84,25	4,34	94,75	10,85	137,50	11,30
		88,75	0,78	91,25	4,12	101,75	10,80	159,50	11,24
		131,75	0,13	134,25	3,03	144,75	10,58	183,50	11,18
		186,50	0,04	189,00	1,80	199,50	10,32	201,50	11,14
		208,50	0,01	211,00	1,36	221,50	10,21		
				235,00	0,93	245,50	10,10		
				253,00	0,68	263,50	10,02		

The slopes of these lines gave the falling rate constants, k_f ; the intercepts are the "critical" moisture contents, W_c .

4.4.3. Effect of Bed Loading on Critical Moisture Content, W_c , and Constant Drying Rate, R_c .

Table 34 tabulates the co-ordinates at the "critical" moisture contents for the different loadings. These values were obtained from Fig. 46.

TABLE 34
Co-ordinates of the Intersection Points of "Constant"
and Falling Rate Periods. (Data for Figs. 47 and 48,
ex Fig. 46)

Exp. No.	Loading, $(D/A \times 10^{-3})$ ($k \text{ g/m}^2$)	"Critical" Moisture Content, W_c (g/g)	"Constant" Drying Rate, R_c ($\text{g/m}^2/\text{hr.}$)
1	0,533	2,85	126,8
2	1,533	4,50	120,0
3	1,799	7,40	93,7
4	4,552	11,62	50,0
5	5,776	11,90	26,8

A log-log plot of the "critical" moisture content W_c against the bed loading D/A is shown in Fig. 47.

The equation of the resulting line was calculated to be

$$W_c = 0,045 (D/A)^{0,655} \quad [\text{equation (53)}]$$

A semi-log plot of the "constant" drying rate R_c against the bed loading D/A is shown in Fig. 48.

The equation of this line gave the relationship

$$R_c = 158e^{-0,00012 D/A} \quad [\text{equation (54)}].$$

The intercept of the latter plot at zero loading ($R_c = 158 \text{ g/m}^2/\text{hr}$) corresponded to the constant rate evaporation of moisture from the filtrate at 65% R.H; 21°C ; and 0,8 m/min air flow velocity.

4.4.4. Effect of Bed Loading on the Falling Rate Period Constant, K_f

The values of the falling rate constant K_f [equation (52A)] were calculated from the slopes of the lines in Fig. 49, as follows:-

TABLE 34A:
Variation of Falling Rate Constant K_f with loading

(Data ex Fig. 49 for Fig. 50)

Exp. No.	Slope	Falling Rate Constant, K_f ($= -2,303 \times \text{slope}$)	Loading (D/A $\text{kg/m}^2 (\times 10^{-3})$)
1	-0,0432	0,0995	0,533
2	-0,00743	0,0171	1,533
3	-0,00266	0,00612	1,799
4	-0,000223	0,000515	4,552
5	-0,000117	0,00027	5,776

Fig. 50 shows a log-log plot of K_f against the loading D/A, from which the following relationship was established:-

$$K_f = 2,78 \times 10^6 \left(\frac{D}{A} \right)^{-2,67} \quad [\text{Equation (55)}].$$

4.4.5. Calculation of Drying Bed Loadings At 65% R.H. and 21°C.

Selecting a range of bed loadings $D/A = 2000 - 4000 \text{ g/m}^2$, [corresponding to the loading rates expected (Section 4.3.4)], values of W_c , R_c and K_f were calculated from the experimental relationships obtained in sections 4.4.3 and 4.4.4 respectively. Table 35 shows the results of these calculations:-

TABLE 35

Drying Bed Loading Calculations at 65% R.H. and 21°C.

Bed Loading $D/A \text{ (g/m}^2\text{)}$	"Critical" Moisture Content $W_c \text{ (g/g)}$	"Constant" Rate of Drying $R_c \text{ (g/m}^2\text{/hr)}$	Falling Rate Period Constant K_f	Drying Time * To Critical Moisture Content, $t_c \text{ (hours)}$
2000	6,54	91	0,0043	153
2500	7,56	79	0,0024	185
3000	8,53	69	0,0015	216
3500	9,43	60	0,00096	237
4000	10,29	52	0,00067	247
* Assuming a wet sludge initially at $W_o = 13,5 \text{ g H}_2\text{O/ g dry solids}$ (93,1% mass/mass moisture content):				

The time to arrive at the critical moisture content at the end of the constant rate period, was calculated as follows:-

$$t_c = \frac{(W_o - W_c) D/A}{R_c} \quad [\text{equation (56)}]$$

The drying time required to reach a desired moisture content W , when $W < W_c$, was obtained from this equation, and is included in Table 36.

Drying times to reach desired moisture contents which are less than the "critical" moisture contents, were calculated from the sum of the

TABLE 36 (FIG. 51)

DRYING TIMES FOR SLUDGE INITIALLY AT MOISTURE CONTENT $W_0 = 13,5 \text{ g H}_2\text{O/g dry solids}$; 65% R.H. & 21°C .

Moisture Content $W \text{ g/g}:-$	DRYING IN 'CONSTANT' RATE PERIOD, t_c				DRYING IN FALLING RATE PERIOD, t_f									Time to 'Critical' Moisture Content $W_c \text{ (hrs)}$
	10	9	8	7	10	9	8,5	8	7	6	5	4	3	
Bed Loading (kg/m^2)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	Time (hrs)	
2,0	77	99	121	143	-	-	-	-	-	20	64	117	186	153
2,5	111	142	174	-	-	-	-	-	33	98	175	270	394	185
3,0	152	195	-	-	-	-	2	35	134	238	362	529	712	216
3,5	204	-	-	-	-	49	109	173	314	477	671	909	1218	237
4,0	-	-	-	-	43	202	288	380	582	816	1093	1434	1876	247

NOTE: (-) = Drying to moisture content W cannot terminate in this period.

TOTAL DRYING TIME, $t_c + t_f$ (FIG. 51)

Moisture Content $W \text{ g/g}:-$	10	9	8,5	8	7	6	5	4	3
BED LOADING (kg/m^2)	Time (Days)	Time (Days)	Time (Days)	Time (Days)	Time (Days)	Time (Days)	Time (Days)	Time (Days)	Time (Days)
2,0	3,2	4,1	4,7	5,0	6,0	7,2	9,0	11,3	14,1
2,5	4,6	5,9	6,5	7,3	9,0	11,8	15,0	19,0	24,1
3,0	6,3	8,2	9,1	10,5	14,6	18,9	24,1	31,0	38,7
3,5	8,5	12,0	14,5	17,1	23,0	29,8	37,8	47,8	60,6
4,0	12,9	18,7	22,3	26,1	34,5	44,3	55,8	70,0	88,5

drying times t_c to "critical" moisture content (Table 35), and drying time in the falling rate period t_f , given by

$$t_f = \frac{2,303}{K_f} \log \frac{(W_c - E)}{(W_f - E)} \quad [\text{from equation (52A)}]$$

Negative values of t_f indicated that the desired final moisture contents were less than the "critical" moisture contents. These values have been omitted from Table 36.

The total drying times required to obtain a range of final moisture contents, $3 < W_f < 10$, were calculated in Table 36 and plotted in Fig. 51.

4.4.6. Sludge Moisture Content for Spadeability.

A mass of drained sludge (approximately 6,5 kg) at moisture content $W_o = 12,50$ g/g, was exposed to the weather under adverse conditions. The drying bed consisted of an open box, 25 cm x 25 cm x 10 cm deep.

A transparent perspex sheet, 60 cm x 60 cm, was supported at a height of 30 cm above the box. The covering perspex sheet was inclined to the horizontal to allow rainwater drainage to flow to the one edge. A maximum-and-minimum thermometer and recording hygrometer were placed beneath the cover. Wind velocities were obtained from the local weather station.

The mass of sludge was examined daily for spadeability. Small core samples were taken through the depth of the bed, for the determination of average moisture content. The following observations were recorded:

Drying Time (days)	Temperature °C.		Relative Humidity %		Wind Velocity km/hour	Sludge Moisture Content, W g/g	Spadeability
	Max.	Min.	Max.	Min.			
1	18,0	11,8	90	53	21,5 NW	10,42	Liquid
2	13,6	10,0	89	83	9,0 NW	9,60	Liquid
3	14,2	11,4	78	75	21,5 NW	8,63	Spadeable
4	11,6	10,0	93	91	calm	7,51	Spadeable
5	12,1	7,4	82	70	calm	7,00	Spadeable
6	12,5	8,2	95	67	calm	5,43	"Dry" Mass
7	15,8	11,0	92	79	13,0 NW	3,82	"Dry" Mass

From the third day of drying onwards, the average mass was sufficiently coherent to be spadeable (W_0 approx. = 8,5 g/H₂O/g dry solids). From the sixth day onwards, the mass had a "dry" consistency, and was readily spadeable. After the seventh day, a strong wind disrupted the apparatus. On the following day, the dried sludge was found in fairly hard lumps, which could be hand-lifted. An average moisture content of the lumps gave $W = 1,90$ g H₂O/g dry solids.

Accepting the maximum moisture content for spadeability as 8,5 g/g, this value is shown in Fig. 51. Drying bed loadings may now be estimated such that a moisture content of less than 8,5 g/g is obtained at 65% R.H. and 21°C. The actual exposure of sludge to wind and radiation produced an increased drying rate, despite the reduced temperatures and higher humidities during this experiment. Thus, a bed-loading of 2,6 kg/m² (Fig. 51) will produce sufficient drying within 7 days of exposure, which will enable the lifting of the sludge from the sand bed.

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