TO MY FATHER

SLUDGE CONDITIONING PRIOR TO ATOMIZED SUSPENSION TECHNIQUE

by

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ABSTRACT

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SLUDGE CONDITIONING PRIOR TO ATOMIZED SUSPENSION TECHNIQUE

The investigations were carried out at the Primary Waste Water Treatment Plant in Beaconsfield, Quebec, and the plant performance and efficiency determined.

The removal values of Suspended Solids and Chemical Oxygen Demand determined, compared favourably with expected values for primary treatment processes, found elsewhere.

Results of experimental tests made at the plant showed that over 10% of solids in the primary sludge could be obtained by maintaining a sludge blanket depth in the thickener of over 3 feet.

Thickening of sludge was controlled by using the concept of Sludge Volume Ratio S.V.R., defined as the volume of sludge blanket held in the thickener, divided by the volume of sludge pumped per day from the thickener.

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CHAPTER I

INTRODUCTION

Before World War II most sewerage systems in Canada were paid for and operated by municipalities for the benefit of the residents of the city. Rising costs and greatly increased residential construction following the war caused many cities to refuse to tax their old residents to supply water and sewer extensions for new sub-divisions, especially beyond the city limits.

The Town of Beaconsfield in the Province of Quebec was one such town, faced in 1955 with a population growing at an average rate of increase of about 10 per cent annually, and consequently faced with the problems of providing water supply and sewerage systems for the residents of the town.

In 1956 H.W. Lea, Consulting Engineer, Montreal,(*26) submitted a report on sanitary sewerage systems to the town. On the basis of his design, the first 8 million gallons per day primary waste treatment plant was built in 1958, capable of treating one-third of the ultimate flow of 24 m.g.d. Waste water is conveyed to the plant by one trunk sewer having a diameter of 36 inches. The waste is treated by sedimentation only, and the effluent is chlorinated before being discharged into Lake St. Louis. Sludge is pumped from the sedimentation tank into a thickener where it is concentrated.

In the conventional primary treatment plant the sludge will be pumped from the thickener to the digester. At the Beaconsfield plant

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* Bibliography Reference

this unit was modified by introducing for the first time in the world, the Atomized Suspension Technique process.

This method was invented and subsequently patented by Dr. W.H. Gauvin in 1952 at the Pulp and Paper Research Institute of Canada(*15). The A.S.T. process permits the evaporation and drying of a liquor or settled sludge of any initial concentration. In 1958 a full-scale A.S.T. unit was installed at the Beaconsfield plant and was put into operation in 1959. After a few months of operation the unit broke down. It was claimed that the initial operation of the unit did not produce satisfactory results due to, among other factors, underdesign of the system and fusion of the pressure nozzles.

In the spring of 1963, the Pulp and Paper Research Institute of Canada approached the Chairman of Civil Engineering and Applied Mechanics, McGill University and requested assistance in the rehabilitation of the A.S.T. unit. Since sludge concentration prior to A.S.T. has not been investigated, it was decided that research work should be done concerning the sludge itself. Additionally, a plant performance study was to be carried out to determine the plant efficiency and, if possible, improve it.

This thesis is, therefore, primarily concerned with these experimental investigations. The following three chapters of the text represent an attempt by the author to present the known principles of waste treatment essential to the experimental work. Much of the subject matter is the work of others and due credit is given by means of references given in Bibliography at the end of this thesis. The remaining chapters present a report on the experimental investigations carried out at the Primary Waste Treatment Plant in Beaconsfield.

CHAPTER II

General Considerations.

The physical conditions in Canada are unlike those of many other countries. Nature has provided an unusually large number of lakes and streams and it is along these that much of the population of the country is located.

In 1960 there were 1,010 sewerage systems in Canada serving 9,059,000 or about 51.2% of the total population. 683 of these systems have waste water treatment installations divided into the following(*8):

346	partial treatment plants
192	waste water ponds or lagoons
145	secondary treatment plants

Since the highest concentrations of population occur in the provinces of Quebec and Ontario, it is not surprising that 70% of the sewerage systems were located in these two provinces.

It may be concluded therefore that little effort has been made in some provinces to construct waste water treatment plants, and either that undue advantage is being taken of these natural conditions or that proper recognition has not yet been given to this problem of waste treatment.

It would seem reasonable to ascribe this limitation in waste water treatment to some of the following causes:

i) The early waste water treatment methods employed were based on English experience and some of them were not suitable under local climatological conditions.

ii) Large bodies of diluting water and absence of water intakes and bathing beaches.

iii) Many sewerage systems built before waste water treatment was well developed have been allowed to continue operations without any improvements.

iv) Lack of public interest in waste water treatment and disposal.

A general review of Canadian practice in the field of waste water treatment has shown that in most cases American methods have been followed very closely. Infact it may be said that Canadian technique relating to waste treatment is very much indebted to the scientific studies emanating from the United States.

Three kinds of wastes are commonly defined, namely:

1. Domestic waste, consists of discharges of spent water from bathrooms, lavatories and kitchens. It is a complex mixture of organic and inorganic matter in many forms including, suspended matter, colloidal and pseudo-colloidal matter, and dissolved matter.

Domestic waste also contains bacteria, viruses and protozoa, some of which are pathogenic in nature.

2. Storm water, represents drainage waters from the land and streets, and may have biochemical oxygen demand, and carry suspended and dissolved matter, and bacteria. Ground water leaking into the sewers and surface water from streets, roofs and paved areas may carry a lot of silt.

3. Industrial wastes, contribute a variety of complex organic and inorganic compounds, such, as for instance, would originate in textile mills, chemical works, gas works, tanneries, slaughter houses, pulp and paper mills, and others.

The problems of waste water treatment are,

i) to remove within economical limitations, the floating and suspended matter.

ii) to remove the remaining fine suspended, colloidal and dissolved matter to levels which would be compatible with the final disposal conditions, and

iii) to disinfect effluent in order to remove bacteria prior to final disposal of plant effluent.

The methods employed for waste water treatment today were introduced on the basis of empirical experimenting. The choice of methods depends on a number of factors which may be listed as follows:-

a) The requirements of local health authorities.

b) The character of wastes.

c) Skill and quality of operation required.

d) Head available for gravity flow through plant and in the final disposal.

e) First cost, and the cost of operation.

f) Ease of increasing capacity.

Basically, the methods or combination of methods employed may be classified as follows:

Primary treatment, removes floating and settleable matter only, while B.O.D. removal is relatively low. The principles employed are basically hydraulic separation and flotation.

Typical units of primary treatment plant would be grit separators, skimming tanks, screens, sedimentation, sludge digestion, and disinfection.

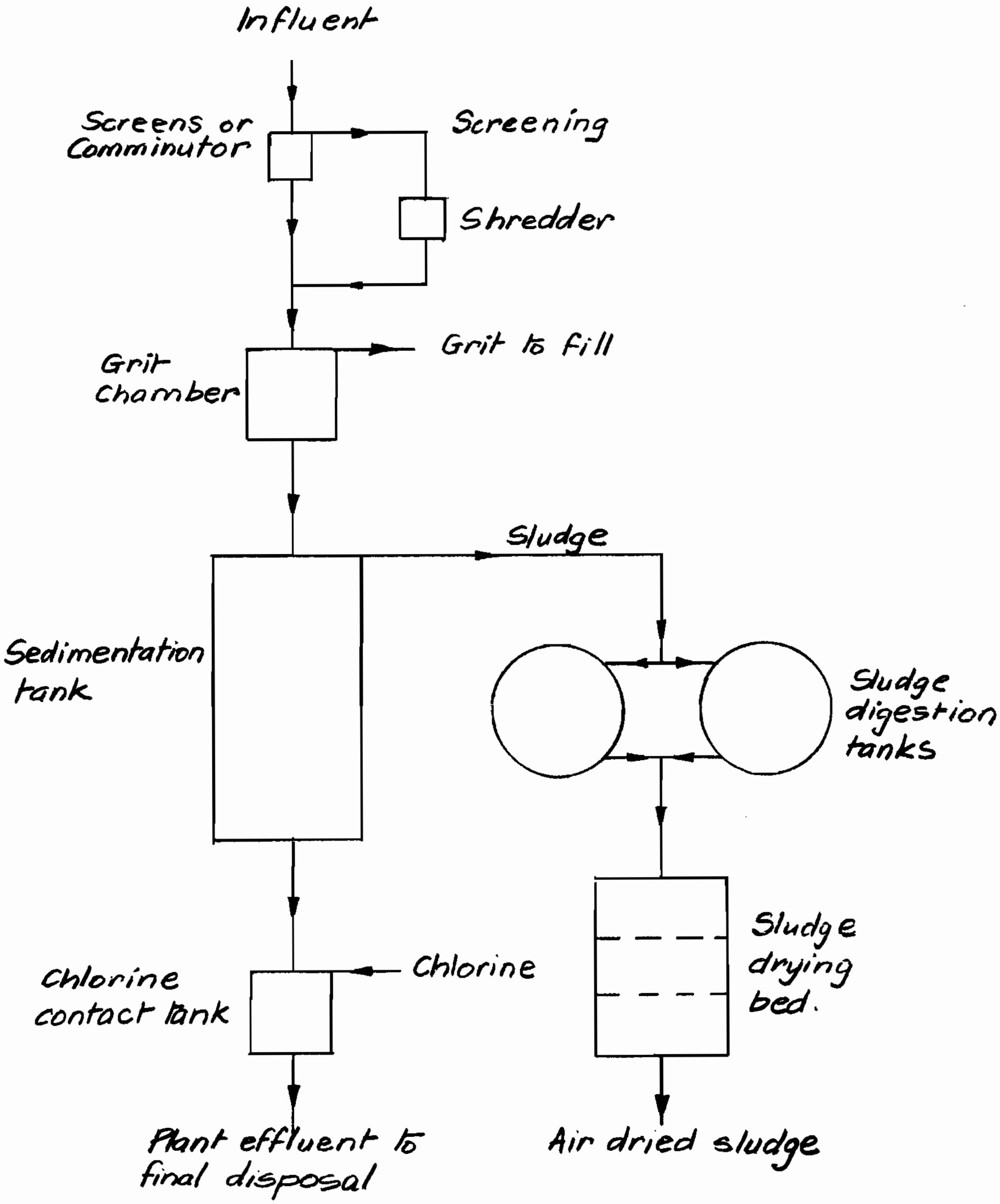


Figure 1. Schematic flow diagram, Primary Treatment Plant

Primary treatment can usually be expected to remove 40 to 70% suspended solids, 25 to 40% B.O.D. and 25-75% bacteria. The removed settleable matter is called raw sludge.

Intermediate Treatment. If removal by plain sedimentation does not satisfy the final disposal conditions, chemical treatment is introduced. The process is analogous to plain sedimentation, but the volume of chemical sludge obtained is larger and heavier than primary sludge.

Removals, of 70-90% of the suspended matter, 50-80% of B.O.D. and 40-80% of bacteria may be expected by the use of this method.

Secondary Treatment. This is an additional treatment of waste by biological methods after primary treatment. The raw waste contains high energy organic matter which is a rich food source for microorganisms such as bacteria, protozoa and others which are found in the waste.

The biological destruction of the organic waste materials is fastest when sufficient of free or dissolved oxygen is available to these microorganisms. The treatment units are trickling filters, activated sludge process, super-aeration, intermittent sand filters and oxidation ponds. Secondary treatment may be expected to remove 80-98% suspended matter, 80-95% B.O.D. and 90-99% bacteria in the waste after primary treatment.

From the statistics of sewerage systems in Canada, it will be seen that raw sewage is still discharged into the great streams, lakes and tidal waters without treatment. Disposal methods of this kind will interfere in the long run with other legitimate water uses.

Disposal of waste waters on land by surface or subsurface irrigation is practiced in some arid and semi arid regions of the world. In most cases the waste water is disinfected before disposal.

Sludge Treatment and Disposal

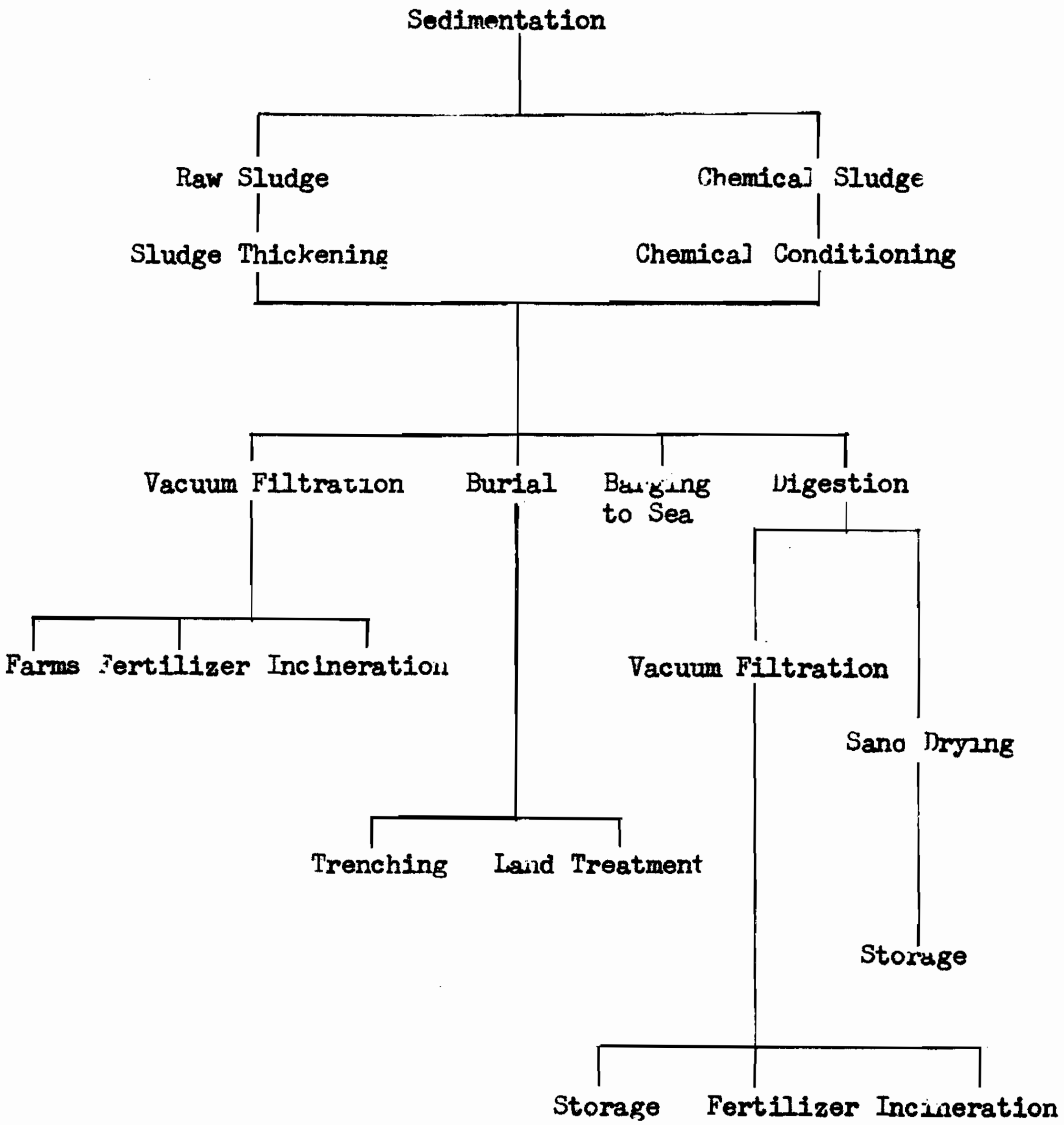
The raw primary sludge contains a large amount of water and, because of the bulkiness of the sludge, and the fact that it is putrescible, it is necessary to reduce the water content and stabilize the organic matter.

The methods employed are extremely varied and Chart No. 1 shows the treatment methods of primary sludge. A detailed discussion of sludge disposal methods is given in Chapter III.

DISINFECTION OF WASTE WATERS

The objectives of waste water disinfection is to destroy the waterborne pathogenic organisms that may be present in order to make the receiving body of water safe for the various uses to which it may be put. Among the pathogenic organisms which may be found in waste waters are cholera, typhoid and paratyphoid, dysentery, amoebic dysentery, and tuberculosis bacteria as well as viruses causing infectious hepatitis and poliomyelitis, and parasitic worms.

The number of the harmless bacteria Eschericia Coli is used as an indication of the efficiency of disinfection and the elimination of pathogenic bacteria. In practice very few plants control their disinfection by the count of coliform organisms.



<u>Chart No. 1.</u> The Primary Sludge Treatment Processes.

The efficiency of disinfection can be determined by analysis of coliform bacteria in the effluent to determine compliance with standards set by some of the regulatory bodies, or by prescribing a definite chlorine residual after a given contact period.

CHAPTER III

SEDIMENTATION

General

In waste treatment the purpose of sedimentation is to remove settleable solids from the waste water. This separation is accomplished in primary settling tanks by gravitation and by natural flocculation or aggregation of the settleable particles.

Removal accomplished by this process is influenced by a large number of factors. These factors may be generalized as follows:

i) The characteristics of the liquid, under which can be included the specific gravity and the viscosity.

ii) The characteristics of the solids which include size, shape, specific gravity, concentration, coagulation, and flocculation.

iii) The characteristics of the design, such as detention period provided, velocity of flow, depth and ratio of length to depth, inlets and outlets, and baffling.

iv) Miscellaneous effects such as currents caused by wind, thermal currents, short-circuiting and others.

A variety of materials that are encountered in waste treatment vary from discrete particles to highly flocculent solids. When a particle is allowed to fall vertically in a still fluid, it is subjected to a viscous drag by the surrounding liquid, which opposes the force of gravity. In addition, the particle continuously displaces the liquid below it, the displaced liquid flows upwards and around it, and by its upward velocity increases the viscous drag. As the particle accelerates and its velocity of fall increases, the drag also increases until it balances the force of gravity, after which the particle settles with a constant terminal velocity.

Many authors agreed that only discrete particles will maintain their individuality during settling whilst flocculent particles will settle at varying settling rates. Thus two classes of particles may be defined.

<u>Class I</u> Particles or floccules which maintain their individuality throughout the settling period, hence settle essentially at a constant rate with respect to the suspending fluid, e.g., sand, silt and fine clay. The density of the particles will range from about 2.65 for fine sand grains down to about 1.03 for mud particles. The size will range from 1 mm. or more for sand to submicroscopic for silt and clay particles.

<u>Class II</u> Particles or floccules which continue to coagulate or flocculate during the settling process with consequent changes in settling rate. These contain proteins and fats with some cellulose and varying amounts of entrained water and some gases. Thus the net density of these particles will vary from less than 1.0 to perhaps 1.2. Their sizes will range from colloidal to several centimeters.

One of the earliest and most widely known attempts to set up a mathematical theory for settling in a sedimentation basin was made by

Allen Hazen in 1904.(*18) He distinguished between quiescent and turbulent flow and restricted his analysis to suspension of Class I particles having a single settling velocity.

According to Hazen, the rate of settlement of particles of sand and silt <u>in still water</u> is approximately as follows:-

Material	Diameter	Rate of Settlement ft/hr.
Coarse sand	1.0 - 0.5	1200 - 636
Fine sand	0.25 - 0.10	320 - 96
Silt	0.05 - 0.005	35 - 0.46
Fine clay	0.001 - 0.0001	0.018 - 0.00018

The principal contribution made was to demonstrate that the extent of the removal of discrete particles in a quiescent basin is independent of the depth of the basin and should be a function of overflow rate and not of the detention time. In 1936 Camp (*7) postulated that essentially the same principle would apply also to Class II particles. For the following two decades, the overflow rate principle was largely accepted for the case of suspension of Class I particles, but there is no complete agreement on the postulates from which Camp derived his conclusion concerning Class II particles.

In 1956 Fitch(*14) presented theoretical and experimental evidence indicating that for class II particles, depth and detention time can be of considerably greater significance than overflow rate in the matter of sedimentation.

A discussion of the subject of sedimentation from a theoretical standpoint may perhaps prove of assistance in this respect.

Class 1 - Clarification

Here the classical laws of sedimentation apply, and the settling is assumed to be unhindered by the presence of other settling particles. From previous discussion the forces acting on a discrete particle in suspension in a quiescent fluid are gravity, viscous drag, and buoyancy.

When a particle moves through a fluid, the resultant force acting.

$$F_R = F_E - F_B - F_D$$

external force minus buoyant force minus frictional drag force, all expressed in lb. force.

The external force, impelling the particle to move is its own weight, and as it is suspended in fluid, the external force must be reduced by buoyancy, or

$$F_E - F_B = (\rho_s - \rho) .g.V \qquad (1)$$

in which ρ_s = mass density of particle
ρ = mass density of fluid
g = gravity constant
V = volume of particle

In 1910 Newton(*28) proposed the law of drag on the assumption that the drag is due to inertia only and hence proportional to the square of the velocity. Newton's Law is generally written as,

$$F_D = C_D \cdot A_c \cdot \rho\, v_s^2/2 \qquad (2)$$

in which C_D = Newton's drag coefficient which in turn, is function of Reynolds Number R
A_c = the projected area of the body in the direction of motion
v_s = the relative velocity between the body and the fluid

The maximum or terminal settling velocity will be reached when the external force minus the buoyancy equals drag force,

i.e., $F_E - F_B = F_D$, hence equating equations (1) and (2)

$$v_s = \sqrt{\frac{2g}{C_D} \cdot (S_s - 1) \cdot \frac{V}{A_c}} \tag{3}$$

in which S_s = specific gravity of particle

For spheres, equation (3) becomes

$$v_s = \sqrt{\frac{4}{3} \cdot \frac{g}{C_D} \cdot (S_s - 1) \cdot D} \tag{4}$$

in which D = particle diameter.

The drag for a small sphere in an incompressible viscous fluid was developed by Stokes(*37) in 1880 completely neglecting the inertia forces as follows

$$F_D = 3\pi \cdot \mu \cdot D \cdot v_s \tag{5}$$

in which μ is the dynamic viscosity of the fluid.

The corresponding settling velocity is given by Stoke's Law as

$$v_s = \frac{g}{18} \cdot (S_s - 1) \cdot \frac{D^2 \rho}{\mu} \tag{6}$$

It may be noted in both equations (4) and (6) that particle size and shape as well as density are predominant variables when dealing with settlement of solids in a still fluid.

Table I. shows settling velocities in inches per minute of particles of different size, in mm. and specific gravity at 50°F.

Table No. I according to Imhoff & Fair (*20)

Diameter	1.0	0.5	0.2	0.1	0.05	0.01	0.005
Quartz sand	330	170	54	16	4	0.2	0.04
Coal	100	50	17	5	1	0.05	0.001
Sewage solids	1-80	0.2-40	0.01-12	0.01-2	0.5	0.02	0.005

Specific gravity of quartz sand = 2.65
that of coal = 1.5
and that of settleable
fresh sewage solids between 1.01 and 1.2

From the above discussion it can be seen that, in a suspension of Class I particles such as grit, silt or sand, each particle maintains its individuality and characteristic settling velocity during sedimentation, and an analysis of the suspension in terms of settling velocities may be used to predict removal and design of settling basins. In order to discuss removal, a number of assumptions must be made.

Notable among these assumptions are the following:

1. The direction of flow in an ideal rectangular continuous flow basin, is horizontal at all times and the velocity of flow is the same in all parts of the settling zone. Therefore each particle of water is assumed to remain in the settling zone for

a time equal to the detention period - namely, the volume of the settling zone divided by the discharge rate.

2. The concentration of suspended particles of each size is the same at all points in the vertical cross-section at the inlet end of the settling zone.

3. Particles are assumed to settle with uniform velocity, without change in size or shape and without interference or coalescence.

4. Particles are removed from suspension when they reach the bottom of the basin or the top of the sludge blanket within the settling zone. The particles once removed cannot be resuspended.

In real rectangular continuous flow basin the above assumptions are subject to contradictions in varying degrees by factual conditions of flocculation, turbulence scour, density currents, inlet and outlet disturbances giving rise to mixing and short-circuiting.

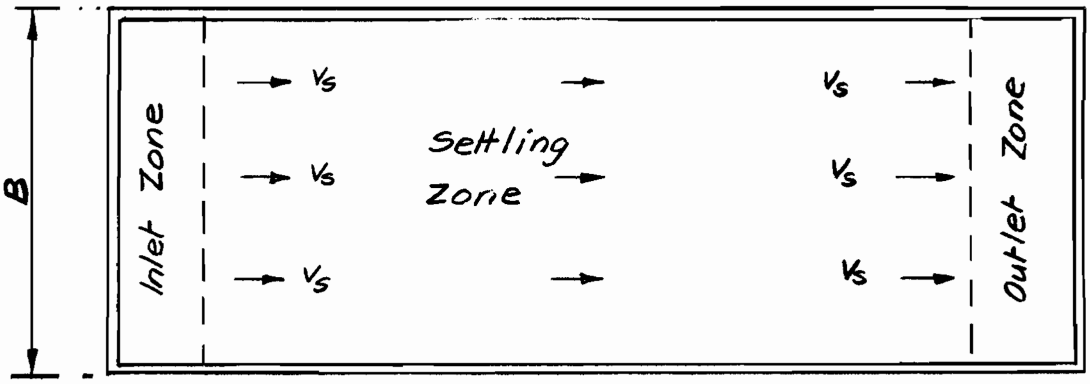
In studying the clarification theory four zones for a continuous settling basin may be considered,

i) An inlet zone in which suspension is dispersed uniformly over the cross-section of the basin.

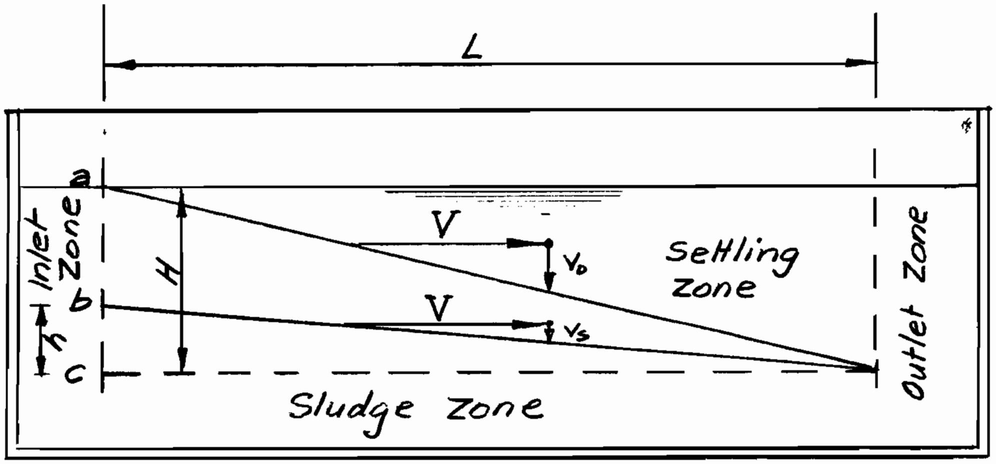
ii) The settling zone in which all settling takes place.

iii) An outlet zone in which the clarified liquid is collected uniformly over the cross-section of the basin and directed to an outlet conduit and,

iv) A sludge zone at the bottom.



PLAN



LONGITUDINAL SECTION

Figure 2. Ideal rectangular settling basin.

It is easily seen from Fig. 2 that if $v_s > v_o$ all particles will be removed and the removal Y/Y_o is unity. On the other hand if $v_s < v_o$ only those particles entering below ab. will be removed.

The removal over length "L" of the settling tank is,

$$\frac{Y}{Y_o} \% = \frac{h}{H} = \frac{v_s \cdot t_o}{v_o \cdot t_o} = \frac{v_s}{Q/A} \qquad (7)$$

in which Q = the rate of flow
A = surface area of settling zone
v_o = overflow rate
t_o = detention time
$\frac{Y}{Y_o}$ = removal ratio of particles having a settling velocity v_s.

The above equation states the following for removal of discrete particles, and unhindered settling.

1. The efficiency of basin is solely the function of the settling velocity of the particles.

2. The efficiency is function of the surface area of the tank and the rate of flow of the basin. These two combined constitute the surface loading or overflow velocity.

3. The efficiency does not depend on the depth of basin and the displacement time or detention period.

4. All particles with velocity $v_s \geqslant v_o$ are removed, and particles with $v_s < v_o$ can be fully captured if false bottoms or trays are inserted in the tank at intervals $h = v_s \times t_o$ ideally. The greater the number of trays, the smaller may be the settling velocity.

One factor in sedimentation is the size-weight composition of a suspension. This term may be expressed as frequency distribution of the settling velocities. Camp(*7) showed that if settling velocity analysis of a suspension is carried out, a cumulative frequency distribution curve may be drawn and from this curve the removal of all particles is

$$P = (1 - P_o) + \frac{1}{v_o}\int_0^{P_o} v_s \, dp \tag{8}$$

in which $(1 - P_o)$ is the removal of particles having $v_s > v_o$ in terms of the total suspension.

$\frac{1}{v_o}\int_0^{P_o} v_s \, dp$ is the removal of particles having $v_s < v_o$ in terms of total suspension.

In vertical tanks only $(1 - P_o)$ is removed.

The foregoing theory will also apply to the settling of discrete particles in ideal radial flow basins, if the assumptions were made that flow is truly horizontal and radial at all points in the basin, and that the velocity is the same at all points equi-distant from the center of the basin. The velocity will decrease however, as the distance from the centre increases and all particles will settle along curved paths.

Class II - Clarification

In establishing the theory of settling of Class I particles, Hazen(*18) applied the behavior of one particle size to an entire material in suspension. Obviously, this application can not be justified when applied to suspension of Class II particles because these particles have varying hydraulic values and their settling velocities are changing continuously. Particles collide when the fast-settling particles collide with the slower ones or when turbulence moves the particles together.

The important factor here becomes contact or possible contact between particles. The number of contacts N per unit volume and time may be estimated. Assume a particle with diameter d' and v_s' coming into contact with particle d" and v_s''. This may take place in a cylinder of diameter d' + d" and height of $v_s' - v_s''$. The height is derived so that one particle will contact the other in a unit time t. If a unit volume contains total of n' particles with d' and n" particles with d", the number of contacts per unit volume and time may be expressed as follows:

$$N = n'n''.(\pi/4).(d' + d'')^2.(v_s' - v_s'') \qquad (9)$$

If these particles are of equal density, Stokes Law may be applied and the final expression for N or number of contacts becomes

$$N = n'n''.\frac{\pi}{72}g.\frac{s - 1}{\nu} \cdot (d' + d'')^3.(d' - d'') \qquad (10)$$

From the above equation it can be seen that the number of contacts per unit volume and time is greater for larger concentration of particles of large size, large relative weight and large size-difference in a fluid of small viscosity, or high temperature.

The above considerations therefore showed that the removal depends on both the clarification rate and the depth. As no satisfactory formulation of this type of clarification exists as yet the settling rate of this class of particle size may be determined by the batch type settling test. The suspension is placed in a settling column and allowed to settle, forming a distinct solid-liquid interface. The initial rate of subsidence of this interface may then be used as the design settling. The form of analysis may be varied. Slade (*34) for example, chose to state his distribution function in terms of the time of settling of each particle from top to bottom of the settling tank, whilst Camp(*7), preferred to state the distribution function in terms of a mass curve. The concentration of the suspended matter may be measured in terms of suspended solids, turbidity, iron, alumina, calcium, hardness, color, or any other index that is reduced by settling.

The initial concentration of liquor in the settling tank must be very small, if free settling of Class II particles is to be considered without appreciable error.

Camp(*7) in his study stated that:

> "The volumetric concentration of suspended solids must exceed about 0.5% before there is an appreciable reduction in settling velocity due to hindered settling. This is comparable to about 1000 ppm of sewage suspended solids."

According to Kalinske(*21), settling commences to be hindered when the solids in suspension occupy about 1% of the original volume of the mixture by weight. This point is reached when there are about 1000 ppm of silt in a suspension of water.

Fair(*13) states that hindered flow persists in sand as long as the volumetric concentration of the sand grains exceeds 4.8%.

Owing to the varying views expressed by other authors, it would appear that it is probably not quite correct to say that there is any definite minimum value of the concentration at which hindered settling in Class II particles commences.

Settling basin performance and design. The previous discussion on the theoretical aspects of sedimentation have been related to conditions that exist in an ideal settling basin.

In actual settling basins, however, the theory must be modified by effects of flocculation, turbulence, scour, currents, inlet and outlet disturbances giving rise to mixing and short-circuiting, and the movement of cleaning mechanisms. In describing and evaluating these effects and in prescribing for their amelioration, Camp(*7) has indicated that,

i) Settling tanks should be designed on the basis of the overflow rate, that is the settling velocity of a particle of sediment of the smallest size theoretically to be 100% removed.

ii) Detention periods are of themselves immaterial; in fact for waste treatment long detention periods may be deleterious. From accumulated data obtained in the U.S., four hours detention period is maximum and most common is two to three hours.

iii) Tank should be no deeper than will be required to prevent scour, and to accommodate cleaning mechanisms.

iv) Tanks should be long and narrow in order to minimize the effects of inlet and outlet disturbances, cross winds, density currents and longitudinal mixing.

A brief discussion of some of the factors mentioned above will be made in order to determine in what ways they affect the removal efficiency of a settling tank.

Flocculation in a sedimentation basin is due to two causes, namely, differences in settling velocities of the particles whereby faster settling particles overtake those which settle more slowly and coalesce with them, and velocity gradients in the liquid which cause particles in a region of higher velocity to overtake those in adjacent stream paths moving at lower velocity.
It is evident that removal of suspended matter will be increased by flocculation.

In an actual settling basin, the velocity is not uniform over a cross-section, even though uniform velocity at inlet and outlet is effected. Velocity at walls and floor of tank is zero because of frictional drag. Hence in most settling tanks the flow will be turbulent. From the results of experiments by Camp,(*7) Dobbins(*11) and others, it was shown in every case that the effect of turbulence on settling of material in suspension is independent of scour and may or may not be accompanied by bed load movement. Thus, if scour and bed load movement are reduced to the minimum, turbulence effect will not affect removal efficiency.

Currents in a settling basin contribute to short-circuiting of flow, e.g.,

i) Eddy currents which are set up by the inertia of the incoming fluid.

ii) Wind induced currents when basins are not covered.

iii) Convection currents that are thermal in origin.

iv) Density currents that cause cold or heavy water to underrun a basin and warm or light water to flow across its surface.

It is known from experiments that some fluids pass through the settling zone of a tank in less time than the detention period. This short-circuiting is due to differences in the velocities and lengths of stream paths. Short-circuiting is accentuated by mixing of the tank contents, movement of cleaning mechanisms, high inlet velocities and by density currents.

The effect of short-circuiting on removal efficiency of a tank may be adverse if strong currents travel along the bottom of the tank at such a rate that the tractive force will dislodge the particles already settled or will carry the particle along as part of the bed load. It is thus necessary to prevent or minimize the differences in the velocities at inlet. One way of effecting uniform velocities in the tank is by introducing endless cells at the inlet to damp out the inlet disturbances.

Types of Settling Tank

The settling basins in most prevalent use today are either rectangular or circular in shape. Settling tanks differ in size, in arrangement of inlets and outlets and in method of sludge removal.

It may be said that the methods of sludge removal often establish the kinds of settling tanks. The following categories can be made:-

i) Tanks that are emptied for sludge removal. These are generally rectangular in plan and cross section with gently sloping bottom. Sludge is generally allowed to accumulate until anaerobic decomposition starts. The basin is then put out of service, the supernatant is drawn off and the sludge is pumped out or withdrawn by gravity for treatment or disposal.

ii) Tanks with sludge scraper mechanisms. The scraper or plows push the sludge along the tank bottom to the sludge sump from where the sludge is withdrawn by pumping or by gravity for treatment or disposal.

iii) Tank with hydrostatic sludge removal. The bottom of the tank is formed into one or more hoppers. Sludge is withdrawn daily from the hoppers without interrupting tank operation.

In the treatment of waste, the quantity of sludges to be handled are generally so great that continuous removal of sludge from the tank is warranted.

Rectangular tanks may be in single units or in a series of units. The length of the tank is usually several times the width. The inlet to the basin is a single pipe or series of parallel inlet pipes. A portion of inlet velocity is dissipated and some horizontal and vertical distribution of flow is accomplished by means of vertical baffle. A conventional rectangular settling tank is shown in Fig. 3. The deposited sludge is moved to a hopper at one end by bottom scrapers

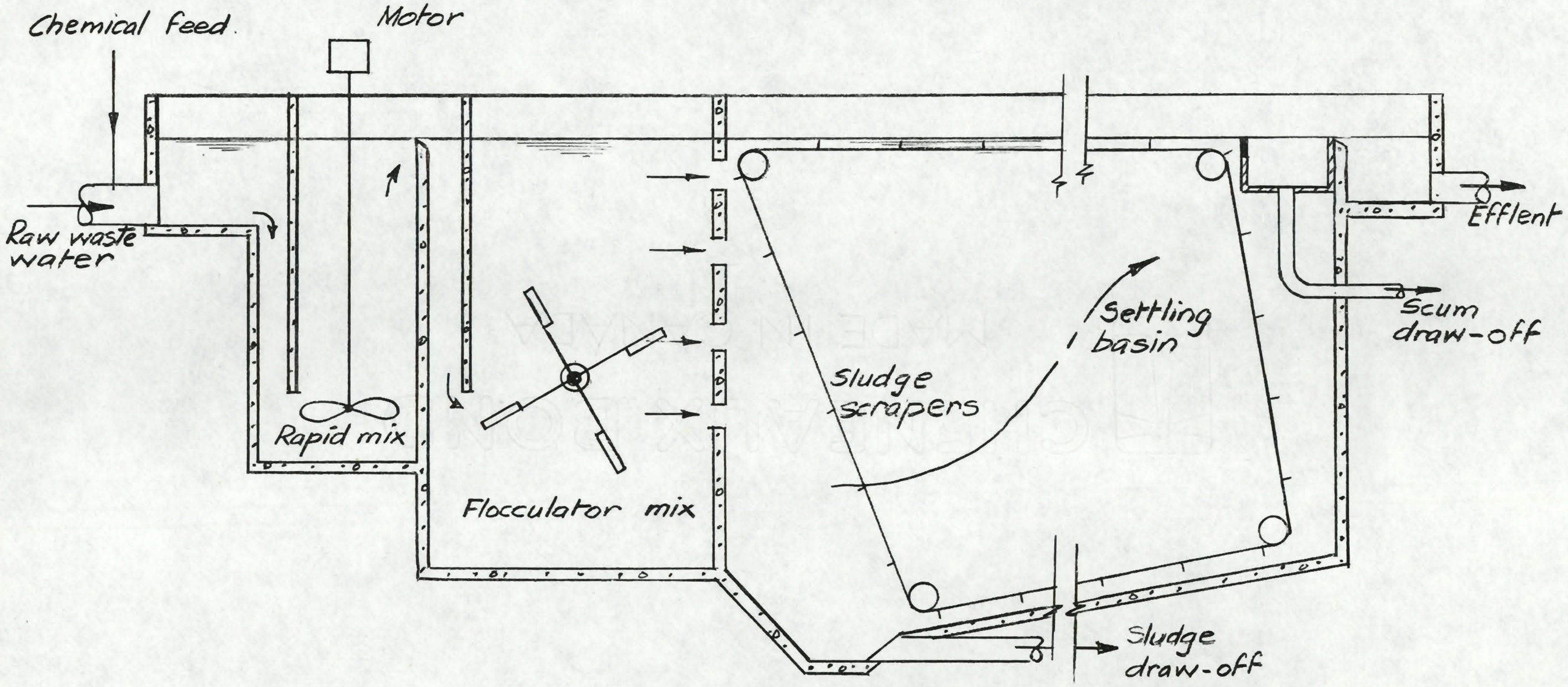


Figure 3. Schematic diagram of Conventional rectangular settling tank

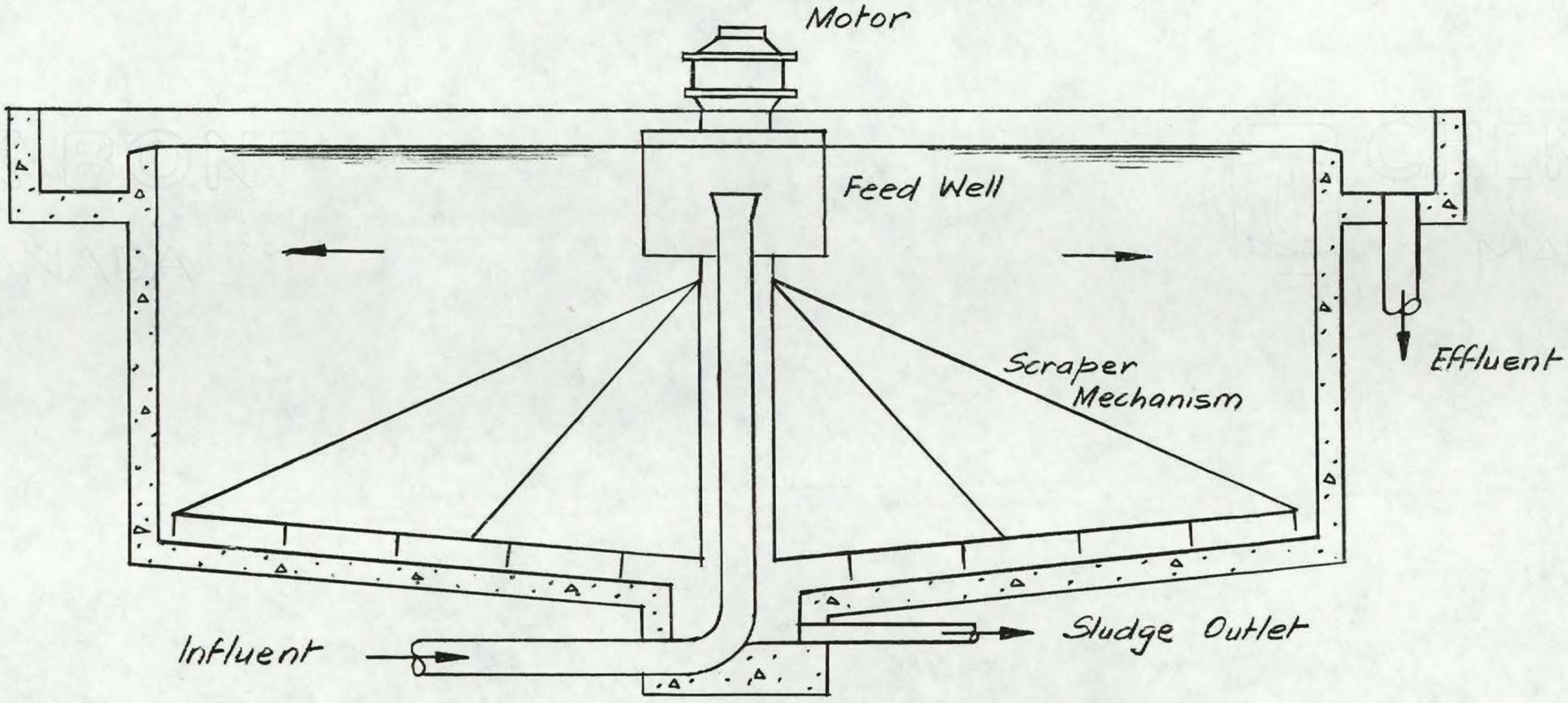


Figure 4. Vertical - Flow settling tank

mounted on a carriage moving on rails fastened to the tank walls. Floating material is moved to the skimmer by the returning scrapers. The scum is removed from the tank through a screen draw-off channel or pipe.

Circular tanks. The term clarifier often is used to describe a circular sedimentation tank . The most common type is a single circular structure in which the flow is introduced at the center into a feed well which dissipates inlet velocities. The sewage moves radially from the feed well to the weir and overflows through at the outside periphery of the tank. The settled solids are raked to the hopper near the tank center by scrapers attached to a drive unit at the center of the tank. The scraper mechanism also carries a skimming device for collecting the floating materials. Fig. 4 shows a conventional circular clarifier.

CHEMICAL PRECIPITATION

In the above discussion, it was stated that flocculation increases the removal efficiency of a settling tank. It is thus only pertinent to discuss briefly the methods used for flocculation and the chemicals most frequently applied.

In 1740 chemical precipitation was tried in Paris, France. Between 1857 and 1880 various methods were tried in England. In 1887 and 1893 chemical precipitation was used in the United States. Interest in this process in the U.S. waned after 1910. In 1929 a revival of interest appeared in the U.S. and by 1956 about twenty municipalities in Canada were known to be using chemical precipitation process.

Chemical treatment of waste water is essentially similar to the chemical treatment of water. In some cases the chemicals are introduced into the suction lines of the pumps. If close control is required the chemicals are added in a mixing chamber which is either constructed with baffled sections to maintain a high degree of turbulence or, preferably, the basin is equipped with a mechanical agitator to insure rapid and uniform mixing regardless of the through put rate through the plant. After addition of chemicals, a period of slow stirring is necessary for build-up of floc particles so that sedimentation can be effective. The detention time will normally range from two to six hours for a rectangular sedimentation basin.

Modern developments have been made to combine mixing, flocculation and sedimentation in one single basin. A single agitator mixes the chemicals and raw waste with previously formed sludge. This aids in more rapid floc formation and greater reduction in detention period required for treatment. Among units used today are Accelator, Precipitator and Reactivator.

Clarification of waste is ineffective unless good floc formation can be obtained. Chemicals used include ferric chloride, ferrous sulphate, alumina sulphate, ferric sulphate, chlorinated copperas, calcium hydrate, calcium oxide, sulphuric acid, sulphur dioxide, calcium hypochlorite, magnesium sulphate and liquid chlorine.

Alum, technically known as aluminum sulphate, is not commonly used in the straight coagulation of waste, but extensively used for the clarification of certain industrial wastes. Alum and activated silica are frequently effective although the dosage required may be much higher than normally used in water treatment.

Ferric chloride is possibly more extensively used than any other chemical for precipitation. It is highly corrosive and is supplied in liquid or crystal form or it can be made locally by the Scott-Darcy process which involves the treatment of scrap iron with a solution of chlorine.

Chlorinated copperas is usually made at the works by treating ferrous sulphate with chlorine. It is invariably used in conjunction with lime.

Calcium hydrate is employed for a variety of reasons and is usually used with the ferrous salt to combine with the excess mineral or acid salts. Occasionally it is used for straight precipitation in which the carbonates normally present in the sewage are precipitated along with much of the coagulable matter in suspension.

Epsom salts or other magnesium compounds are frequently effective, particularly if the waste has a high pH value.

Chlorine. The application of chlorine either in the form of bleaching powder or as chlorine gas has been most consistently successful in sewage treatment.

A few years ago it was thought that chemical treatment would outpace all other types of purification but this prediction did not materialize. It seems probable that the high cost of chemicals, coupled with the fact that efficiency of sewage treatment is largely dependent upon the correct application of chemicals which normally involves high maintenance cost, is one of the big reasons for the slowing up in straight chemical treatment.

In the foregoing chapter, plain sedimentation and chemical precipitation have been investigated in some detail. Hazen, Camp and many others in the course of their studies made references to the application of their theories to sedimentation of sewage solids. It must be said that waste water is an extremely variable material changing in quantity and quality and in various characteristics which affect its settleability hourly, daily and seasonally. The results of the application of a purely theoretical basis of analysis to a particular sewage under certain fixed or assumed conditions is valueless, and even misleading when considered in terms of a varying material under varying external influences. It is worth quoting the observation made by Professor Schroepfer(*33) after his studies of 30 sedimentation plants scattered throughout the United States.

> "Despite the apparent simplicity of the theories of sedimentation and the clear relation of overflow rates to removal ratios, attempts to compare the performances of settling tanks in sewage works have been disappointing. There appears to be little or no correlation in actual operating records between removal efficiencies and overflow rates or detention periods."

CHAPTER IV

SLUDGE TREATMENT & DISPOSAL

Sludge is the settleable suspended solids removed from the waste water by sedimentation. When regularly removed from the sedimentation tank the sludge is termed fresh since it has not undergone decomposition. Its quality depends upon many factors among which are:

i) Composition, state, strength, constituents and settleability of the waste water entering the tank.

ii) Characteristics and design of the settling tank.

iii) Methods of settling tank operation.

Sludge creates a most difficult problem in waste disposal because its polluting potential is greater than that of liquid fraction. The amount of sludge to be treated and disposed of is great due to its volume. Sludge forms loose structures of particulate and flocculant matter with included water. The pore structure and space is large, and therefore the water content is high, producing many times the volume of the solids actually, forming the sludge.

The observation of Van Kleeck(*39) on the quality of raw sludge settled from waste largely of domestic origin is summarized in Table II.

Table II

Characteristics of Raw Sludge from Plain Sedimentation Tanks

Quality	Description	Remarks
Physical texture	Non-uniform, lumpy	-
Color	Brown	Dyes from industrial waste may color the sludge or it may be dark gray or black in color if septic.
Density	Average solids content of 3 to 8%	A good quality for drawing should average 5 to 7% solids.
Odor	Normally offensive	May have little or no odor in presence of such industrial wastes as metallic salts.
pH	Usually range 5.5-6.5 average value 6.0	Affected by industrial wastes and septicity of sludge.
Total alkalinity	Range 300-1000 mg/l, usually 500-800 mg/l	-
Mineral or non-volatile solids content	Around 30% of total dry solids	If appreciably more than 30% presence of grit should be suspected.
Septicity	Non-septic	Septic raw sludge is, however, rather common.
Sludge volume	From 70-700cu.ft/mill. gal. of sewage treated. Average about 250-350 cu.ft/mill. gal.	High volumes indicate too thin a sludge is being drawn. Low volumes indicate weak sewage, poor tank efficiency or sludge building up in tank.
Digestibility	Normally easily digested.	Non-digesting raw solids indicate harmful industrial wastes.
Grease content	Normally 15-20 mg/l.	Appreciably higher figures are indicative of industrial wastes.
Drainability on sand beds.	Poor	-

Characteristics and quantity

Standard Methods(*1) for the examination of water and waste water prescribe a number of test procedures for sludge analyses. The main items of interest here are specific gravity, moisture and solids contents, solids characteristics, acidity, alkalinity, nitrogen, digestability and volatile acids. Numerous investigations have supplied enough of generalized data for estimates of quantity and quality applicable to domestic waste. Certain properties of sludge deserve a special consideration.

From the engineering viewpoint the moisture content of sludge is its most important characteristic. The solids from the settling tank form liquid sludge with a 97% to 99.5% moisture content. It is obviously desirable to reduce the moisture content to facilitate handling and disposal of the sludge.

If sludge contains a weight of W_s solids and weight of W of water, then the percentage moisture content

$$P\% = \frac{W}{W + W_s} \times 100 \tag{11}$$

Sludge solids content then is

$$(100 - P)\% = \frac{W_s}{W + W_s} \times 100 \tag{12}$$

Specific gravity of dry sludge is given as

$$S_s = \frac{100\, S_f\, S_v}{100 S_v + P_v\, (S_f - S_v)} \tag{13}$$

in which S_s = specific gravity of dry sludge
S_f = specific gravity of fixed solids, usually 2.5
S_v = specific gravity of volatile solids, usually 1.0
P_v = % of volatile solids in sludge

With the above values, equation (13) becomes

$$S_s = \frac{250}{100 + 1.5P_v} \tag{13a}$$

Specific gravity of wet sludge may be given as follows:

$$S = \frac{100\ S_w S_s}{P S_s + (100 - P)S_w} \tag{14}$$

and as $S_w = 1$, usually, hence equation (14) becomes

$$S = \frac{25{,}000}{250P + (100 - P)(100 + 1.5P_v)} \tag{14a}$$

Volume of wet sludge is given as

$$V = \frac{W_s}{W} \cdot \frac{100S_w + P(S_s - S_w)}{(100 - P)\ S_s\ S_w} \tag{15}$$

Dewatering of sludge will reduce its volume, and adding subscript zero to the designation values above, the volume of partially dewatered sludge V, may be expressed as follows:

$$V = V_o \frac{100S_s + P(S_s - S_w)}{100S_w + P_o(S_s - S_w)} \cdot \frac{(100 - P_o)}{(100 - P)} \tag{16}$$

When specific gravity of total sludge solids is close to specific gravity of water, or unity,

$$V = V_o \frac{100 - P_o}{100 - P} \tag{16a}$$

The equation (16a) indicates that the loss of water from sludge changes its volume approximately in the ratio of its solids concentration. It is thus advantageous to reduce the moisture content as much as possible in order to reduce sludge volume and effect economy in sludge handling.

Measurement of the quantities of sludge that are produced by different treatment processes is a routine undertaking of waste water treatment works. Estimates of expected quantities may be made by analyses of the waste water and from a knowledge of the removals of solids effected by different treatment processes and the changes produced by different sludge-treatment methods.

Estimates of the associated volumes must depend upon general information relating to the water or solids content of the sludge and to the proportions and specific gravities of the constituent volatile and fixed solids.

Calculation of actual quantities of sludge rests fundamentally on the base values for the composition of domestic waste water given in Table XXVI p. 190 and the efficiency of removals mentioned earlier in Chapter II.

The commonly observed weights and volumes of domestic waste water sludge are shown in Table III.

Sludge treatment methods

The basic methods employed in sludge treatment are varied and discussion of some of them will be given.

TABLE III

Weight and Volumes of Domestic Waste Water Sludge from Plain Sedimentation(*20)

Waste water flow = 100 gals. per capita per day.

	DRY SOLIDS			WET SLUDGE			Fuel value
	grams /c/d	lb/100 person per day	lb/mg sewage	Solids %	cu.ft/1000 persons daily	gal./mg. sewage	BTU/lb of dry solids
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)
1. Wet primary thin	54	119	1190	2.5	76	4,700	8,000
2. Wet primary thick	54	119	1190	5	38	2,360	8,000
3. Wet digested	34	75	750	13	9	584	4,000
4. Air dried digested	34	75	750	45	4.5	250	4,000

(c) = $\frac{1000 \times (b)}{454}$ (d) = 10 x (c). (e) Common values. (f) Closely $\frac{(c)}{0.624(e)}$

(g) = 6.25 x 10(f) (h) Commonly observed volatile matter content.

In checking the sewage sludge at Beaconsfield for the present population of 12,000, values for wet digested sludge were taken, giving:-

wt. of sludge on dry basis = 900 lbs.
wt. of wet sludge = 674 gallons

The calculated weight of wet sludge compares favourably with pumping test* carried out in August 1963.

Fuel value = 4,000 x 900 = 3,600,000 BTU.

---

* Table No.XIII shows pumping periods investigated.

1. Sludge Thickening.

Means of sludge thickening are often used to reduce the cost of handling and disposing of voluminous quantities of sludge. The water content in the flocculent mass is hard to remove, and even after thickening it is very high. The ultimate concentration obtained is effected by various factors among which are,

i) Nature of settleable solids, their density, shape, flocculent structures, electrostatic charges and other surface characteristics.

ii) The concentration of settleable solids in the original suspension.

iii) The dissolved substances in the intersticial blanket.

iv) Temperature

v) Depth of the sludge blanket.

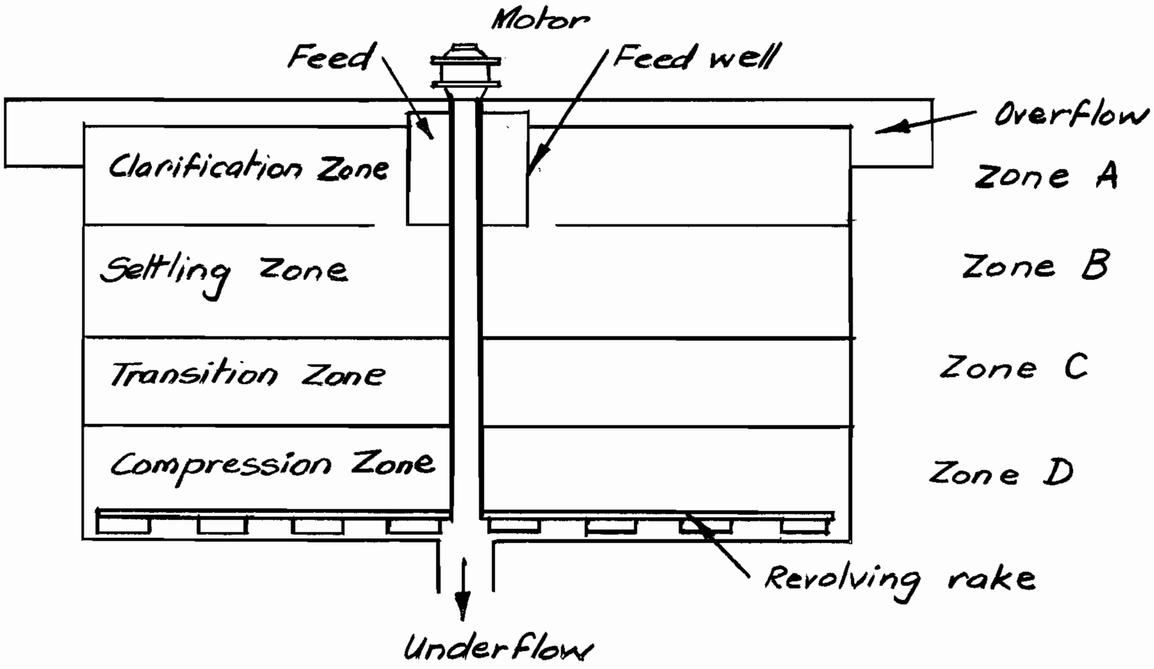
vi) Surface area of the blanket.

vii) Time allowed for compaction of sludge.

viii) Structural modification by mechanical action.

a) Settling may be defined as the reduction of moisture content of a semi-liquid mass by gravity sedimentation. This process is carried on in a thickener unit.

A thickener normally operates as a continuous process unit with a sludge inflow near the top, a thickened underflow at the bottom and a clarified flow a short distance above the inlet. The depth of the thickener may be considered to be divided into four horizontal zones, located one above the other as shown in Fig. 5.



Four Zones in Continuous Thickener

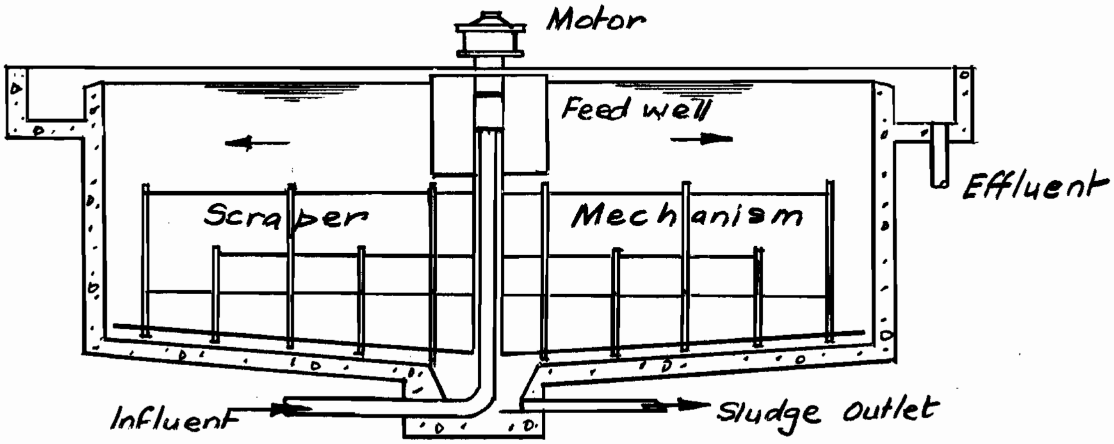


Figure 5. Representative design of Thickener

Zone A extends from the feed level to the overflow level and is called the clarification zone. During steady state operation the larger part of the liquid in the feed rises through the clarification zone and overflows the weir. The rest of the liquid and the solids move downward and are discharged through the bottom.

Zone B extends from the level where the feed is introduced at the bottom of feed well to the top of the transition zone. The feed well may be said to permit dilution of the feed slurry with the clarified liquid in Zone A. Zone B may be called the zone of uniform settling rate. The solids in the feed slurry settle through this zone to the transition and compression zone.

Zone C is the transition zone, and it is the zone in which the particle concentration changes from hindered settling to compression zone settling.

Zone D is the region of compression, generally the concentrated slurry in this zone is moved to a central discharge by the gentle action of a rake mechanism.

The solids content of the sludge influent into the thickener will generally range from about 800 to 5000 parts per million, and the volumetric concentration of the particles is so great that the initial subsidence represents a process of hindered settling.

A discussion of the mechanism of settling was first described in 1917 by Coe and Clevenger(*9). They stated that when a suspension is allowed to settle quiescently in a container the interface between liquid and solids falls at a uniform rate. At the same time the compression zone builds up from the bottom of the container. After a period of time, the rate begins to diminish, indicating that the interface has reached the top of the transition zone of variable composition. The decline in settling rate continues, and when the interface reaches the top of the compression zone, further fall results from a consolidation of the suspension. Application of rake to improve thickening in the compression zone was later demonstrated by Bull and Darby(*6) in 1926, and subsequently verified by others including Kammermeyer(*22) in 1941 and Commings(*10) in 1954.

In 1952, Kynch(*25) presented a theoretical analysis of the thickening process based on the assumption that the velocity of a settling particle is a function solely of the concentration of particles in the immediate locality.

Talmage and Fitch(*38) later in 1956 applied the results of Kynch analysis to the design of thickener and developed method for determining the required cross-sectional area.

In order to understand the mechanism of thickening process the zones B and D in Figure 5 will be considered separately under the headings:-

Hindered settling. In the settling of particles in a sedimentation tank, Stokes(*37), Camp(*7) and others based their work on free settling, but in the thickening process, settling of particles cannot be called free settling. Close proximity of particles in the interface leads to a rate of settling less than that predicted by Stokes(*37) Law. Settling is therefore considered as hindered. Empirical relationships for this phase of settling process were developed by Kermack, McKendrick and Ponder(*23) while others like Egolf(*12) and Robinson(*31) developed semi-empirical formulae by simply modifying Stokes'(*37) Law for the settling velocity of a particle.

Robinson(*31) proposed:-

$$\frac{dH}{dt} = K' \cdot d^2 \frac{(\rho_s - \rho_c)}{\mu} \tag{17}$$

in which H = height of sludge column, slurry interface
t = time
K' = constant
ρ_c = density of suspension
μ = viscosity of suspension

The above equation merely alters Stokes' Law in its final form and when dealing with waste water, both values of "d" and $(\rho_s - \rho_c)$ are questionable entities.

The work of other researchers frequently is presented in the following form.

$$\frac{v}{v_c} = f(e) \tag{18}$$

in which v_c = the hindered velocity and
f(e) = a function of e the fractional volume of dispersion medium (dimensionless)

Steinour(*36) for example, found $f(e) = e^2 10^{-1.82(1 - e)}$

Fair and Gayer(*13) suggested $f(e) = e^{4.5}$ for suspension of discrete particles.

Comings(*10) utilized the relationship

$$\frac{v}{v_c} = \frac{\mu_e}{\mu_c}$$

derived from Robinson's(*31) equation and Stokes' (*37) Law. μ_e from above is the viscosity of the dispersing medium.

A study of the experiments made by Rudolfs(*32) and Camp(*7) indicated that for a mixed liquor of 10,000 ppm, hindered settling persisted between velocities of 0.008 to 0.15 centimeters per second whereas for mixed liquor of 2,000 p.p.m., free settling persisted in the range of 0.14 to 0.27 cm. p.sec. In a later experiment by McNown(*27), he showed that settling velocity in suspension of 1,000 p.p.m. by volume is decreased by as much as 13% whereas at a volumetric concentration of 10,000 p.p.m. the settling velocity is decreased by as much as 25%.

A review of Knych(*25) analysis will perhaps help in understanding the quantitative mechanism of settling of particles in zone B.

If one considers a thin layer in the transition zone of Figure 5 with concentration C moving up with velocity $\overline{U}$ ft/sec., it follows that the particle settling into this layer will have a concentration of (C - dc) lb/cu.ft. and a settling velocity of $(v + dv + \overline{U})$ ft/sec. with respect to the layer. The concentration of solids settling out of this layer will be C with settling velocity $(v + \overline{U})$ ft/sec. with respect to the layer.

Since the concentration of the layer is constant, the quantity of solids settling into the layer must be equal to the quantity of solids settling out of the layer and a material balance can therefore be made.

$$(C - dc)\,A\,.\,t\,(v + dv + \overline{U}) = C\,.\,A\,.\,t\,(v + \overline{U}) \tag{19}$$

in which

A = area of transition layer at time t.

Simplifying and solving for $\overline{U}$ (neglecting second order terms

$$\overline{U} = C\frac{dv}{dc} - v \tag{20}$$

According to Kynch's postulate $v = f(\text{concentr. } C)$ and now through substitution

$$\overline{U} = Cf'(C) - f(C) \tag{21}$$

From above, the concentration of the layer, C = constant, and, since $f(C)$ and $f'(C)$ have fixed values, $\overline{U}$ also must remain constant. It is a fact that $\overline{U}$, or the rate at which the layer of concentration C travels up through the suspension, remains constant. This fact can be used to determine the solids concentration of the layer at the upper boundary of a settling suspension.

Suppose H_o and C_o are the initial height and concentration of a slurry in a batch settling. The total weight of the suspended solids in the slurry is $H_o\,.\,A\,.\,C_o$. If C_2 is the concentration of the layer reaching the liquid solid interface at time t_2, then the quantity of solids that has passed through this layer is $C_2A.t_2(v_2 + \overline{U}_2)$. This must equal the total weight of solids in the column,

$$C_oH_oA = C_2A\,t_2(v_2 + \bar{U}_2) \tag{22}$$

The rate at which any layer propagates upward has been found to be constant. Therefore, if the interface at the time t_2 is at a height H_2,

$$\bar{U}_2\, t_2 = H_2$$

Substituting this in the equation (22)

$$C_2 = \frac{C_oH_o}{H_2 + v_2t_2} \tag{23}$$

If the initial concentration of a batch settling rate is known, curve can be constructed showing height of solid-liquid interface versus time and from the graph the economic height H_1 can be found.

$$H_1 = H_2 + v_2t_2$$

This is the height the slurry would occupy if all the solids present were at the same concentration as the layer of solid-liquid interface.

Equation (23) becomes

$$C_2 = \frac{C_oH_o}{H_2 + H_1 - H_2} \qquad \text{hence,}$$

$$C_2H_1 = C_oH_o \tag{24}$$

Talmage and Fitch (*38) followed up Kynch's theory (*25) and utilized equation (24) to obtain

$$A = \frac{t_u}{C_o H_o} \tag{25}$$

in which

A = the unit area of a layer in a continuous flow thickener required to pass one ton of solids of concentration C_2 in one day. t_u is the critical time taken for the quantity of solids in a batch test to subside past the layer of concentration C_2 in a continuous flow thickener.

In 1963 Behn and Liebman (*4) also applied equation (24) to the design of thickeners and obtained

$$A_2 = \frac{Q_o C_o}{v_2} \cdot \left[\frac{1}{C_2} - \frac{1}{C_u} \right] \tag{26}$$

in which

Q_o = the volumetric feed rate in the continuous flow thickener.
C_u = the underflow solids weight concentration.

<u>Compression zone</u>. When hindered settling progresses to the stage when the particles in suspension are in mutual contact, further settling bears little relationship to Stokes'(*37) Law.

Roberts(*30) developed an empirical formula for analyzing the compression zone as follows,

$$(D - D_\infty) = (D_o - D_\infty)\, e^{-kt} \tag{27}$$

in which

D = dilution factor at time t, weight of fluids divided by weight of solids
D_o and D_∞ are dilution factors at time zero and infinity.
K = constant.

In 1953 Behn(*4) obtained the equation

$$\log (D - D_{\infty}) = K t \log (D_o - D_{\infty}) \tag{28}$$

in which the constant

$$K = \frac{K^*(\rho_s - \rho)g}{H_c W} \tag{29}$$

where K^* = permeability constant

From the foregoing discussion a qualitative and quantitative mechanism of suspension thickening has been presented. The behaviour of solids during settling is considered to be hindered due to high solids concentration in the liquor, the rate of settling being less than Stokes' terminal velocity(*37) of a single particle.

Hinderance is deemed due to mutual interference of the particles. The ratio of free to hindered settling velocity has generally been taken as a function of the fractional volume of dispersion medium. From the hindered zone the particles settle through the transition zone to the compression zone. Theoretical analysis of the zone mechanism based on works by Kynch(*25), Talmage(*38) and others have been used in the field of thickener design, although in actual thickener operation the four zones are not distinct. Experiments on actual thickeners have shown that the passage from the hindered settling zone through the transition zone to the compression zone is not well defined.

A detailed discussion of the thickening process has been made because of its relevant bearing on the experimental investigation carried out in this work.

(b) Pressurized flotation. The process employs the use of pressurized air for separation of solids from sludge with or without the use of coagulating chemicals. When chemicals are used these react with alkaline substances in the sludge to produce numerous small gas bubbles which cause the sludge particles to rise. Up to date there has not been enough investigations and operational experience for effective evaluation of this process.

2. Sludge Conditioning.

The term sludge conditioning as commonly used in waste water treatment relates to a process by which waste sludge is conditioned. Two methods of sludge conditioning are generally recognized:

a) Chemical coagulation. When certain chemicals are added to raw sludge they form relatively insoluble flocs which coalesce suspended and colloidal particles in the sludge. This action facilitates the extraction of moisture from the sludge by filtration. The chemicals used are the same as those described previously in chapter III. Ferric chloride is, however, the most widely used coagulant for chemical sludge conditioning.

b) Elutriation. This process was first studied by Genter(*16) and comprises essentially the removal of soluble decomposition products normally present in sludge by washing with water comparatively free of such compounds. In some cases washing of the sludge is done with plant effluent using mechanical or diffused air agitation. There are four steps in the process,

i) The sludge is diluted with water.

ii) The water and sludge are mixed thoroughly and rapidly to form a weaker solution of the dissolved decomposition products.

iii) The washed sludge is removed from the mixture by sedimentation.

iv) The weaker solution is decanted and usually returned to the primary settling tank.

Elutriation may reduce the alkalinity of the sludge and therefore less coagulant would be required for further conditioning.

3. Digestion

The most common method of processing sludge is by digestion of the organic solids resulting in stable matter which can be economically disposed of by drying or burning without creating a nuisance. Digestion is accomplished in covered heated or unheated digesters.

The earliest large scale tanks for digestion of sludge were built of earth at Birmingham, England in 1913. Open masonry tanks without mechanical equipment were built in Baltimore, United States of America, in 1910. After World War I experiments were made in the United States with mechanically cleaned digestion tanks; first uncovered and then with fixed covers for the collection of gas. Since the first heated digestion tanks went into operation in the United States about 1926, the process has grown in popularity and is in use in most waste treatment plants in North America.

A brief resume of the mechanism of digestion may perhaps prove of assistance in this respect.

Raw or concentrated sludge contains abundant food for bacteria and other microorganisms which are already present in the waste. These living microorganisms break down complex organic matter into simple, more stable substances. In the absence of oxygen within the closed digester, only the facultative microorganisms survive and multiply. In raw domestic sludge the volatile content is about 70% and the inorganic content is 30%. Much of the water in the sludge is "bound" water which will not separate from the sludge solids. The facultative microorganisms break down the complex molecular structure of these solids, setting free the "bound" water and obtaining oxygen and food for their life existence.

The initial attack is brought about by extracellular enzymes elaborated by the bacteria. These enzymes hydrolyze the organic matter to simple soluble compounds which are utilized in the synthesis of protoplasm of the bacteria. The cellulose and starches are hydrolyzed to the simple sugars, while the proteins are broken to the amino acids. Only the fatty acids are not attacked by the extracellular enzymes.

In the anaerobic environment the bacteria do not have an unlimited hydrogen acceptor and the extent of metabolism is definitely limited. The final hydrogen acceptors are carbon, sulphur, nitrogen and phosphorous. The pattern of metabolism leads to the formation of acids, which results in lowering of the pH as the concentration becomes sufficiently high. The build-up of acid end products has resulted in the first phase of the digestion process to be known as the acid phase.

The high acid concentration retards further bacterial metabolism because of the low pH and the build-up of an end product in the system. Essentially the biological system approaches equilibrium. A second group of bacteria develops which can utilize the organic acids. As this group increases, the acids are metabolized to carbon dioxide and methane. Metabolism of the amino acids results in liberation of ammonia, which in turn neutralizes a portion of the remaining acids. In this way the pH rises thus creating favorable living conditions for bacterial growth. This second phase of digestion is known as the methane phase.

The methane fermentation quickly lowers the excess acids and permits further degradation of the more complex organics. The methane bacteria break down the fatty acids, using carbon dioxide as their hydrogen acceptor and water as the oxygen donor. The acetic acid is also broken down to methane and carbon dioxide by direct metabolism.

The third phase of digestion is a period of intensive digestion, stabilization and gasification. In this phase the more resistant nitrogenous materials such as higher proteins and amino acids are attacked by the bacteria given off methane and carbon dioxide.

As long as the digester is able to maintain a balanced bacterial population of acid formers and methane formers, there are few operating problems. The organic matter added to the digester would be quickly converted to methane and carbon dioxide. Unfortunately a sudden addition of a large quantity of readily degradable organic matter quickly results in the formation of excess acids with its resultant depression of pH and bacterial activity.

The microorganisms are highly specialized bacteria. The most active bacteria are the facultative bacteria and the obligate anaerobic bacteria. The acid formers are mostly the facultative bacteria. The methane formers are a small specialized group of bacteria that are obligate anaerobes. There is a third group of bacteria which occurs in anaerobic digestion, the sulphur bacteria. These bacteria utilize sulphates as their hydrogen acceptor with the production of hydrogen sulphide as the reduced end product.

Once the microorganisms have finished digesting the organic solids, the sludge is removed from the digester and dewatered by gravity or vacuum filtration. The dried sludge can then be disposed of on the land where microorganisms break down the small quantity of organic material left.

Considering the microbiology of anaerobic digesters and the biochemical reactions they bring about, it should be possible to postulate how a digester should be constructed to yield the maximum stabilization for the least effort. The few changes in anaerobic digesters in the past few years have shown that by making the environment more favorable it has been possible to reduce the digestion period down to as low as six days.

The anaerobic digester should be operated as a complete mixing type of treatment unit in order that microbial population could be kept relatively uniform, both in total mass and in species. This means uniform feeding and complete mixing at all times in all points of the tank. Constant feeding helps to eliminate shock loadings and hence sudden increases in volatile acids. Once the organic matter has been added to the digester, it must be completely dispersed to all points in the tank. This can be accomplished only with high-speed mixing.

The rate of biological reaction is clearly a function of temperature. This has been shown in anaerobic digestion where heat is added to elevate the tank temperature to between 33° and 37°C. One of the most important advances in heating digesters has been the development of the external heat exchanger to replace the internal hot-water coils. One of the things that external heat exchangers opened up for reconsideration is the case of thermophilic digestion. Early research had shown that thermophilic digestion at 55°C was much more rapid than at 35°C, but it was not practical because of the high rate of sludge drying on the heating pipes. With external heat exchangers and high sludge recycle rates it is possible to raise the temperature of the digester to 55°C.

The results of a year's operation as reported by Anderson(*3) indicated that the high-rate digester would digest raw sludge at loading rates in excess of three times those possible with conventional digesters.

4. Sludge Dewatering

The concentrated sludge obtained by utilizing thickening, conditioning and digestion methods, contains too much water to permit economical and convenient final disposal. Moisture may be further removed by employing mechanical methods.

Some of the dewatering processes presently in use will be described below:

a) Vacuum filtration. The process is essentially the removal of moisture from a layer of sludge. The sludge is drawn by suction onto a porous medium usually a cloth of cotton wool, synthetic fibres, plastic or a stainless steel mesh, or, a double layer of stainless steel coil springs. A spadable, friable sludge cake is finally produced and this can be heat dried, incinerated or used as fill.

The successful dewatering of sludge by this process requires proper conditioning of the sludge prior to filtration. The choice of conditioning methods and amounts of chemicals is generally predicted by performing filtration tests in the laboratory.

The rotary continuous vacuum filters are generally classified into the drum type and the horizontal belt type.

The drum type is essentially a drum over which is laid the filtering medium. The filtering medium is cotton or woolen cloth of suitable weight and weave. The cloth is stretched and wired over a supporting layer of copper mesh which covers the sides of the drum and overlies a series of cells running the length of the drum. These cells can be placed under vacuum or pressure.

The drum is suspended horizontally so that about one quarter of its diameter is immersed in a trough containing sludge slurry. A vacuum of sufficient magnitude is applied to the submerged cells to attach a mat of sludge of suitable thickness to the cloth. The emerging mat is placed under a drying vacuum and the sludge liquor is drawn into the vacuum cells and drained from them for treatment and disposal with other sludge liquors.

The dried cake is removed from the drum by a scraper just before it enters the sludge trough again.

The horizontal belt type is manufactured in Canada and closely resembles a conveyor belt. At the wet end, coagulated sludge with a moisture content of 95% to 96% is introduced. This sludge is retained on an endless belt of woolen filter cloth. The filter medium bearing its sludge load and supported by a rubber suction mat moves across a battery of vacuum exhaust ports. These ports are of equal width to the filter cloth and suction mat. They are joined to a common vacuum and filtrate liquor manifold.

b) Centrifuging. This is a liquid-solid separating process which has long been widely accepted by industries for sludge dewatering. Up to the present time there are about 20 waste water treatment plants in North America employing centrifuges. It can therefore be said that sludge dewatering by this process is still in the experimental stage. There are a few known types of centrifuges now in operation, among which are solid bowl centrifuge and sonic screens.

The Bird continuous solid bowl centrifuge consists essentially of two rotating elements. The bowl is driven by a motor and the conveyor is driven through a gear system that revolves the conveyor at slightly lower speed than the bowl. Sludge is fed through a feed pipe inside the conveyor from where it passes through ports into the pool of liquid maintained in the bowl. Solids are settled against the wall of the bowl by centrifugal force and picked up by the screw conveyor which moves it across to the discharge end. The clarified liquid leaves by way of effluent ports in the bowl. White(*40) in his experiment of this type of centrifuge showed that a concentrated sludge of 28 to 32% solids contents were obtained.

The sonic screens method of dewatering of sludge is relatively new and is in the experimental stage. When raw sludge is passed through a sonic screen apparatus, dewatering results from longitudinal and transverse electromagnetic vibrations in the sonic range. The transverse waves produce an extremely rapid dewatering rate while the longitudinal waves agglomerate and concentrate the solids. The basic unit consists of screens vibrating in several planes simultaneously. One installation used in the Wuppertal-Buchenhofen plant in West Germany(*24) consists of the following separate pieces of apparatus:

i) Coarse screen with 8 x 24 mm. mesh.

ii) Sonic wet screen with mesh varying as selected from 2 to 12 mm.

iii) Three sonic screens with square mesh varying between 0.5 mm to 0.1 mm.

iv) Roll press with recirculation system of the press water.

Keiss(*24) reported that from tests carried out on the installation, the coarse screen removed 2% of the total solids; the sonic wet screen removed 23% of the total solids, while the sonic screens removed 55% of the total solids. The discharged sludge contained approximately 75% to 80% water and this was further pressed to produce a further reduction in moisture content to about 60% to 65%.

The results of many experiments carried out on the above mentioned centrifuges have shown that moisture content may be reduced to a point where the total solids content of the dewatered sludge is in the range of 65 to 70%. Frequently however the concentrates from these processes have relatively high solids content compared with the filtrate from vacuum filters.

c) Freezing. The freezing of sludge to facilitate further dewatering has not been practised in North America. Nevertheless, many a waste treatment plant operator has observed that frozen sludge dewaters very quickly and after thawing the solids make humus, having lost the sticky characteristic of partially dried sludge. It has been speculated that the freezing action breaks the cell walls that retain the internal moisture in sludge.

5. Sludge Drying and Incineration

The principal bases of design for units devoted to sludge drying and incineration are as follows: The quantity and characteristics of the sludge, the fuel available, the schedule of operation, and the proposed method of handling and storing materials and disposing of the end products. Among the methods used today for sludge drying and incineration are the following.

a) Air drying. This is a process whereby sludge is air-dried on beds of sand or other granular materials. The sludge depth depends on the kind and character of the sludge. As the liquor percolates through the sand, evaporation becomes an important part of the dewatering process and helps to dry up the sludge. The dried sludge reaches a moisture content of 50% to 60%, shrinks about 65% in bulk, and may be removed for final disposal after ten days to three weeks.

The beds usually consist of a bottom layer of gravel of uniform size about one foot deep, over which is laid, 6 to 9 inches of clean sand. Farm tile underdrains are laid in the gravel layer in a manner which provides positive drainage. Most sand drying beds are used in conjunction with small waste water treatment plants where mechanical dewatering processes are too expensive, and adequate economic and soil conditions exist.

b) Heat drying. Sludge mostly used for commercial fertilizers are often dried by heat in a drier. A concentration of 50% to 90% solids content may be obtained. There are two types of driers used, the kiln drier common to the lime and cement industry, and the flash drier or cage mill system. Drying in all cases is promoted by adding a sufficient amount of previously dried sludge to the incoming cake in order to reduce the concentration of the mixture to about 50%.

The purpose of heat drying of sludge may be summarized as follows:

i) To reduce the moisture content of the dewatered sludge so that its volume is reduced to a practical minimum for economical disposal.

ii) To retain the fertilizing properties of the dewatered sludge.

iii) To destroy microorganisms capable of producing diseases.

iv) To retain and improve its soil conditioning properties, and

v) To reduce odors.

The rotary kiln drier consists of rotating cylinder revolving at speeds of 4 to 8 r.p.m., with heated gases entering at one end with the sludge cake. Baffles or flights help break up and mix the drying sludge as it passes from inlet to outlet of the kiln. The dried sludge is granular in shape and may contain clinker-like masses which are ground before use as a soil conditioner.

The flash drier subjects the cake to high temperatures for a short period of time. The cake is then mixed with some dry sludge and fed into the drying system. Here the mixture is moved at a velocity of several thousand feet per minute in a stream of gas having a temperature of about 1000°F. The passage through this high temperature zone takes a few seconds. During this time the moisture is reduced to about 10%. The most widely used means of collecting dried sludge from the system are high-efficiency centrifugal dust collectors. These units usually consist of a large number of small diameter cyclones which give high efficiencies even on the fine dust obtained from the combustion.

Stacks are used to disperse the gases given off from the system. The exit temperature is about 180°F and none of the fertilizing properties in the original cake are known to be destroyed. The dried sludge may be disposed of as soil conditioner or incinerated for conversion to ash.

Some recent innovations of heat treatment of sludge are spray drying, and the Porteous process.(*5)

Spray drying of wet sludge in a heated tower has been tried with indifferent success. The process described by Dr. Gauvin(*15), consists of a spray evaporator 5 feet internal diameter by 7 feet 6 inches high. Hot gases at 1000°F and a 4% sludge injected at the throat of a venturi, enter at the top of the evaporator. Water is evaporated from the atomized particles and passes off with the hot gases. The exit temperature of the spent gases is about 300°F. The dried sludge dropping to the bottom of the tower have about 11% solids concentration and this may be further incinerated or used as a fertilizer.

The Porteous process is an English development that has met with considerable success in England. The process takes it name from W.K. Porteous, the inventor, and essentially depends on steam heating the sludge about 15 minutes. This action alters the gel-like structure of the sludge, thereby reducing hydration and water affinity of the solids. The heated sludge mixture can therefore be separated very easily by simple settlement and decantation. The thickened residue can be further dewatered by filter-press methods. Among the advantages attributed to the process are:-

i) Freedom from weather conditions.

ii) Freedom from waste sludge smells.

iii) Low plant and operation costs.

iv) No chemicals required for sludge conditioning.

v) Elimination of drying beds.

vi) Complete sterilization of all sludge.

c) Incineration. Attempts to burn air-dried sludge out of doors have been carried on for over 60 years but have met with limited success only, due to problems encountered with respect to smoke, odor, and weather conditions. After the First World War attempts were made to burn waste sludge in combination with other wastes or combustible materials. The first few years prior to 1935 saw the first effective and practical work done to burn sludge in an incinerator. After 1935 some municipalities in North America used incineration process as the method of final sludge disposal. In fact there is no known practical method today that so nearly approaches the ultimate in waste solids reduction as incineration.

The purpose of incineration may be summarized as follows:

i) Complete destruction of all organic materials, killing of all microorganisms and reduction of the volume of the sludge to a minimum.

ii) Control by burning all gases released from the sludge, and,

iii) Production of inert material which can be disposed of readily and economically.

Incineration is done by burning dewatered sludge to ash at temperatures between 1,250° to 1600°F. This may be accomplished in a cyclone or flash type furnace installed in connection with either a flash drier or a kiln; the multiple hearth furnace, or an evaporator.

In most cases heat drying precedes incineration as in rotary driers and multiple-hearth furnace. When heat drying is separated from incineration, the preheated sludge is burnt in suspension in a reactor at a higher temperature as in the Zimmerman process.(*42) A recent innovation is the Atomized Suspension Technique(*15) for complete combustion of dewatered sludge.

In the flash drier, the sludge is first predried with the hot gases returned from the combustion chamber. The pre-dried sludge is transferred into the fire box of the flash incinerator where the sludge is reduced to ash. A portion of the dried material has to be returned to the filter cake so that the wet filter cake can be properly handled in the flash drier.

The Herreshoff Multiple-Hearth Incinerator (*41) was first used in the chemical industry in 1885 for the roasting of pyrites in sulphuric acid plants. Little change was required in this type of unit to adopt it for sludge burning. In the multiple hearth furnaces the wet filter cake is introduced at the top hearth and by means of rabble arms, the sludge is moved to each successive next lower hearth. Combustion takes place in the third or fourth hearth from the top. Ash is removed from the lowest hearth. The temperatures in the various hearths range between 800°F at the top hearth to 1600°F in the middle hearth and falling down to about 600°F on the bottom hearth. The exhaust gases are used to preheat air returned to the top of the hearth.

Ash which is one of the by-products of incineration is disposed of in many cases as fill. Average ash amounts to 0.05 lb/cap/day, and its weight is about 40 to 50 lbs. per cu.ft.

The Zimmerman Process consists of preheating the sludge to 180°F at pressure of about 1200 pounds per square inch. The organic matter of the sludge is chemically oxidised to carbon dioxide in aqueous phase by dissolved oxygen in a specially designed reactor. The effluent, containing chiefly mineral matter, is withdrawn continuously and can be lagooned or used as a fill material. Up to date only one pilot plant using this process is in operation in Greater Chicago. It may be concluded that sufficient data and information are not available to allow detailed design or economic analyses of the process by consulting engineers.

The Atomized Suspension Technique was invented by Dr. W. Gauvin(15) in 1952 at the Pulp and Paper Research Institute of Canada, and subsequently patented. The process permits the evaporation and drying of a liquor or sludge of any initial concentration. This is followed, if so required, by chemical treatment of the dried solids in the form of suspended particles, through a reactor tower. Since this technique is used for incinerating thickened sludge obtained from Beaconsfield treatment plant, Quebec, a brief review of the essential characteristics of the process will be made.

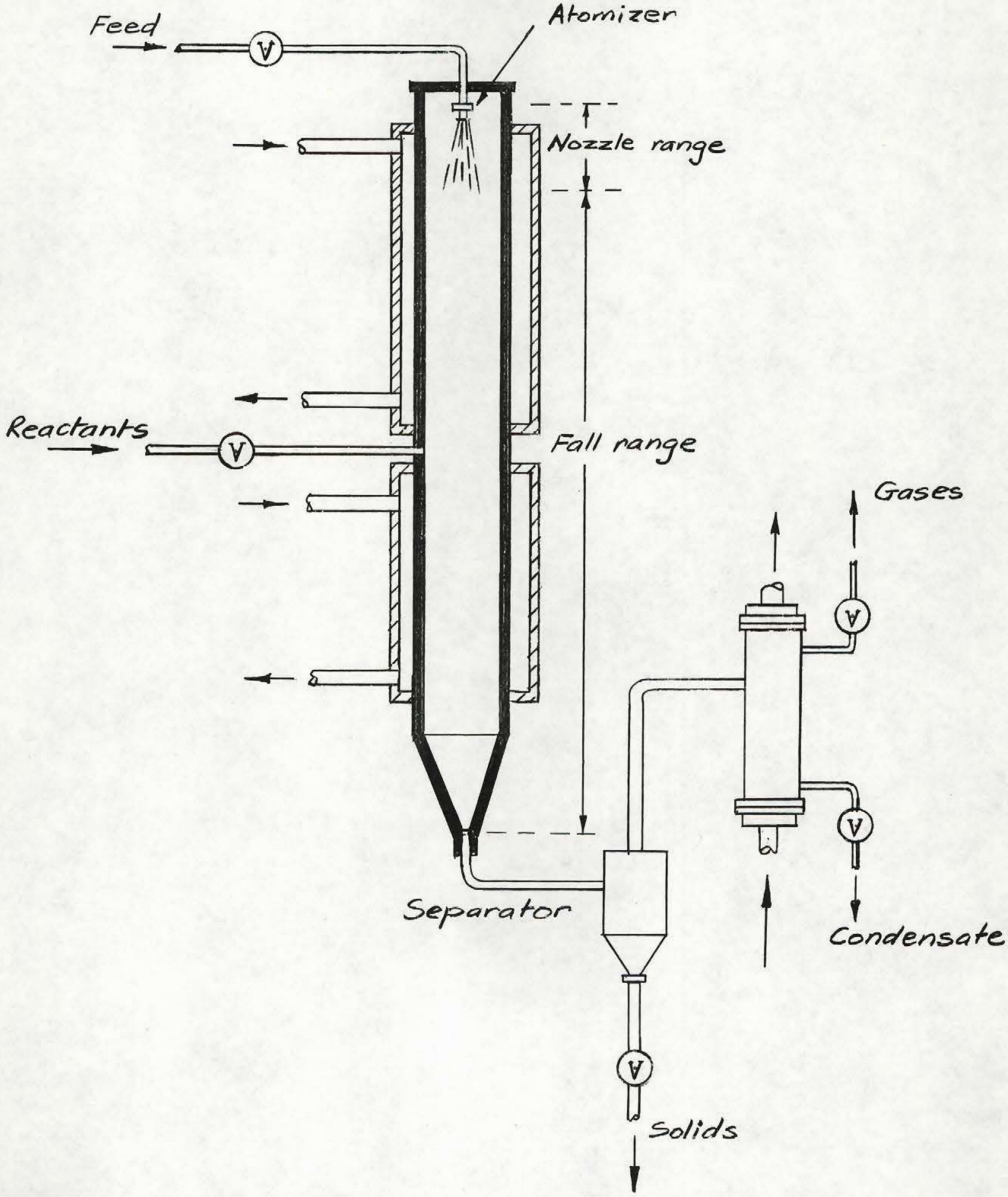


Figure 6. Atomized Suspension Reactor and Auxiliaries.

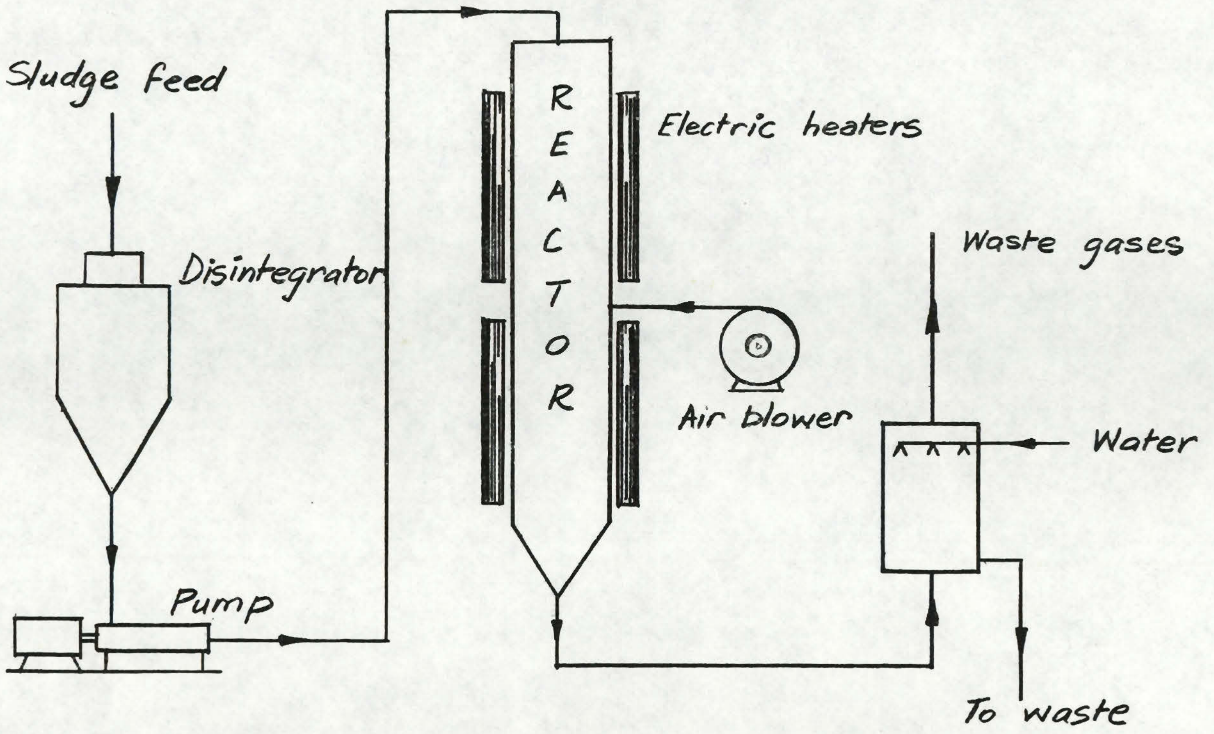


Figure 7. Schematic diagram - electrically-heated A.S.T. Sewage Sludge Disposal Plant.

Fig. 6 shows a diagrammatic sketch of the A.S.T. The technique consists of atomizing the concentrated sludge into the top of a reactor, the walls of which are maintained at temperature of about 1400°F by hot gases circulating through a jacket.

In the nozzle range, the finely divided droplets, of an average diameter of about 20 to 25 microns, quickly decelerate from a high initial velocity imparted by the atomizer to their terminal settling velocity, at which point the fall range begins. The droplets then become dispersed in the water vapor produced by their own evaporation, and the suspension thus created flow down the reactor in very nearly streamline motion. Evaporation of the dispersed droplets is completed in a short time because of the large surface area available for the heat and mass transfer. Evaporation is followed by drying while the cloud of the dispersed sludge droplets moves down the upper portion of the reactor.

At about midheight of the reactor tower air or oxygen is admitted and combustion reduces the organic matter to a harmless ash. Leaving the reactor at the bottom, the suspension consists in general of a solid residue which is recovered in cyclone collectors, a large quantity of steam which is condensed and utilized, and by-product gases which can be further processed for recovery or led away for disposal. The whole process takes only 15 seconds and is claimed superior to conventional methods of sludge disposal.

6. Sludge Disposal

The disposal of sludge by removal in liquid form to farms for use as a fertilizer and soil conditioner is an ancient practice which according to history had its birth in the Orient and in later years was practised in England and in Germany.

The final disposal of either liquid or partly dewatered sludge is influenced by:

i) Available land areas for dumping sludge cake or lagooning wet sludge.

ii) Local market possibilities for use of sludge as a soil conditioner. These possibilities include more than proximity to local farms.

iii) Suitability for discharge of wet digested or raw sludge into nearby bodies of water.

iv) The character and composition of the sludge, whether raw or digested.

A review of the principal methods of sludge disposal will be made below.

The use as fertilizer or solid conditioner is the commonest method of disposal for all classes of sludge for agricultural purposes.

Sludge contains so much water that liquid sludge disposal requires the handling of excessive quantities. The manpower required in handling the liquid sludge is however much less than that required in handling the dry sludge in the drying beds. When used as fertilizers the sludge is generally transported to land in trucks fitted with watertight boxes.

Some authors have made continued references to the neglect of waste sludge in the field of agriculture. As long as waste sludge continues to be viewed as a relatively dangerous substance, its final deposition will be presumed to be accompanied with great risk, unless, prior to final disposal, it is rendered completely innocuous by the destruction of all its original, dangerous characteristics.

Many farmers neglect the use of sludge as fertilizer or soil conditioner chiefly as the result of the habit of rating the value of a manure in terms of the constituent elements. The principal objections to the use of sludge may be summarized as follows:

i) Large quantity of fluid bulk must be transported unless pipelines can be laid to nearby agricultural fields.
ii) Difficulties often arose in the uniform application of sludge to the area to be fertilized.
iii) There is greater likelihood of objectionable odors, and
iv) there is a possibility of pathogens being present in the sludge.

In spite of these objections, sludge is used as fertilizers and soil conditioner. The treatment plant at Beaconsfield, Quebec, produces thickened sludge which is removed periodically from the thickener by trucks and sold as fertilizer to farmers in the Laurentians.

Sludge is particularly adapted to lawns, golf courses, pastures and meadows as well as to flower and vegetable gardens.

Rawn(*29) reported that trees fertilized with sludge had a healthier foliage both in amount and color and retained their leaves for a longer time in the fall than nearby unfertilized trees. Sludge has been used as fertilizer for citrus groves for about 13 years with convincing results. Sludge deepens the green color of grass and stimulates a luxuriant growth. For flower beds it provides a much needed humus for the summer months as well as a moderate but long-yielding amount of nitrogen. In vegetable gardens, sludge is beneficial for corn, potatoes, beans, spinach, asparagus and vine crops.

From the foregoing, it may be said that sludge has not been more widely used as fertilizer or soil conditioner, because of its source and the fear of producing offensive odors. Since properly digested, dried and stored primary sludge has been established as safe, hygienically, for application to the soil, there appears little room for argument from the standpoint of health. With regard to odor, many of the more common animal manures including, cow, pig and sheep wastes have as much or more odor than sludge especially if applied before thoroughly rotted or composited. Finally, since the method of disposal is not without potential dangers, it is necessary to make a very thorough study whenever it is considered as a method of disposal.

Incineration of raw or digested sludge is extensively used by large municipalities as it proves to be the most economical method of sludge disposal. The ash produced is inert and free from pathogens. The ash may be used for fill or merely sluiced to the ground surface nearby. It has little value as a fertilizer.

The barging of sludge to sea has been practiced for several years by large coastal cities in the United States. New York, Los Angeles and Miami are but a few of these cities which are using this method of sludge disposal.

Burial of sludge is not feasible in most circumstances. Where large and isolated areas are available, burial may be accomplished by discharging raw sludge into trenches. To control odor, these trenches are generally covered with earth as the sludge is placed into them.

Lagooning of sludge is used extensively in the southwest section of the United States where the monthly temperatures average about 60°F. Very few municipalities in Canada have used lagoons for sludge disposal. Lagoons receiving sludge may act as storage basins, and in this respect some local odor nuisance is likely. Lagooning is obviously the easiest and cheapest method of digested sludge disposal where its use is practicable.

Raw sludge, either wet or dewatered is used for fill only where it is controlled as a sanitary land fill. In most cases the sludge is covered promptly with earth or other suitable materials. The need for covering with earth or other suitable materials, however, depends on location and the odor problems anticipated.

In conclusion it is pertinent to quote the famous expression of R.H. Gould(*17) that,

> "The ultimate disposal of solids removed from waste water is the tail that wags the dog as far as most treatment plants are concerned."

It is encouraging to note that since the above statement was made, (1942) the various methods found and many researches carried out have added enough of experience and knowledge to the subject of sludge disposal to enable sanitary engineers to apply more rational methods.

CHAPTER V

THE SCOPE AND OBJECTIVES OF THE EXPERIMENTAL INVESTIGATIONS

As stated earlier in Chapter I, the Pulp and Paper Research Institute of Canada approached the Chairman of Civil Engineering and Applied Mechanics, McGill University, and requested assistance in the rehabilitation of the A.S.T. unit at the Waste Water Treatment Plant in Beaconsfield, Quebec. The main research work carried out in 1963 was the investigation concerning sludge conditioning. In 1964 the research work was expanded to cover the overall plant performance. Theoretical basis in connection with the research work have been given in Chapters III and IV.

The waste water treatment plant at Beaconsfield has been in operation for the past six years. Primary treatment of waste is provided, consisting of plain sedimentation, followed by chlorination of the plant effluent. The primary sludge removed from the clarifier is concentrated before final disposal.

The units installed to accomplish primary treatment of wastes and sludge treatment and disposal are of sufficient capacities to serve a population of 15,000.(*26) It was assumed that this figure would be reached by 1966, when additional units could be added.

In January 1964 it was estimated that the population now served by the plant is approximately 12,000. The average flow of waste water entering the plant is 1.75 m.g.d. with a maximum daily flow of 3.5 m.g.d.

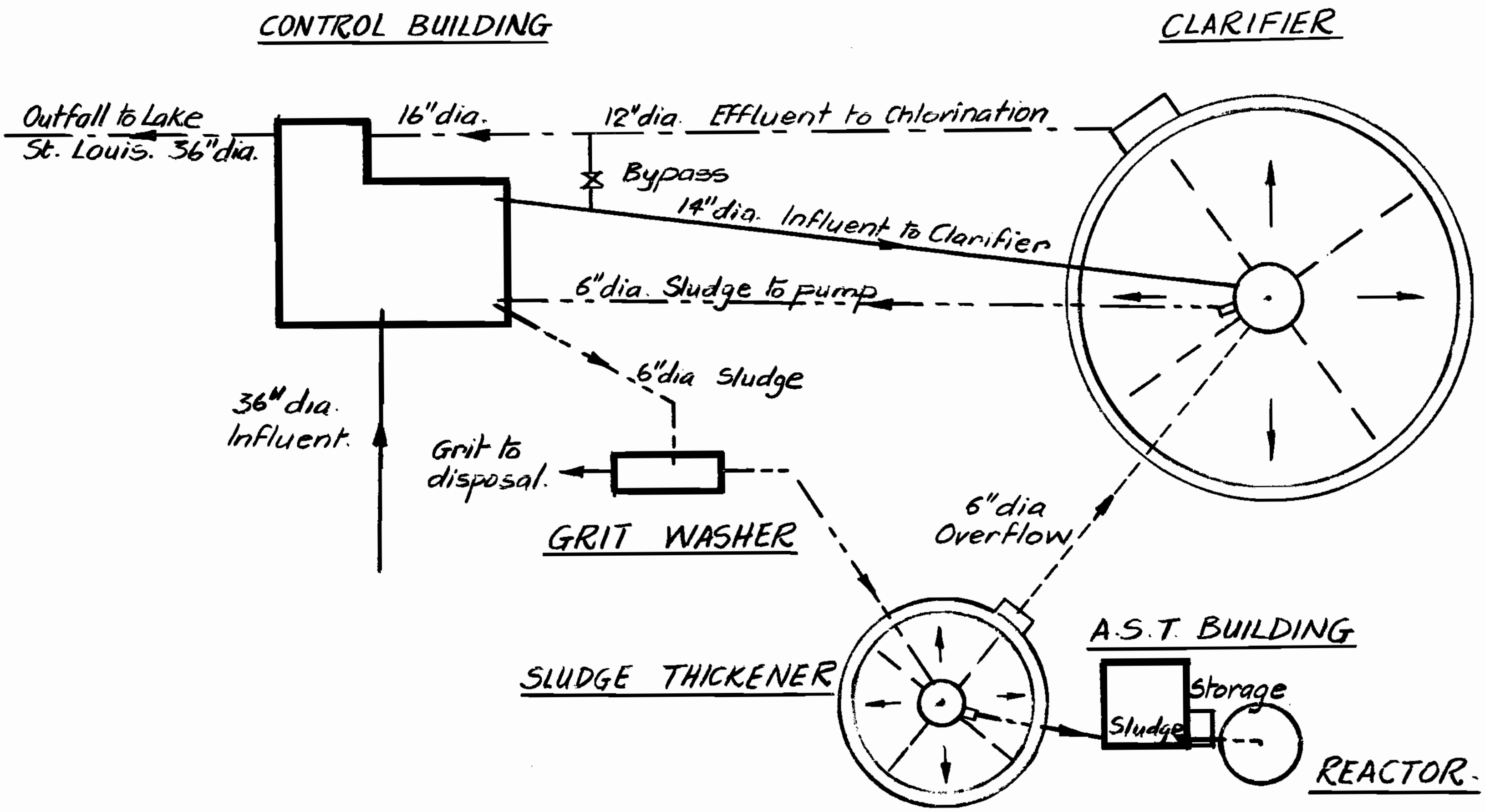


Figure 8. PLAN
not to scale

TOWN OF BEACONSFIELD - PRIMARY TREATMENT PLANT

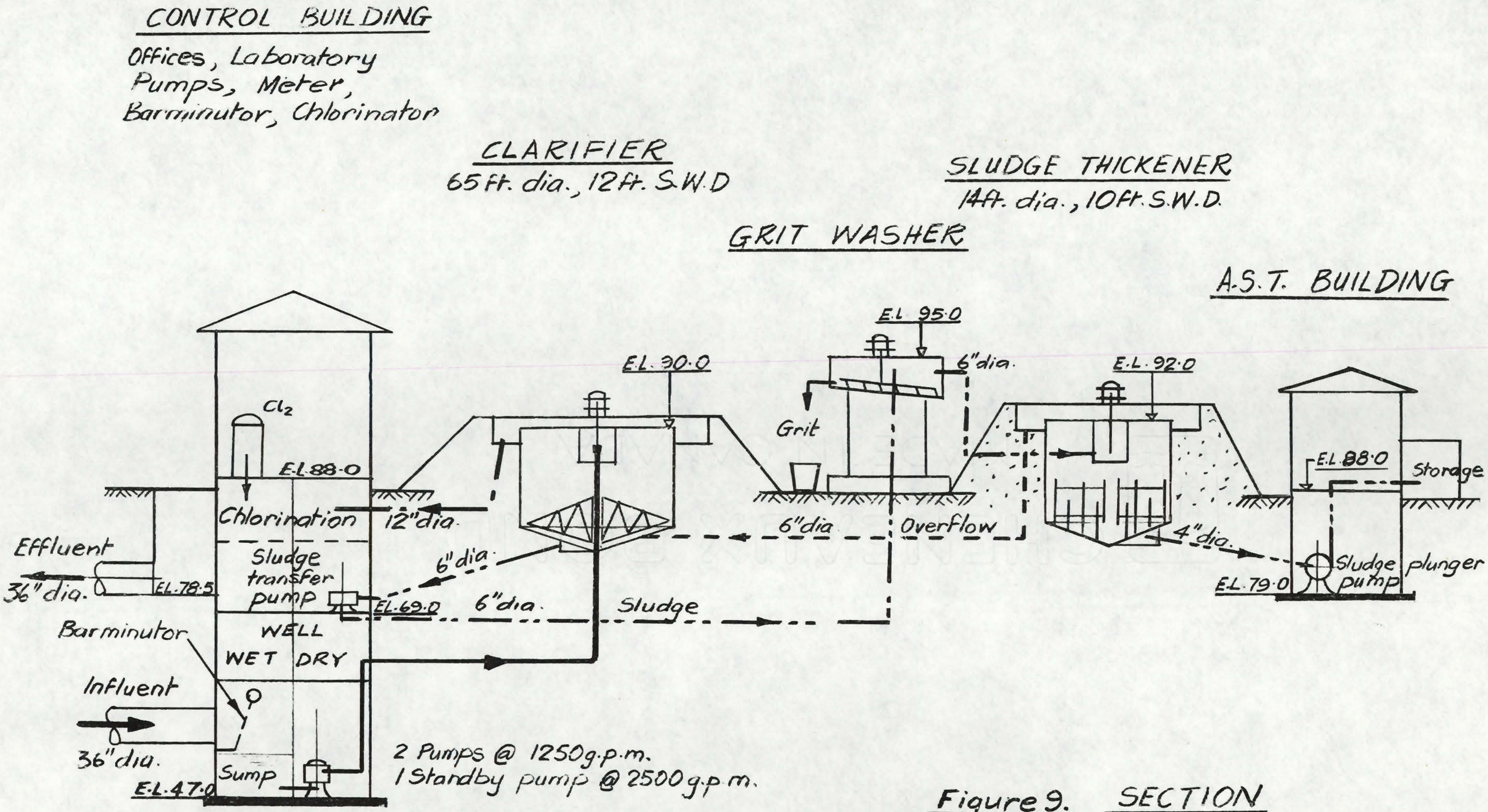


Figure 9. SECTION
not to scale

TOWN OF BEACONSFIELD - PRIMARY TREATMENT PLANT

A flow diagram of the plant is shown in Fig. 8. The 36-inch influent sewer carries the waste water into a sump situated in the basement of the Control Building. The basement is divided into the wet side and the dry side. The waste water passes from the sump through a barminutor into the wet well. An emergency bypass for the barminutor has been provided, consisting of a manually cleaned bar screen.

From the wet well the waste water is lifted by two electrically driven pumps with capacity of 1250 g.p.m. each, and one large gasoline driven standby pump with capacity of 2500 g.p.m. The discharge header of the pumps conveys the flow to the clarifier.

The clarifier effluent is discharged over a circular weir on the periphery of the clarifier and is collected in a central sump. From the sump, the effluent flows by gravity through a 16-inch pipe into the chlorine contact chamber. The effluent is chlorinated and a contact time of about 15 minutes is provided in the contact chamber. The chlorinated effluent is then discharged into Lake St. Louis through a 36-inch outfall sewer.

The raw sludge removed from the clarifier is pumped to the grit washer by a centrifugal sludge transfer pump located in the basement of the control building. After separation of grit, the remaining sludge flows from the grit washer by gravity into the thickener.

The liquid separated from the thickened sludge overflows a circular weir on the periphery of the thickener and is returned by gravity through a 6-inch pipe to the clarifier. The thickened sludge is first pumped into a storage tank from where the sludge is drawn by suction hose into trucks and then removed for final disposal. The present method of disposal of the thickened sludge will be changed as soon as the A.S.T. unit is put into operation.

The application of the A.S.T. method requires the provision of a control building adjacent to the reactor where the actual burning of sludge takes place. In the A.S.T. building are located the sludge plunger pump, small laboratory for sludge examination and other A.S.T. equipment.

The investigation concerning sludge conditioning was started in May 1963. The following three months constituted a trial period during which the equipment was tested and procedures developed for handling the concentrated sludge obtained from the thickener.

At the start of the investigation it became apparent that there would be a number of problems to be solved prior to the actual research work.

> The pumping of sludge from the thickener through the sludge plunger pump had been stopped in 1960 when the A.S.T. unit was taken out of operation. Removal of sludge was carried out after 1960 by stopping the operation of the thickener and then pumping the sludge into trucks. This operation was carried out on the average once in three weeks, when the sludge depth in the thickener was around 8 feet.
>
> It was obvious that in order to obtain normal operating conditions during the future operation of A.S.T. unit this method of removal of sludge would have to be changed. Arrangements were therefore

made to put the sludge plunger pump into service and to obtain a storage tank which would hold the sludge pumped from the thickener. Final removal of the sludge from the storage tank would be made by the Sanitank Company, Quebec, on the average four times a week.

During the initial operation of the plunger pump, it became evident that the concentration of sludge in the thickener would be limited by possible mechanical difficulties in sludge handling. With sludge concentration of about 18% solids and with discharge pressures above those provided for in the design, the shear pin of the sludge pump failed. It was therefore concluded that the pump would be inadequate for thickener operation at higher sludge concentration of over 18% solids. The experimental work therefore had to be modified so that the maximum depth of sludge blanket maintained in the thickener would yield a solid concentration of the sludge below 18%. Pumping of sludge was carried out on a 2 hour cycle, with the rate of sludge withdrawal being proportional to the volume of the sludge blanket. Pumping rate on the average was 10 g.p.m.

The pumping of sludge from the clarifier to the thickener was practiced intermittently during the day. Cross-section sampling of the sludge blanket in the thickener with respect to solids-concentration revealed deposition of alternate light and heavy layers of solids.

In order to provide for more uniform concentration throughout the sludge blanket in the thickener, it was decided to operate the sludge transfer pump continuously during a 12-hour or 24-hour pumping schedule. Under each of these conditions the sludge blanket was sectioned and it was found that the alternate layers of light and heavy sludge had practically been eliminated.

Another reason for continuous primary sludge pumping was to maintain a steady flow of fresh primary solids to the thickener.

During and following the storms that occurred in a 12-hour trial pumping, it was found necessary to maintain continuous pumping from the clarifier to prevent accumulation of silt carried into the plant from the sewer system, and consequent damage to the primary collector mechanism. The silt load, on entering the thickener dropped rapidly near the center of the tank and did not impose excessive torsional stress on the thickener mechanism, provided that the sludge was withdrawn to prevent accumulations.

The effect of silt load on the sludge was known to increase the solids content of the sludge by as much as 3% and to decrease the volatile content of the sludge by about 15%. Another effect of excessive accumulation of sludge blanket was the rising of portions of dense sludge to the surface of the thickener, especially during the warmer weather.

On the basis of the above observations it was decided to maintain a 6 foot depth of sludge blanket in the thickener, in order to minimize the amount of solids carried by the thickener effluent, and also to obtain an underflow sludge concentration of below 18% solids.

At the start of the investigation it was found that there was no sludge measuring device at the plant. Attempts were made to obtain quotations for installing either a Parshal flume, a Venturi meter, or a magnetic flow meter on the discharge side of the sludge transfer pump. The high cost of any of the equipment mentioned above made it impossible to continue further investigation.

The possibility of determining the flow into the thickener at the overflow weir of the grit washer was considered. This was found not to be feasible owing to the fluctuations of flow caused by the reciprocating action of the rakes of the washer. It was therefore decided to fasten aluminum metal strips 6 inches high to the existing weir around the thickener. The overflow from the thickener was discharged through a rectangular weir cut in the strip. A stilling well which consisted of a piece of pipe was provided for head measurements.

One basic assumption was made that the quantity of influent into the thickener was equal to the quantity of effluent. The underflow was so small that this could be neglected. This was confirmed by test during the month of August 1963 when the total underflow from the thickener was 17,335 gallons or 559 g.p.d. average compared with thickener overflow of 6.975 m.g. or 225,000 g.p.d. avgr., measured over the rectangular weir.

It was found that the calculated flow of effluent from the thickener at normal operating condition of the plant was 157 g.p.m. Attempts were made to throttle the delivery valve of the sludge transfer pump in order to reduce the pumping rate to 60 g.p.m. The minimum flow obtained was still considerably above 60 g.p.m.

Since the design figure for the thickener was given as 60 g.p.m. with surface loading of 560 g.p.d./sq.ft., it was evident that the sludge transfer pump was being operated at very low pumping efficiency. After discussion with the Town Engineer, it was agreed that this condition of high rate of pumping started in 1960 after the installation of the grit washer mechanism. Because of the high flows into the thickener it was necessary to change the electric motor of the pump, since the old one was running at a current of 5 amperes instead of the rated 3 amperes.

Three suggestions were made in order to achieve the design thickener loading:

1) Recirculate part of the sludge and allow only 60 g.p.m. into the thickener. The cutting down of the flow could be done at the grit washer.
2) Introduce a belt drive on the electric motor, thereby reducing the speed of the pump and thus reducing the pumping rate to about 60 g.p.m.
3) Introduce a variable speed drive on the motor which will make it possible to vary the thickener loading and thus provide more variables for the experimental work.

The investigation to establish overall efficiency for the plant was started in May 1963, till October of the same year. The research work was recommenced in April 1964.

From records available, calculations were made to find out the minimum and peak loadings of the clarifier during the past four years.

The pertinent daily variations were as follows:

Table IV.

Year	Sewage flow m.g.d.	Flow c.f.s.	* Surface loading gpd/sq.ft.	**Detention period hours	*** Removal %
1960 min. flow Sept. 10th	0.83	1.54	250	7.2	68
max. flow Mar. 20th	2.20	4.09	662	2.7	60
1961 min. flow Jan. 2nd	1.10	2.04	331	5.4	67
max. flow Apr. 17th	2.69	5.00	810	2.2	57.5
1962 min. flow Oct. 5th	0.756	1.404	228	7.9	68
max. flow Apr. 10th	3.17	5.88	955	1.9	55
1963 min. flow Feb. 3rd	0.66	1.225	198	9.0	68
max. flow Apr. 8th	2.16	4.010	650	2.8	61

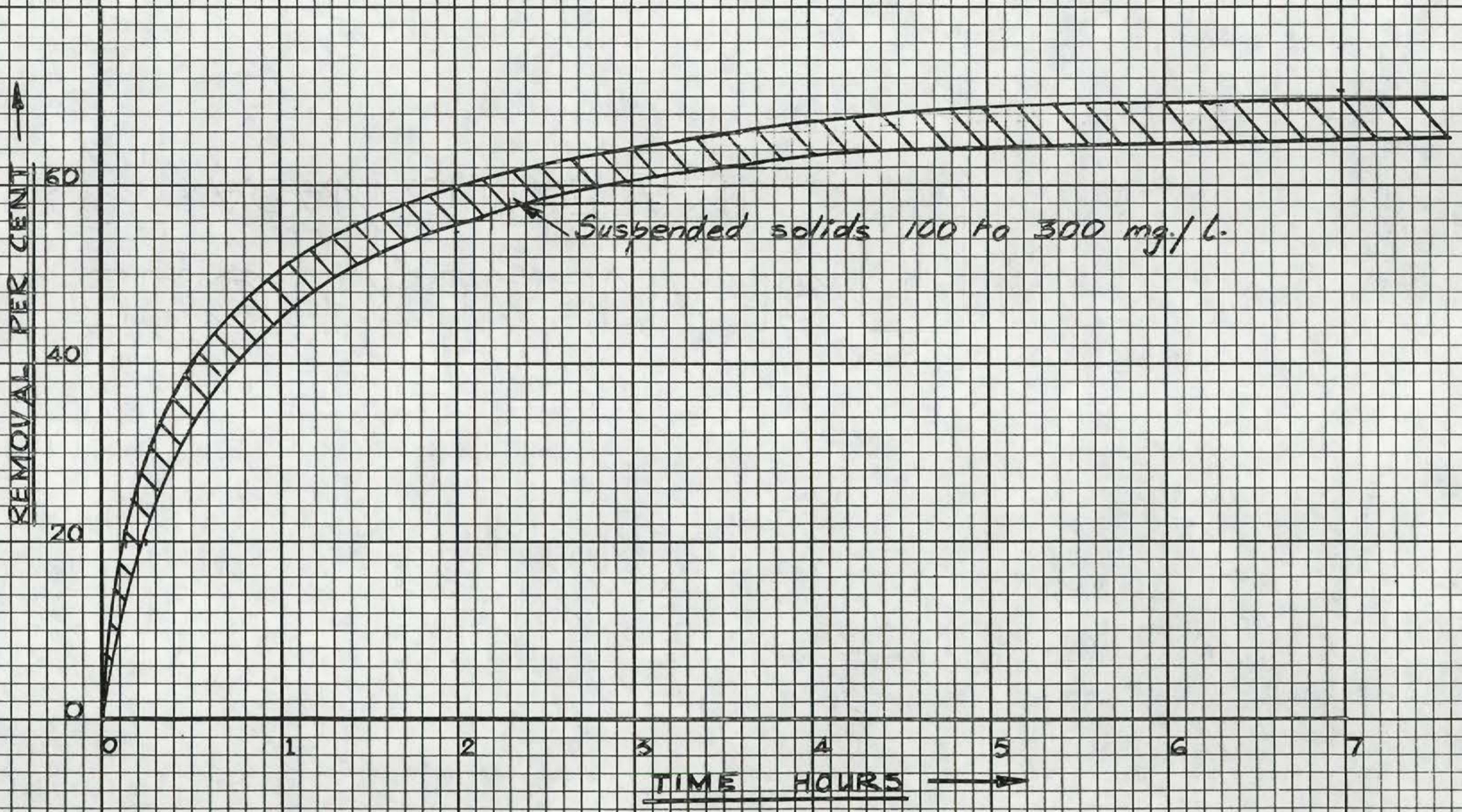
* Surface loading g.p.d./sq.ft $= \dfrac{\text{Flow, g.p.d.}}{\text{Area of tank, sq.ft.}} = \dfrac{\text{Flow, g.p.d.}}{\pi \times 32.5^2}$

** Detention period hrs. $= \dfrac{\text{Volume of tank, ft.}^3}{\text{Flow, c.f.s.}} = \dfrac{\pi \times 32.5^2 \times 12}{3600 \times \text{Flow, c.f.s.}}$

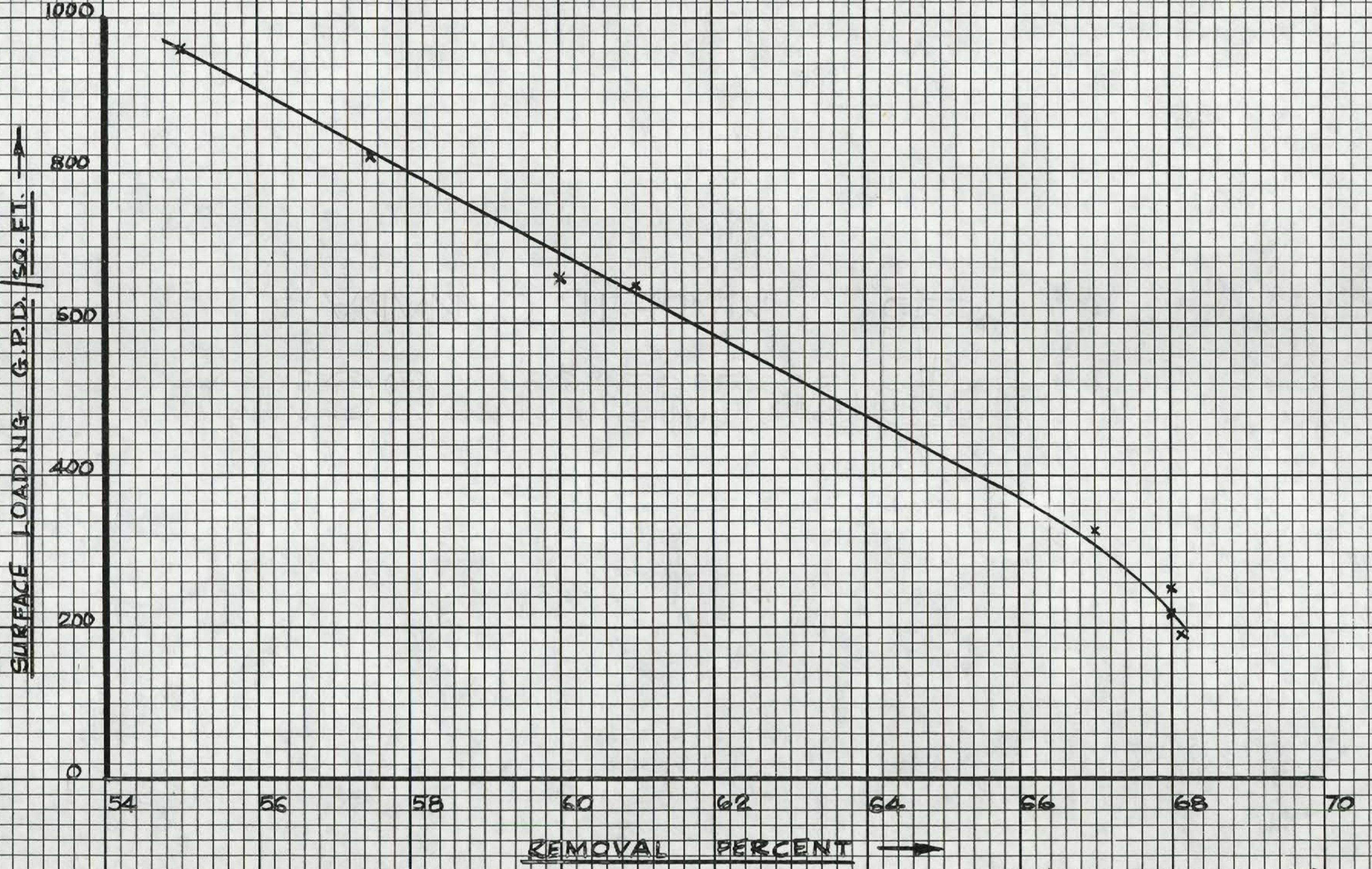
*** Removal figures obtained by interpolating graph 1 Removal % versus Detention time, hrs., after Fair & Geyer. Fig. 22-12 (*13).

The graph No. 2 shows surface loadings g.p.d./sq.ft. against removal % (calculated) of settleable suspended matter of the clarifier. This investigation is of little value because the removals were not representative of actual clarifier operation, and a general picture only may be obtained from the above work. This showed that the average removal of the

Graph I. Removal of suspended solids from waste water in Primary settling tanks after Fair & Geyer *13



Graph 2. Surface Loading of Clarifier for different calculated percentage solids removal between 1960 & 1963



TOWN OF BEACONSFIELD

clarifier at maximum loading in the past four years was about 58%. During periods of heavy storms, the clarifier was bypassed and waste water was chlorinated before discharge into the Lake St. Louis.

The clarifier loading was controlled by the waste water lift pumps and these in turn were switched in and out by depth electrodes in the wet well. Thus the clarifier loading was directly controlled by the amount of waste water coming into the plant. Furthermore, since there was no holding or storm storage unit at the plant, it was imperative that the clarifier loading could not be regulated. Experimental work would therefore be carried out and flow measurements taken over 24 hours on the specific days of testing.

During the month of December 1963 the town installed a belt drive on the motor of the sludge transfer pump. This reduction of motor speed produced a sludge pumping rate of 80 g.p.m. and thickener surface loading of 745 g.p.d./sq.ft. It was agreed that this would be the only loading rate that would be used for future tests.

Seven sampling points were to be decided upon and samples would be taken hourly over a 24 hour testing period. Analyses of the samples were to be made in order to determine not only the efficiency of the separate treatment units but also the overall efficiency of the treatment plant.

From the foregoing, the factors to be evaluated are as follows:

1. Organic and mineral components as well as the solids content of the thickened sludge.

2. The effect of the thickness of sludge blanket maintained in the thickener on the solids concentration of the sludge.

3. The effect of the solids concentration on the volatile content.

4. The effect of consolidation time on the sludge removed in the clarifier.

5. The effect of the sludge volume ratio on the solids concentration.

6. Establishing unit efficiency, and

7. Establishing overall efficiency of the plant and investigation concerning improvement of it.

CHAPTER VI

APPARATUS

Description of units at Beaconsfield waste water treatment plant are as follows:

Control Building. The following are housed in the control building:-

(a) The general office.

(b) Small laboratory

(c) Lockers and washrooms.

(d) Storeroom and repair shop.

(e) Barminutor and bypass screens.

(f) Waste water lift pumps.

(g) Sludge transfer pump.

(h) Chlorination unit.

(j) Kennison nozzle metering unit.

Barminutor. This unit provides continuous and automatic screening and comminuting of coarse suspended matter without their removal from the flow. The barminutor installed at the plant is a model "B" type manufactured by Chicago Pump Company. It has a screen with 3/8" openings and is periodically cleaned by electrically powered mechanism. A bypass bar screen is provided which can be manually cleaned.

Pumps. Three high lift pumps are now in operation which lift influent from the wet well to the clarifier. These pumps are of centrifugal type and manufactured by the Smart-Turner Machine Co. Ltd., Hamilton, Canada. The pumps are installed in the dry well of the control building. Two small pumps have a rated capacity of 1250 g.p.m. at total dynamic head of 60 feet, and the large gasoline driven standby pump has a rated capacity of 2500 g.p.m. at T.D.H. of 60 ft.

The sludge transfer pump is of side suction, centrifugal type manufactured by The Smart-Turner Machine Co. Ltd. and this is located in the store room in the basement of the control building. The pump has a rated capacity of 60 g.p.m. at T.D.H. of 25 ft.

The sludge pump is located in a pump pit in the A.S.T. building. It is of a piston plungertype and manufactured by Ralph B. Carter Co., Hackensack, N.J., U.S.A. It has capacity ranging with stroke length of the piston. The maximum rated capacity is 75 g.p.m. at T.D.H. of 30 ft.

All pumps in use at the plant are motor driven while internal combustion standby unit is provided for the large raw waste water pump.

Meters. The plant effluent is measured at the end of the pipe carrying the effluent from the clarifier into the chlorine contact chamber. The metering unit consists of a 20-inch Kennison nozzle with range of 0.2 to 3.5 m.g.d. and an indicating and totalizing flow recorder with a range of 0.2 to 3.5 m.g.d. The Kennison nozzle and recorder were manufactured by B-I-F Industries, Inc., Providence, Rhode Island, U. S. A.

Influent to the thickener is measured by blocking off the existing weir and installing a temporary rectangular weir on the periphery of the thickener. The weir dimension being 12-inch long by 4-inch high. A stilling well is provided for head measurements.

Chlorinators. One visible flow chlorinizer is installed in the chlorine room. The unit was manufactured by B-I-F. Industries, Inc. It accurately controls and meters chlorine gas under vacuum in the dry, non-corrosive state, producing and deliverying chlorine-water solution to the effluent in the contact chamber. The feed is from 0.4 to 500 lbs. per 24 hours.

The Clarifier. No. DOL-547-1 was manufactured by Dorr-Oliver-Long Ltd., Orillia, Ontario, Canada. It is a concrete structure 65 feet diameter by 12 feet side water depth. The center column carries a rotating scraper and scum removal mechanisms. A steel walkway bridge is provided which extends from one side of the clarifier to the drive platform. The waste water enters the clarifier at the feed well and flow is horizontal and radial toward the periphery of the tank.

The thickener is a Dorr Densludge thickener No. DOL-547-2 and is a concrete structure 14 ft. diameter by 10 ft. side water depth. The center column carrying a rotating scraper mechanism. A steel walkway bridge is provided which extends from one side of the thickener to the drive platform.

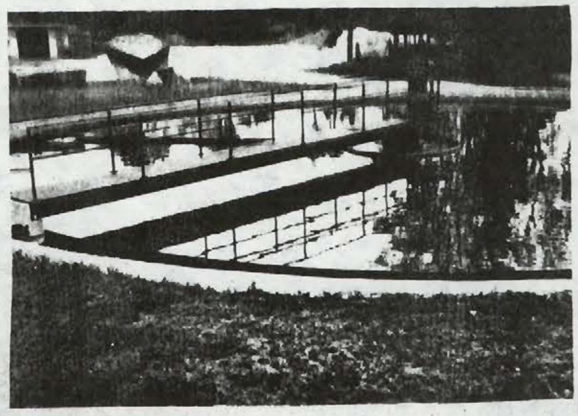
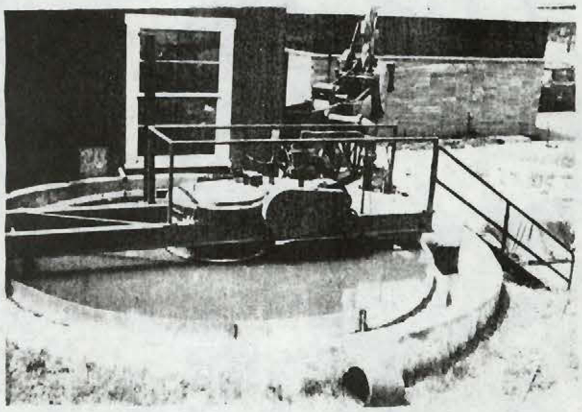


PLATE 1

Clarifier

PLATE 2

Thickener



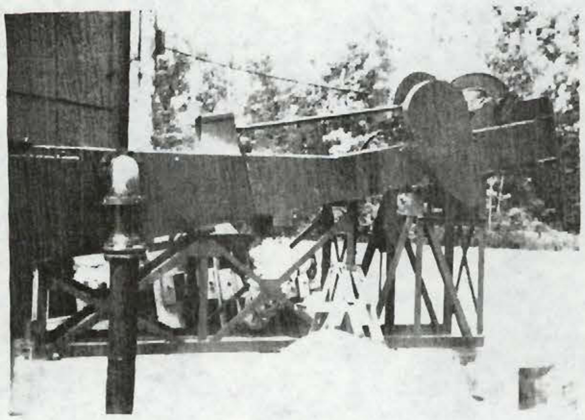


PLATE 3

Grit Washer

The grit washer is a Dorrco Grit Washer No. DOL-1339-1. It consists of a set of mechanically operated reciprocating blades installed in an inclined rectangular settling box. Sludge is fed continuously about midway in the grit washer pool. The coarse suspended matter such as sand and gravel settle to the bottom of the tank and are removed up the slope by successive passes of the blades and discharged in a drained condition. Fine solids kept in suspension by the motion of the mechanism, overflow the end of the tank into the influent pipe to the thickener.

CHAPTER VII

PROCEDURE

The experimental work concerning the sludge conditioning was started in August 1963. The variables considered were as follows:

1. Removal of sludge from the clarifier. This consists of the following,

   a) Continuous pumping of sludge for 24 hours, with thickener influent at 157 g.p.m.

   b) Continuous pumping of sludge for 24 hours, recirculating part of sludge pumped, at the grit washer and allowing only 60 g.p.m. into thickener.

   c) Continuous pumping of sludge for 12 hours (8 a.m. - 8 p.m.) with thickener influent at 157 g.p.m.

2. Maintenance of sludge level in the thickener. This consists of operating the thickener at sludge depths of 3 feet and 6 feet.

Depending on the variables above, moisture content and mineral and volatile contents of the thickened sludge may vary. Laboratory analyses were therefore required.

In summary, there were three operating conditions of the clarifier, and for each of these conditions there were two sludge blanket depths maintained in the thickener - making in all 6 test runs.

The conditioning of sludge in the thickener was carried on for 6 days during each test run. The days were made up of 2 days preparatory work before test, 2 days testing and collection of samples, and 2 days after testing for checking of experiments. Results of each day of test were to be computed separately.

For any given set of conditions of the variables mentioned above, the sludge plunger pump was operated for 10 minutes at intervals of 2 hours during the day of testing. Test period was from 8 a.m. to 10 p.m.

Catch samples of thickened sludge pumped were collected in common galvanized steel pails at 2, 4 and 8 minutes of each 10 minutes pumping. Two other samples were obtained by compositing part of the 3 samples taken above. The catch samples were analyzed on site to determine the moisture content. The two composited samples were analyzed in the laboratory. Ash analyses were later carried out, on all 5 dried samples.

The thickener loading was determined by measuring the depth of effluent over the rectangular weir of the thickener. Measurements to obtain the depth of flow were carried out by means of an engineer's level and staff.

The depth of flow was taken before each pumping of sludge from the thickener. A mean of all depth readings was used for calculating the thickener loading.

The waste water flow measurements were taken hourly over the day of test in order to find the hourly and daily flows into the Treatment Plant.

The experimental work concerning the Plant's efficiency was started in April 1964. The units to be investigated were the clarifier, the thickener and chlorination units. Seven sampling points were established and samples of waste water and sludge were drawn from these points for laboratory analyses.

The influent samples were taken from Station 1 which was a point in the wet well immediately after the barminutor.

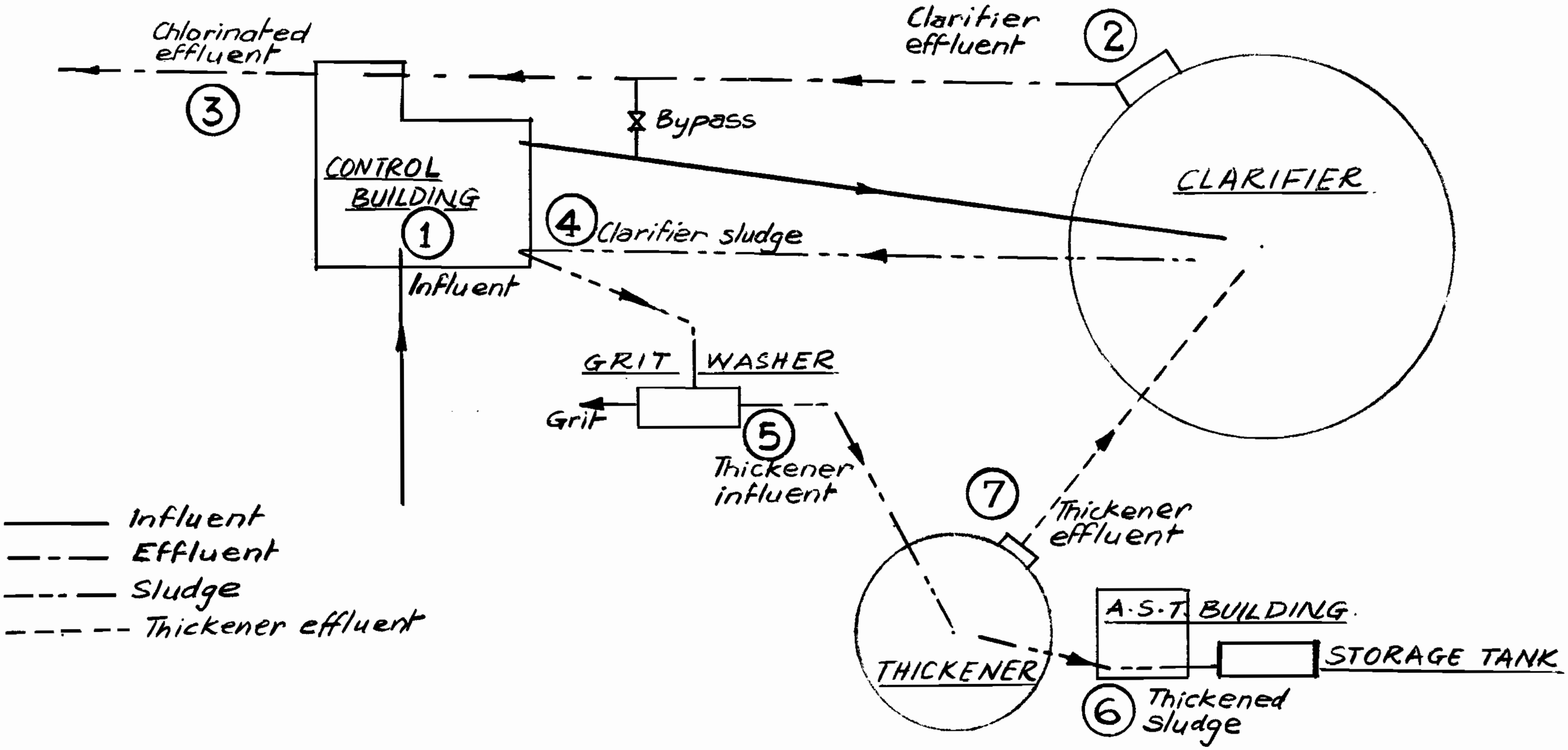


Figure 10. SAMPLING STATIONS AT BEACONSFIELD TREATMENT PLANT.

The clarifier effluent samples were taken at the effluent sump of the clarifier. This sump was taken as Station 2.

The chlorinated effluent samples were drawn from Station 3, which was the manhole leading to the trunk mains carrying the chlorinated effluent from the contact chamber to the outfall.

Station 4 was a point on the delivery side of the sludge transfer pump and the samples taken were called the clarifier sludge.

Station 5 was located at the overflow weir of the grit washer and samples collected were considered as influent samples to the thickener.

The thickened sludge samples were obtained from station 6 which was a 2 inch valve connected to the delivery pipe of the sludge plunger pump.

Station 7 was at the rectangular weir of the thickener. The thickener effluent samples were collected at this point.

Eight tests were to be carried out at normal operating conditions of the plant. Each test was to be carried out for one day. Three days were allowed before each test to condition the plant and especially to establish a definite sludge depth in the thickener.

During the period of testing, the thickener loading was maintained at about 745 gals/sq.ft./day or 80 g.p.m. This value was established by measurement of thickener overflow at the rectangular weir. As mentioned previously, the sludge transfer pump was fitted with a belt drive motor and this effect produced a reduction of pumping rate. The influent flow measurements were taken every hour over a 24 hour test period. Samples of waste water and sludge were collected every hour at each of the 7 sampling stations and composited at the end of each test. The composite samples were then analyzed to determine the settleable suspended matter, the total solids content, the suspended solids content, the C.O.D. and pH values. In addition, the organic and mineral contents of total and suspended solids were determined. Other physical examinations of the waste water and sludge samples were made during each test.

CHAPTER VIII

OBSERVATIONS

The observations for sludge conditioning have been classified according to the test number and only the summary observations, rather than the complete weighing reports are tabulated. The summary observations are included in Appendix E.

Samples of the actual observation forms together with sample calculations are shown in the tabulations which follow in pages 89 to 116.

The mean of the values for solids content as well as for inorganic content of the samples were found by using the method of moving averages. The standard fluctuation and coefficient of fluctuation of each sample taken during a single pumping are shown.

The mean, standard fluctuation and coefficient of fluctuation of an average sample taken over one day's test have been computed and are shown at the bottom of each set of observations.

The results of tests Nos. 1 to 6 are shown in Table No. V. The variations of waste water flow, volatile matter and solids content have been plotted for Tests Nos. 1 to 6 and are shown in graphs Nos. 21 to 30.

From series of tests carried out on thickener operation, a table of observations of sludge blanket depths and corresponding solids and volatile content together with the sludge volume ratio has been obtained. The tabulated observations are shown in Table X.

The observations taken during the tests on the units of the Treatment Plant are included in Appendix E. The variations of waste water flow during these tests are shown in graphs Nos. 31 to 38. The results of removal efficiencies of the units of the Treatment Plant are shown in Table No. VII.

TEST No. 4 October 23, 1963

Waste water flow	1.09 m.g.d.	Thickener underflow	530 g.p.d.
Sludge pumping into thickener	12 hours	Depth of sludge in thickener	6 feet
Thickener loading	850 gals/sq.ft/d.	Specific gravity	1.033

SOLIDS CONTENT - First day.

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	A-1	17.4			
	A-2	15.8			
	A-3	15.6	15.79	0.823	5.22
	A-4	15.65			
	A-5	15.81			
10 a.m.	B-1	14.4			
	B-2	14.6			
	B-3	14.6	14.57	0.222	1.52
	B-4	14.49			
	B-5	14.69			
12 noon	C-1	12.8			
	C-2	13.6			
	C-3	13.6	13.72	0.544	3.95
	C-4	14.00			
	C-5	14.69			

Observations (continued)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
2 p.m.	D-1	13.2			
	D-2	13.4			
	D-3	14.0	13.843	0.44	3.2
	D-4	14.2			
	D-5	13.88			
4 p.m.	E-1	13.8			
	E-2	13.6			
	E-3	13.6	13.641	0.158	1.16
	E-4	13.65			
	E-5	13.88			
6 p.m.	F-1	13.4			
	F-2	13.6			
	F-3	13.4	13.634	0.394	2.89
	F-4	13.96			
	F-5	14.10			
8 p.m.	G-1	13.2			
	G-2	13.2			
	G-3	13.58	13.539	0.398	2.94
	G-4	13.88			
	G-5	13.61			

Observations (continued)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
10 p.m.	H-1	13.46			
	H-2	13.65			
	H-3	13.75	13.683	0.11	0.8
	H-4	13.68			
	H-5	13.62			
Average for the day			13.771	0.465	3.38

In the test itself, single observations of solids content showed a coefficient of fluctuation from ± 0.8% to ± 5.2%. These are the result of accidental errors in solids content determination. While it is impossible to establish a correction for any one observation which is subject to accidental error, a series of such observations taken collectively tend to obey the law of probability.

The coefficient of fluctuation of an average sample taken from all samples collected over the day is ± 3.38%. It may be concluded that this coefficient of fluctuation in solids content depended on the precision of measurement and it would seem reasonable to assume that all other observations of solids content carried out under the given set of conditions would have about the same range of coefficient of fluctuation.

As mentioned earlier, equivalent determinations of coefficient of fluctuations have been made for all tests 1-6 and the results have been presented in a summarized form in the observations pertaining to each of these tests. (Appendix E).

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuation ± F	Coefficient of Fluctuation C = ± F/M %.
8 a.m.	A-1	34.57			
	A-2	31.54			
	A-3	31.81	32.04	2.35	7.30
	A-4	32.53			
	A-5	30.90			
10 a.m.	B-1	31.72			
	B-2	30.93			
	B-3	30.83	30.49	1.75	5.74
	B-4	29.28			
	B-5	30.30			
12 noon	C-1	29.56			
	C-2	31.24			
	C-3	29.96	30.04	1.79	5.97
	C-4	29.26			
	C-5	29.32			
2 p.m.	D-1	27.29			
	D-2	30.54			
	D-3	28.57	29.14	3.33	11.42
	D-4	29.03			
	D-5	29.28			

Observations (continued)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuation ± F	Coefficient of Fluctuation C = ± F/M %.
4 p.m.	E-1	28.32			
	E-2	28.87			
	E-3	29.65	29.31	0.555	1.89
	E-4	29.53			
	E-5	29.17			
6 p.m.	F-1	29.28			
	F-2	28.46			
	F-3	30.56	29.53	3.01	10.02
	F-4	29.04			
	F-5	29.88			
8 p.m.	G-1	29.27			
	G-2	29.51			
	G-3	28.85	29.37	1.14	3.88
	G-4	29.83			
	G-5	30.19			
10 p.m.	H-1	28.45			
	H-2	29.31			
	H-3	28.07	28.62	1.88	6.56
	H-4	28.89			
	H-5	28.31			
Average for the day			29.50	0.635	2.15

SAMPLE CALCULATION

TEST No. 4 October 23, 1963

a) Solids Content - Calculations of Moving Averages

Magnitude	First Reduction			Second Reduction		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
8 a.m.						
17.4						
15.8	33.2					
15.6	31.4	64.6	16.15			
15.65	31.25	62.65	15.66	31.81		
15.81	31.46	62.71	15.68	31.34	63.15	15.79
10 a.m.						
14.4						
14.6	29.0					
14.6	29.2	58.2	14.55			
14.49	29.08	58.29	14.57	29.12		
14.69	29.18	58.27	14.57	29.14	58.26	14.57
12 noon						
12.8						
13.6	26.4					
13.6	27.2	53.6	13.4			
14.0	27.6	54.8	13.7	27.1		
14.69	28.69	56.29	14.07	27.77	54.87	13.72

Observations (continued)

Magnitude	First Reduction			Second Reduction		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
2 p.m.						
13.2						
13.4	26.6					
14.0	27.4	54.0	13.5			
14.2	28.2	55.6	13.9	27.4		
13.88	28.08	56.28	14.07	27.97	55.37	13.843
4 p.m.						
13.8						
13.6	27.4					
13.6	27.2	54.6	13.65			
13.65	27.25	54.45	13.61	27.26		
13.88	27.53	54.78	13.70	27.31	54.57	13.641
6 p.m.						
13.4						
13.6	27.0					
13.4	27.0	54.0	13.50			
13.96	27.36	54.36	13.59	27.09		
14.10	28.06	55.42	13.855	27.445	54.535	13.634
8 p.m.						
13.2						
13.2	26.4					
13.58	26.78	53.18	13.295			
13.88	27.46	54.24	13.56	26.855		
13.61	27.49	54.95	13.74	27.30	54.155	13.539

Observations (continued)

Magnitude		First Reduction			Second Reduction	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
10 p.m.						
13.46						
13.65	27.11					
13.75	27.40	54.51	13.63			
13.68	27.43	54.83	13.71	27.34		
13.62	27.30	54.73	13.68	27.39	54.73	13.683
Average for the day.						
15.79						
14.57	30.36					
13.72	28.29	58.65	14.662			
13.843	27.563	55.853	13.963	28.625		
13.641	27.484	55.047	13.762	27.725	56.35	14.09
13.634	27.275	54.759	13.689	27.451	55.176	13.794
13.539	27.173	54.448	13.612	27.301	54.752	13.688
13.683	27.222	54.395	13.599	27.211	54.512	13.628
14.09						
13.794	27.884					
13.688	27.482	55.366	13.842			
13.628	27.316	54.798	13.699	27.541		13.771

b) <u>Inorganic Content - Calculations of Moving Averages</u>

Magnitude	First Reduction			Second Reduction		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
8 a.m.						
34.57						
31.54	66.11					
31.84	63.35	129.46	32.365			
32.53	64.34	127.69	31.922	64.287		
30.90	63.43	127.77	31.942	63.864	128.151	32.04
10 a.m.						
31.72						
30.93	62.65					
30.83	61.76	124.41	31.102			
29.28	60.11	121.87	30.47	61.572		
30.30	59.58	119.69	29.922	60.392	121.964	30.49
12 noon						
29.56						
31.24	60.80					
29.96	61.20	122.00	30.50			
29.26	59.22	120.42	30.105	60.605		
29.32	58.58	117.80	29.45	59.555	120.16	30.04
2 p.m.						
27.29						
30.54	57.83					
28.57	59.11	116.94	29.235			
29.03	57.60	116.71	29.18	58.415		
29.28	58.31	115.91	28.98	58.16	116.575	29.144

Observations (continued)

Magnitude	First Reduction			Second Reduction		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
4 p.m.						
28.32						
28.87	57.19					
29.65	58.52	115.71	28.93			
29.53	59.18	117.70	29.425	58.355		
29.17	58.70	117.88	29.47	58.895	117.25	29.31
6 p.m.						
29.28						
28.46	57.74					
30.56	59.02	116.76	29.19			
29.04	59.60	118.62	29.655	58.845		
29.88	58.92	118.52	29.63	59.285	118.13	29.53
8 p.m.						
29.27						
29.51	58.78					
28.85	58.36	117.14	29.285			
29.83	58.68	117.04	29.26	58.545		
30.19	60.02	118.70	29.675	58.935	117.48	29.37
10 p.m.						
28.45						
29.31	57.76					
28.07	57.38	115.14	28.785			
28.89	59.96	114.34	28.585	57.37		
28.31	57.20	114.16	28.54	57.125	114.495	28.624

Observations (continued)

Magnitude		First Reduction			Second Reduction	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Average for the day						
32.04						
30.49	62.53					
30.04	60.53	123.06	30.765			
29.144	59.184	119.714	29.929	60.694		
29.31	58.454	117.638	29.410	59.339	120.033	30.008
29.53	58.84	117.294	29.324	58.734	118.073	29.518
29.37	58.90	117.74	29.435	58.759	117.493	29.373
28.624	57.994	116.894	29.224	58.659	117.418	29.355
30.008						
29.518	59.526					
29.373	58.891	118.417	29.604			
29.355	58.728	117.619	29.405	59.009		29.505

SAMPLE CALCULATIONS

Test No. 4 First Day

a) Solids Content - Calculations of Standard Fluctuation, F, and Coefficient of Fluctuation, C.

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\Sigma \Delta''^2$	F $\pm\sqrt{\frac{\Sigma\Delta''^2}{n-2}}$	C $\pm$ F/M
8 a.m.			Mean = 15.79			
17.4						
15.8	-1.6					
15.6	-0.2	1.4	1.96			
15.65	0.05	0.25	0.063			
15.81	0.16	0.11	0.012	2.035	0.832	5.22
10 a.m.			Mean = 14.57			
14.4						
14.6	0.2					
14.6	0.0	-0.2	0.04			
14.49	-0.11	-0.11	0.012			
41.69	0.20	0.31	0.096	0.148	0.222	1.52
12 noon			Mean 13.72			
12.8						
13.6	0.8					
13.6	0.0	-0.8	0.64			
14.0	0.4	0.4	0.16			
14.69	0.69	0.29	0.084	0.884	0.544	3.95

Observations (continued)

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\Sigma\Delta''^2$	F $\pm\sqrt{\frac{\Sigma\Delta''^2}{n-2}}$	C $\pm$ F/M
2 p.m.			Mean = 13.84			
13.2						
13.4	0.2					
14.0	0.6	0.4	0.16			
14.2	0.2	-0.4	0.16			
13.88	-0.32	-0.52	0.27	0.59	0.444	3.2
4 p.m.			Mean = 13.64			
13.8						
13.6	-0.2					
13.6	0.0	0.2	0.04			
13.65	0.05	0.05	0.003			
13.88	0.23	0.18	0.032	0.075	0.158	1.16
6 p.m.			Mean = 13.64			
13.4						
13.6	0.2					
13.4	-0.2	-0.4	0.16			
13.96	0.56	0.36	0.13			
14.10	0.14	-0.42	0.176	0.466	0.394	2.89

Observations (continued)

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\sum\Delta''^2$	F $\pm\sqrt{\frac{\sum\Delta''^2}{n-2}}$	C $\pm F/M$
8 p.m.			Mean = 13.54			
13.2						
13.2	0.0					
13.58	0.38	0.38	0.144			
13.88	0.30	-0.08	0.006			
13.61	-0.27	-0.57	0.325	0.475	0.398	2.94
10 p.m.			Mean = 13.68			
13.46						
13.65	0.19					
13.75	0.10	-0.09	0.008			
13.68	-0.07	-0.17	0.029			
13.62	-0.06	0.01	0.000	0.037	0.111	0.8
Average for the day						
			Mean 13.77			
15.79						
14.57	-1.22					
13.72	-0.85	0.37	0.137			
13.843	0.123	0.973	0.945			
13.641	-0.202	-0.325	0.106			
13.636	-0.005	0.197	0.039			
13.539	-0.097	-0.092	0.009			
13.683	0.144	0.241	0.058	1.294	0.465	3.38

b) <u>Inorganic Content</u> - Calculations of Standard Fluctuation, F, and Coefficient of Fluctuation, C.

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\Sigma\Delta''^2$	F $\pm\sqrt{\frac{\Sigma\Delta''^2}{n-2}}$	C $\pm F/M$
8 a.m.			Mean = 30.04			
34.57						
31.54	-3.03					
31.81	0.27	3.30	10.9			
32.53	0.72	0.45	0.202			
30.90	-1.63	-2.35	5.510	16.612	2.35	7.82
10 a.m.			Mean = 30.49			
31.72						
30.93	-0.79					
30.83	-0.10	0.69	0.476			
29.28	-1.55	-1.45	2.100			
30.30	1.02	2.57	6.600	9.176	1.75	5.74
12 noon			Mean = 30.04			
29.56						
31.24	1.68					
29.96	-1.28	-2.96	8.75			
29.26	-0.70	0.58	0.336			
29.32	0.06	0.76	0.578	9.664	1.79	5.97

Observations (continued)

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\Sigma\Delta''^2$	F $\pm\sqrt{\frac{\Sigma\Delta''^2}{n-2}}$	C $\pm F/M$
2 p.m.			Mean = 29.14			
27.29						
30.54	3.25					
28.57	-1.97	-5.22	27.3			
29.03	0.46	2.43	5.9			
29.28	0.25	-0.21	0.044	33.244	3.33	11.42
4 p.m.			Mean = 29.30			
28.32						
28.87	0.55					
29.65	0.78	0.23	0.0053			
29.53	-0.12	-0.90	0.81			
29.17	-0.36	-.24	0.058	0.921	0.56	1.89
6 p.m.			Mean = 29.53			
29.28						
28.46	-0.82					
30.56	2.10	2.92	8.5			
29.04	-1.52	-3.62	13.1			
29.88	0.84	2.36	5.58	27.18	3.01	10.02

Observations (continued)

Magnitude	Difference Δ'	Difference Δ''	Δ''^2	$\Sigma\Delta''^2$	F $\pm\sqrt{\frac{\Sigma\Delta''^2}{n-2}}$	C $\pm F/M$
8 p.m.			Mean = 29.37			
29.27						
29.51	0.24					
28.85	-0.66	-0.90	0.81			
29.83	0.98	1.64	2.69			
30.19	0.36	-0.62	0.385	3.885	1.14	3.88
10 p.m.			Mean = 28.85			
28.45						
29.31	0.86					
28.07	-1.24	-2.10	4.4			
28.89	0.82	2.06	4.25			
28.31	-0.58	-1.40	1.96	10.61	1.88	6.5
Average for the day			Mean = 29.50			
30.04						
30.49	0.45					
30.04	-0.45	-0.90	0.81			
29.14	-0.90	-0.45	0.202			
29.30	0.16	1.06	1.12			
29.53	0.23	0.07	0.005			
29.37	-0.16	-0.39	0.152			
28.85	-0.52	-0.36	0.130	2.419	0.635	2.15

SAMPLE OBSERVATIONS

Test No. 4 First Day

Solids Content - using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES: A-1-2-3								
8.20	0	0	9.00	0	0	9.42	0	0
8.26	6	11.8	9.20	20	64.4	9.47	5	9.8
8.30	10	26.6	9.25	25	74.2	9.52	10	29.6
8.35	15	45.2	9.28	28	77.8	9.57	15	49.0
8.40	20	61.8	9.30	30	79.6	10.02	20	67.2
8.45	25	73.2	9.32	32	81.2	10.07	25	76.4
8.50	30	79.4	9.35	35	83.0	10.12	30	81.0
8.52	32	80.8	9.38	38	84.0	10.17	35	83.6
8.55	35	82.2	9.40	40	84.2	10.20	38	84.2
SAMPLES: B-1-2-3								
10.24	0	0	11.05	0	0	11.50	0	0
10.29	5	9.0	11.10	5	10.0	12.05	15	49.4
10.34	10	27.8	11.15	10	28.8	12.10	20	65.8
10.39	15	46.6	11.20	15	48.0	12.15	25	77.4
10.44	20	63.6	11.25	20	64.0	12.18	28	81.6
10.49	25	76.2	11.30	25	75.6	12.20	30	83.0
10.54	30	82.8	11.35	30	82.0	12.22	32	84.2
10.56	32	84.0	11.39	34	84.2	12.24	34	85.0
10.58	34	85.0	11.41	36	85.0	12.26	36	85.4
11.00	36	85.6	11.46	41	85.4	12.28	38	85.4

Observations (continued)

Time.	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist. %
SAMPLES: C-1-2-3								
12.30	0	0	1.10	0	0	1.51	0	0
12.35	5	9.4	1.25	15	51.2	2.01	10	31.2
12.50	20	66.6	1.30	20	68.6	2.06	15	51.0
12.55	25	78.6	1.35	25	79.6	2.11	20	68.6
1.00	30	85.0	1.38	28	82.6	2.16	25	80.2
1.02	32	86.2	1.40	30	84.0	2.21	30	84.8
1.04	34	86.8	1.43	33	85.8	2.23	32	85.6
1.06	36	87.0	1.45	35	86.2	2.25	34	86.2
1.07	37	87.2	1.46	37	86.4	2.27	36	86.4
SAMPLES: D-1-2-3								
2.31	0	0	3.07	0	0	3.43	0	0
2.36	5	10.6	3.27	20	71.6	3.48	5	10.4
2.42	11	36.0	3.32	25	81.2	3.58	15	50.8
2.46	15	52.4	3.35	28	83.8	4.03	20	69.4
2.51	20	71.2	3.37	30	85.0	4.08	25	81.2
2.56	25	81.6	3.39	32	86.0	4.13	30	86.0
3.00	29	85.2	3.41	34	86.6	4.15	32	86.0
3.02	31	86.4	3.44	37	86.6	4.18	35	86.0

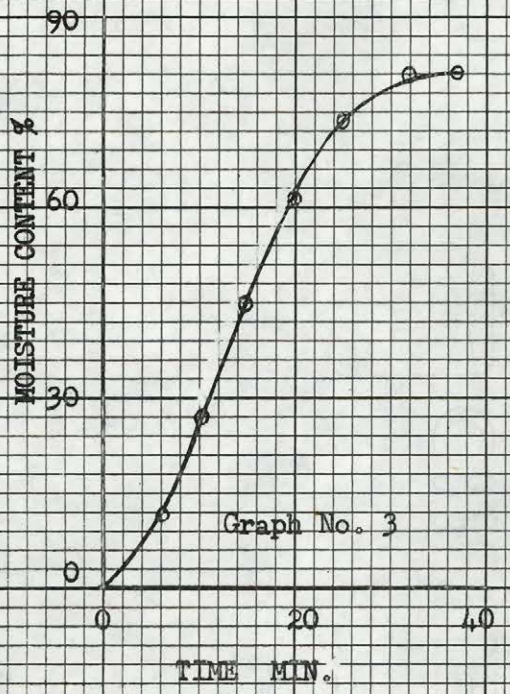
Observations (continued)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist. %
SAMPLES: E-1-2-3								
4.18	0	0	4.52	0	0	5.25	0	0
4.23	5	10.8	5.12	20	72.2	5.46	21	69.6
4.28	10	32.4	5.17	25	81.2	5.50	25	78.0
4.33	15	54.0	5.20	28	85.0	5.53	28	81.4
4.38	20	72.8	5.22	30	86.2	5.55	30	83.2
4.43	25	81.6	5.24	32	86.4	5.59	34	85.6
4.48	30	86.0	5.26	34	86.4	6.01	36	86.2
4.50	32	86.2	5.28	36	86.4	6.03	38	86.2
SAMPLES: F-1-2-3								
6.06	0	0	6.50	0	0	7.30	0	0
6.11	5	10.4	7.00	10	30.2	7.50	20	68.0
6.16	10	31.2	7.05	15	50.2	7.53	23	75.4
6.21	15	51.4	7.10	20	67.8	7.56	26	79.8
6.26	20	69.8	7.15	25	79.2	8.00	30	84.0
6.31	25	79.4	7.18	28	82.8	8.02	32	85.4
6.36	30	84.0	7.20	30	84.4	8.04	34	86.4
6.38	32	85.2	7.22	32	85.2	8.06	36	86.6
6.40	34	86.2	7.24	34	86.2	8.08	38	86.6
6.42	36	86.4	7.26	36	86.4			

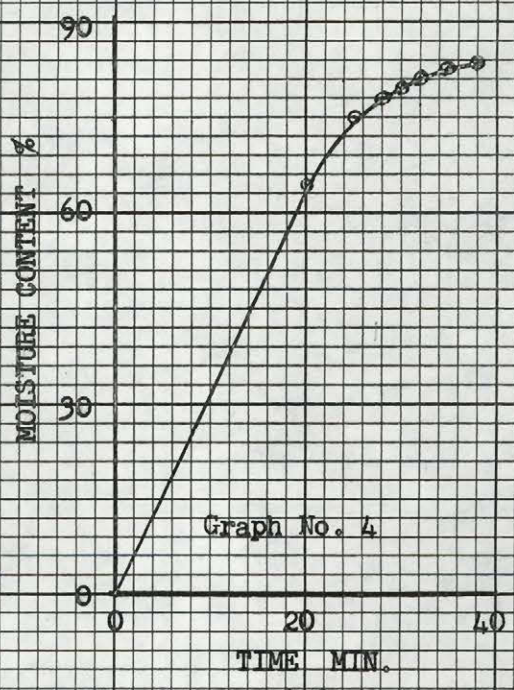
N.B. Analyses of samples not shown above were carried out at the Pointe Claire Laboratories of PPRIC.

TEST No. 4 - FIRST DAY
8 a.m. October 24, 1963.

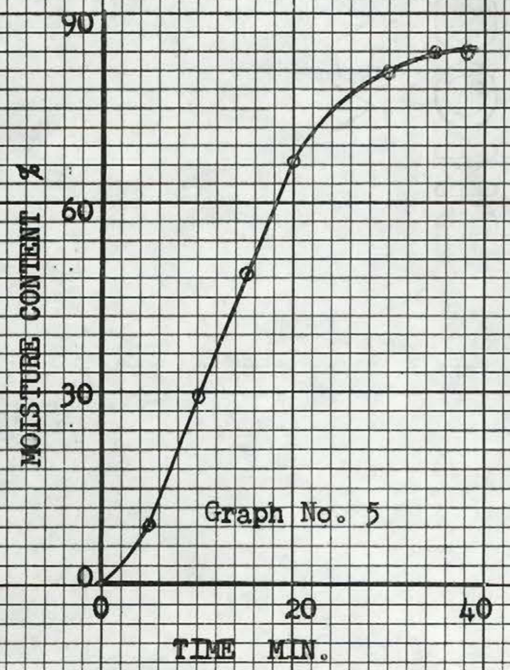
SLUDGE DRYING CURVES



A-1

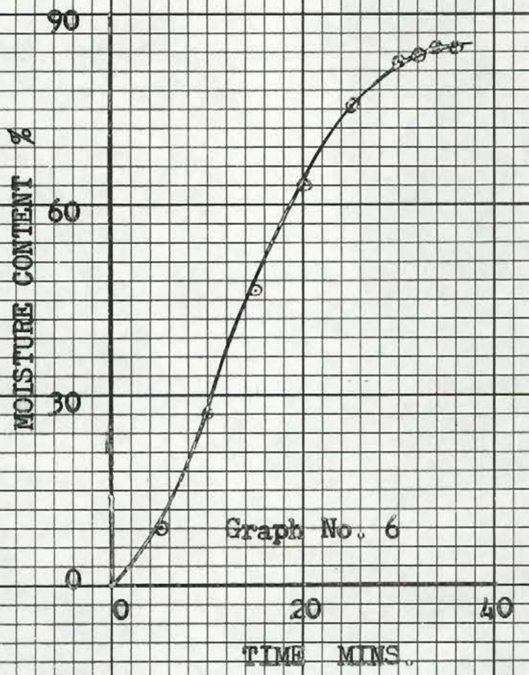


A-2



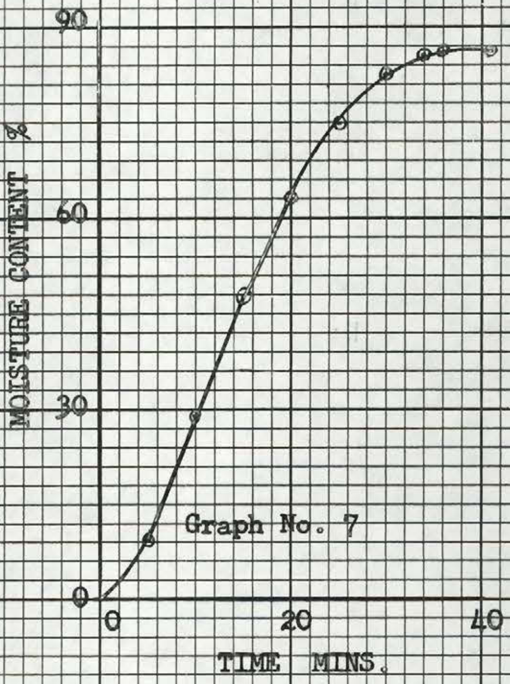
A-3

OBSERVATIONS (CONT'D.) - 10 a.m. October 24, 1963.



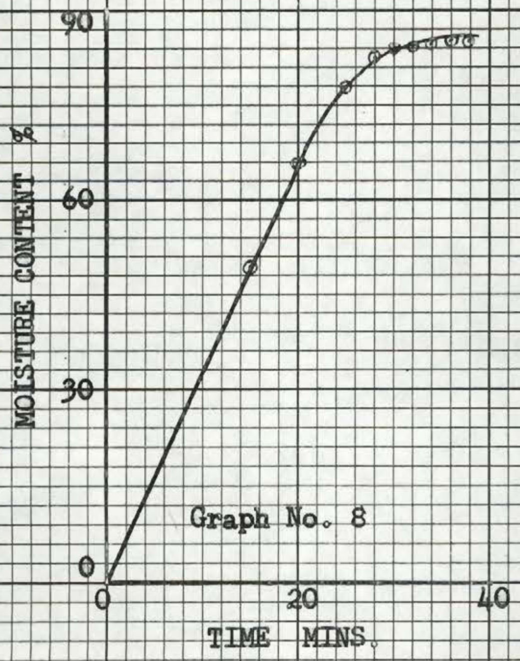
Graph No. 6

B-1



Graph No. 7

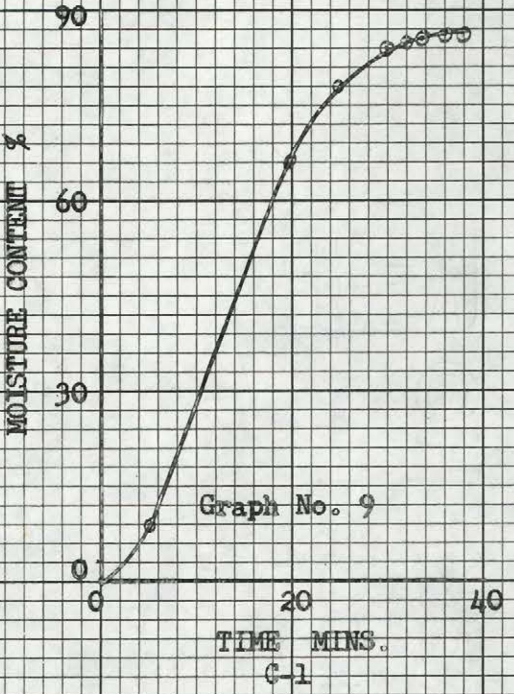
B-2

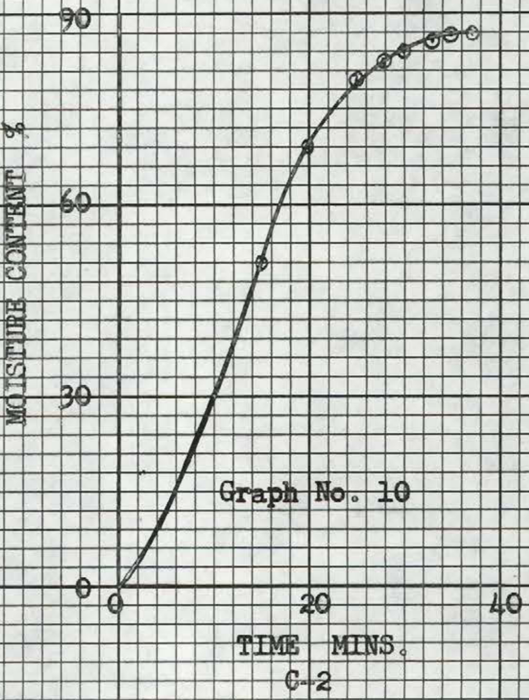


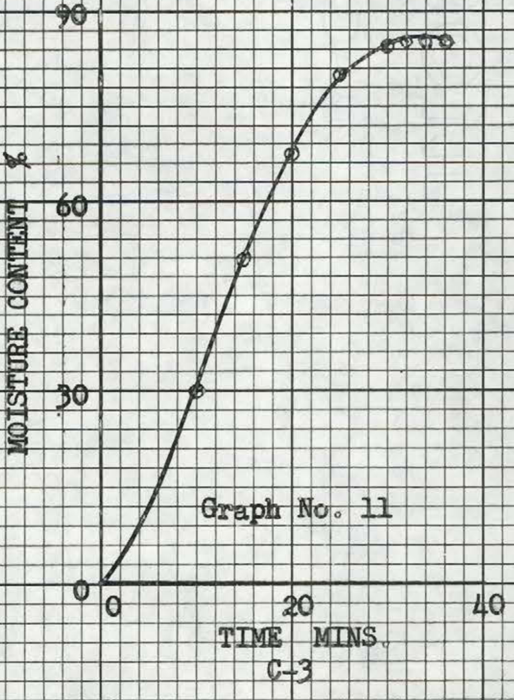
Graph No. 8

B-3

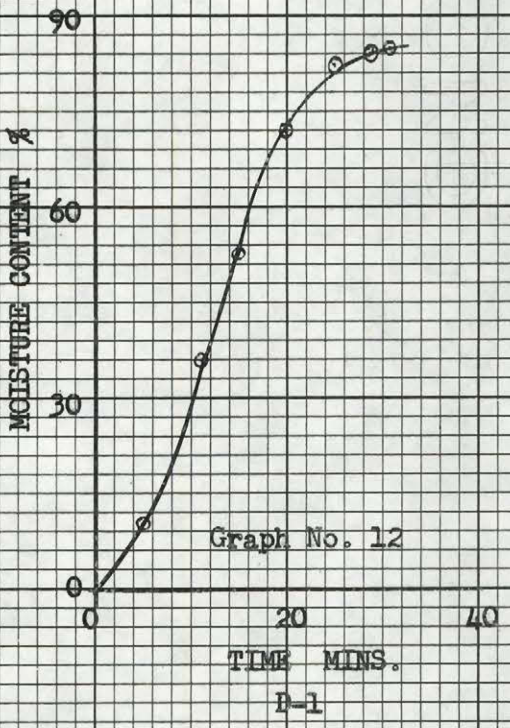
OBSERVATIONS (CONT'D.) - 12 noon October 24, 1963

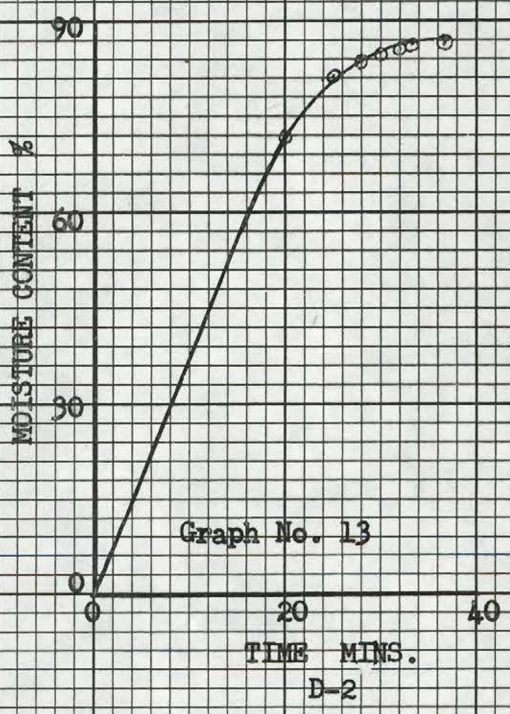


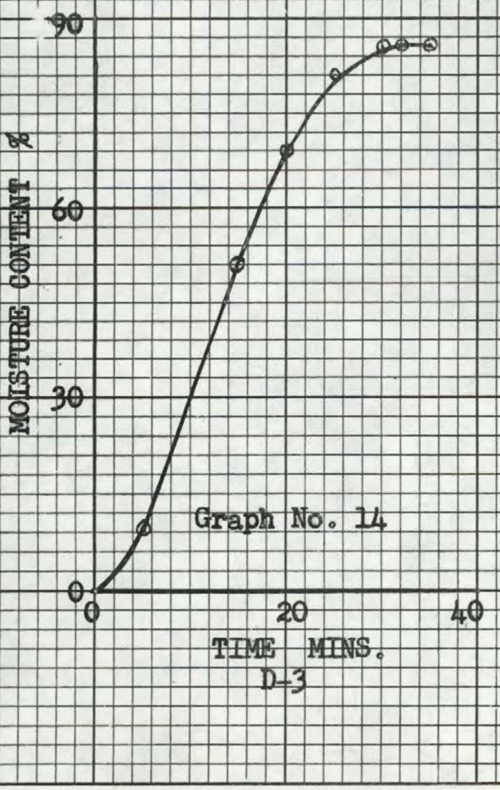




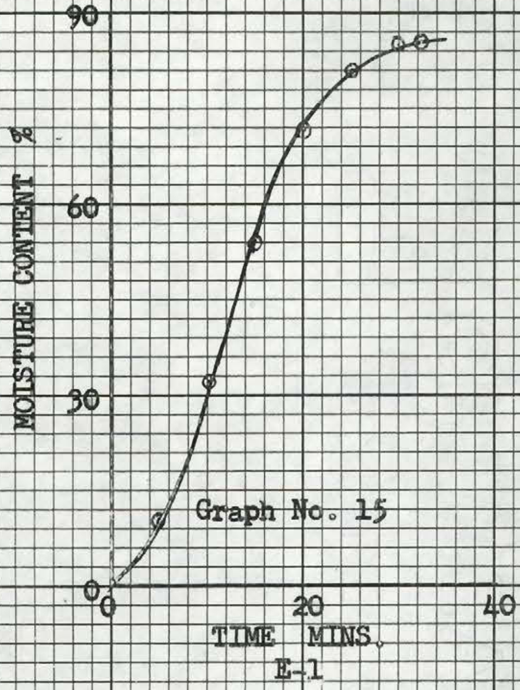
OBSERVATIONS (CONT'D.) - 2 p.m. October 24, 1963


Graph No. 12

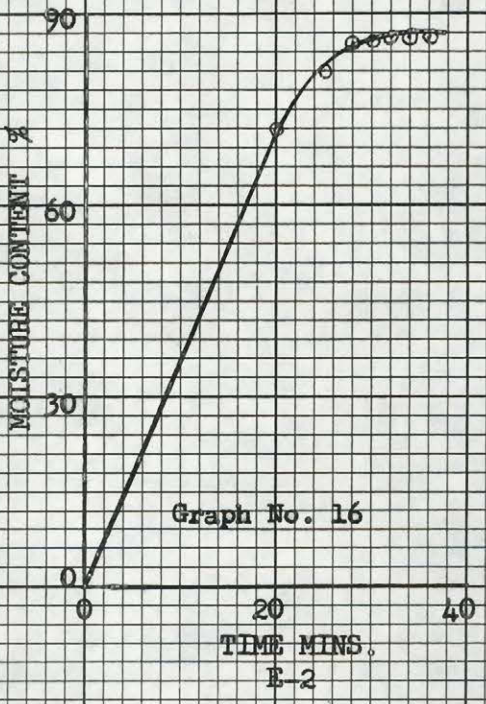

Graph No. 13


Graph No. 14

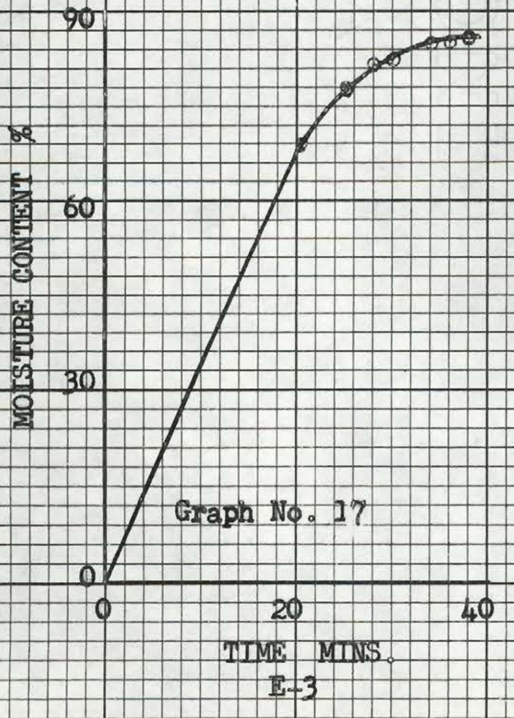
OBSERVATIONS (CONT'D.) - 4 p.m. October 24, 1963.



Graph No. 15

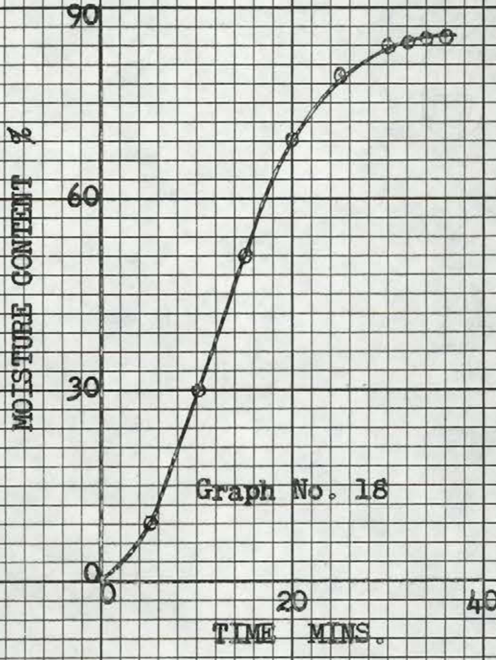


Graph No. 16



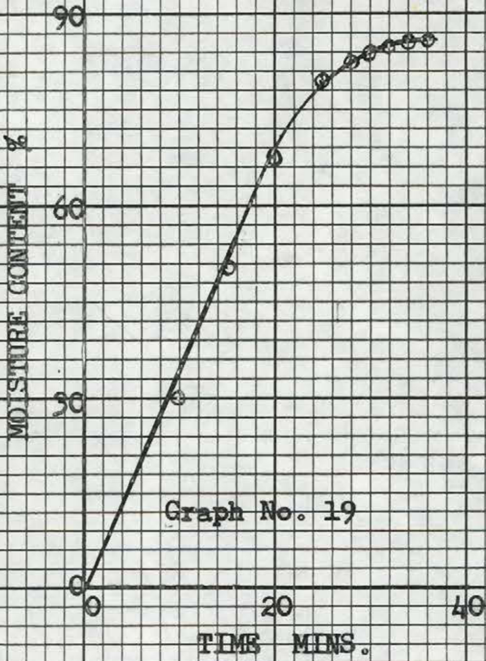
Graph No. 17

OBSERVATIONS (CONT'D.) - 6 p.m. October 24, 1963



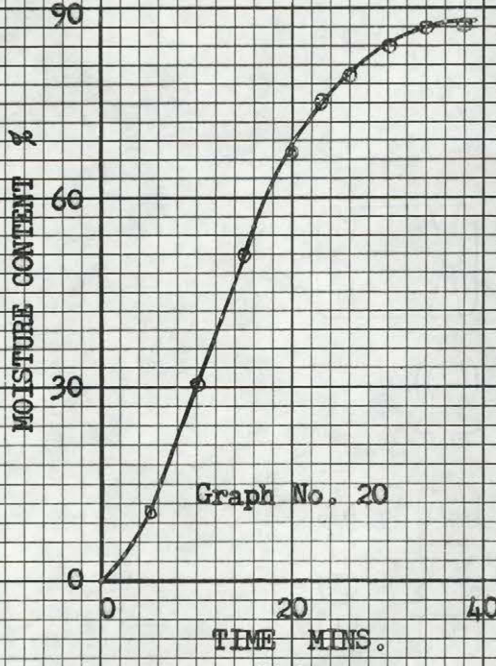
Graph No. 18

F-1



Graph No. 19

F-2



Graph No. 20

F-3

SAMPLE OBSERVATIONS

Test No. 4 First Day

Solids Content - Using Standard Laboratory Procedure (*1)

Time	Sample mark	Container empty (zero wt) grams	Zero wt. + sludge sample grams	Zero wt. + dry solids grams	Wt. of sludge sample grams	Wt. of dry solids grams	Moisture content %	Solids content %
8 a.m.	A-4	1.4853	24.6649	5.1124	23.1796	3.6271	84.35	15.65
	A-4	1.4824	24.7493	5.1599	23.2669	3.6775	84.19	15.81
10 a.m.	B-4	1.4735	24.3986	4.7974	22.9251	3.3239	85.51	14.49
	B-4	1.4708	24.7445	4.8340	23.2737	3.3632	85.31	14.69
12 noon	C-4	1.4664	24.9464	4.7524	23.4800	3.2860	86.00	14.00
	C-4	1.4688	25.6256	5.0164	24.1568	3.5476	85.31	14.69
2 p.m.	D-4	1.4718	24.336	4.7178	22.8618	3.2460	85.80	14.20
	D-4	1.4698	24.0119	4.5994	22.5421	3.1296	86.12	13.88
4 p.m.	E-4	1.4674	23.0668	4.4162	21.5994	2.9488	86.35	13.65
	E-4	1.4670	22.7724	4.4248	21.3054	2.9578	86.12	13.88
6 p.m.	F-4	1.4712	27.0684	5.0452	25.5972	3.5740	86.04	13.96
	F-4	1.4724	24.8146	4.7640	23.3722	3.2916	85.90	14.10
8 p.m.	G-4	1.4850	22.6492	4.4224	21.1642	2.9374	86.12	13.88
	G-4	1.4868	21.9974	4.2779	20.5106	2.7911	86.39	13.61
10 p.m.	H-4	1.4880	25.4480	4.8940	23.9600	3.4060	86.32	13.68
	H-4	1.4879	25.0043	4.6906	23.5164	3.2027	86.38	13.62
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)

Note

Columns (1) (2) and (3) are self-explanatory.

Columns (4) and (5) are the 1st and 2nd weighings respectively.

Columns (6) and (7) are computed and self-evident.

Column (8) is 100 - Column (9).

Column (9) is Column (7) divided by Column (6) multiplied by 100%.

SAMPLE OBSERVATIONS

Test No. 4 First Day

Inorganic Content - using Standard Laboratory Procedure (*1)

Time	Sample mark	Container empty (zero wt) grams	Zero wt. + solid sample grams	Zero weight + inorganic grams	Weight of dry solids grams	Weight of inorganic grams	Volatile content %	* Inorganic content %
8 a.m.	A-4	16.7879	18.1252	17.2000	1.3373	0.4121	69.18	30.82
	A-4	16.5439	18.2473	17.0719	1.7034	0.5280	69.01	30.99
10 a.m.	B-4	16.9678	18.6478	17.4775	1.6800	0.5099	69.66	30.34
	B-4	16.0240	17.5202	16.4766	1.4962	0.4526	69.75	30.25
12 noon	C-4	16.3625	18.0055	16.8517	1.6430	0.4892	70.23	29.77
	C-4	15.5248	17.3868	16.3468	1.4620	0.4220	71.14	28.86
2 p.m.	D-4	16.1051	17.4961	16.5103	1.3900	0.4042	70.92	29.08
	D-4	15.6526	17.0353	16.0462	1.3827	0.3936	71.53	28.47
4 p.m.	E-4	15.9581	17.4523	16.3928	1.4942	0.4347	70.91	29.09
	E-4	16.3633	17.7491	16.7684	1.3858	0.4051	70.76	29.24
6 p.m.	F-4	15.9251	17.3824	16.3670	1.4573	0.4419	69.68	30.32
	F-4	16.0451	17.2800	16.4087	1.2349	0.3636	70.56	29.44
8 p.m.	G-4	15.7599	16.9315	16.0966	1.1716	0.3367	71.26	28.74
	G-4	15.5016	16.6270	15.8301	1.1254	0.3285	70.81	29.19
10 p.m.	H-4	15.8946	17.1181	16.2391	1.2235	0.3445	71.84	28.16
	H-4	16.8590	18.0789	17.2061	1.2199	0.3471	71.55	28.45
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)

Note

Columns (1), (2) and (3) are self-explanatory.

Columns (4) and (5) are the 1st and 2nd weighings respectively.

Columns (6) and (7) are computed and self-evident.

Column (8) is 100 - Column (9)

Column (9) is Column (7) divided by Column (6) x 100%.

N.B. * Average of ash contents to be included in sample calculations for standard fluctuation and coefficient of fluctuation.

TABLE V

RESULTS OF OBSERVATIONS ON SLUDGE CONDITIONING

Item:-	TEST NO. 1		TEST NO. 2		TEST NO. 3		TEST NO. 4		TEST NO.5	TEST NO. 6
Date:	28.8.63	29.8.63	3.9.63	4.0.63	10.10.63	11.10.63	23.10.63	24.10.63	24.9.63	29.9.63
Pumping into thickener	Continuous pumping at 150 gpm				12 hours pumping at 150 gpm 8 a.m. - 8 p.m.				Continuous pumping at 60 gpm, part of primary sludge recirculated.	
Sludge depth in thickener, ft.	3	3	6	6	3	3	6	6	3	6
Sewage flow, mgd.	1.14	1.23	1.12	1.21	1.13	1.15	1.09	0.91	1.23	1.39
Sludge underflow, gpd.	805	830	645	640	820	930	530	615	835	635
Solids content %	15.18	14.48	15.60	15.69	11.36	10.99	14.08	14.31	11.30	13.43
Volatile content %	59.24	59.53	60.04	59.02	59.45	61.42	70.25	70.68	67.40	67.14
Specific gravity	1.034	1.037	1.048	1.046	1.035	1.031	1.033	1.029	1.024	1.059
Sludge volume ratio, days	3.58	3.43	8.95	9.00	3.51	3.10	10.85	9.35	3.45	9.07
Wt. of dry solids lb/mg. of sewage	1,107	1,013	940	870	852	916	707	994	786	650
Wt. of organics lb/mg. of sewage	655	605	563	512	505	562	497	700	530	438

OBSERVATIONS TABLE VI

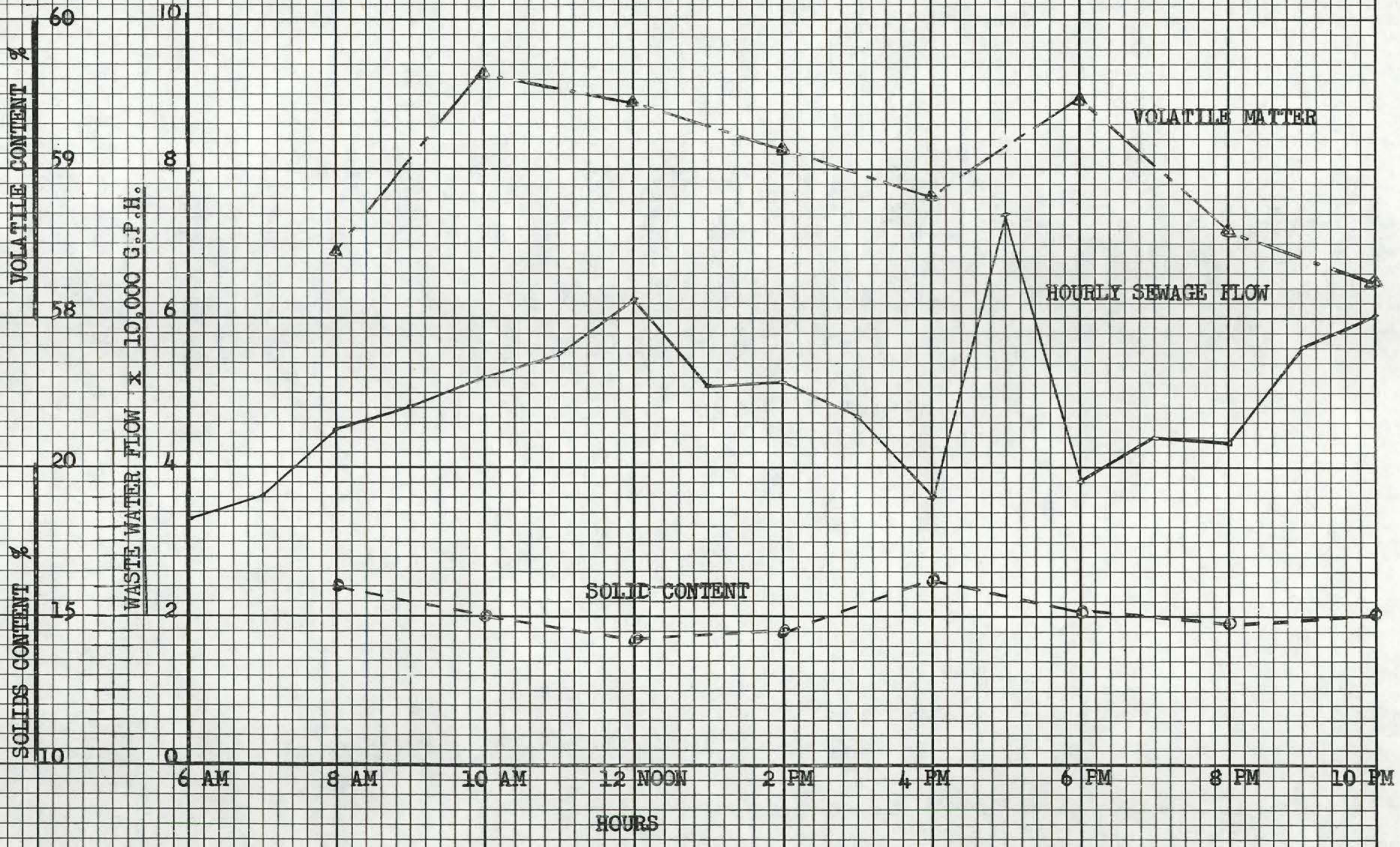
WASTE WATER FLOW - Quantity x 100 gallons

1963

		August		September				October			
		28	29	3	4	24	29	10	11	23	24
A.M.	1	468	450	480	430	529	563	422	430	507	330
	2	430	377	450	356	444	503	365	400	398	340
	3	388	322	398	359	383	530	350	440	320	270
	4	388	452	295	306	371	572	358	396	344	330
	5	409	456	420	318	431	520	375	368	456	290
	6	329	470	400	335	426	388	370	380	526	285
	7	356	468	384	288	446	453	423	378	931	285
	8	450	526	373	338	540	555	384	338	427	339
	9	479	664	488	665	480	499	633	674	430	506
	10	515	538	488	586	564	593	399	432	470	465
	11	552	636	637	636	750	530	604	483	500	465
	12	625	566	494	577	442	610	587	479	600	451
P.M.	1	510	544	473	609	550	588	598	676	450	466
	2	514	535	555	621	616	460	378	502	410	498
	3	468	629	524	542	567	621	561	468	440	370
	4	366	395	496	678	477	623	503	516	458	260
	5	736	622	366	474	510	546	547	567	487	477
	6	382	503	469	650	534	517	454	551	316	396
	7	445	538	399	513	627	634	419	523	305	403
	8	435	497	552	601	500	652	470	460	439	414
	9	560	528	473	590	494	731	515	547	287	452
	10	606	376	618	558	559	530	590	539	546	447
	11	465	594	451	516	569	671	510	430	343	406
	12	554	630	507	529	519	961	444	523	489	445

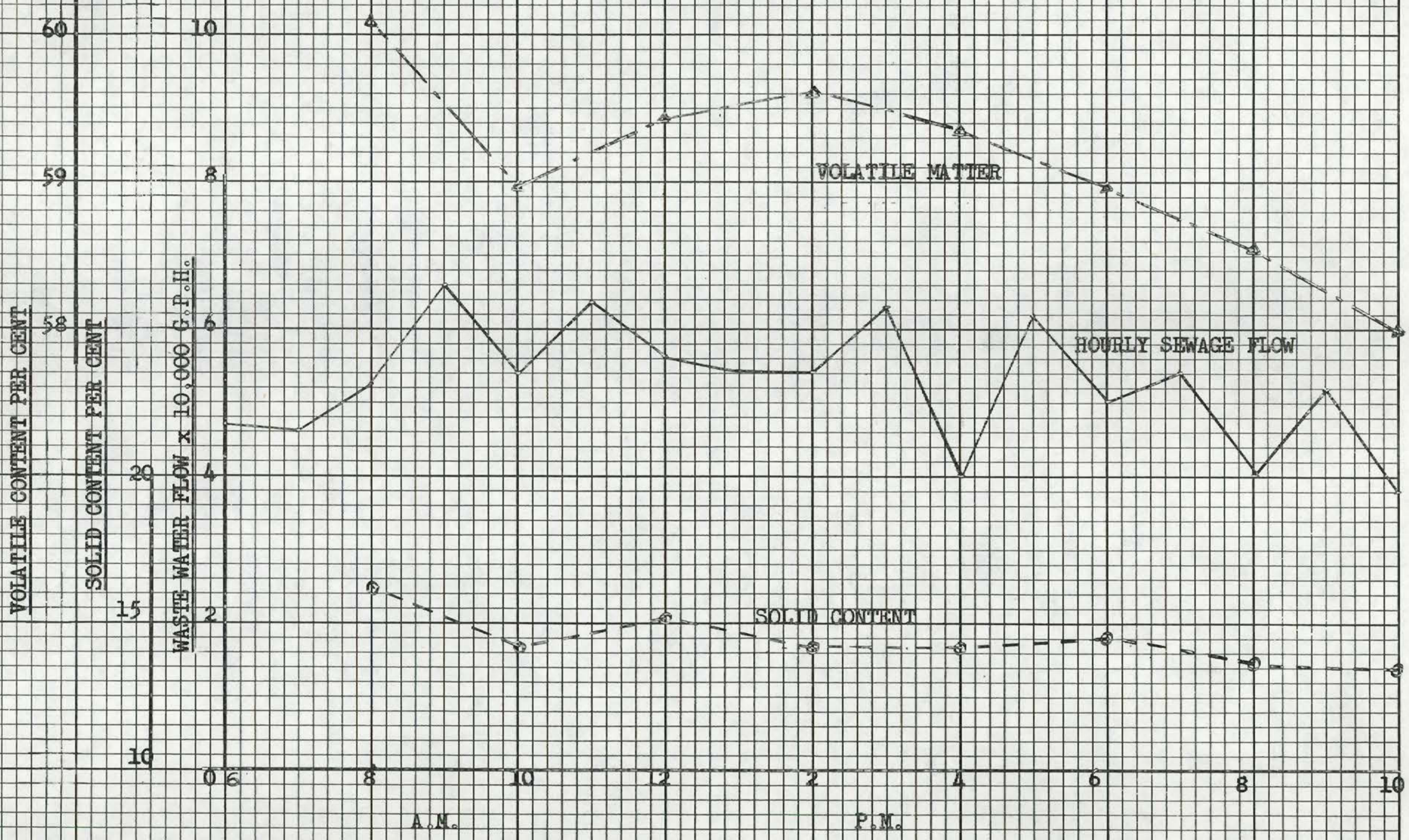
VARIATIONS OF WASTE WATER FLOW; VOLATILE MATTER & SOLID CONTENT - Graph No. 21

TEST NO. 1 - First Day, August 28th, 1963



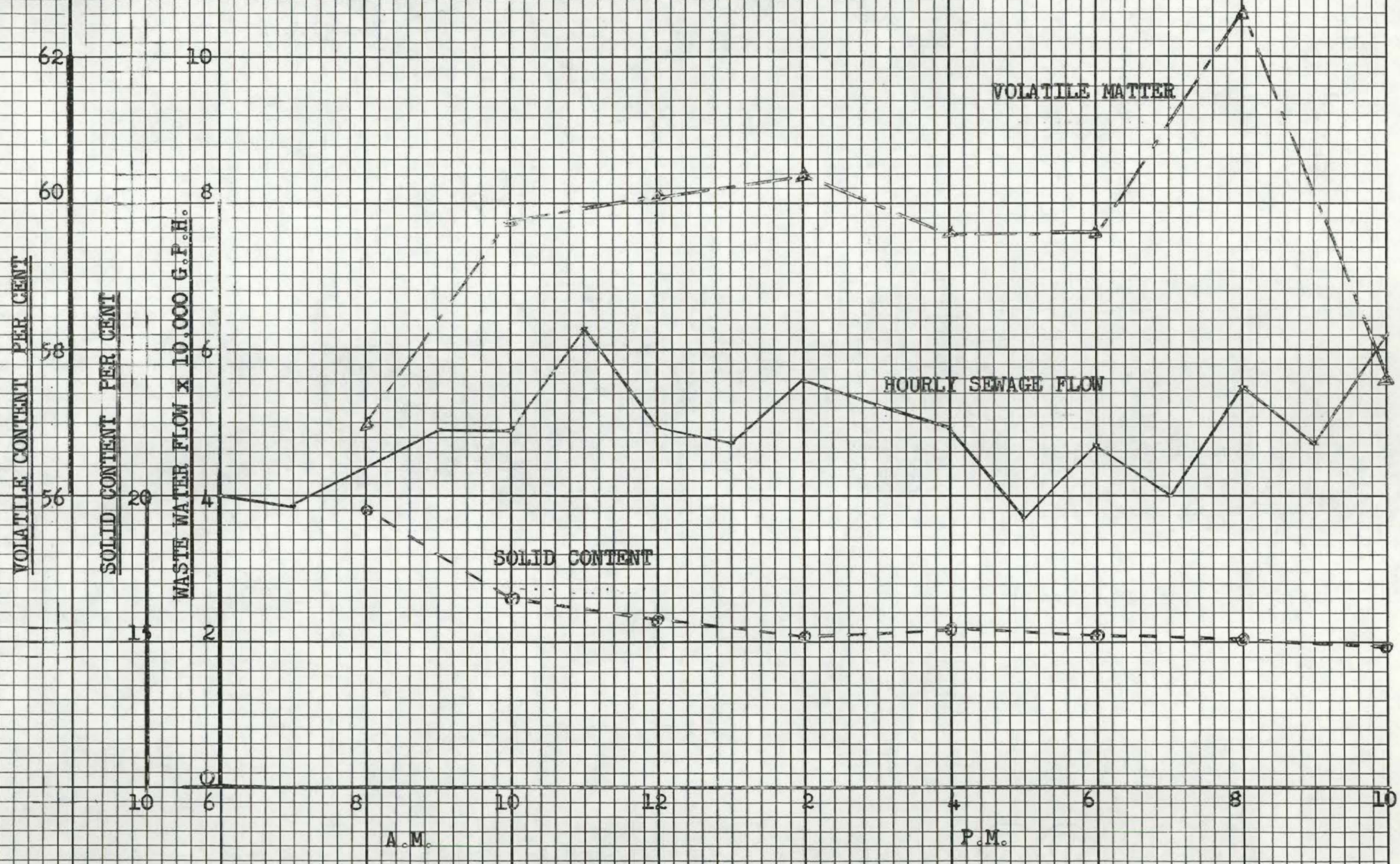
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER & SOLID CONTENT — Graph No. 22

TEST NO. 1 - Second Day, August 29th, 1963



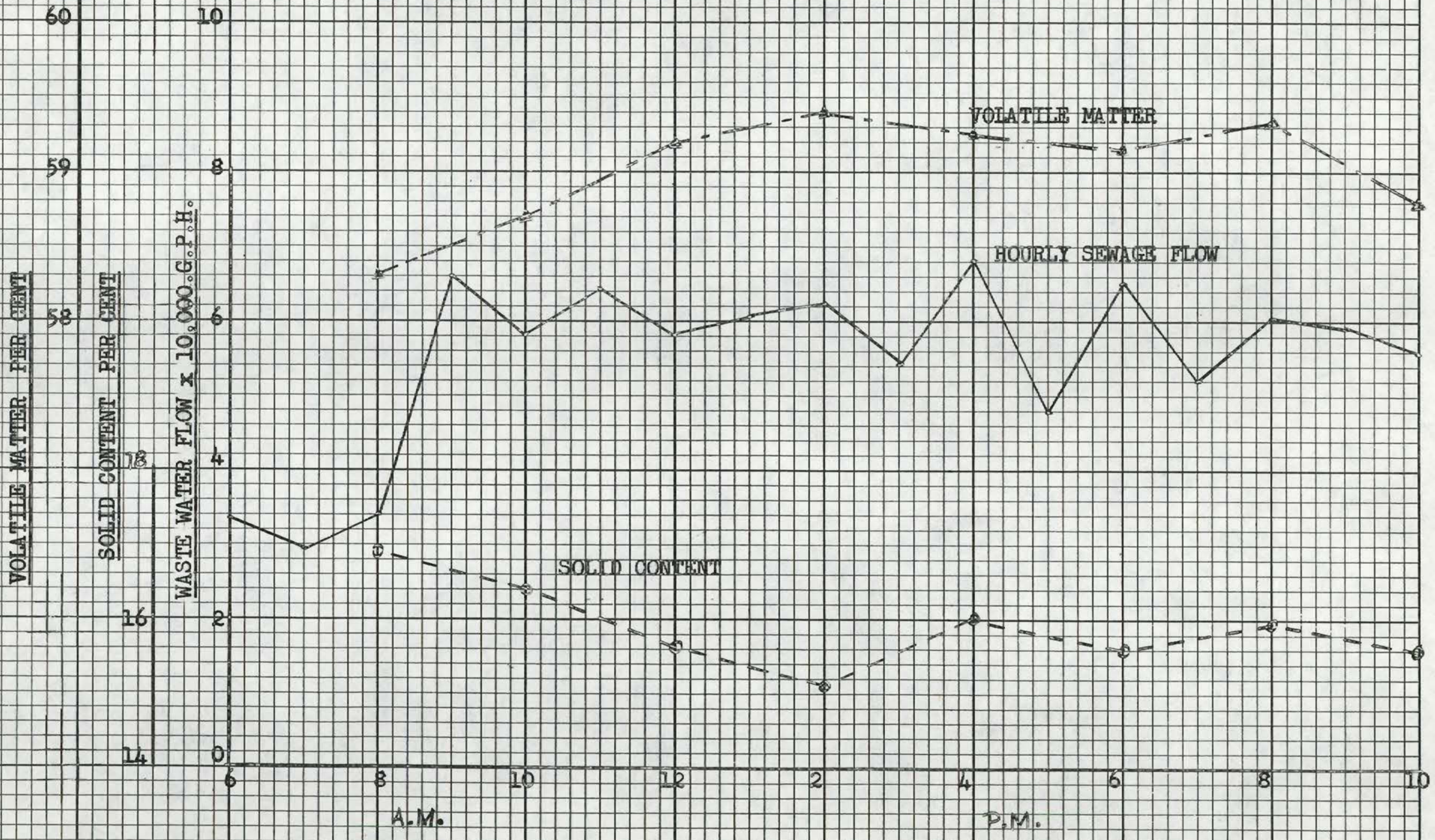
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER & SOLIDS CONTENT - Graph No. 23

TEST NO. 2 - First day, September 3, 1963



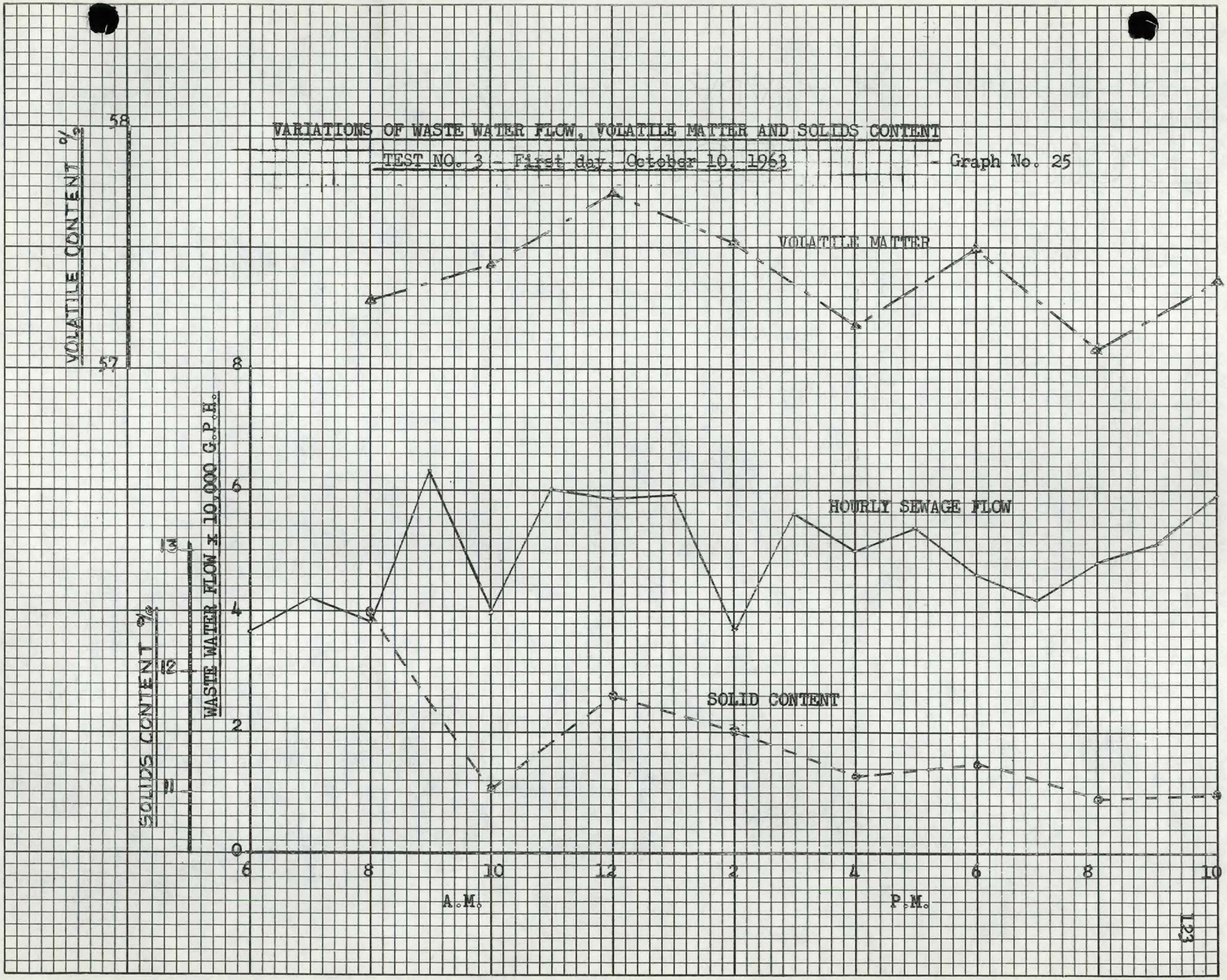
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER & SOLIDS CONTENT - Graph No. 24

TEST NO. 2 - Second Day, September 4, 1963.



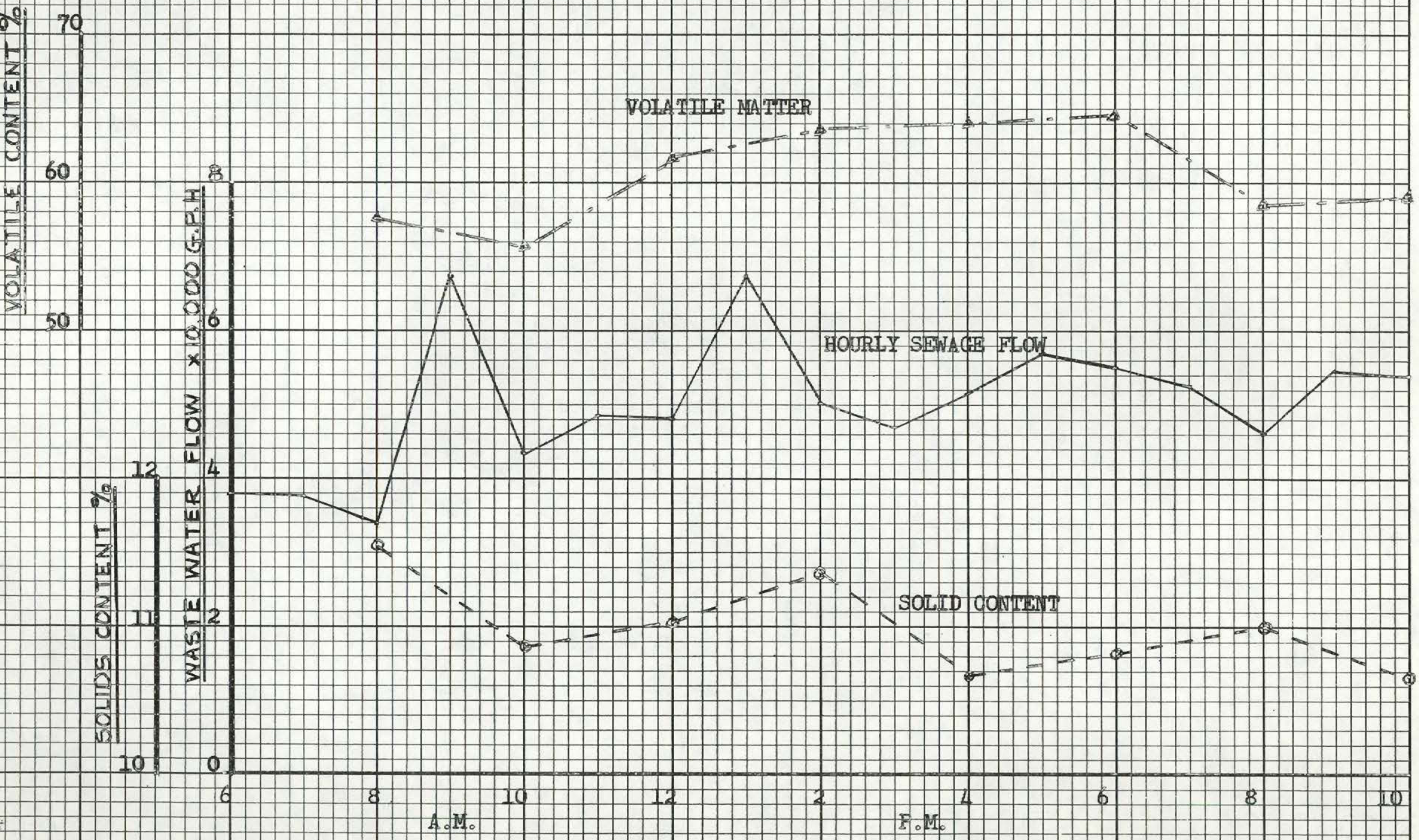
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER AND SOLIDS CONTENT

TEST NO. 3 - First day, October 10, 1963 - Graph No. 25



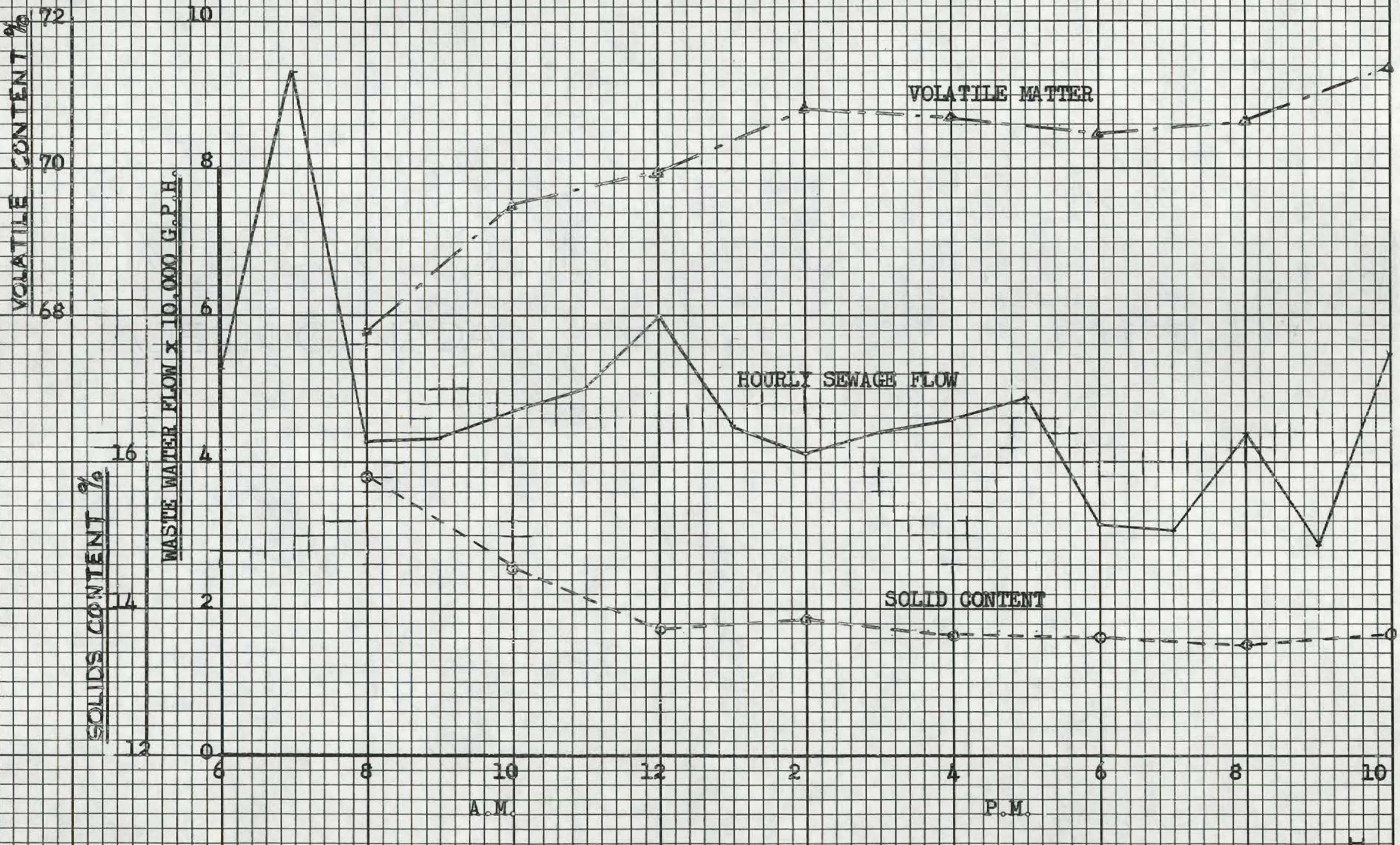
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER AND SOLIDS CONTENT - Graph No. 26

TEST NO. 3 - Second day, October 11, 1963



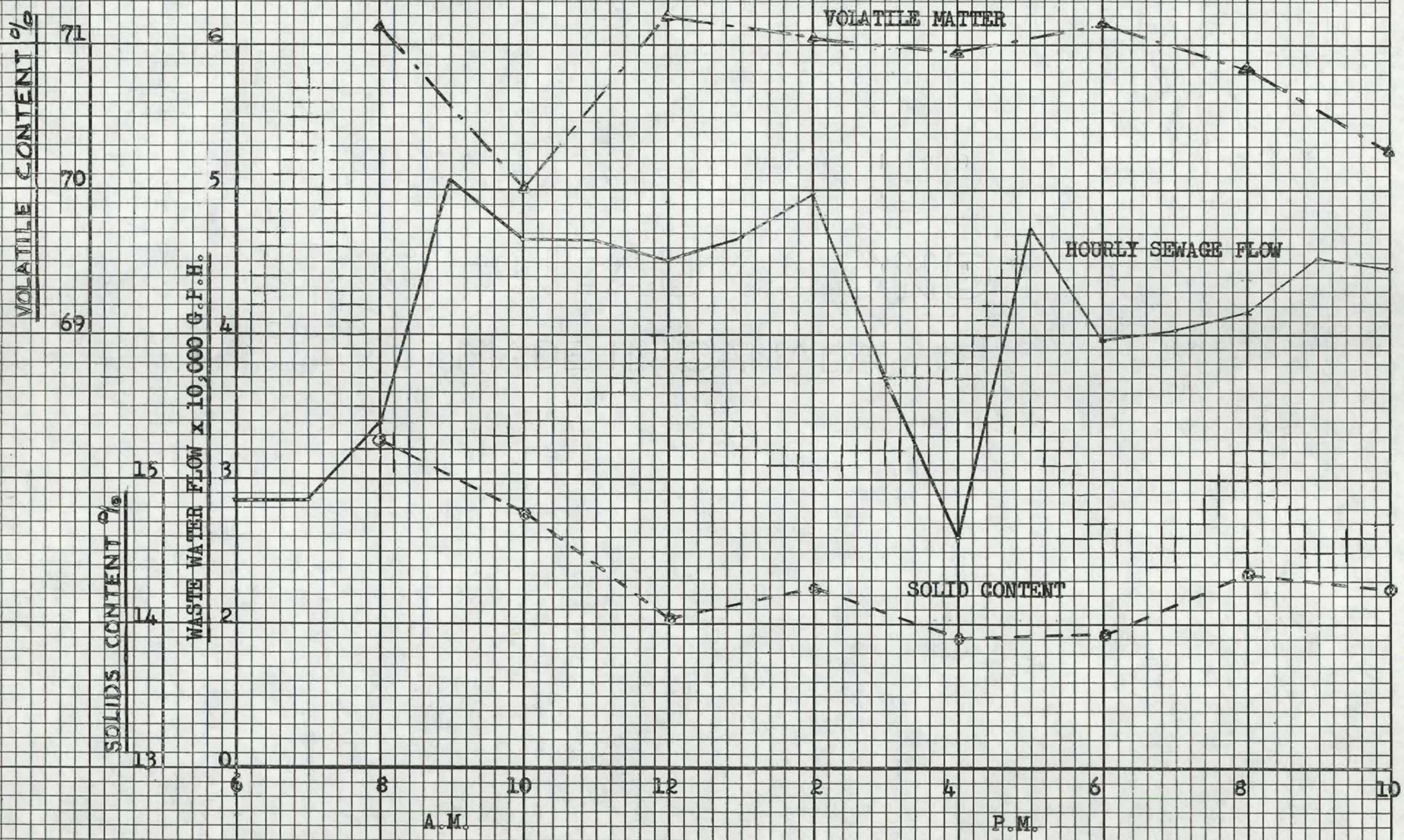
VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER AND SOLIDS CONTENT - Graph No. 27

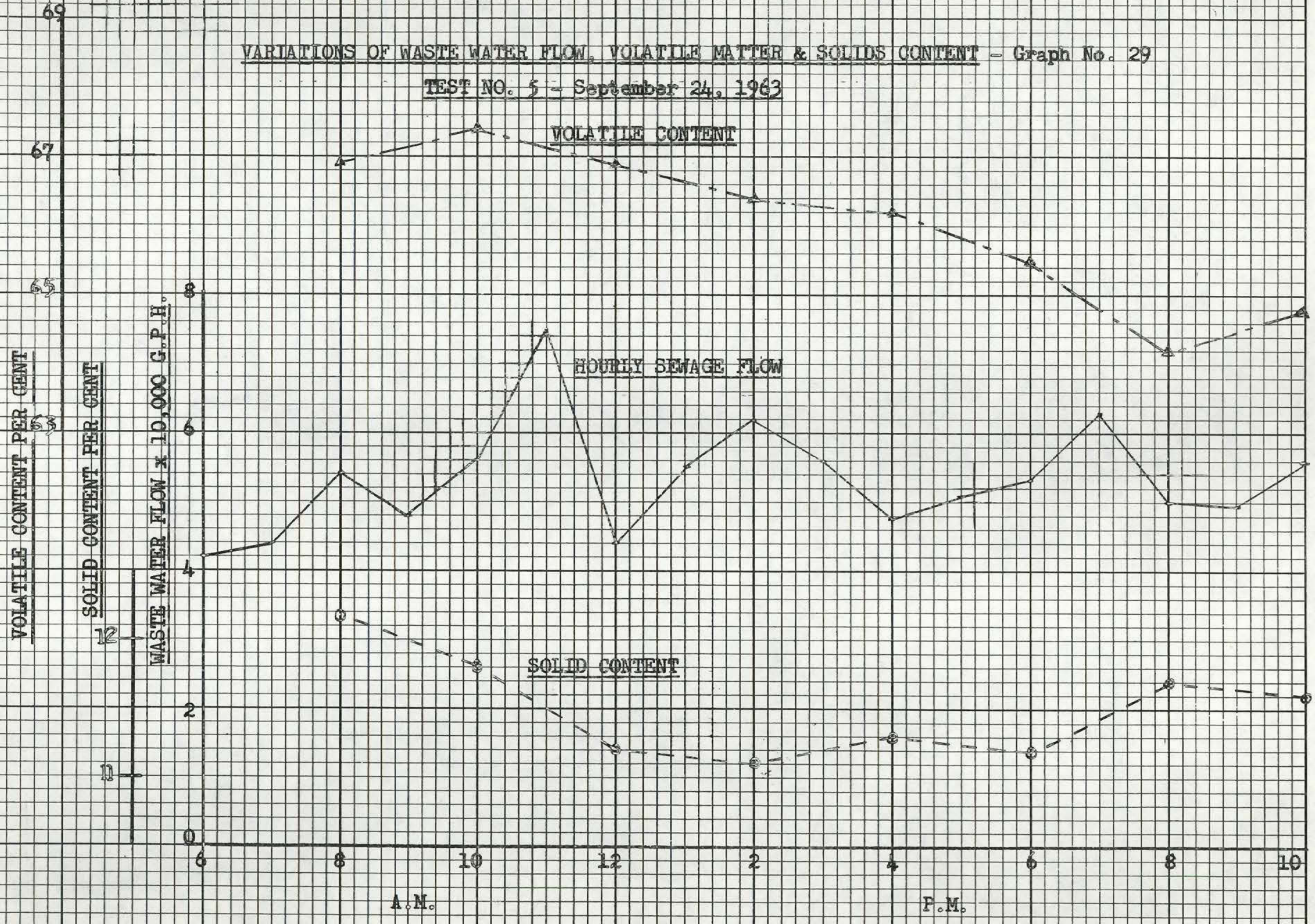
TEST NO. 4 - First day, October 23, 1963



VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER AND SOLIDS CONTENT - Graph No. 28

TEST NO. 4 - Second Day, October 24, 1963




VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER & SOLIDS CONTENT - Graph No. 29

TEST NO. 5 - September 24, 1963

VARIATIONS OF WASTE WATER FLOW, VOLATILE MATTER & SOLID CONTENT - Graph No. 30

TEST NO. 6 - September 29, 1963

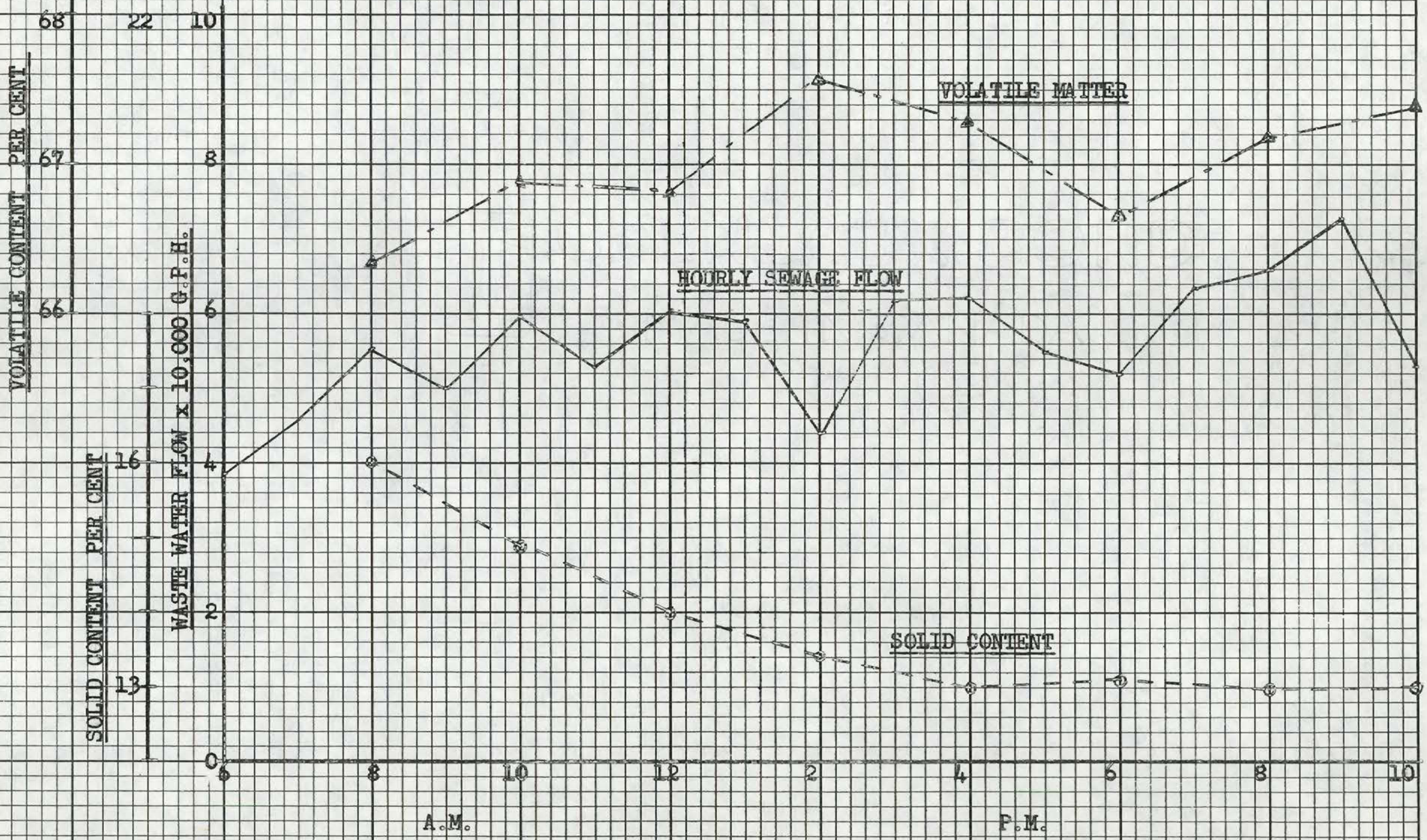


TABLE VII - RESULTS OF OBSERVATIONS ON TREATMENT PLANT UNITS

Date	Test No. 7 18.5.64	Test No. 8 31.5.64	Test No. 9 8.6.64	Test No. 10 21.6.64	Test No. 11 28.6.64	Test No. 12 5.7.64	Test No. 13 12.7.64	Test No. 14 19.7.64
Waste Water Flow m.g.d.	1.28	1.18	1.09	1.96	1.02	1.08	1.36	1.03
CLARIFIER								
Surface loading g.p.d./sq.ft.	385	355	328	286	307	328	410	310
Detention period, hours	1.94	2.11	2.28	2.59	2.44	2.30	1.82	2.41
Suspended solids removal %	67	63	47	55	60	66	47	60
C.O.D. removal %	18	21	36	20	38	29	18	34
Sludge - solids content %	0.65	1.94	1.28	1.04	1.14	1.70	1.60	1.70
THICKENER								
Sludge depth feet	6	4	5	4	4	3	3	5
Sludge - solids content %	13.2	11.7	12.2	11.8	11.9	14.0	10.3	13.0
Volatile content %	62.60	67.03	75.0	77.54	73.60	67.07	78.2	67.18
S.V.R.	8.73	4.85	6.32	4.67	4.26	3.08	2.94	5.25
CHLORINATION								
C.O.D. removal %	49	46	65	29	-	32	30	42
PLANT								
Suspended solids removal %	61	66	41	42	-	63	57	59

OBSERVATIONS

WASTE WATER FLOW

TABLE VIII

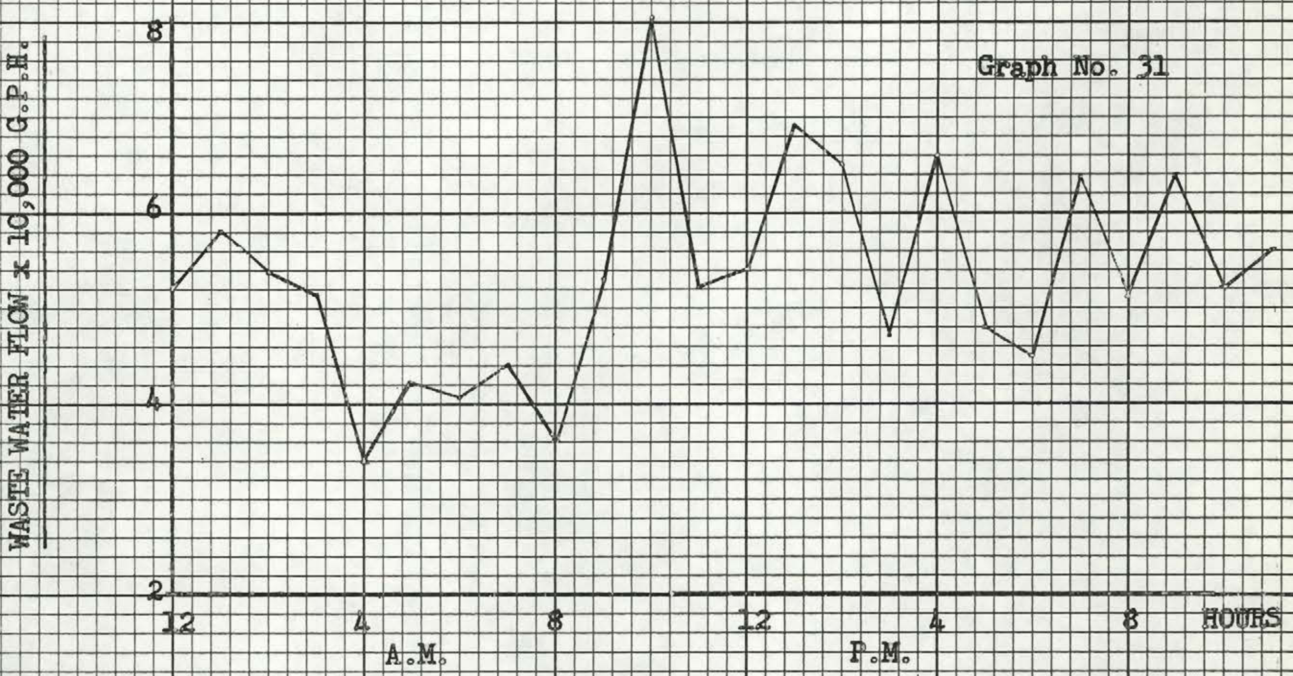
Quantity x 100 gallons

1 9 6 4

		May		June			July		
		18	31	8	21	28	5	12	19
A.M.	1	580	469	454	403	426	451	580	400
	2	535	460	408	352	396	330	527	387
	3	515	579	332	318	381	340	516	363
	4	338	409	390	376	189	320	534	345
	5	442	620	203	299	316	400	409	348
	6	415	360	362	214	333	390	376	337
	7	449	294	361	294	216	465	344	310
	8	357	430	418	410	380	473	407	363
	9	533	414	475	454	380	406	520	398
	10	806	466	610	403	380	445	603	459
	11	515	504	610	492	440	620	692	489
	12	538	544	595	481	510	420	680	546
P.M.	1	713	512	505	525	480	520	595	498
	2	651	560	578	442	390	470	710	450
	3	477	480	477	439	460	610	660	486
	4	665	510	345	442	430	363	685	463
	5	481	520	513	430	495	556	660	500
	6	451	545	498	384	335	343	532	410
	7	639	440	440	400	410	429	550	450
	8	514	525	640	460	367	472	624	480
	9	639	540	359	450	327	470	634	470
	10	521	460	542	420	562	446	521	410
	11	556	470	376	450	462	431	543	490
	12	525	641	553	429	558	629	628	485
Totals		12,835	11,752	10,944	9,667	9,623	10,799	13,630	10,337

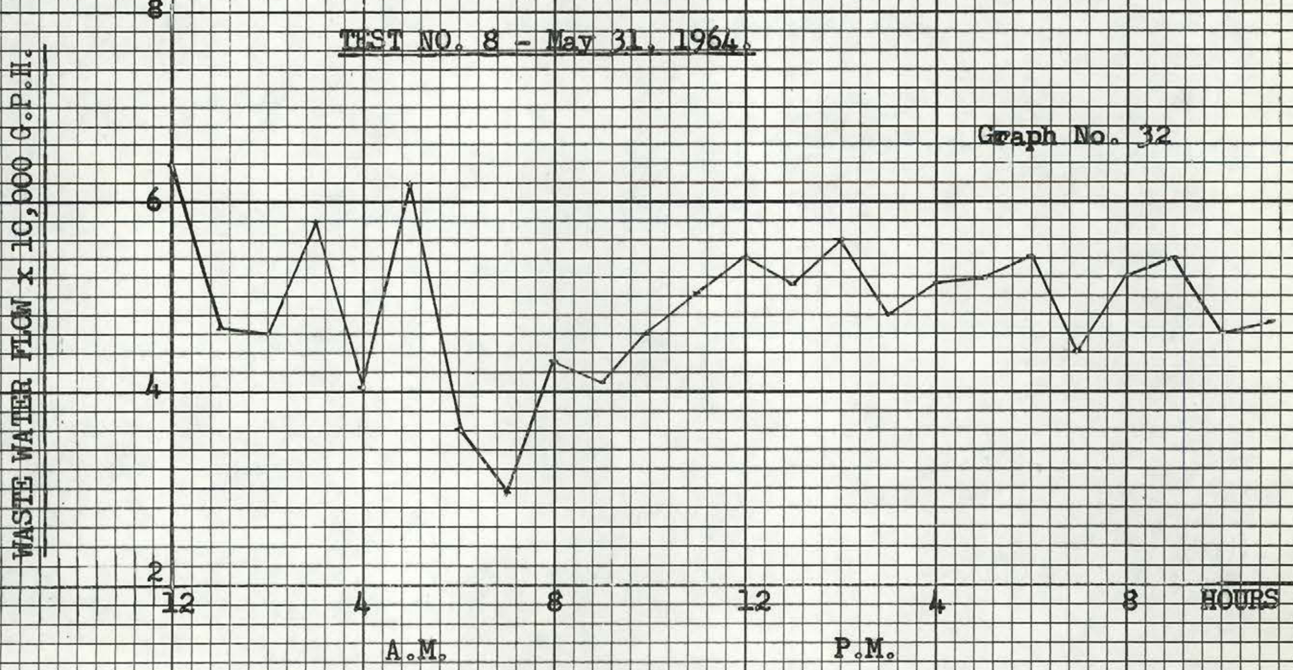
VARIATIONS OF WASTE WATER FLOW

TEST NO. 7 - May 18, 1964.



Graph No. 31

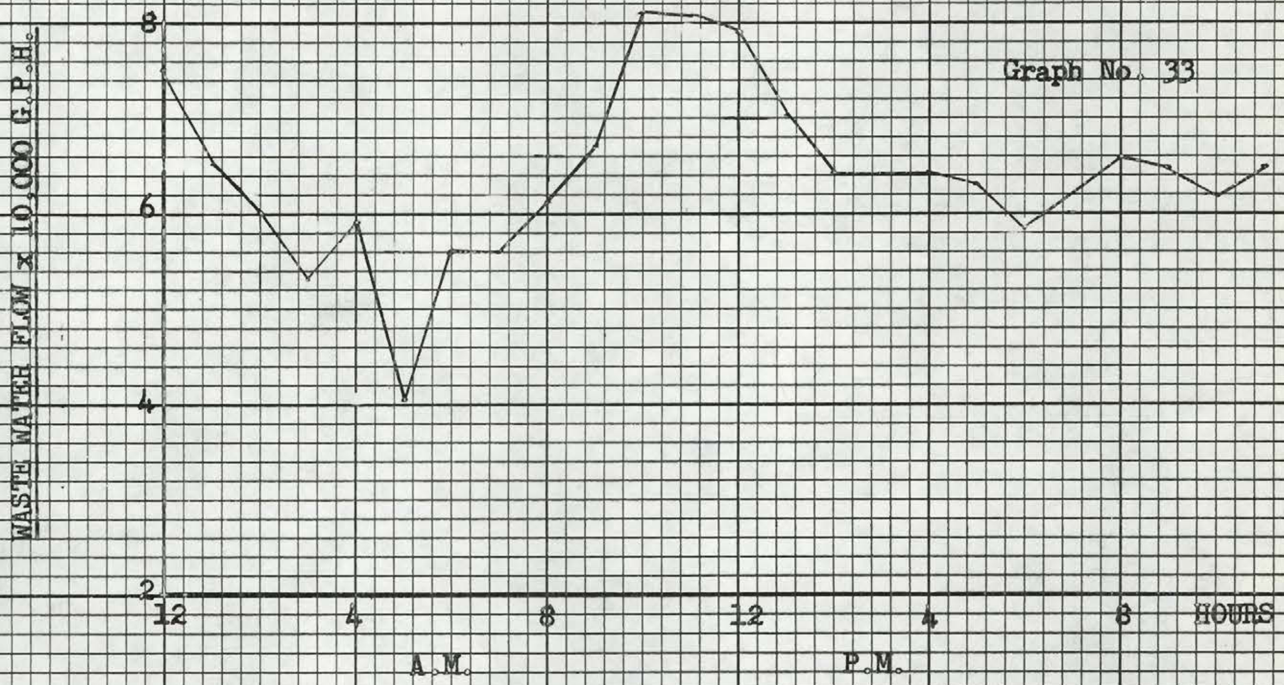
TEST NO. 8 - May 31, 1964.



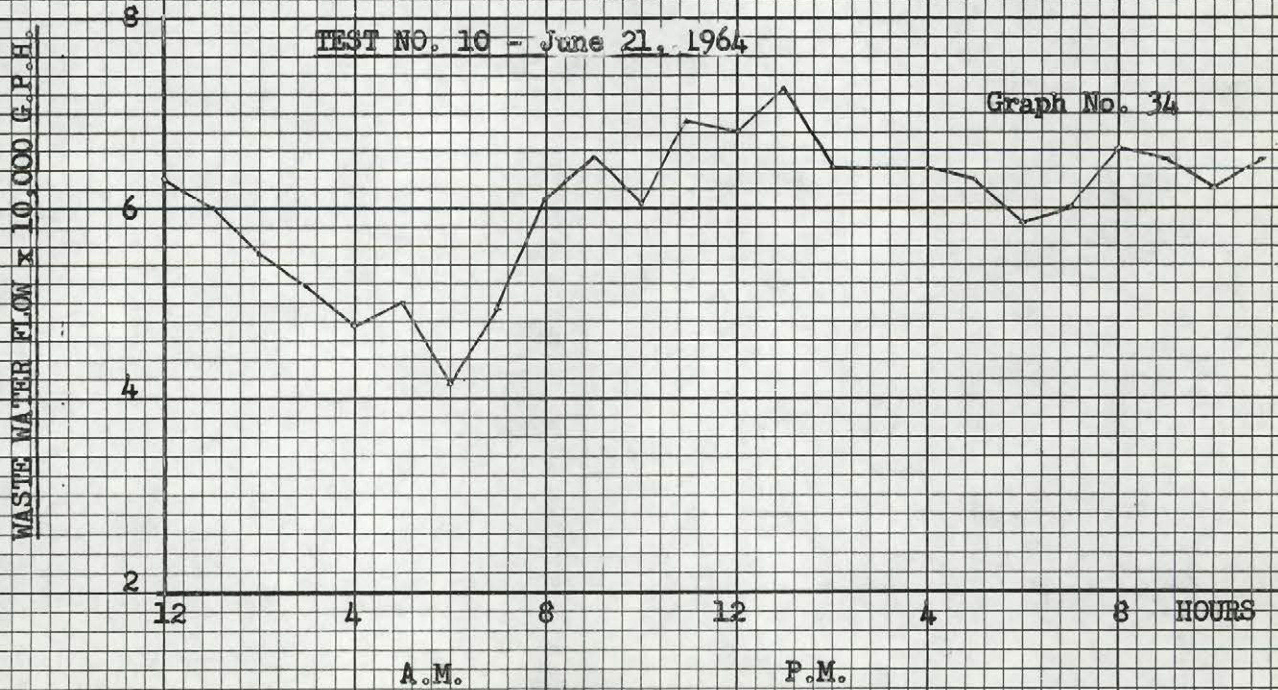
Graph No. 32

VARIATIONS OF WASTE WATER FLOW

TEST NO. 9 - June 8, 1964



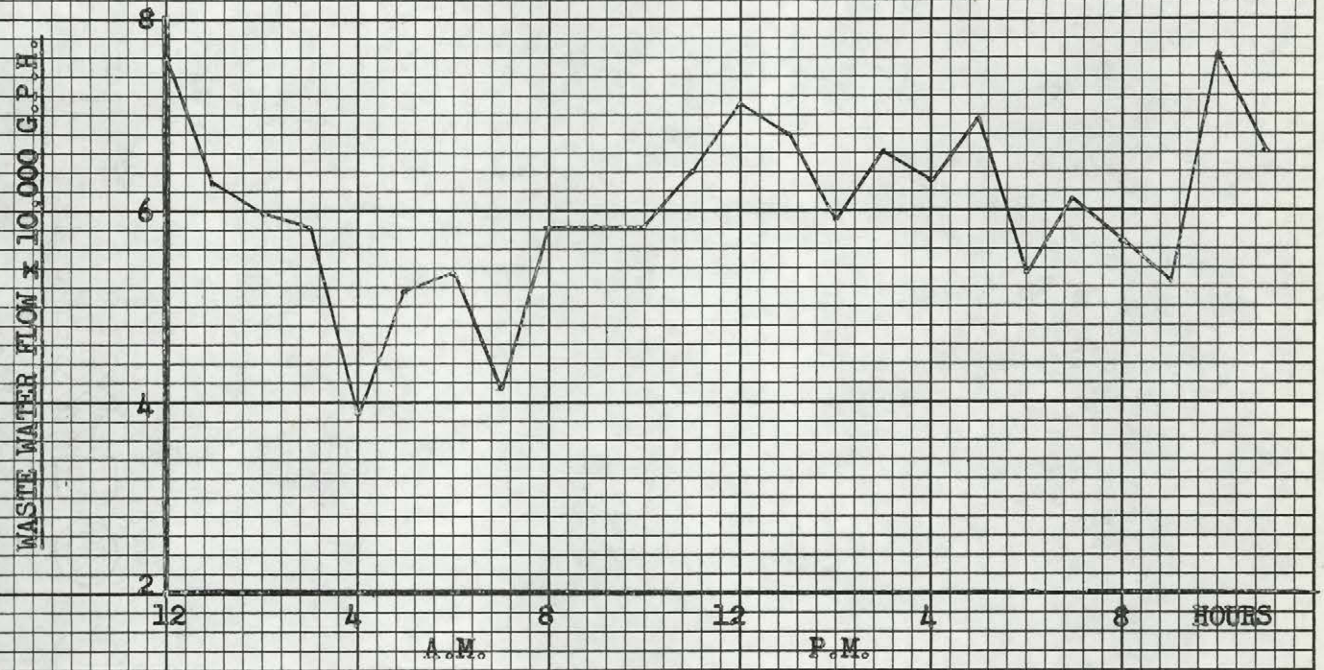
TEST NO. 10 - June 21, 1964



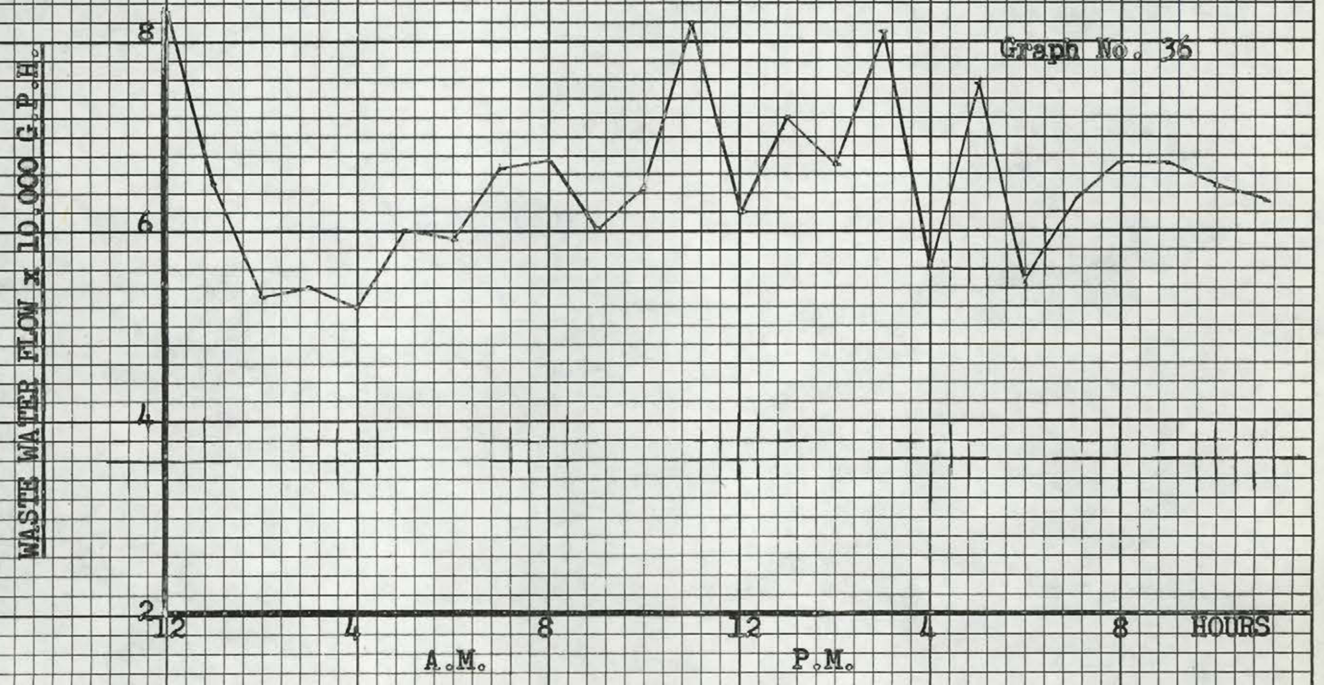
VARIATIONS OF WASTE WATER FLOW

TEST NO. 11 - June 28, 1964

Graph No. 35



Graph No. 36

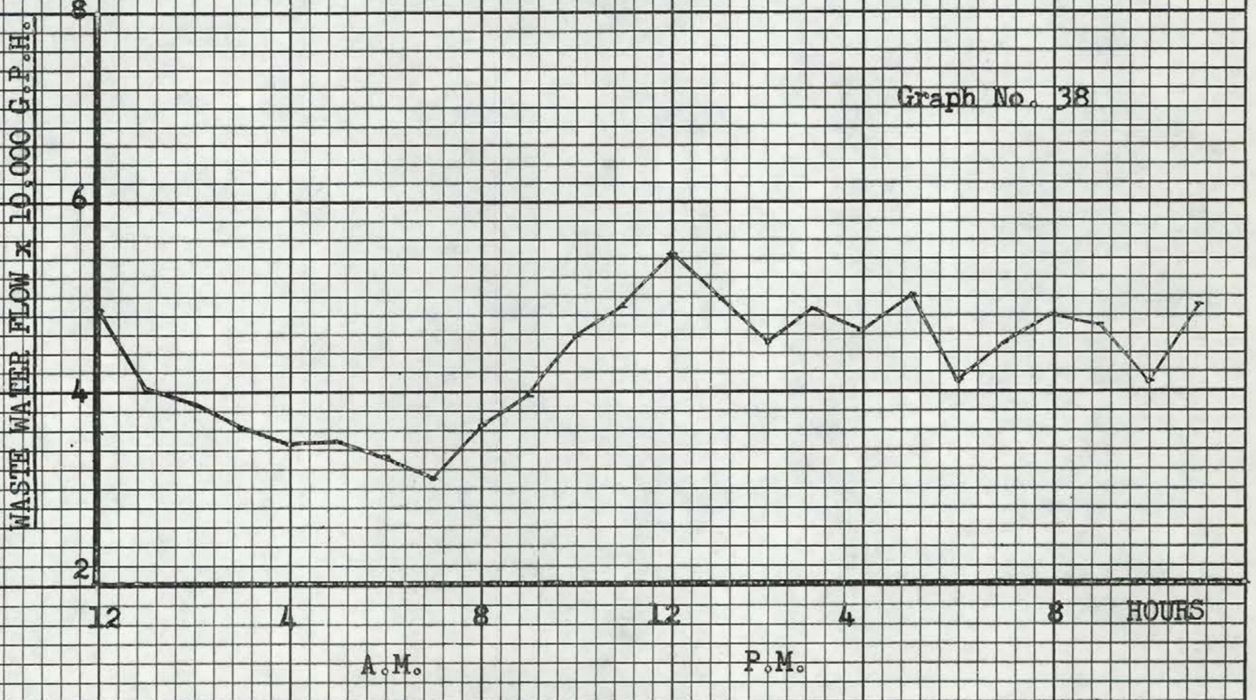


VARIATIONS OF WASTE WATER FLOW

Test No.

Graph No. 37

Graph No. 38



CHAPTER IX

DISCUSSION OF RESULTS

The results of three months of thickening primary sludge are presented in Table V. It was found that during the 24-hour pumping of primary sludge, the thickened sludge removed from the thickener averaged about 6.5 lbs. of solids per sq.ft. of thickener area per m.g. of waste water treated. The hydraulic loading rate of the thickener was 1400 g.p.d. per sq.ft.

During the tests of 12-hour pumping, thickened sludge removal averaged about 5.6 lbs. of solids per sq.ft. per m.g. of waste water treated and the liquid loading rate of the thickener was 700 gallons per sq.ft. per day.

The tests carried out when part of the primary sludge was recirculated gave average sludge removal of about 4.7 lbs. of solids per sq.ft. per m.g. waste water at hydraulic loading rate into thickener of 560 gallons per sq.ft. per day.

In all cases mentioned above the removal of organics average about 65% of the total solids removed.

Because of great variation in the concentration of the sludge pumped to the thickener, it was agreed that the analyses of influent to the thickener would not be carried out during the test runs 1 to 6.

A few settling analyses, however, were made on grab samples of the primary sludge at the overflow weir of the grit washer and the results showed that suspended solids content on the average was 15,000 p.p.m.

A settling test was carried out by allowing 1000 ml. of primary sludge sample to settle for 1 hour in a graduated 1 liter cylinder. The quantity of sludge which settled out at the end of 1 hour was read and recorded as the settleability of primary sludge in ml/l.

Settleability of primary sludge during test runs 1 to 6 averaged 150 ml/l. Other tests carried out in 1964 gave average settleability of primary sludge of 100 ml/l. and suspended solids content of 10,000 p.p.m. or 1% of solids. The latter tests were carried out by compositing samples taken hourly over a 24-hour period. More reliability was therefore placed on the results obtained in 1964.

The results of three months of tests of the waste water treatment plant are presented in Table VII and Appendix E. It was found that the thickener retained in the concentrated sludge over 99% of the solids pumped from primary clarifier.

Thickener effluent averaged about 10% of the plant influent and carried from 60 to 110 p.p.m. of suspended solids. Another factor affecting the effluent quality was the quantity of sludge held in the thickener. In test No. 2 it was found by physical examination that the effluent quality was adversely affected by the sludge blanket depth of 6 feet. A much clearer effluent was obtained during test No. 1 with sludge blanket depth of 3 feet. The effluent obtained during test No. 6, when part of the primary sludge was recirculated, was also much clearer than the effluent obtained during test No. 2.

It may be said, therefore, that in order to maintain good effluent quality in the thickener overflow, both the liquid loading rates as well as the sludge blanket depth must be reduced.

During tests Nos. 1 and 2, high sludge concentrations were obtained, namely, 14.83% of solids at 3 feet and 15.64% of solids at 6 feet sludge blanket depth. The rate of sludge pumping was 700 gallons per m.g. of waste water treated for test No. 1 and averaged 550 gallons per m.g. of waste water for test No. 2.

The tests Nos. 3 and 4 indicated that sludge concentration averaged 11.17% and 14.20% of solids at sludge blanket depths of 3 and 6 feet, respectively.

It was therefore concluded that the results obtained during tests Nos. 1 and 2 were greatly affected by the storms that occurred during the latter part of August 1963. Experiments made during the month of July 1964 also showed sludge concentration of 14% of solids at 3 feet sludge blanket depth.

Since the tests Nos. 1 and 2 were made during a rainy period, results of sludge concentration indicated the influence of silt accumulation. Average value of inorganic component of the sludge was 40.62%, representing the conditions which would occur during and after every rainy period.

Tests. Nos. 3 and 4 represented two different conditions, namely a rainy period flow and the dry weather flow. Due to absence of excessive amounts of silt or inorganics in general, the values of the organic and inorganic components of the sludge in test No. 4 were 70.46% and 29.54% respectively. Test No. 3 indicated that while the organic and inorganic components approximated the values obtained in tests Nos. 1 and 2, the higher loading of the clarifier had resulted in lower removal efficiency.

The sludge concentrations obtained in tests Nos. 5 and 6 gave values of 11.3% of solids at 3 feet sludge blanket depth and 13.43% of solids at 6 feet depth. These values were comparable with those obtained for tests Nos. 3 and 4. It should be noted that during these test runs, the loading of the thickener was kept at 60 g.p.m., while the rest of the sludge pumped was returned to the wet well of the plant and, consequently, to the clarifier. It is thought that the effect of recirculation of part of the primary sludge caused re-suspension of previously settled-out solids, and may be one of factors producing lower solids content of the thickened sludge.

In Table IX is shown the Sludge Volume Ratio (S.V.R.) which is defined as the volume of sludge blanket in the thickener divided by the daily volume of sludge removed from the thickener. This relationship, which has the dimension of days was used as a relative measure of the average solids detention period time in the sludge blanket. In order to use the true solids detention time, an inventory of the weight of dry solids in the thickener would have to be made frequently. The weight of dry solids so established divided by the rate of removal of solids from the thickener would constitute the actual solids detention time. Such a method of control would be so laborious as to be impractical. Therefore, as a measure of solids detention time, the sludge volume ratio was used as a practical means of controlling the thickening process. The maximum S.V.R. used was governed by the mechanical limitations of the equipment and to obtain a satisfactory thickener effluent quality.

Table VII shows that the clarifier removed from 47% to 67% of the suspended solids while the C.O.D. removals varied from 18% to 38%. Clarifier sludge concentration averaged 1.4% of solids. It was found that clarifier sludge concentration varied considerably reaching even 0.5% of solids when the sludge transfer pump was operating at 157 g.p.m. A low pumping rate and a longer detention period in the clarifier would most probably raise the concentration of primary sludge.

During the tests made operating the sludge transfer pump 12 hours a day only, it was found that the 12-hour pumping of clarifier sludge was fully acceptable and did not cause any difficulty in plant operation.

During 24-hour pumping schedule, it was found that the primary sludge was very diluted and contained high percentage of water.

The thickener operation was also investigated during the tests on plant treatment efficiencies. The sludge blanket depths maintained during the tests Nos. 7 to 14 ranged from 3 feet to 6 feet. Sludge concentration averaged 10% of solids at 3 feet sludge blanket depth, 11% solids at 4 feet, 12.5% solids at 4 feet and 14.0% solids at 6 feet sludge blanket depth.

The chlorination unit increased the efficiency of the plant as indicated by the C.O.D. removals of from 29% to 65%. It was, however, found that suspended solids in the chlorinated effluent sometimes exceeded the values obtained in clarifier effluent.

In the course of plant maintaining operations it was found that some settling took place also in the chlorination chamber during plant overload period. It was thought that surge-removal of the sediment from the chlorination chamber accounted for the relatively high amount of suspended matter in the plant effluent.

TABLE NO. IX - OBSERVATIONS ON SLUDGE CONDITIONING

Date	Sludge Blanket Depth	Sludge Underflow g.p.d.	S.V.R.	Solids Content %	Volatile Content %
4.5.64	2'-0"	1060	1.81	5.10	82.85
5.5.64	2'-0"	980	1.96	5.70	82.20
7.5.64	2'-3"	920	2.44	6.00	80.30
18.6.64	2'-9"	865	3.05	9.81	79.70
10.10.63	3'-0"	820	3.51	11.36	59.45
11.10.63	3'-0"	930	3.10	10.95	61.42
22.6.64	3'-0"	850	3.40	10.00	77.00
* 5.7.64	3'-0"	935	3.08	14.00	67.07
* 28.8.63	3'-0"	805	3.58	15.18	59.24
* 29.8.63	3'-0"	830	3.43	14.48	59.53
24.9.63	3'-0"	835	3.45	11.30	67.40
1.6.64	3'-9"	800	4.50	11.20	69.23
12.5.64	4'-0"	800	4.80	10.14	70.4
31.5.64	4'-0"	790	4.85	11.70	67.03
21.6.64	4'-0"	820	4.67	11.30	77.54
25.6.64	4'-0"	810	4.74	11.60	77.93
28.6.64	4'-0"	900	4.26	11.90	73.60
30.5.64	4'-6"	740	5.84	12.70	62.50
26.6.64	4'-9"	685	6.65	12.60	75.02
14.5.64	5'-0"	700	6.85	12.90	63.20
8.6.64	5'-0"	760	6.32	12.20	75.00
3.7.64	5'-0"	680	7.06	12.80	72.24
21.5.64	5'-6"	700	7.55	12.80	69.90
18.5.64	6'-0"	660	8.73	12.90	62.60
23.10.64	6'-0"	530	10.85	13.77	70.25
24.10.64	6'-0"	615	9.35	14.11	70.68
20.5.64	6'-0"	680	8.46	13.20	62.60
29.9.63	6'-0"	635	9.07	13.43	67.14

* Values disregarded because results of solids content were high and indicated periods of storms.

TABLE X - EFFICIENCIES FOR PRIMARY WASTE WATER TREATMENT PROCESSES

	Suspended Solids %	B.O.D. %
Imhoff & Fair (*20)	40 - 70	25 - 40
Schroeper (*33)	65	36
Fair & Geyer (*13)	40 - 70	25 - 40

Table X shows removal percentages of suspended matter and B.O.D. that may be expected in Primary Treatment Processes.

The overall efficiencies of the primary treatment plant at Beaconsfield gave average removals of 55% of suspended solids and 42% of C.O.D. These values compare favourably with expected values for primary treatment processes, obtained from the operational experience of similar plants.

Curve Fitting

Graphical methods have been used to present the experimental data in Table IX.

Three types of curves were considered and their equations are,

$$y = bx^n$$

$$y = a + bx \text{ and}$$

$$y = \frac{x}{a + bx}$$

The above equations can be transformed to equations of straight lines.

The straight lines of best fit for the three graphs plotted were obtained by employing the method of least squares.

From the equation of the line of best fit the value of y may be obtained for any given value of x on the assumption that the least squares line is the best representation of the relation between x and y.

In determining a particular curve which is that of best fit to the graph No. 39 - Solids Content vs. Sludge Blanket Depths - the root-mean-square of the corrections for the three equations were computed and compared.

For equation $y = bx^n$, the root mean square or the standard error of estimate was found to be 0.356.

For the equation $y = a + bx$, the standard error of estimate was 1.190, and,

For the equation $y = \frac{x}{a + bx}$, the standard error of estimate was 0.037.

Since equation $y = \frac{x}{a + bx}$, gave the smallest value for the standard error of estimate, it was decided to accept this equation as that of curve of best fit to the points plotted in graph No. 39.

The method of least squares is based upon the law of chance or random sampling and is designed to make the sum of the squares of the differences, or residuals between observed and calculated values a minimum.

Thus if R_n = the residual of the n^{th} pair of observations (x_n, y_n) which are to be fitted by the equation

$$y = a + bx, \text{ then the residual}$$

$$R_n = a + bx - y_n.$$

The term $(a + bx_n)$ is the value of y calculated to be paired with the value x_n, whereas y_n is the observed value of y. The parameters a and b are to be chosen in such a way that the sums of the squares of the residuals are to be a minimum.

$$R^2 = \sum \left[(a + bx) - y\right]^2 = \text{a minimum}$$

To meet this requirement, the first derivatives of $\sum R^2$ with respect to a and b are set equal to zero or

$$\frac{d\left[\sum R^2\right]}{da} = 2\sum\left[R\,\frac{dR}{da}\right] = 2\sum\,(a + bx - y) = 0$$

$$\frac{d\left[\sum R^2\right]}{db} = 2\sum\left[R\,\frac{dR}{db}\right] = 2\sum\,(a + bx - y) = 0$$

For n pairs of observed values the following simultaneous equations exist.

$$\text{I}\quad na + b\Sigma x - \Sigma y = 0$$

$$\text{II}\quad a\Sigma x + b\Sigma x^2 - \Sigma xy = 0$$

Dividing both equations by n we have the two normal equations.

$$\text{I}\quad a + \frac{b\Sigma x}{n} - \frac{\Sigma y}{n} = 0$$

$$\text{II}\quad \frac{a\Sigma x}{n} + \frac{b\Sigma x^2}{n} - \frac{\Sigma xy}{n} = 0$$

Thus we have 2 equations and 2 unknowns and the values of a and b are determined from

$$a = \frac{\Sigma x^2\Sigma y - \Sigma x\,\Sigma xy}{r\Sigma x^2 - \left[\Sigma x\right]^2}$$

$$b = \frac{r\Sigma xy - \Sigma x\Sigma y}{r\Sigma x^2 - \left[\Sigma x\right]^2}$$

To illustrate the application of the method more completely, the values of sludge blanket depths and corresponding solids content as tabulated in Table IX have been used. The equation chosen being $y = \dfrac{x}{a + bx}$

TABLE XI - $\frac{\text{SLUDGE BLANKET DEPTH}}{\text{SOLIDS CONTENT}}$ $\frac{x}{y}$ VS. SLUDGE BLANKET DEPTH 'x

Observed "x"	Observed $\frac{"x"}{y}$	x^2	$x.\frac{x}{y}$	Calculated $\frac{x}{y}$
2.0	0.392	4.00	0.784	0.298
2.0	0.351	4.00	0.702	0.298
2.25	0.375	5.08	0.844	0.306
2.75	0.281	7.58	0.773	0.322
3.00	0.264	9.00	0.792	0.330
3.00	0.274	9.00	0.822	0.330
3.00	0.300	9.00	0.900	0.330
3.00	0.266	9.00	0.798	0.330
3.75	0.335	14.10	1.256	0.354
4.00	0.395	16.00	1.580	0.362
4.00	0.342	16.00	1.368	0.362
4.00	0.354	16.00	1.416	0.362
4.00	0.345	16.00	1.380	0.362
4.00	0.336	16.00	1.344	0.362
4.50	0.355	20.30	1.598	0.378
4.75	0.377	22.60	1.791	0.386
5.00	0.388	25.00	1.940	0.394
5.00	0.410	25.00	2.050	0.394
5.00	0.391	25.00	1.955	0.394
5.50	0.430	30.30	2.365	0.410
6.00	0.465	36.00	2.790	0.426
6.00	0.436	36.00	2.616	0.426
6.00	0.425	36.00	2.550	0.426
6.00	0.455	36.00	2.730	0.426
6.00	0.446	36.00	2.676	0.426
Totals 104.50	9.188	478.96	39.820	9.194
Means 4.18	0.368	19.16	1.593	0.368

which can be written as

$$\frac{x}{y} = a + bx.$$

The variables are $\frac{x}{y}$ values which are plotted on the ordinate axis, and x values are plotted on the abscissa axis.

Substituting the values obtained from Table XI 'n the equations I and II,

$$\text{I} \quad a + 4.18b - .368 = 0$$

$$\text{II} \quad a + \frac{19.16}{4.18} b - \frac{1.593}{4.18} = 0$$

whence $a = 0.234$

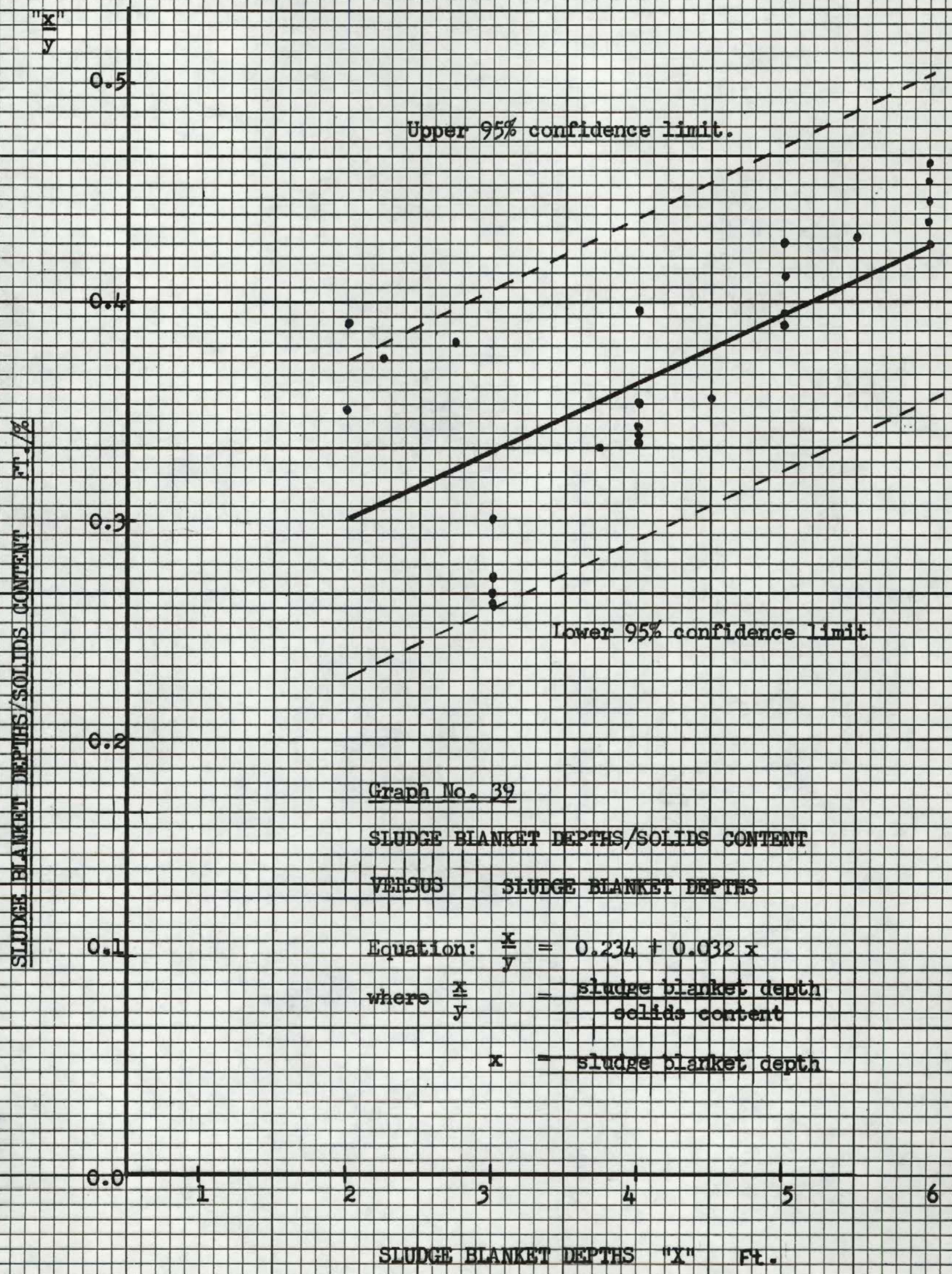
$b = 0.032$

The equation of the line of best fit is

$$\frac{x}{y} = 0.234 + 0.32x$$

Using this equation, values of $\frac{x}{y}$ were calculated for the observed values of x and entered in the preceding Table XI. A summation of these values of calculated $\frac{x}{y} = 9.194$, therefore, the average value of calculated $\frac{x}{y} = 0.368$ which equals the average value of observed $\frac{x}{y}$.

The straight line $\frac{x}{y} = 0.234 + 0.032x$ may now be drawn representing the line of best fit through the plotted observation points for the solids content and sludge blanket depths. The result of this procedure is shown on the next page.



Graph No. 39

SLUDGE BLANKET DEPTHS/SOLIDS CONTENT

VERSUS SLUDGE BLANKET DEPTHS

Equation: $\frac{x}{y} = 0.234 + 0.032\,x$

where $\frac{x}{y}$ = $\frac{\text{sludge blanket depth}}{\text{solids content}}$

x = sludge blanket depth

TABLE XII - ADJUSTMENT OF OBSERVATIONS TO LEAST SQUARES LINE - $\frac{\text{SLUDGE BLANKET DEPTH}}{\text{SOLIDS CONTENT}}$ VS SLUDGE BLANKET DEPTH

	Observed $\frac{x}{y}$	Calculated $\frac{x}{y}$	Correction to Observed x/y, d	d^2
	0.392	0.298	-0.094	0.00884
	0.351	0.298	-0.053	0.00281
	0.375	0.306	-0.069	0.00476
	0.281	0.322	+0.041	0.00168
	0.264	0.330	+0.066	0.00436
	0.274	0.330	+0.056	0.00314
	0.300	0.330	+0.330	0.00090
	0.266	0.330	+0.019	0.00036
	0.335	0.354	-0.033	0.00109
	0.395	0.362	+0.020	0.00040
	0.342	0.362	+0.008	0.00006
	0.354	0.362	+0.017	0.00029
	0.336	0.362	+0.026	0.00068
	0.355	0.378	+0.023	0.00053
	0.377	0.386	+0.009	0.00008
	0.388	0.394	+0.006	0.00004
	0.410	0.394	-0.016	0.00026
	0.391	0.394	+0.003	0.00001
	0.430	0.410	-0.020	0.00040
	0.465	0.426	-0.039	0.00152
	0.436	0.426	-0.010	0.00010
	0.425	0.426	+0.001	0.00000
	0.455	0.426	-0.029	0.00084
	0.446	0.426	-0.020	0.00040
Totals:	9.188	9.194	+0.325 to -0.383	0.03355
Means:	0.368	0.368		0.00134

The standard error of estimate for the given data shown in Table-XII is

$$\sigma_{\frac{x}{y}} = \pm\sqrt{0.00134}$$
$$= \pm\ 0.037$$

From experimental data shown in Appendix E it was found that coefficient of variation of an average sample taken from each day's samples was within ± 5%. Since analyses of samples were carried out under fixed laboratory conditions, it was agreed that most of the points plotted in Graph No. 39 should lie between lines parallel to the least squares line of best fit, one at 2 $\sigma_{\frac{x}{y}}$ units above and one at 2 $\sigma_{\frac{x}{y}}$ units below, measured vertically.

95.00% of all cases would lie between ± 1.96 $\sigma_{\frac{x}{y}}$. ± 0.073 units each side of the least squares line, measured vertically.

From the equation of the least squares line of best fit estimate x/y for x = 6

$$\frac{x}{y} = 0.426$$

$$\frac{x}{y} + \sigma_{\frac{x}{y}} = 0.426 + 0.073$$
$$= 0.499$$

$$\frac{x}{y} - \sigma_{\frac{x}{y}} = 0.426 - 0.073$$
$$= 0.353$$

So the probability that the correct $\frac{x}{y}$ lies between 0.499 and 0.353 is 95%.

Equivalent calculations have been carried out for the solids content and volatile content relationship as well as for the sludge volume ratio vs. solids content.

The equation $y = a + bx$ was used in finding the curve of best fit for the Graph No. 40 - Solids Content vs. Volatile Content. Calculations made are shown in Tables XIII and XIV.

The equation $y = \frac{x}{a + bx}$ was used as the equation of the curve of best fit for the Graph No. 41 - Solids Content and Sludge Volume Ratio relationship. The calculations made are shown in Tables XV and XVI.

Two equations $y = \frac{x}{a + bx}$ and $y = a + bx$ were used in obtaining the curves of best fit for the Graphs Nos. 42 and 43 respectively. Graph No. 42 shows Suspended Solids Removal vs. Detention Period. Graph No. 43 shows C.O.D. Removals vs. Detention Period. These observations of C.O.D., suspended solids removals and detention period concerned the clarifier operation.

TABLE XIII - VOLATILE CONTENT VS. SOLIDS CONTENT

	Observed "x" Solids Content	Observed "y" Volatile Content	x^2	xy	Calculated "y"
	5.10	82.85	26.00	423.00	81.97
	5.70	82.20	32.50	468.00	80.91
	6.00	80.30	36.00	481.80	80.38
	9.80	79.70	96.00	780.00	73.66
	11.36	59.45	129.00	675.00	70.91
	10.95	61.42	120.00	673.00	71.63
	10.00	77.00	100.00	770.00	73.31
	11.30	67.40	128.00	761.00	71.01
	11.20	69.23	125.50	775.40	71.19
	10.14	70.40	102.50	713.90	73.06
	11.70	67.03	137.00	784.30	70.31
	11.30	77.54	128.00	876.00	71.01
	11.60	77.93	134.50	904.00	70.48
	11.90	73.60	142.00	875.00	69.95
	12.70	62.50	161.50	793.80	68.54
	12.60	75.02	158.50	946.00	68.72
	12.90	63.20	166.50	815.00	68.19
	12.20	75.00	148.50	915.00	69.42
	12.80	72.24	164.00	925.00	68.36
	12.80	69.90	164.00	895.00	68.36
	12.90	62.60	166.50	807.50	68.19
	13.77	70.25	190.00	970.00	66.65
	14.11	70.68	200.00	997.50	66.05
	13.20	62.60	174.00	826.50	67.66
	13.43	67.14	180.50	904.00	67.25
Totals:	281.46	1777.18	3311.00	19,755.70	1777.17
Means:	11.258	71.087	132.44	790.228	71.087

Substituting the values obtained from Table XIII, the two equations I and II are

$$a + 11.258b - 71.087 = 0$$

$$11.258a + 132.44b - 790.228 = 0$$

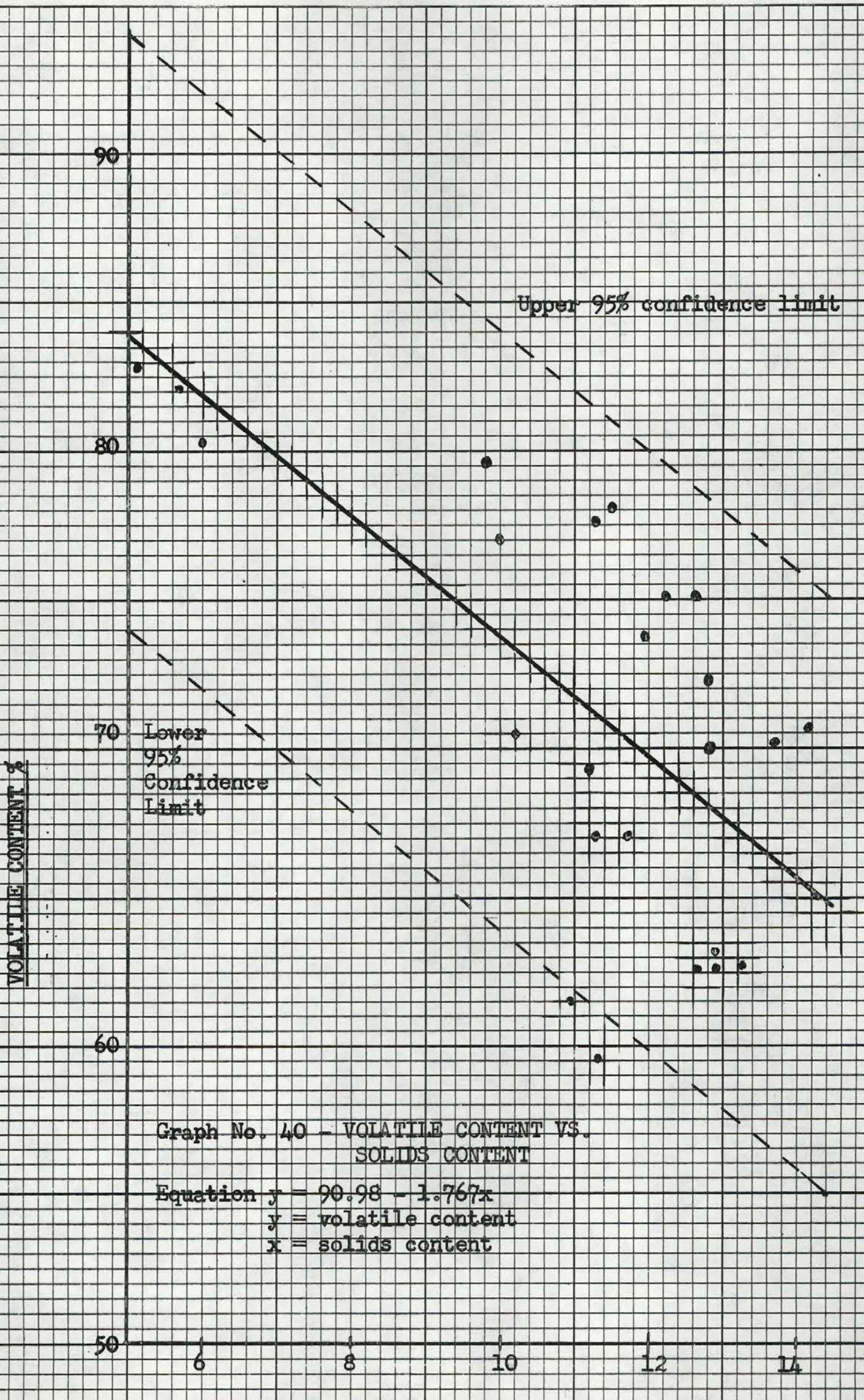
Solving for a and b we have

$$a = 90.98$$

$$b = 1.767$$

Therefore, the equation of the line of best fit is

$$y = 90.98 - 1.767x.$$



Graph No. 40 - VOLATILE CONTENT VS. SOLIDS CONTENT

Equation y = 90.98 - 1.767x
y = volatile content
x = solids content

TABLE XIV - ADJUSTMENT OF OBSERVATIONS TO LEAST SQUARES LINE - SOLIDS CONTENT VS. VOLATILE CONTENT

	Observed y	Calculated y	Correction to Observed y d	d^2
	82.85	81.97	- 0.88	0.7744
	82.20	80.91	- 1.29	1.6641
	80.30	80.38	+ 0.08	0.0064
	79.70	73.66	- 6.04	36.4816
	59.45	70.91	+ 11.46	131.3316
	61.42	71.63	+ 10.21	104.2441
	77.00	73.31	- 3.69	13.6161
	67.40	71.01	+ 3.61	13.0321
	29.23	71.19	+ 1.96	3.8416
	70.40	73.06	+ 0.70	0.4900
	67.03	70.31	+ 3.28	10.7584
	77.54	71.02	- 6.52	42.5104
	77.93	70.48	- 7.45	55.5025
	73.60	69.95	- 3.65	13.3225
	62.50	68.54	+ 6.04	36.4816
	75.02	68.72	- 6.30	39.6900
	63.20	68.19	+ 4.99	24.9001
	75.00	69.42	- 5.58	31.1364
	72.24	68.36	- 3.88	15.0544
	69.90	68.36	- 1.54	2.3716
	62.60	68.19	+ 5.59	31.2481
	70.25	66.65	- 3.60	12.9600
	70.68	66.05	- 4.63	21.4369
	62.60	67.66	+ 5.06	25.6036
	67.14	67.25	+ 0.11	0.0121
Totals:	1777.18	1777.17	- 55.05 to	668.4706
Means:	71.087	71.087	+ 53.09	26.7388

95% confidence limit $y \pm 1.96 \times \sqrt{26.7388}$

$= y \pm 10.135$

TABLE XV - $\frac{\text{SLUDGE VOLUME RATIO}}{\text{SOLIDS CONTENT}}$ VS. SLUDGE VOLUME RATIO

	Observed "x" Sludge Volume Ratio	Observed $\frac{"x"}{y}$ S.V.R./Solids Content	x^2	$x \cdot \frac{x}{y}$	Calculated $\frac{"x"}{y}$
	1.81	0.355	3.276	0.643	0.2711
	1.96	0.344	3.842	0.6742	0.2792
	2.44	0.407	5.954	0.9931	0.3052
	3.05	0.311	9.303	0.949	0.3381
	3.51	0.309	12.320	1.085	0.3629
	3.10	0.283	9.610	0.877	0.3408
	3.40	0.340	11.560	1.156	0.3570
	3.45	0.305	11.903	1.052	0.3597
	4.50	0.402	20.250	1.809	0.4164
	4.80	0.473	23.040	2.270	0.4326
	4.85	0.415	23.532	2.013	0.4353
	4.67	0.413	21.809	1.929	0.4256
	4.74	0.409	22.468	1.939	0.4294
	4.26	0.358	18.148	1.525	0.4034
	5.84	0.460	34.106	2.686	0.4888
	6.65	0.528	44.223	3.511	0.5325
	6.85	0.531	46.923	3.637	0.5433
	6.32	0.518	39.942	3.274	0.5147
	7.06	0.552	49.844	3.897	0.5546
	7.55	0.590	57.003	4.455	0.5811
	8.73	0.677	76.213	5.910	0.6448
	10.85	0.788	117.723	8.550	0.7593
	9.35	0.663	87.423	6.199	0.6783
	8.46	0.641	71.572	5.423	0.6302
	9.07	0.675	82.265	6.122	0.6632
Totals:	137.27	11.7470	904.237	72.578	11.7475
Means:	5.4908	0.4699	36.1695	2.9031	0.4699

Substituting the values obtained from Table XV in the equations I and II

$$\text{I} \quad a + 5.4908b - 0.4699 = 0$$

$$\text{II} \quad 5.4908a + 36.1695b - 2.9031 = 0$$

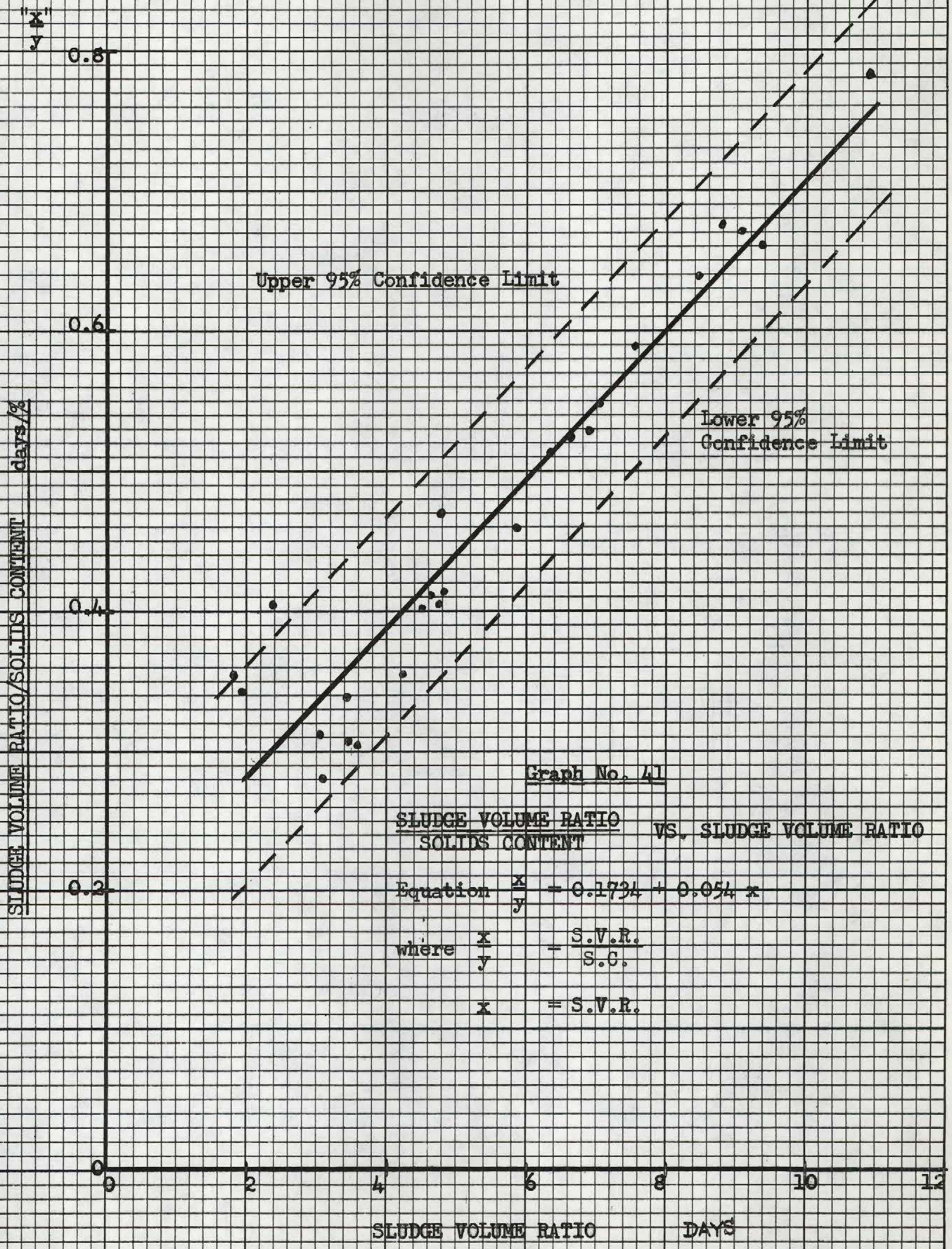
The values of a and b are determined as

$$a = 0.1734$$

$$b = 0.0540$$

Therefore, the equation of the line of best fit is

$$\frac{x}{y} = 0.1734 + 0.054x.$$



Graph No. 41

$\frac{\text{SLUDGE VOLUME RATIO}}{\text{SOLIDS CONTENT}}$ VS. SLUDGE VOLUME RATIO

Equation $\frac{x}{y} = 0.1734 + 0.054\ x$

where $\frac{x}{y} = \frac{S.V.R.}{S.C.}$

$x = S.V.R.$

TABLE XVI - ADJUSTMENT OF OBSERVATIONS TO LEAST SQUARES LINE - S.V.R./ SOLIDS CONTENT VS. SLUDGE VOLUME RATIO

	Observed $\frac{x}{y}$	Calculated $\frac{x}{y}$	Correction to Observed $\frac{x}{y}$ d	d^2
	0.355	0.2711	- 0.0839	0.007039
	0.344	0.2792	- 0.0648	0.004199
	0.407	0.3052	- 0.1018	0.010363
	0.311	0.3381	+ 0.0271	0.000734
	0.309	0.3629	+ 0.0539	0.002905
	0.283	0.3408	+ 0.0578	0.003341
	0.340	0.3570	+ 0.0170	0.000289
	0.305	0.3597	+ 0.0547	0.002992
	0.402	0.4164	+ 0.0144	0.000207
	0.473	0.4326	- 0.0404	0.001632
	0.415	0.4353	+ 0.0203	0.000412
	0.413	0.4256	+ 0.0126	0.000159
	0.409	0.4294	+ 0.0204	0.000416
	0.358	0.4034	+ 0.0454	0.002061
	0.460	0.4888	+ 0.0288	0.000829
	0.528	0.5325	+ 0.0045	0.000020
	0.531	0.5433	+ 0.0123	0.000151
	0.518	0.5147	- 0.0033	0.000011
	0.552	0.5546	+ 0.0026	0.000007
	0.590	0.5811	- 0.0089	0.000079
	0.677	0.6448	- 0.0322	0.001037
	0.788	0.7593	- 0.0287	0.000824
	0.663	0.6783	+ 0.0153	0.000234
	0.641	0.6302	- 0.0108	0.000117
	0.675	0.6632	- 0.0118	0.000139
Totals:	11.7470	11.7475	- 0.3866 to	0.040197
Means:	0.4699	0.4699	+ 0.3871	0.001608

95% Confidence limit gives $\frac{x}{y} \pm 1.96 \times \sqrt{.001608}$

$$= \frac{x}{y} \pm 0.0786$$

TABLE XVII - $\frac{\text{DETENTION PERIOD "x"}}{\text{SUSPENDED SOLIDS REMOVAL "y"}}$ VS. DETENTION PERIOD "x"

	Observed "x"	Observed "$\frac{x}{y}$" $\times 10^{-2}$	x^2	$x \cdot \frac{x}{y}$ $\times 10^{-2}$	Calculated "$\frac{x}{y}$" $\times 10^{-2}$
	1.94	2.8955	3.7636	5.6173	3.5000
	2.11	3.3492	4.4521	7.0668	3.7337
	2.28	4.8511	5.1984	11.0605	3.9666
	2.59	4.7091	6.7091	12.1966	4.3913
	2.44	4.0667	5.9536	9.9227	4.1858
	2.30	3.4848	5.2900	8.0150	3.9940
	1.82	3.8723	3.3124	7.0476	3.3364
	2.41	4.0167	5.8081	9.6802	4.1447
Totals:	17.89	31.2454	40.4863	70.6067	31.2525
Means:	2.2363	3.91	5.0608	8.83	3.91

Substituting these values in the equations I and II

I $a + 2.2363b - 0.0391 = 0$

II $2.2363a + 5.0608b - 0.0883 = 0$

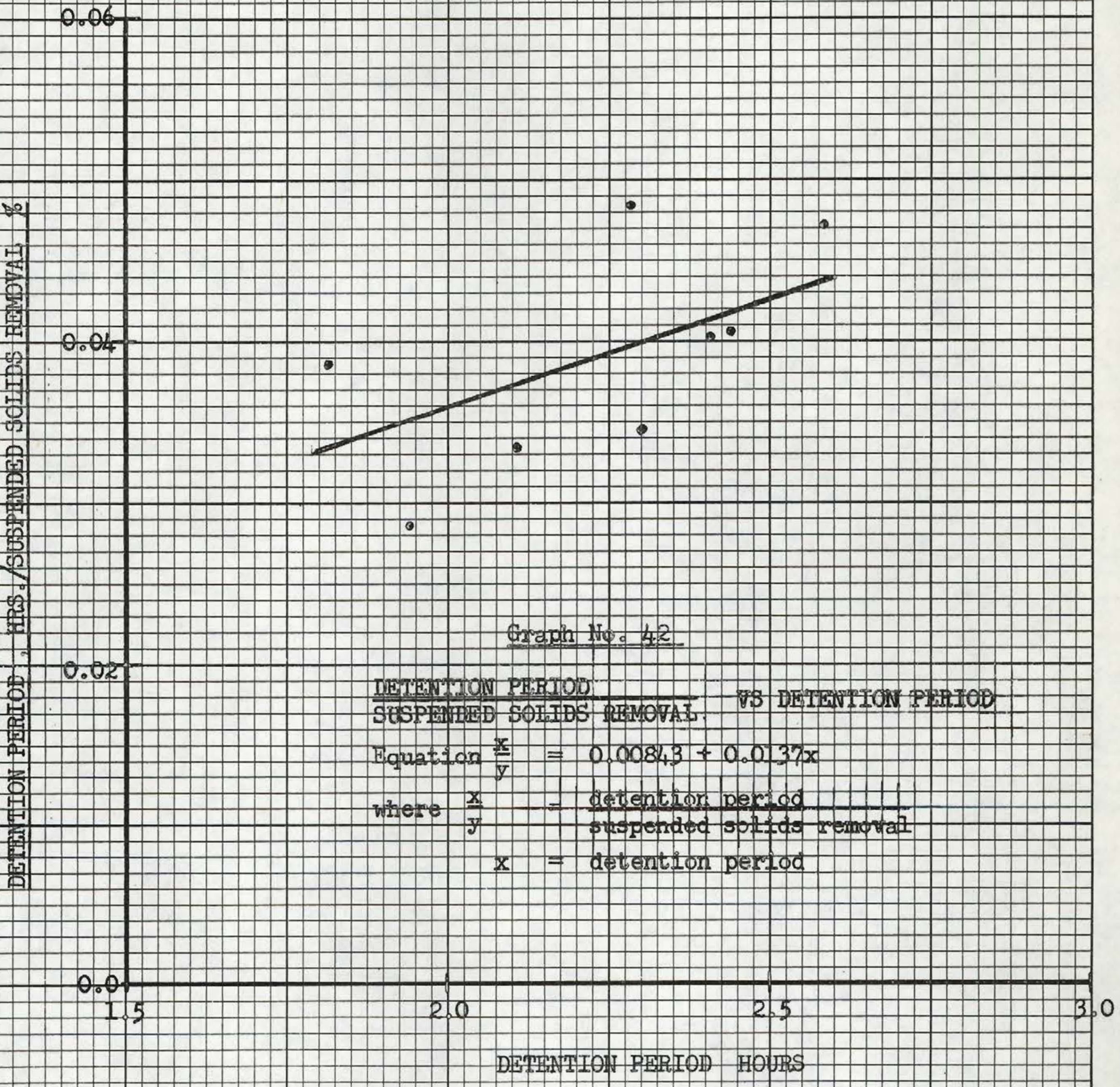
whence

$a = 0.00843$

$b = 0.0137$

and the linear equation of the line of best fit is

$$\frac{x}{y} = 0.00843 + 0.0137x$$



Graph No. 42

$\frac{\text{DETENTION PERIOD}}{\text{SUSPENDED SOLIDS REMOVAL}}$ VS DETENTION PERIOD

Equation $\frac{x}{y} = 0.00843 + 0.0137x$

where $\frac{x}{y} = \frac{\text{detention period}}{\text{suspended solids removal}}$

x = detention period

TABLE XVIII - DETENTION PERIOD "x" VS. C.O.D. REMOVAL "y"

	Observed "x"	Observed "y"	x^2	xy	Calculated "y"
	1.94	18	3.7636	34.92	21.3758
	2.11	21	4.4521	44.31	24.4453
	2.28	36	5.1984	82.08	27.5424
	2.59	20	6.7081	51.80	33.1648
	2.44	38	5.9536	92.72	30.4443
	2.30	29	5.2900	66.70	27.9051
	1.82	18	3.3124	32.76	19.19934
	2.41	34	5.8081	81.94	29.9002
Totals:	17.89	214	40.4863	487.23	213.977
Means:	2.2363	26.7500	5.0608	60.9038	26.747

Substituting these values in the equations I and II

$$\text{I} \quad a + 2.2363b - 26.75 = 0$$

$$\text{II} \quad 2.2363a + 5.0608b - 60.9038 = 0$$

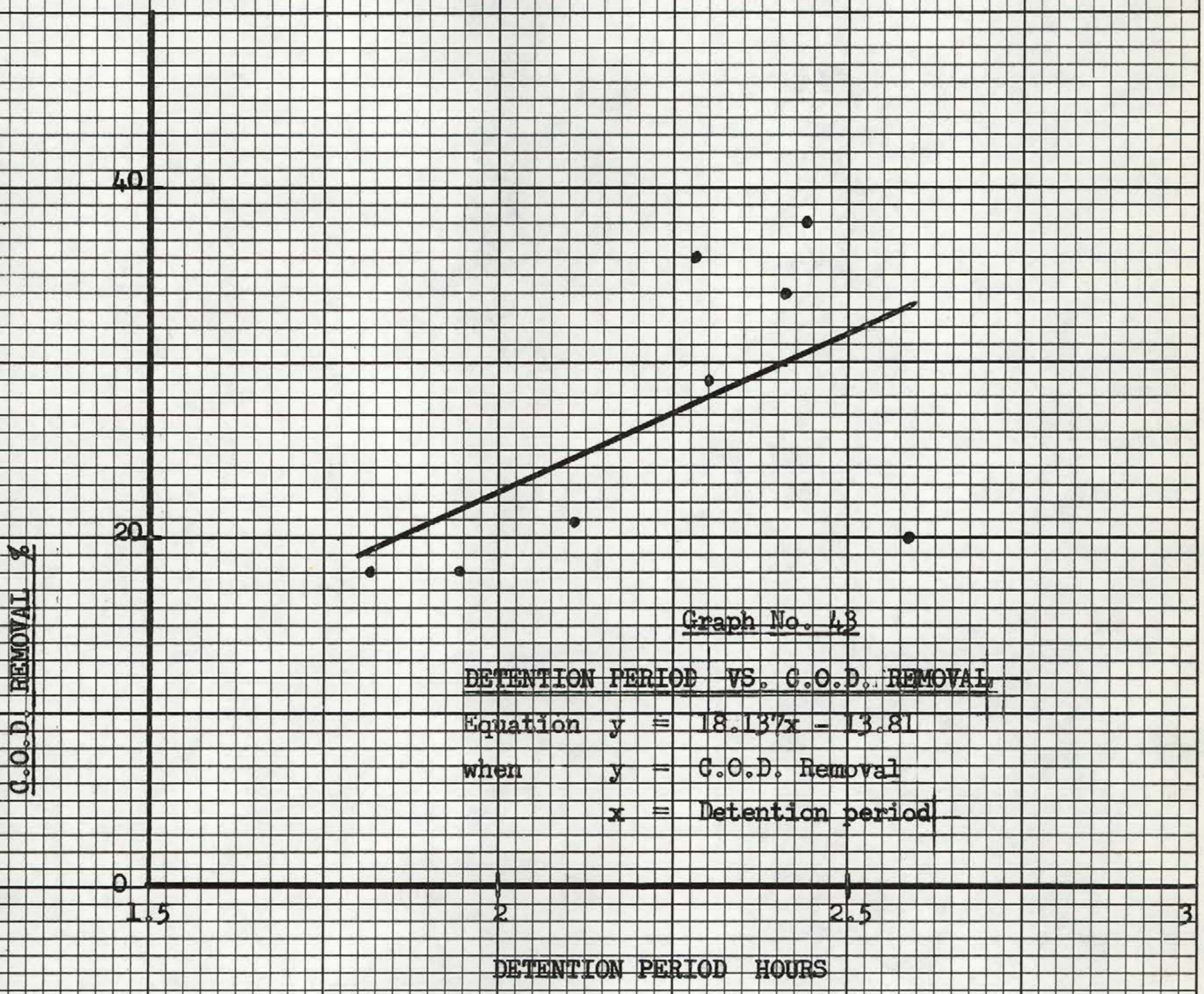
whence

$$a = -13.81$$

$$b = 18.137$$

The linear equation of the time of best fit is

$$y = -13.81 + 18.137x$$



Graph No. 43

DETENTION PERIOD VS. C.O.D. REMOVAL

Equation y = 18.137x - 13.81

when y = C.O.D. Removal

x = Detention period

An examination of the graph No. 39 indicated that a linear relationship existed between the sludge blanket depth in the thickener and solids content of the thickened sludge. The slope of the graph showed that an increase in sludge blanket depth/solids content would indicate also an increase in sludge blanket depth.

An examination of the graph No. 40 indicated a linear relationship between volatile content and solids content of the thickened sludge. The slope of the graph showed that at low solids content the volatile content of the solids tend to be high as contrasted with low volatile content at high solids content. For any value of solids content, the volatile content has a value of more than 60% of it.

Graph No. 41 showed the relationship between the $\frac{\text{Sludge Volume Ratio}}{\text{Solids Content}}$ and Sludge Volume Ratio. The graph is linear, indicating a rise in solids content with an increase in the sludge volume ratio.

If a graph of Solids Content vs. Sludge Volume Ratio were to be constructed, it would be seen that if the sludge volume ratio were decreased below some critical value, a marked reduction in solids content would result. Experiments were carried out to determine the rate of sludge consolidation during the first few hours of the thickening operation with a view toward approximating the critical S.V.R.

The results of two experiments carried out were questionable and were not recorded but they demonstrated that in each case there was a rapid initial consolidation period of about 2 days, followed by an abrupt decrease in the rate of consolidation. This is believed to have occurred when water no longer escaped freely from the sludge mass under the given conditions of the tank and the sludge thereafter was in the state of extended period of compression under its own weight.

In order to maintain high concentrations of sludge, the thickening process must be controlled so that the solids to be thickened remained in the thickener for a long enough period to have obtained this condition.

Graph No. 42 shows Detention Periods of the Clarifier versus $\frac{\text{Detention Period}}{\text{Suspended Solids Removal}}$. A linear relationship was obtained giving an average detention period of 2.2 hours and average suspended solids removal of 58%.

Graph No. 43 shows the relationship between C.O.D. Removal and Detention Period. An average C.O.D. removal of 27% for the clarifier was obtained.

It can be seen from the graphs Nos. 42 and 43 that observations of C.O.D. and suspended solids removals were obtained for clarifier's detention periods ranging from 1.82 to 2.59 hours. These observations, though few, were sufficient for obtaining average values of detention period, C.O.D. and suspended solids removals.

It is believed that if a series of observations were made on the lines of the results computed for the graphs Nos. 42 and 43, it would be possible to obtain similar relationship as that obtained by Fair and Geyer(*13) and shown in Graph 1 on Page 75.

CHAPTER X

CONCLUSIONS

The results of investigations regarding the Waste Water Treatment Plant of the Town of Beaconsfield, Quebec, and the method used for sludge conditioning can be summarized as follows:

1. A linear relationship exists between solids content of the thickened sludge and the sludge blanket depths in the thickener.

   Average values of solids content obtained for different depths of sludge blanket in the thickener were 6.7%, 10.0%, 11.0%, 12.5% and 14.0% of solids for 2, 3, 4, 5, and 6 feet sludge blanket depths, respectively.

2. Thickener operations at a high thickener liquid loading of 1400 g.p.d./sq.ft. and a sludge blanket depth of 6 feet resulted in poor effluent quality.

   Clearer effluent was obtained when thickener loading was reduced to 560 g.p.d./sq.ft. and sludge blanket depth maintained at 6 feet.

3. A linear relationship exists between solids content and volatile content of the thickened sludge. It was found that the solids content varied inversely as the percentage of the volatile content in the thickened sludge.

4. From a practical point of view, the results obtained during 12 hour-pumping of clarifier sludge indicated that a 12-hour primary sludge transfer period is fully acceptable and did not cause any diffi-

culty in plant operation. This, in turn, will mean reduction in plant operation cost.

5. The thickening process was controlled by using the concept of sludge volume ratio, defined as the volume of sludge blanket held in the thickener divided by the volume of sludge pumped per day from the thickener. In conjunction with this control, the depth of sludge blanket was kept low, thus aiding in the attainment of higher sludge concentration. During cold weather the S.V.R. was varied from 8 to 10 days. During the warmer weather when biological activity increased, resulting in gas formation, the range of S.V.R. was reduced to between 3 and 5 days.

6. The investigations concerning the performance and efficiency of the clarifier gave the following average data:

i) Detention period, 2.2 hours varying from 1.82 to 2.59 hours.

ii) Surface loading, 338 g.p.d./sq.ft. varying from 286 to 410 g.p.d./sq.ft.

iii) Suspend solids removals, 58% varying from 47% to 67% and

iv) C.O.D. removals, 27%, varying from 18% to 38%.

7. The average performance data for the treatment plant were found by using values obtained from analyses of influent and chlorinated effluent samples, and are as follows:

i) Suspended solids removal, 55% varying from 41% to 66%.

ii) C.O.D. removals, 42% varying from 29% to 65%.

These values compare favourably with expected values for primary treatment processes, obtained from the operational experience of similar plants.

8. It is concluded that the clarifier, thickener and chlorination units are adequate for treating waste water from the Town and these units are not overloaded except during periods of storms.

9. The average value of dissolved solids in the clarifier effluent is 521 p.p.m. This indicates, a secondary treatment process may eventually be needed for subsequent treatment of the effluent.

Future Investigation

The results of the present experiments and the experimental conditions themselves are suggesting the nature of future investigations.

The writer intends to offer a few suggestions regarding experimental conditions that may be anticipated in advance of future investigations, namely,

1. It would seem unnecessary to investigate sludge concentration below a sludge blanket depth of 3 feet in the thickener. Results of tests on sludge concentration below the 3 feet sludge blanket depths were highly inconsistent.

   It is believed that the concentrated sludge reaches the state of extended period of compression only after 3 feet sludge blanket depth.

2. Attempts should be made to obtain thickener performance data at a loading of 547 g.p.d./sq.ft. and lower. This would require recirculation of sludge, which may be done in the following manner.

   a) Recirculation from the grit washer to the clarifier. This could be done by connecting a 3" diameter pipe directly from the drain connection flange of the grit washer to the clarifier. A valve connected to this pipe would provide a control for varying the thickener loading rates.

b) Recirculation from the grit washer to the wet well. This could be done by opening the weir plates over the organic overflow weir in the grit washer and allowing part of the clarifier sludge to overflow back to the wet well.

It is believed that the first method would produce better results since the sludge would only be recirculated through the sludge transfer pump. The second method would pass the sludge through the low lift pumps and thus effect destruction of large sludge flocs.

These arrangements, if carried out, would enable the Town to run the sludge transfer pump at high pumping rate, thus preventing the repeated clogging of the 150 ft. long suction line from the clarifier. Furthermore, it would be possible to regulate the flow to the thickener via the valve on the new line connection from the grit washer to the clarifier, or via the weir openings on the organic overflow weir in the grit washer.

3. It had been shown that a certain solids content could be maintained in the thickened sludge by proper maintaining of sludge blanket depth and sludge volume ratio. Trial runs would be required to show whether the A.S.T. can handle the solids content produced.

4. Additional problems would arise when the Town decides to install a secondary treatment process. The combination of primary and secondary sludges would change the moisture content of the thickened sludge and thus the general consistency would be changed as well.

APPENDIX A

Data obtained in preliminary study and investigation.

Preliminary investigations and study of contemplated experiments had served to establish the following:

Waste Water Flow. Records of waste water flow to the treatment plant at Beaconsfield were studied for the years 1960 to 1963. The waste water flows were compared with corresponding water consumption figures. H.W. Lea(*26) suggested in his report that an estimate of the ultimate peak sanitary waste flow from the Town can be made by taking the anticipated ultimate peak water consumption and by adding to it an allowance for unavoidable infiltration of ground water.

The infiltration of ground water may take place through pipe joints, breaks in pipes, manholes and improperly plugged house connection branches provided for future development of the area. An allowance for infiltration of 100 g.p.d. per capita was assumed during the design of the treatment plant.

A study of the information made available indicated that no attempt has been made to investigate the effect of precipitation and infiltration on the waste water flow to the plant.

The observation of monthly flows of waste water, water consumption and rainfall have been tabulated in Table XIX and plotted in graph No. 44. An inspection of this graph will show the following:

TABLE XIX - MONTHLY FLOWS OF WASTE WATER, WATER CONSUMPTION & RAINFALL

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
1960 Sewage m.g.	46.50	56.00	65.72	-	-	42.10	42.48	34.91	31.92	37.11	57.96	41.17
Water m.g.	22.97	17.60	17.67	18.10	21.51	25.88	29.92	33.24	20.44	20.66	19.29	20.42
Rainfall inches	0.40	1.66	0.75	3.41	1.88	2.58	3.48	1.19	2.27	4.34	3.33	0.33
1961 Sewage m.g.	38.52	37.59	67.27	29.72	64.24	51.58	41.32	44.16	49.62	44.81	37.82	40.24
Water m.g.	23.32	25.37	22.62	18.95	21.53	22.75	23.15	26.14	25.87	28.70	22.70	21.42
Rainfall inches	0.07	2.13	0.79	2.94	2.46	5.17	3.07	5.41	0.83	2.55	1.39	1.76
1962 Sewage m.g.	43.48	41.75	61.17	78.21	72.26	48.35	42.74	45.09	35.65	31.23	46.45	40.35
Water m.g.	21.11	20.87	27.96	21.30	26.71	39.40	29.22	23.11	22.64	21.92	27.12	31.68
Rainfall inches	1.55	0.26	0.10	3.59	1.54	3.06	5.03	2.36	2.72	3.80	0.93	0.27
1963 Sewage m.g.	32.40	24.20	37.19	49.67	51.00	35.74	28.87	37.53	38.37	33.44	41.91	39.76
Water m.g.	30.58	22.56	22.86	23.51	26.83	35.24	28.35*	28.35*	28.35*	28.35*	28.35*	28.35*
Rainfall inches	0.05	0.02	1.14	3.15	2.25	3.61	2.47	5.88	4.64	0.55	5.52	0.40

* Average figures estimated.

Graph No. 44 - MONTHLY FLOWS OF WASTE WATER, WATER CONSUMPTION & RAINFALL

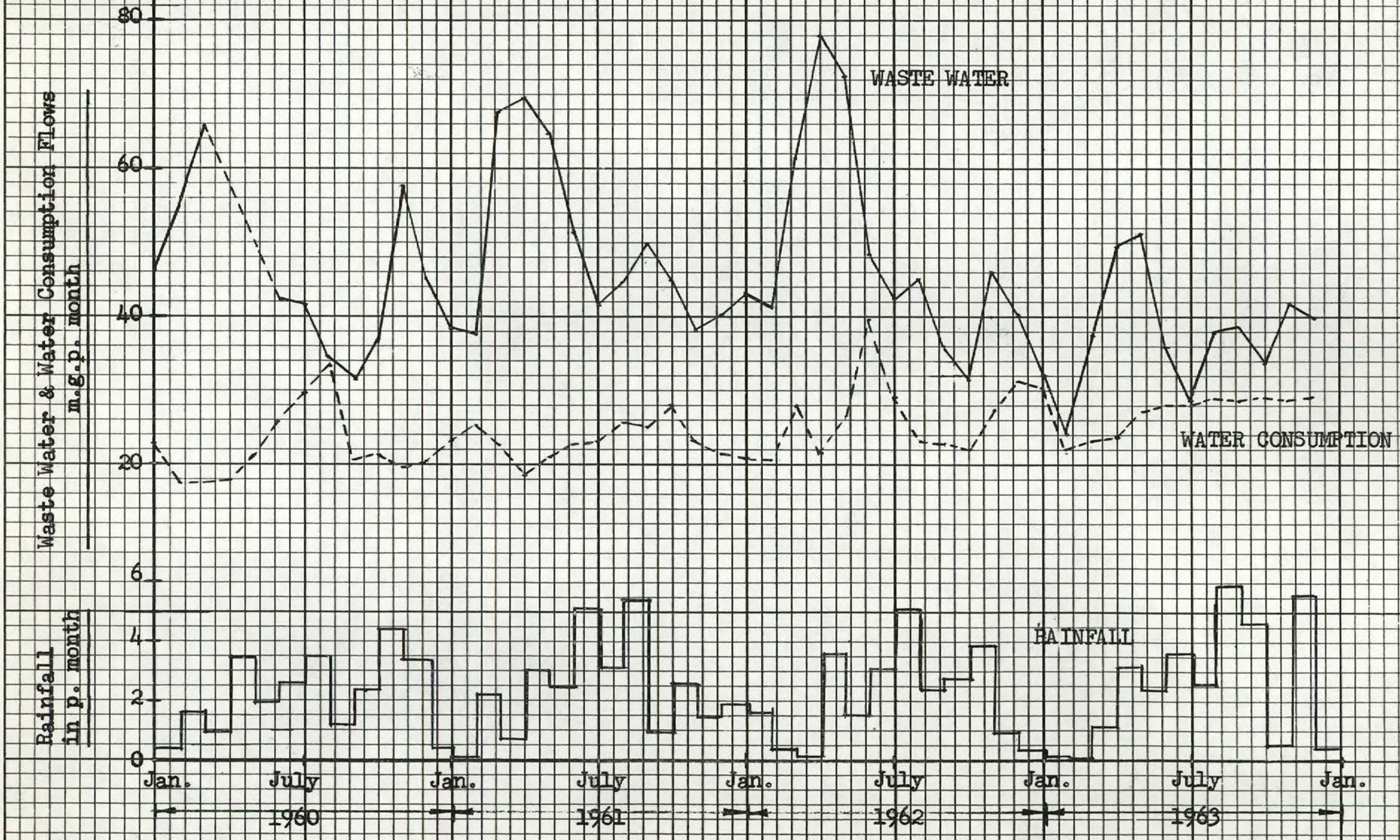


TABLE XX - CALCULATED INFILTRATION DURING RECORDED MONTHLY PEAK WASTE WATER FLOWS

Date	Recorded waste water flow	Recorded water consumption	Calculated infiltration g.c.d.
March 1960	65.72	17.67	164
November 1960	57.91	19.29	136
April 1961	69.72	18.95	166
September 1961	49.62	25.87	78
January 1962	43.48	21.11	64
April 1962	78.21	21.30	168
August 1962	45.09	23.11	63
November 1962	46.45	27.12	57
May 1963	51.00	26.83	65
September 1963	38.37	28.35	28
November 1963	41.91	28.35	38

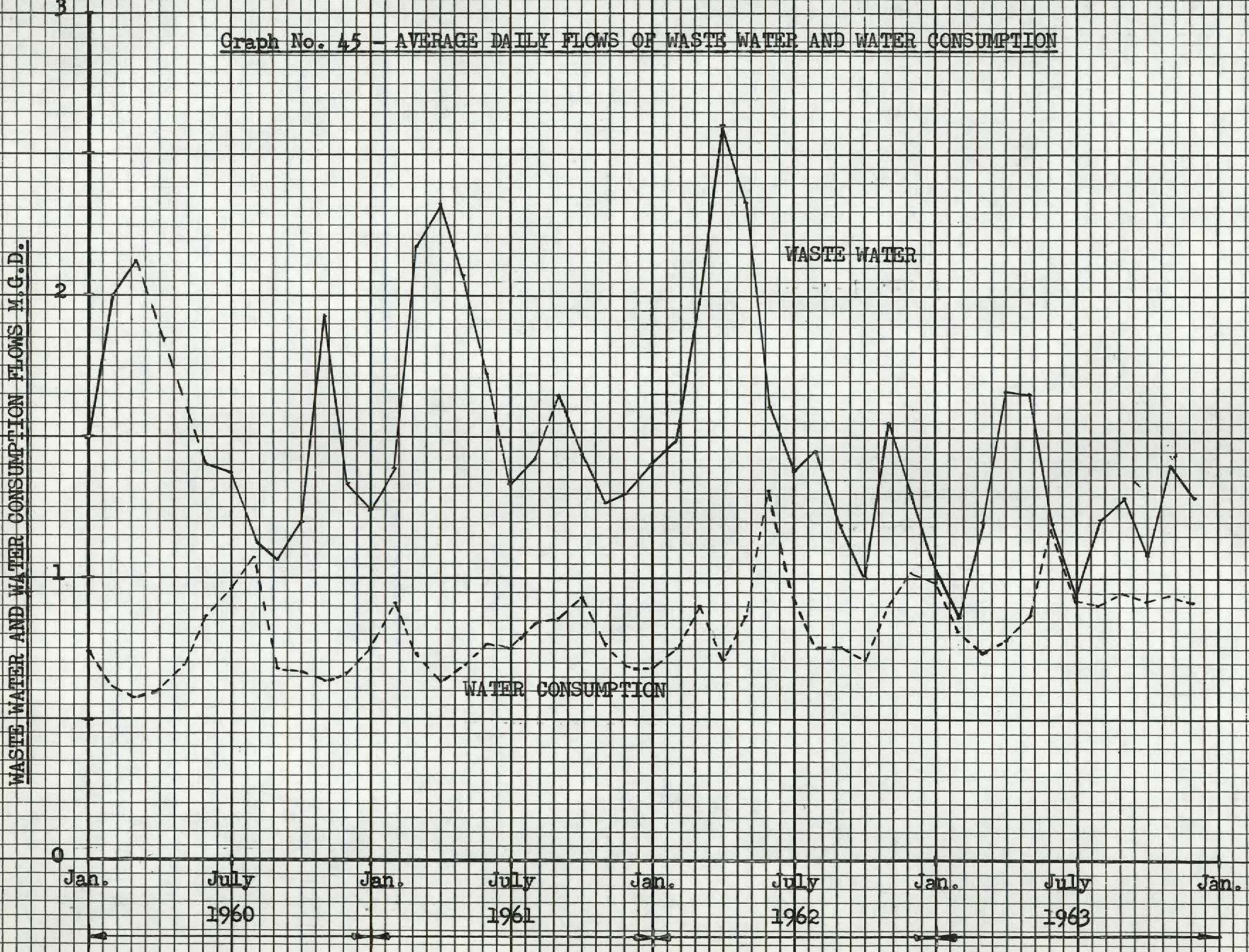
TABLE XXI - POPULATION FIGURES

Year	Population
1960	9,500
1961	10,200
1962	11,300
1963	12,000

TABLE XXII - AVERAGE DAILY FLOWS OF WASTE WATER AND WATER CONSUMPTION

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
1960												
Waste water m.g.	1.50	2.00	2.12	-	-	1.40	1.37	1.13	1.06	1.20	1.93	1.33
Water "	0.74	0.61	0.57	0.60	0.69	0.86	0.96	1.07	0.68	0.67	0.64	0.66
1961												
Waste water m.g.	1.24	1.39	2.17	2.32	2.07	1.72	1.33	1.42	1.65	1.44	1.26	1.30
Water "	0.75	0.91	0.73	0.63	0.69	0.76	0.75	0.84	0.86	0.93	0.76	0.69
1962												
Waste water m.g.	1.40	1.49	1.97	2.61	2.33	1.61	1.38	1.45	1.19	1.01	1.55	1.30
Water "	0.68	0.75	0.90	0.71	0.86	1.31	0.94	0.75	0.75	0.71	0.90	1.02
1963												
Waste water m.g.	1.05	0.86	1.20	1.66	1.65	1.19	0.93	1.21	1.28	1.08	1.40	1.28
Water "	0.99	0.81	0.74	0.78	0.87	1.18	0.92	0.91	0.95	0.92	0.94	0.91

Graph No. 45 - AVERAGE DAILY FLOWS OF WASTE WATER AND WATER CONSUMPTION



The observations of average daily flows of waste water and water consumption have been presented in Table XXII and plotted in graph No. 45. An inspection of the graph will show the following:

TABLE XXIII-CALCULATED INFILTRATION DURING RECORDED DAILY PEAK WASTE WATER FLOWS

Date	Recorded waste water flow m.g.d.	Recorded water consumption m.g.d.	Calculated Infiltration g.c.d.
March 1960	2.12	0.57	163
November 1960	1.93	0.64	136
April 1961	2.32	0.63	166
September 1961	1.65	0.86	78
April 1962	2.61	0.71	168
November 1962	1.55	0.90	57
April 1963	1.66	0.78	74
September 1963	1.28	0.95	28
November 1963	1.40	0.94	38

N.B. During periods of high precipitation, the waste water flows exceeded 3.5 m.g.d. and calculations for these periods have not been made. Calculated infiltration values were based on population figures shown in Table XXI.

It was evident from graphs Nos. 44 and 45 that during 1960, 1961 and early 1962, the peak waste water flows were high and were due to infiltration into the sewer system. Attempts were made to plot the rainfall figures and to find the contribution these played in raising the waste water flows.

In 1962, the Town attempted to seal the faulty joints that were found on closed circuit TV inspection of some sections of the sewer system. This joint sealing operation has been extended to the present time, covering a considerable section of the sewer system. As the result, a marked reduction in peak flows has been achieved as can be seen after November 1962 in graph No. 44.

The actual infiltration averaged 150 g.c.d. in 1960, 122 g.c.d. in 1961, 103 g.c.d. in 1962 and only 45 g.c.d. in 1963.

It might be concluded that infiltration into the sewers has been reduced considerably since the time the Town started sealing of faulty joints detected during inspection.

Though the value of 100 g.c.d. has been used by several consulting engineers in the designs of treatment plants, it has been shown that the actual infiltration figures have been reduced from 150 g.c.d. in 1960 to 45 g.c.d. in 1963.

It is believed that infiltration effect cannot be entirely eliminated in the sewer system, and allowance for infiltration should be considered in the future design of secondary treatment process.

APPENDIX B

CALIBRATIONS OF EQUIPMENT

1. Thickener

In order to determine the sludge blanket depths in the thickener, a measuring rod having a flat plate at the end was used. The rod was calibrated in feet from 0 to 11. The 11 foot mark on the rod corresponded to a zero sludge blanket depth while the 0 foot mark on the rod corresponded to the 11 foot sludge blanket depth in thickener.

Since the cross-sectional area of the thickener was 154 sq.ft., 1 foot division on the rod was equivalent to sludge volume of 960 gallons.

The graph No. 46 shows the calibration curve of the thickener when holding sludge.

2. Ground Storage Tank

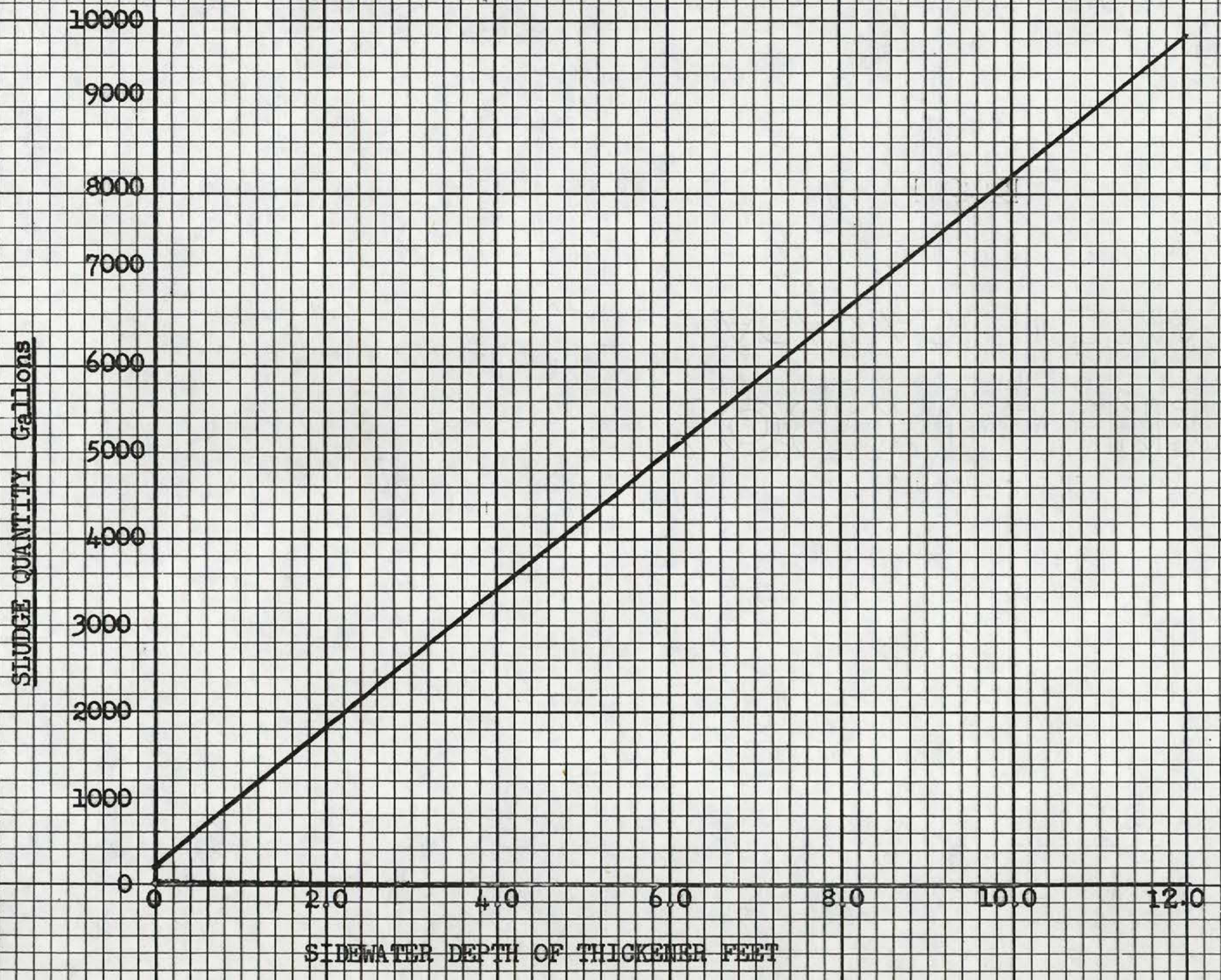
The ground storage tank is 5 feet in diameter by 15 feet long. A calibration curve of this tank is shown in graph No. 47.

3. Rectangular Weir

The rating curve of the rectangular weir used on the thickener was obtained by integrating the expression

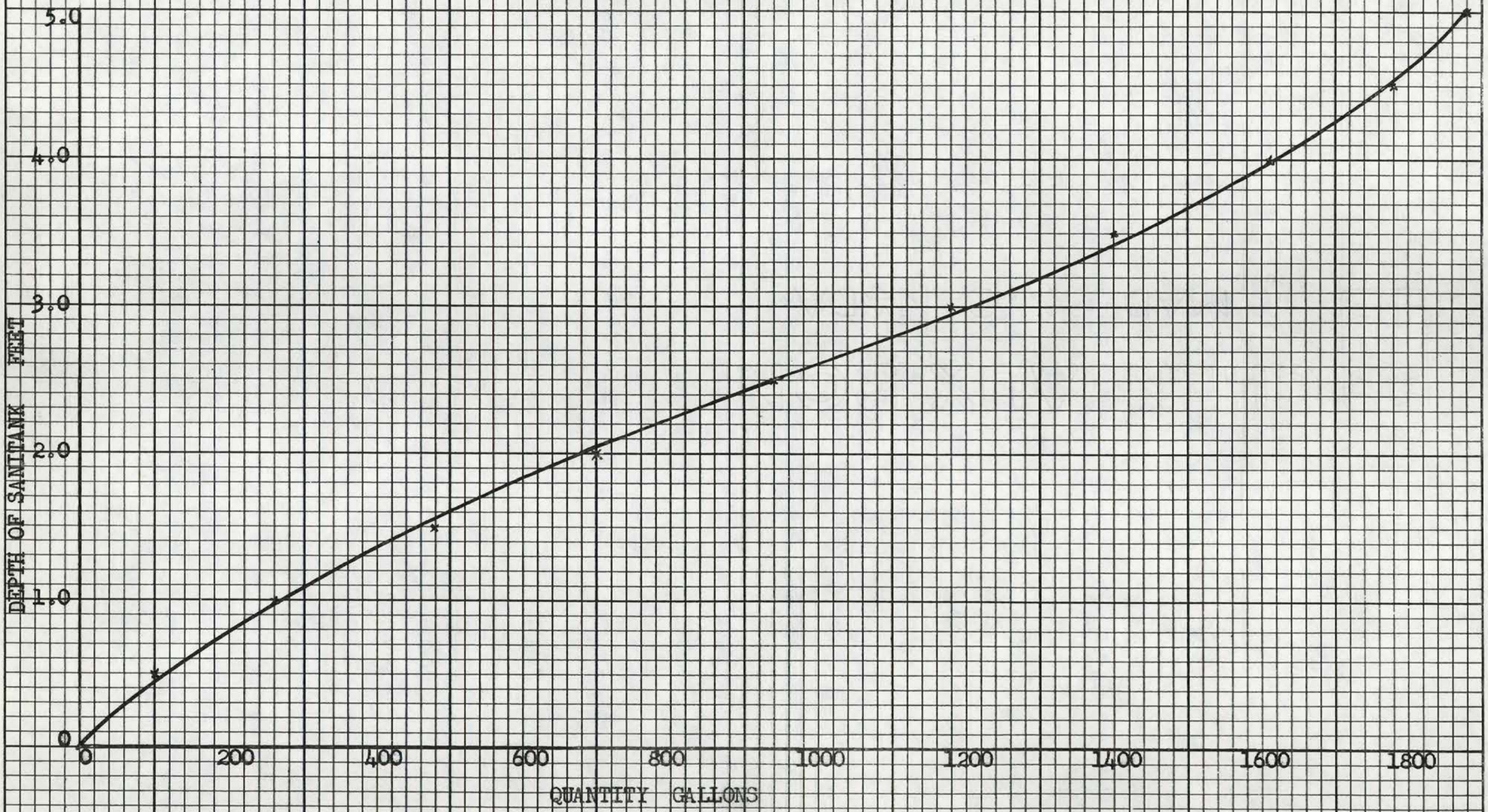
$$dQ = vdA = \sqrt{2gh}\ w.\ dh, \text{ where}$$

Q = instantaneous rate of flow, ft^3/sec.
v = instantaneous velocity of flow ft/sec. $= \sqrt{2gh}$
A = cross-sectional area of flow $ft^2 = w.dh$
g = acceleration due to gravity, ft/sec^2
h = elevation of pond above weir crest. ft.
w = surface width of stream cross-section passing through rectangular weir, ft.



Graph No. 46 - CALIBRATION CURVE OF THICKENER WHEN HOLDING SLUDGE

Graph No. 47 - CALIBRATION CURVE OF THE GROUND STORAGE TANK



Using the necessary conversion factors to yield flow rate in units of gallons per minute a rating equation of

$$Q = 3.33\ (W - 0.2H)\ H^{3/2}$$

was obtained where

Q = instantaneous rate of flow, c.f.s.
H = head on weir, ft.
W = width of effluent cross-section passing through weir ft.

Experiments were carried out to determine the flows over the rectangular weir and to obtain a rating curve for the weir.

Table XXIV shows the results of observed heads over weir at different openings of the delivery valve of the sludge transfer pump.

The possibility of measuring the discharge directly was considered but as mentioned earlier the cost of such metering equipment was too high.

Graph No. 48 shows the rating curve for the rectangular weir.

TABLE XXIV - OBSERVATIONS OF HEADS OVER RECTANGULAR WEIR

Valve opening	Observed "H" ft.	Calculated Flow
Full opening	0.262	157 g.p.m.
3/4 opening	0.221	124 g.p.m.
1/2 opening	0.210	115 g.p.m.
Full opening with part of sludge recirculated	0.134	60 g.p.m.
1964 pumping rate	0.163	80 g.p.m.

PLATE 4

Beaconsfield Waste Treatment Plant

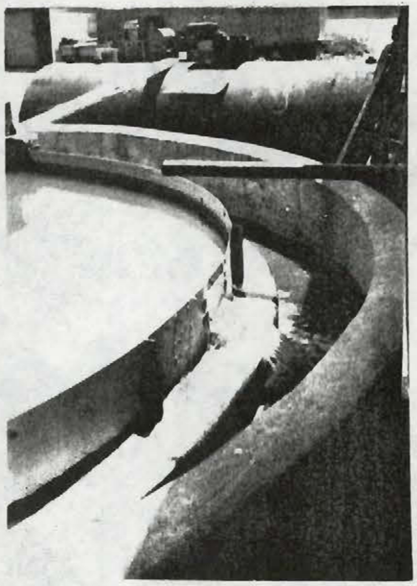


PLATE 5

12 inch rectangular weir on thickener periphery

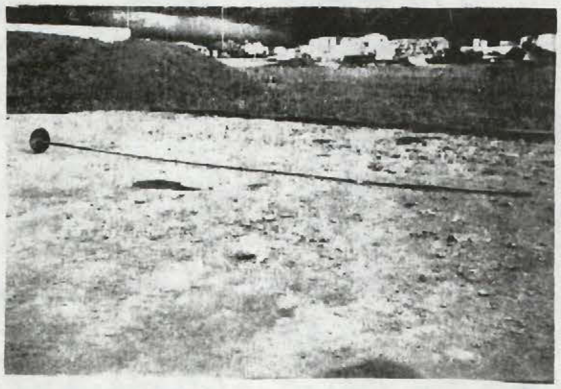
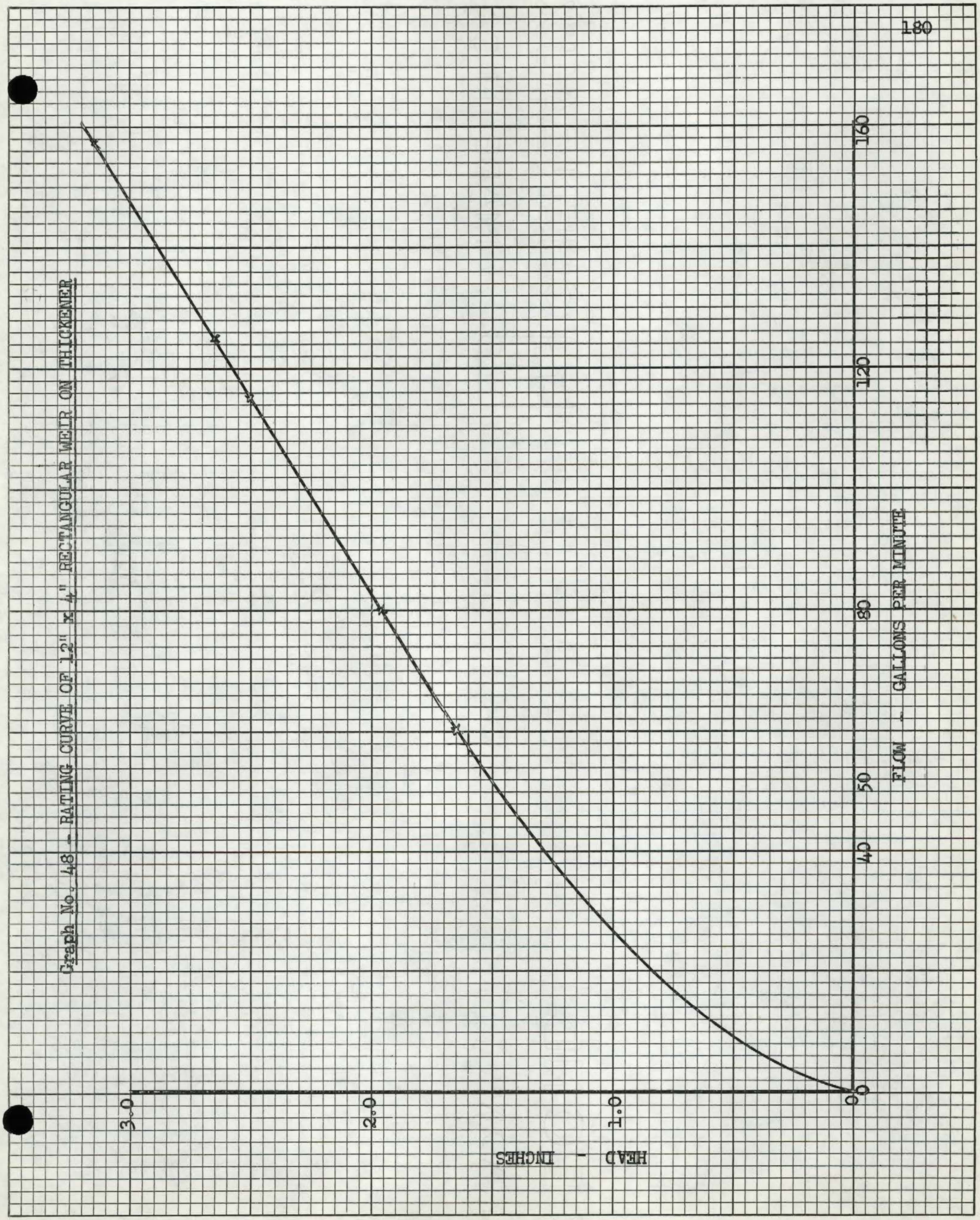


PLATE 6

Thickener
Sludge Depth Measuring Rod



Graph No. 48 - RATING CURVE OF 12" x 4" RECTANGULAR WEIR ON THICKENER

APPENDIX C

EXPERIMENTAL PROCEDURES

The procedures of the experiments carried out have been described while little explanation was given about sampling, pumping, measurements and chemical analyses of waste water and sludge. Discussion of all these matters is given below.

Sampling. In a waste flow in a sewer there are variations in quantity, quality, time, and place. Wastes at the surface of flow may be of floating, nature, at the bottom, wastes may consist of settling solids and sandy and gritty materials and at any intermediate point there may be found a mixture of both.

There is no ideal place for sampling, and as a compromise, sample would therefore be taken at a point slightly beneath surface where turbulence is mixing the wastes.

The two types of sampling in use are called grab or catch sampling and composite sampling.

Grab samples may be obtained at any moment but the data would be applicable to the time of sampling only, and to then prevailing conditions.

Composite sample is a mixture of grab samples taken at different times. The volume of grab samples would be at least 120 ml., obtained using a wide mouth bottle having a diameter of 35 mm. at the mouth. Basically a 24-hour period is the basis for obtaining a composite sample, taking grab samples each hour, and mixing these at the end of sampling period, or combining them in a single bottle as collected.

It is also desirable to combine the individual samples in volumes proportional to flow. A final volume of about 1/2 to 2/3 gallons would be sufficient.

Automatic sampling is done frequently today using different types of mechanical equipment.

Preservation of samples to be used depends on purpose of analyses. Samples of waste water, effluents and sludge should be analyzed as soon as possible after collection. Since no single method of preservation is entirely satisfactory, the method of preservation would be chosen with due regard to the examinations to be made. B.O.D. analyses would require that samples be immediately chilled to 3°-4°C and kept at this temperature during compositing period. Samples for determining C.O.D. by the dichromate method would be preserved by adding sufficient H_2SO_4 to produce a final acidity of pH 2-3. Sludge sample examination would be made as soon as possible.

A description of sampling stations at the waste water treatment plant in Beaconsfield has been given earlier in Chapter VII and in Figure 10. Sludge samples were taken every 2 hours and analyzed. Waste water samples were analyzed using a 24-hour composite sample.

<u>Pumping</u>. Special arrangements were made to run only the pumps that would affect the loadings of the thickener. The operation of the low lift pumps was not to be disturbed since these pumps are set automatically to operate with the flow of waste water coming into the plant.

Generally reciprocating plunger pumps are used for raw sludge pumping but the sludge transfer pump at the plant is of a centrifugal type.

During the preliminary investigation of the treatment plant at Beaconsfield, it was found that the pumping rate of the sludge transfer pump was about 157 g.p.m. Some attempts to regulate the pump flow by throttling the discharge valve resulted in reduction of efficiency, pump clogging and loss of sufficient suction to draw the sludge from the clarifier sump.

Attempts were later made in December 1963 to reduce the speed of the sludge transfer pump by introducing a brake drum on the shaft to the motor of the pump. A pumping rate of 80 g.p.m. has now been effected by this method.

Under normal operating conditions, clarifier sludge would be pumped on 24-hour a day period. During tests Nos. 3 and 4, the sludge transfer pump was run 12 hours a day only.

A reciprocating plunger pump situated in the pump pit in the A.S.T. building transfers the thickened sludge from the thickener to the ground storage tank. The stroke of the pump is reduced so as to produce a pumping rate of sludge of about 10 g.p.m.

The following procedure has been followed when pumping sludge from the thickener into the ground storage tank.

1. Ascertain whether and/or how much sludge should be pumped.
2. Ascertain whether there is room enough in the storage tank.
3. If sludge is to be pumped, close the main electrical switch in the pit, and open the 10-inch main valve between the thickener and the pump. The valve would be fully opened after 34 full turns of the valve handle.

4. Open the connecting rod oiler and finger tighten the grease cup.
5. Start the pump, and keep pumping until the sludge becomes thin or watery as would be shown by grab samples obtained from the 2-inch sampling valve located upstairs.
6. When the sludge becomes thin, stop the pump, and restart it after about 30 minutes.
7. Take samples immediately from upstairs sampling valve.
8. If the sludge is once again thick, then keep pumping until the sludge becomes thin, stop the pump, wait 30 minutes and repeat these cycles of operation until the sludge becomes repeatedly thin immediately after restarting the pump.
9. To shut down for the day, open the main switch and close the main valve and connecting rod oiler.

Note: The capacity of the sanitank truck is 1200 gallons and the capacity of the ground storage tank is 1860 gallons. About 1000 to 1200 gallons of 10% sludge should be pumped per day.

During test periods, pumping of thickened sludge was carried out on time clock basis at intervals throughout the day. Normally a working day of 8 a.m. - 10 p.m. is adopted for the tests on sludge conditioning.

Trial pumping of sludge was carried out during the month of August 1963 to determine the daily quantity of sludge to be withdrawn from the thickener. The results of observations are shown in Table XXV and graphs Nos. 49 and 50.

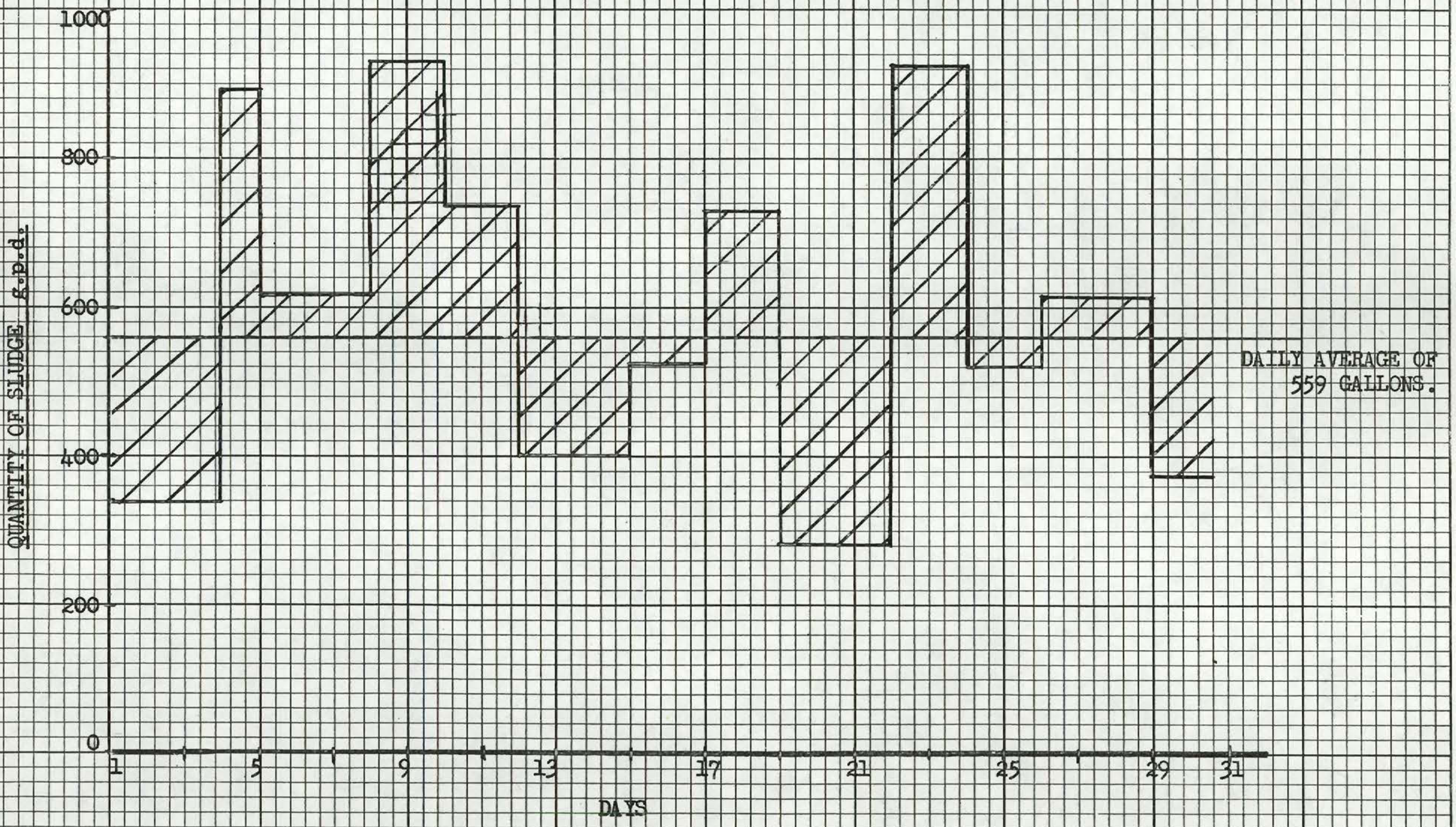
TABLE XXV - SLUDGE PUMPING FOR AUGUST 1963

Date	Gallons Pumped	Average G.P.D.
August 1-3	1020	340
4	890	890
5-7	1850	617
8-9	1860	930
10-11	1475	738
12-14	1200	400
15-16	1050	525
17-18	1460	730
19-21	850	283
22-23	1850	925
24-25	1040	520
26-28	1850	617
29-31	1120	373
Totals	17,335 gallons	
Average	559 g.p.d.	

N.B. The sludge depth in the thickener was maintained throughout the month at 4 feet.

Graph No. 49 - SLUDGE PUMPED FROM THICKENER DURING THE MONTH OF AUGUST 1963

Graph No. 50 - VARIATION IN DAILY QUANTITY OF SLUDGE PUMPED FROM THICKENER, AUGUST 1963



APPENDIX D

PHYSICAL AND CHEMICAL ANALYSES

a. Waste Water Analyses

The temperature of waste water is normally slightly higher than that of water supply because of heat added during the use of water. Abnormally low temperature would indicate infiltration of ground water or surface water, high temperature on the other hand may indicate presence of industrial wastes. With a rise in temperature, viscosity decreases, and settling progresses rapidly. Biological activity also increases with rise in temperature.

During tests of the treatment units at Beaconsfield, the temperatures of wastes and sludge were taken. Temperature measurements were made with a good grade mercury filled centigrade thermometer and expressed in degrees centigrade.

The color of waste is usually gray. Black color and scum indicate progressed decomposition. Waste is almost odorless but the odor may be soapy. When waste is septic the odor which results would be hydrogen sulphide, indol, scatol and putrid.

The basic chemical analyses required for design of a primary treatment plant may be given as follows:

1. B.O.D.
2. Suspended and dissolved solids
3. Total solids
4. C.O.D.

5. Settleable suspended solids
6. Total nitrogen
7. Alkalinity
8. Chlorine residual
9. Chlorine demand
10. pH.

General data of analyses are given in Table XXVI. The table shows wider limits which may be expected due to great variation in the composition and concentration of the waste.

TABLE XXVI - CHEMICAL ANALYSES DATA OF WASTE WATER

mg/l.

Sewage	Strong	Average	Weak
Solids, total	1,200	830	200
volatile	700	480	120
fixed	500	350	80
Suspended, total	500	295	100
volatile	400	215	70
fixed	100	85	30
Settleable solids	300	180	50
B.O.D.$_5$	300	180	50
Nitrogen	85	50	25
Chlorides	175	100	15

In terms of grams/capita/day.

		B.O.D.
Solids, total	250	54
Suspended, total	90	42
volatile	65	
fixed	25	
Dissolved, total	160	12
volatile	80	
fixed	80	

The separate items in Table XXVI may be discussed as follows:

Total solids are the residue on evaporation of a waste water sample. Determination of total solids may be useful control of plant operation. The results for total residues would however be subject to considerable error because of losses of volatile compounds during evaporation and of carbon dioxide during ignition. In the interpretation of results, these possible sources of errors would have to be recognized.

The procedure for determining total solids of waste is as follows:

100 ml. of composited sample is evaporated in an ignited and tared dish, the sample is then dried for 1 hour at 103°C in a conventional oven.

$$\text{mg/l residue on evaporation} = \frac{\text{mg. residue} \times 1000}{\text{ml. sample}}$$

Total volatile and fixed residues are determined by igniting the residue at 600°C in an electric muffle furnace to constant weight for 1 hour. The loss on ignition is reported as mg/l. volatile solids, and the residue as mg/l. fixed solids.

The suspended solids are determined by filtering wastes through an asbestos mat and the increase in weight represents the total suspended matter, which again can be divided into organic and inorganic by ignition.

The apparatus used for making the asbestos mat consists of a 25 ml. Gooch crucible and suction equipment. The reagent is asbestos cream made by adding 10 grams acid - washed, medium fiber asbestos to 1 liter of distilled water.

In order to prepare the mat, the crucible is first filled with asbestos cream, and allowed to stand for 2 minutes before applying suction. After the water has been drawn through the mat, the suction is left on while the crucible is filled with distilled water. This is likewise drawn through the mat.

The procedure of washing the mat is repeated twice more. The crucible with the mat is then dried and ignited in a muffle furnace at 600°C for 1 hour. After removal from the furnace the crucible is partially cooled in air then in a desiccator.

Before filtering the waste sample the crucible is weighed. Then 100 ml. well composited sample is filtered through the weighed Gooch crucible, using suction. Leaving the suction on, 10 ml. of distilled water is used to wash the residue in order to remove soluble salts. The crucible and solids are then dried at 103°C for 1 hour, allowed to cool to room temperature in a desiccator before weighing.

$$\text{mg/l total suspended matter} = \frac{\text{mg. suspended solids} \times 1000}{\text{ml. sample}}$$

The volatile and fixed suspended matter are determined by the same method discussed already under volatile and fixed residues of total solids.

Settleable matter is determined by permitting 1 liter of waste sample to settle in standard Imhoff cone for 1 hour. The volume of settleable matter in the cone is recorded as ml/l. The remainder is the unsettleable suspended matter.

The term "biochemical oxygen demand" (B.O.D.) is the amount of oxygen required by bacteria present in waste to oxidize organic matter into simpler substances with the release of energy. The B.O.D. test may be considered as wet oxidation procedure in which the living organisms serve as a medium for oxidation of organic matter to CO_2 and H_2O. There is a quantitative relation between the amount of O_2 required to convert a given amount of organic matter to CO_2 and H_2O and this is the basis for B.O.D. determination.

The B.O.D. then is governed by bacterial population, type, number and temperature. Temperature effects are kept constant by carrying out all analyses at 20°C which is about the median value for most waste waters.

Theoretically an infinite time is required for completion of oxidation but practically 20 days is considered to be a limiting time.

B.O.D. can be measured directly in a few samples, but a dilution procedure is usually required.

Direct method would apply if $B.O.D._5$ does not exceed 7 mg/l. and nearly complete saturation would be obtained by diffusion of air in the sample at 20°C.

Dilution method would give B.O.D. of the diluted sample as a direct ratio of the percentage of waste in dilution. 10% dilution would use oxygen at 1/10 rate of the 100% sample.

The results of B.O.D. tests carried out on wastes treated at Beaconsfield treatment plant are given in Table XXVII.

TABLE XXVII - RESULTS OF B.O.D. TESTS ON WASTE AT BEACONSFIELD TREATMENT PLANT

Date	Sampling at Station 1 Influent	Sampling at Station 2 Clarifier Effluent	Sampling at Station 3 Chlorinated Effluent
B.O.D. (5 days at 20°C) p.p.m.			
12/9/63	97.87	23.49	24.42
23/10/63	64.60	44.2	36.35
-/11/63	35.1	38.8	22.50

N.B. Tests were carried out by Warnock Hersey Company Ltd., Quebec.

The Chemical Oxygen Demand has been used as a means of measuring the pollution strength of wastes. It is based upon the fact that all organic compounds with few exceptions can be oxidized to CO_2 and H_2O by strong oxidizing agent under acid conditions. Lignin and glucose are oxidized completely. As a result C.O.D. values are greater than B.O.D. and may be much greater when significant amounts of biologically resistant organic matter is present.

The major advantage of C.O.D. test is its short time interval required for evaluation — 3 hours rather than 5 days. It was decided that C.O.D. test would be substituted for B.O.D. in the tests that were carried out at Beaconsfield treatment plant.

The oxidizing agents employed in C.O.D. determination are many and varied, but potassium dichromate has been found to be the most practical of all since it is capable of oxidizing a wide variety of organics almost completely to CO_2 and H_2O. Other reagents would be sulphuric acid, standard ferrous ammonium sulphate solution and ferroin indicator solution.

The procedure for C.O.D. determination is as follows; 50 ml. sample is placed in a round bottom flask and 25 ml. standard dichromate solution is added. To the mixture is added 75 ml. H_2SO_4. The reflux mixture is thoroughly mixed, before heat is applied. The flask is then attached to the Friedrichs condenser and the mixture is refluxed for 3 hours.

The condenser is used to intercept escaping volatile materials. During oxidation pH must be small, acid conditions govern, and temperature must be high. After refluxing of the mixture, the content in the flask is washed with distilled water, into a 500 ml. conical flask. The mixture is then diluted to about 350 ml. with distilled water and the excess dichromate in the mixture is titrated with standard ferrous ammonium sulphate using ferroin indicator. The color change would be sharp, going from a blue green to a reddish blue.

A blank consisting of 50 ml. distilled water instead of the sample together with the reagents is also refluxed in the same manner.

$$\text{mg/l C.O.D.} = \frac{(a - b)c \times 8000}{\text{ml. sample}} - d.$$

where a = ml. $Fe(NH_4)_2\,(SO_4)_2$ used for blank.
b = ml. $Fe(NH_4)_2\,(SO_4)_2$ used for sample
c = normality $Fe(NH_4)_2(SO_4)_2$
d = Cl correction = mg/l . Cl x 0.23

The pH value of a solution is the logarithm of the reciprocal of its hydrogen ion concentration. Although it has little bearing on the strength of sewage, yet it often provides a convenient control in chemical treatment processes.

The pH values of wastes at Beaconsfield are determined by the glass electrode method. The electrode systems are calibrated against buffer solutions of known pH value.

b. Sludge Analyses.

From previous discussion in the text it was shown that moisture content is of great importance in sludge thickening. Apart from moisture content determination, it may be necessary to know both the mineral and organic contents of the sludge.

Moisture content determination was carried out both on the spot by a Cenco moisture balance and in the laboratory by the standard method.

The Cenco moisture balance as the name suggests is a device for determination of the moisture content of various materials which have been finely divided by grinding, pulverizing or other methods. Its application has been adaptable for finding the moisture content of waste water sludge. The moisture balance is effective for any material from which the moisture can be removed by the heat of the infra red lamp, providing the heat does not change the chemical composition of the material. It employs a sensitive torsional balance for weighing the sample. A scale, graduated into divisions from 0 to 100%, is mounted on a large drum, and readings can be taken continuously. The reading at any given time during heating of the sample would be the percentage loss in weight of the sample due to loss of moisture.

In order to successfully use the balance it is necessary to calibrate it and decide on a definite procedure for obtaining the moisture content of a sludge sample.

After determining moisture contents of over 400 sludge samples, the writer decided on the following procedure:

1. Turn the scale lamp on by means of the toggle switch at the right.

2. Rotate the scale until the 100% mark coincides with the index by turning the scale adjusting knob, which is located on the right side of the unit. This would establish a reference point.

3. Move the pointer to the index by turning the pointer adjusting knob which is located on the left side of the unit. The knob would be rotated in a direction opposite to that in which the pointer would move to coincide with the index.

4. Rotate the scale until the 0% mark coincides with the index. This would preset the amount of unbalance. The pointer would then be above the index.

5. Raise the lamp housing and carefully distribute the sludge sample on the sample pan until the pointer returns to the index. Approximately 25 grams of sludge sample would be needed. This weight of material would then correspond to 100 divisions of the scale.

6. Lower the lamp housing, and turn lamp on by means of the toggle switch at the left. Start a stop clock at the same time in order to prepare a drying curve (percent moisture content against time mins.) for the sludge sample.

7. Readjust the powerstat control to 120. The sample would begin to lose moisture and the pointer would rise above the index. Readings of moisture content % at intervals of 5 minutes are taken from 0 to 20 minutes. After about 23 minutes of drying the

powerstat control is readjusted to 90 and readings are continued at intervals of 2 minutes from the 23rd minute to the 36th minute. The sample would be completely dried after 36 minutes and the weight would stop changing. Wait for one to three minutes before recording final moisture content.

8. Switch off the infra red lamp and remove the dry sample for ash analysis.

Note: Discoloration of the sample or smoke would indicate the release of volatile matter other than moisture and would yield erroneous results.

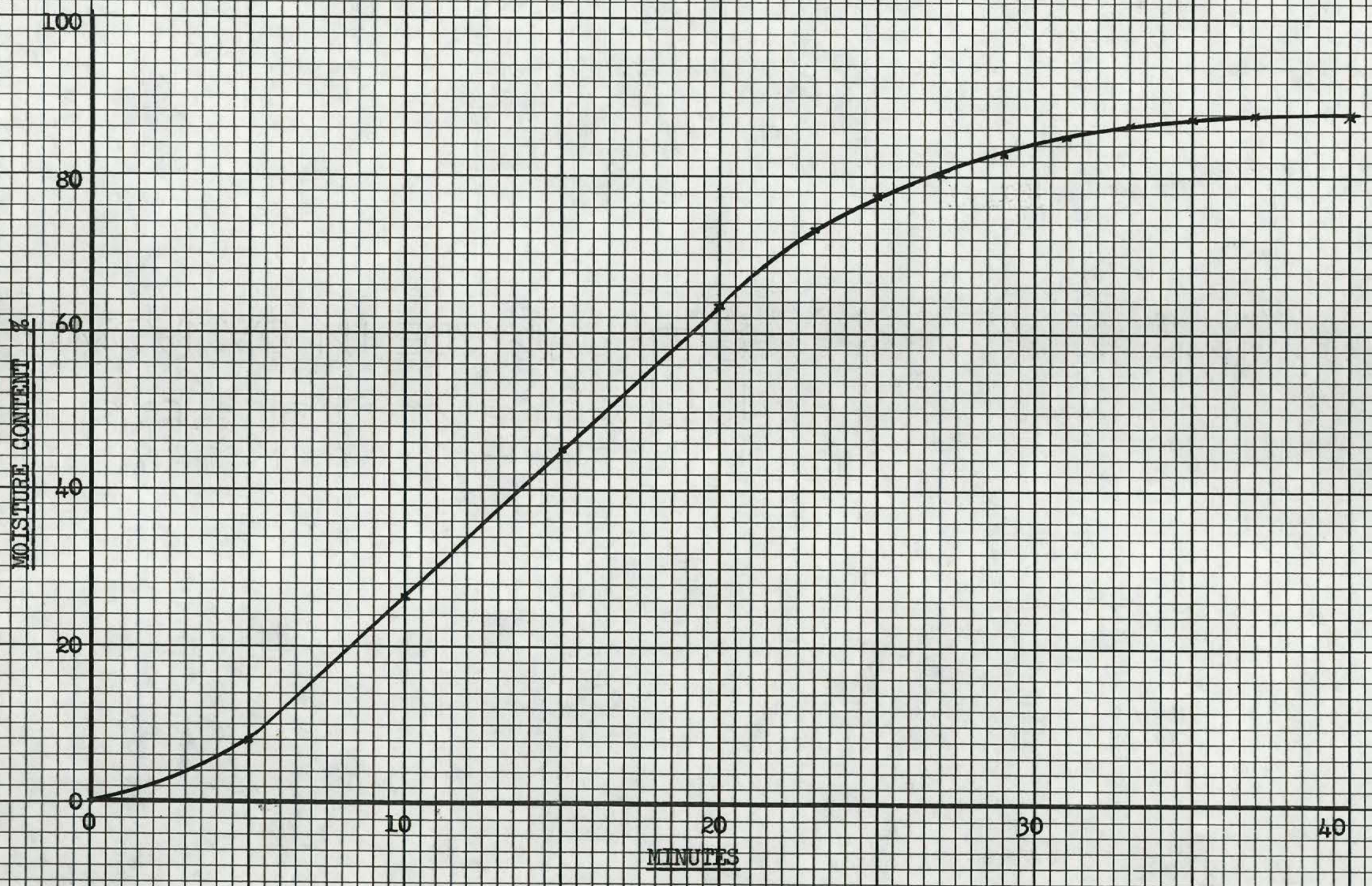
Sample readings and drying curves on sludge sample taken for a typical moisture content determination are shown below:

TABLE XXVIII - READINGS TAKEN DURING SLUDGE DRYING ON CENCO MOISTURE BALANCE

May 18, 1964.

TIME	MINUTES	MOISTURE CONTENT %
9.00	0	0
9.05	5	8.0
9.10	10	26.6
9.15	15	45.2
9.20	20	63.6
9.23	23	73.2
9.25	25	77.6
9.27	27	80.4
9.29	29	83.0
9.31	31	85.0
9.33	33	86.4
9.35	35	87.6
9.37	37	88.0
9.40	40	88.0

Graph No. 51 - DRYING CURVE OF SLUDGE SAMPLE USING CENCO MOISTURE BALANCE



Determination of total residue and moisture content by laboratory procedures are as follows:

About 25 grams of sludge sample is weighed in a tared evaporating dish. This is evaporated to dryness in a water bath, and then dried in an oven at 103°C for 1 hour. The residue is then cooled in a desiccator before weighing. Prolonged heating would produce loss of volatile organic matter which would give erroneous results.

The volatile residue is determined by igniting the residue obtained above in an electric muffle furnace at 600°C for 1 hour. The ash is cooled in a desiccator and reweighed. The results obtained are reported as percent ash and volatile solid.

The specific gravity of the sludge is determined by comparing the weight of a volume of the sample with that of an equal volume of distilled water.

$$\text{specific gravity} = \frac{\text{weight of sludge sample}}{\text{weight of distilled water}}$$

The suspended matter of sludge is determined by the aluminum dish method. The apparatus consists of aluminum dish with a perforated bottom, filter paper (Whatman No. 1), sponge rubber ring to fit rim of aluminum dish, Buchner funnel and a filter flask.

The dish and filter paper are first dried in an oven, cooled and weighed. The apparatus is then assembled so that about 20 in. of vacuum is applied on the filter paper. About 50 ml. sludge sample is applied to the dish. After water had been extracted, the sample is dried in an oven at 103°C for 1 hour and then cooled and weighed.

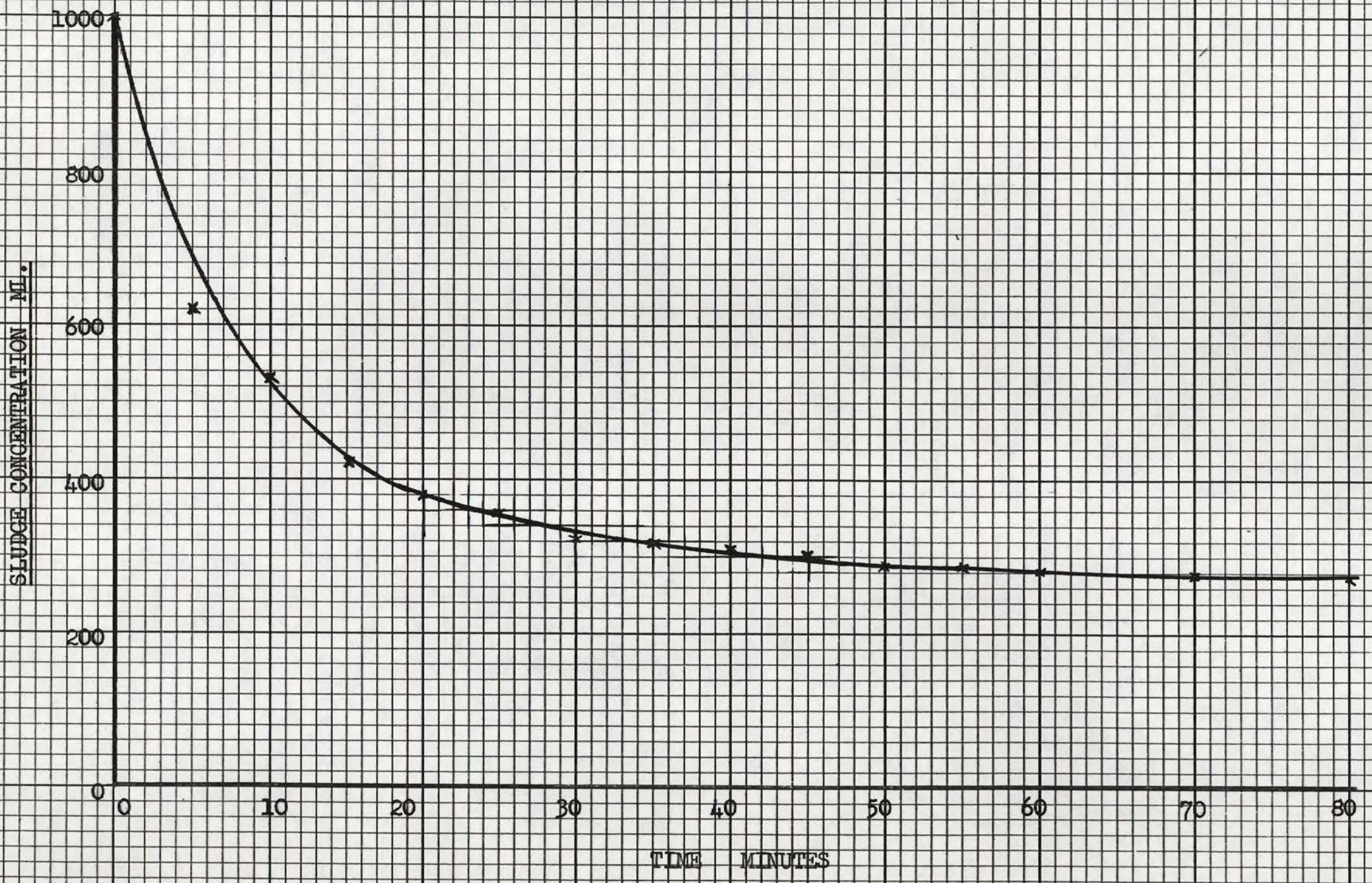
$$\text{mg/l total suspended solids} = \frac{\text{mg suspended solids x 1000}}{\text{ml. sample}}$$

The settleability of the sludge from the primary tank is determined by allowing 1000 ml. of sludge sample to settle in a 1 liter graduated cylinder. The volumes occupied by the sludge at various intervals of time are recorded and these values are plotted. A curve indicating rate of settling is then obtained. Results of a typical settling analysis are shown in Table XXIX and a settling curve has been drawn in graph No. 52.

TABLE XXIX - RESULTS OF SETTLING ANALYSIS ON SLUDGE SAMPLE - 28 AUGUST 1963

Time	Minutes	Concentration of sludge ml.
4.00	0	1000
4.05	5	620
4.10	10	530
4.15	15	420
4.20	20	380
4.25	25	360
4.30	30	340
4.35	35	320
4.40	40	310
4.45	45	300
4.50	50	290
4.55	55	285
5.00	60	280
5.10	70	270
5·20	80	270

Graph No. 52 – SETTLING CURVE OF SLUDGE SAMPLE



The pH determinations of sludge samples were carried out on the liquors which separate from sludge samples. The glass electrode method was used for obtaining pH values of sludge samples.

The temperature of the sludge at the time of collection would be recorded to the nearest degree centigrade with a good mercury filled centigrade thermometer.

APPENDIX E

EXPERIMENTAL DATA

The results of experiments on sludge conditioning are included in pages 205 to 267, and those on the waste water treatment plant at Beaconsfield are included in pages 268 to 283.

TEST NO. 1 August 28th, 1963.

Waste water flow 1.143 m.g.d. Thickener underflow 805 g.p.d.

Sludge pumping into thickener 24 hours Depth of sludge in thickener 3 ft.

Thickener loading 1400 gals./sq.ft./d. Specific gravity 1.034

SOLIDS CONTENT - First Day.

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	DA-1	13.50			
	DA-2	14.80			
	DA-3	15.00	16.16	4.91	30.4
	DA-4	19.69			
	DA-5	17.17			
10 a.m.	DB-1	15.20			
	DB-2	15.20			
	DB-3	15.20	15.16	0.604	4.0
	DB-4	14.93			
	DB-5	15.67			
12 noon	DC-1	14.20			
	DC-2	13.80			
	DC-3	15.00	14.30	1.94	13.45
	DC-4	13.80			
	DC-5	14.26			
2 p.m.	DD-1	15.50			
	DD-2	14.00			
	DD-3	15.00	14.49	2.25	15.58
	DD-4	13.88			
	DD-5	14.88			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	DE-1	15.20			
	DE-2	16.20			
	DE-3	16.20	16.30	0.966	5.94
	DE-4	16.71			
	DE-5	15.98			
6 p.m.	DF-1	15.80			
	DF-2	14.80			
	DF-3	15.80	15.37	1.700	11.10
	DF-4	15.09			
	DF-5	15.71			
8 p.m.	DG-1	14.80			
	DG-2	14.60			
	DG-3	15.60	14.95	1.61	10.75
	DG-4	14.44			
	DG-5	14.56			
10 p.m.	DH-1	13.41			
	DH-2	14.94			
	DH-3	15.16	15.157	1.07	7.05
	DH-4	15.80			
	DH-5	15.20			
Average for the day			15.18	1.414	9.34

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	DA-1	42.35			
	DA-2	41.71			
	DA-3	40.48	41.54	2.27	5.45
	DA-4	42.53			
	DA-5	42.48			
10 a.m.	DB-1	38.45			
	DB-2	39.35			
	DB-3	39.61	40.32	2.60	6.44
	DB-4	42.48			
	DB-5	41.75			
12 noon	DC-1	39.98			
	DC-2	40.20			
	DC-3	39.52	40.51	4.40	10.85
	DC-4	42.72			
	DC-5	39.44			
2 p.m.	DD-1	40.15			
	DD-2	40.96			
	DD-3	38.96	40.83	4.30	10.55
	DD-4	42.98			
	DD-5	43.63			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	DE-1	41.55			
	DE-2	39.57			
	DE-3	40.49	41.12	2.42	5.86
	DE-4	43.03			
	DE-5	43.04			
6 p.m.	DF-1	39.64			
	DF-2	40.69			
	DF-3	39.27	40.56	2.97	7.33
	DF-4	41.99			
	DF-5	42.92			
8 p.m.	DG-1	38.93			
	DG-2	40.64			
	DG-3	41.40	41.34	0.55	1.34
	DG-4	42.15			
	DG-5	43.01			
10 p.m.	DH-1	41.40			
	DH-2	42.58			
	DH-3	40.82	41.74	3.56	8.54
	DH-4	42.67			
	DH-5	40.48			
Average for the day			40.82	0.874	2.14

SAMPLE OBSERVATIONS

Test No. 1 - First Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time.	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	DA-1-2-3							
8.05	0	0	8.50	0	0	9.32	0	0
8.10	5	10	8.55	5	9	9.37	5	9.0
8.15	10	26	9.00	10	26	9.40	8	22.0
8.20	15	42.6	9.05	15	42.6	9.45	13	41.4
8.25	20	62.0	9.10	20	58.0	9.50	18	57.0
8.30	25	73.0	9.15	25	71.8	9.55	23	75.4
8.35	30	81.0	9.20	30	82.0	9.58	26	80.6
8.37	32	86.4	9.23	33	85.2	10.00	28	83.0
8.40	35	86.5	9.25	35	85.2	10.05	33	85.0
SAMPLES	DB-1-2-3							
10.13	0	0	10.51	0	0	11.35	0	0
10.18	5	9.2	10.56	5	9	11.40	5	9.6
10.20	7	18.6	11.01	10	25	11.45	10	28.0
10.25	12	37.4	11.06	15	40.0	11.50	15	48.0
10.30	17	55.6	11.11	20	57.0	11.55	20	66.0
10.35	22	72.4	11.16	25	65.0	12.00	25	78.6
10.40	27	82.0	11.21	30	70.0	12.03	28	82.4
10.45	32	84.6	11.25	34	79.0	12.05	30	84.0
10.47	34	84.8	11.29	38	84.6	12.07	32	84.6
10.49	36	84.8	11.31	40	84.8	12.10	35	84.8

OBSERVATIONS (CONT'D.)

SAMPLES DC-1-2-3

Time	Mins	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
12.14	0	0	12.50	0	0	1.27	0	0
12.20	6	16.0	12.55	5	11.0	1.30	3	4.0
12.25	11	37.0	1.00	10	33	1.36	9	28.0
12.30	16	57.4	1.05	15	54	1.40	13	45.0
12.35	21	75.4	1.10	20	72.8	1.50	23	79.0
12.40	26	82.2	1.15	25	82.2	1.54	27	83.6
12.44	30	85.4	1.18	28	85.0	1.56	29	84.8
12.46	32	85.6	1.20	30	86.0	1.58	31	85.0
12.47	33	85.6	1.25	35	86.2	2.00	33	85.0

SAMPLES: DE-1-2-3

4.07	0	0	4.45	0	0	5.21	0	0
4.12	5	14	4.50	5	15.8	5.26	5	3.5
4.17	10	30.4	4.55	10	34.0	5.31	10	30.0
4.22	15	50.6	5.02	17	67.6	5.36	15	56.0
4.27	20	68.4	5.05	20	74.0	5.41	20	72.2
4.32	25	80.4	5.10	25	83.6	5.46	25	79.6
4.37	30	85.0	5.13	28	84.6	5.51	30	84.0
4.40	33	85.8	5.15	30	84.8	5.55	34	84.8

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	DF-1-2-3							
6.06	0	0	6.42	0	0	7.25	0	0
6.11	5	9.4	6.47	5	10.4	7.30	5	11.2
6.17	11	34.4	6.52	10	32.4	7.37	12	41.8
6.26	20	67.4	6.57	15	53.0	7.40	15	53.0
6.30	24	77.8	7.02	20	71.6	7.45	20	69.4
6.36	30	84.0	7.07	25	82.0	7.50	25	79.0
6.38	32	84.2	7.12	30	85.0	7.55	30	83.8
6.40	34	84.2	7.15	33	85.2	8.00	35	84.2
SAMPLES	DG-1-2-3							
8.06	0	0	8.42	0	0	9.19	0	0
8.11	5	12.0	8.47	5	11.4	9.24	5	12.0
8.16	10	32.0	8.51	9	27.6	9.29	10	32.6
8.21	15	53.6	8.56	14	48.8	9.34	15	53.4
8.26	20	71.4	9.01	19	68.4	9.39	20	71.4
8.31	25	80.4	9.06	24	79.4	9.44	25	79.6
8.36	30	85.2	9.11	29	84.6	9.50	31	84.0
8.39	33	85.2	9.15	33	85.4	9.55	36	84.4

N.B. **Analyses of samples not shown above were carried out in the Pointe Claire Laboratories of P.P.R.I.C.**

TEST No. 1 August 29th, 1963.

Waste water flow 1.23 m.g.d. Thickener underflow 830 g.p.d.

Sludge pumping into thickener 24 hours Depth of sludge in thickener 3 ft.

Thickener loading 1400 gals./sq.ft/d. Specific gravity 1.037

SOLIDS CONTENT - Second Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	JA-1	16.5			
	JA-2	16.1			
	JA-3	16.4	16.33	0.462	2.83
	JA-4	16.37			
	JA-5	16.55			
10 a.m.	JB-1	13.97			
	JB-2	13.84			
	JB-3	14.16	14.30	0.534	3.74
	JB-4	14.90			
	JB-5	14.95			
12 noon	JC-1	15.50			
	JC-2	15.0			
	JC-3	15.4	15.27	0.608	3.99
	JC-4	15.34			
	JC-5	14.98			
2 p.m.	JD-1	14.4			
	JD-2	14.4			
	JD-3	14.4	14.34	0.439	3.07
	JD-4	14.13			
	JD-5	14.57			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	JE-1	14.2			
	JE-2	14.0			
	JE-3	14.6	14.13	1.46	10.32
	JE-4	13.53			
	JE-5	14.18			
6 p.m.	JF-1	14.6			
	JF-2	13.8			
	JF-3	15.0	14.73	1.33	9.03
	JF-4	15.09			
	JF-5	15.45			
8 p.m.	JG-1	14.6			
	JG-2	14.0			
	JG-3	13.6	13.79	0.344	2.5
	JG-4	13.62			
	JG-5	14.01			
10 p.m.	JH-1	13.83			
	JH-2	13.14			
	JH-3	13.47	13.564	1.16	8.55
	JH-4	14.16			
	JH-5	13.16			
Average for the day			14.476	1.67	11.52

INORGANIC CONTENT - Second Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	JA-1	40.44			
	JA-2	40.47			
	JA-3	40.40	39.87	2.5	6.27
	JA-4	38.30			
	JA-5	40.02			
10 a.m.	JB-1	40.39			
	JB-2	41.39			
	JB-3	41.00	41.02	1.16	2.83
	JB-4	40.66			
	JB-5	41.78			
12 noon	JC-1	39.02			
	JC-2	39.47			
	JC-3	41.51	40.57	2.41	5.93
	JC-4	40.40			
	JC-5	41.51			
2 p.m.	JD-1	41.05			
	JD-2	39.42			
	JD-3	40.49	40.41	2.47	6.14
	JD-4	41.45			
	JD-5	39.09			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations + F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	JE-1	40.23			
	JE-2	39.32			
	JE-3	40.70	40.63	1.34	3.29
	JE-4	41.51			
	JE-5	42.25			
6 p.m.	JF-1	40.98			
	JF-2	40.46			
	JF-3	40.95	41.05	0.61	1.48
	JF-4	41.49			
	JF-5	42.33			
8 p.m.	JG-1	40.98			
	JG-2	40.84			
	JG-3	40.84	41.50	1.98	4.77
	JG-4	43.01			
	JG-5	42.53			
10 p.m.	JH-1	41.04			
	JH-2	42.17			
	JH-3	41.78	42.04	2.5	5.94
	JH-4	42.96			
	JH-5	40.40			
Average for the day			40.70	0.688	1.69

SAMPLE OBSERVATIONS

Test No. 1 - Second Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES JA-1-2-3								
8.15	0	0	9.00	0	0	9.43	0	0
8.20	5	11.0	9.05	5	11.5	9.48	5	12.0
8.25	10	27.4	9.10	10	28.4	9.53	10	28.0
8.30	15	24.0	9.15	15	53.0	9.58	15	52.0
8.35	20	63.0	9.20	20	66.4	10.03	20	68.0
8.40	25	72.0	9.25	25	77.0	10.08	25	79.6
8.45	30	79.0	9.30	30	82.4	10.13	30	83.8
8.50	35	83.2	9.35	35	85.2	10.18	35	84.4
8.56	41	84.4	9.40	40	85.6	10.23	40	84.4
SAMPLES JC-1-2-3								
12.04	0	0	12.42	0	0	1.18	0	0
12.11	7	20.6	12.47	5	10.8	1.23	5	11.0
12.17	13	45.0	12.52	10	30.0	1.28	10	30.2
12.21	17	61.0	12.57	15	50.4	1.34	16	53.4
12.24	20	71.0	1.02	20	68.4	1.38	20	66.0
12.29	25	80.0	1.07	25	79.6	1.43	25	77.0
12.32	28	83.4	1.12	30	84.4	1.48	30	82.4
12.34	30	84.6	1.14	32	85.2	1.50	32	83.8
12.37	33	85.4	1.15	33	85.4	1.58	40	84.8

OBSERVATIONS (CONT'D.)

SAMPLES JD-1-2-3

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
2.04	0	0	2.44	0	0	3.20	0	0
2.09	5	12.0	2.50	6	15.4	3.25	5	12.0
2.14	10	25.6	2.54	10	30.6	3.30	10	32.6
2.19	15	48.4	2.59	15	50.0	3.35	15	52.0
2.24	20	64.0	3.04	20	68.0	3.40	20	71.0
2.29	25	77.4	3.09	25	80.8	3.45	25	81.2
2.34	30	84.0	3.14	30	85.4	3.50	30	85.2
2.39	35	85.6	3.17	33	85.6	3.53	33	85.6

SAMPLES JE-1-2-3

4.04	0	0	4.42	0	0	5.25	0	0
4.10	6	15.0	4.47	5	11.0	5.35	10	32.6
4.15	11	35.4	4.52	10	31.6	5.40	15	53.4
4.20	16	57.0	4.59	17	57.6	5.45	20	71.2
4.25	21	73.0	5.02	20	71.6	5.50	25	81.0
4.28	24	82.6	5.07	25	80.8	5.55	30	85.0
4.30	26	83.8	5.13	31	85.2	5.58	33	85.4
4.35	31	85.0	5.18	36	86.2	5.59	34	85.4
4.37	33	85.2	5.20	38	86.4	6.00	35	85.4

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES JF-1-2-3								
6.07	0	0	6.47	0	0	7.25	0	0
6.15	8	22.0	6.55	8	24.6	7.35	10	30.4
6.20	13	42.2	7.00	13	45.0	7.40	15	49.0
6.27	20	68.8	7.05	18	64.4	7.45	20	66.6
6.32	25	80.2	7.10	23	78.4	7.50	25	79.6
6.35	28	83.2	7.15	28	84.8	7.55	30	84.0
6.40	33	85.6	7.20	33	86.2	7.58	33	84.8
6.42	35	85.8	7.22	35	86.4	8.00	35	85.0
6.44	37	85.8	7.23	36	86.4	8.02	37	85.0
SAMPLES JG-1-2-3								
8.06	0	0	8.46	0	0	9.29	0	0
8.11	5	11.0	8.51	5	11.2	9.40	11	37.8
8.16	10	31.6	8.56	10	32.0	9.45	16	59.4
8.21	15	52.4	9.01	15	53.8	9.51	22	78.4
8.26	20	70.6	9.06	20	71.8	9.55	26	84.2
8.32	26	82.4	9.11	25	82.4	9.59	30	86.0
8.36	30	84.0	9.16	30	84.6	10.01	32	86.4
8.39	33	84.8	9.19	33	85.2	10.02	33	86.4

N.B. Analyses of samples not shown above were carried out at the Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 2 September 3rd, 1963.

Waste water flow 1.12 m.g.d. Thickener underflow 645 g.p.d.

Sludge pumping into thickener 24 hours Depth of sludge in thickener 6 ft.

Thickener loading 1400 gals./sq.ft/d. Specific gravity 1.048

SOLIDS CONTENT - First Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	A-1	15.8			
	A-2	18.4			
	A-3	20.6	19.79	1.23	6.2
	A-4	20.8			
	A-5	20.41			
10 a.m.	B-1	16.0			
	B-2	16.8			
	B-3	16.4	16.56	0.79	4.78
	B-4	16.55			
	B-5	17.08			
12 noon	C-1	15.8			
	C-2	15.2			
	C-3	16.4	16.56	0.79	4.78
	B-4	16.55			
	B-5	17.08			
2 p.m.	D-1	16.0			
	D-2	14.8			
	D-3	15.8	15.30	2.3	15.0
	D-4	14.68			
	D-5	16.11			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	E-1	15.8			
	E-2	15.0			
	E-3	15.8	15.70	1.02	6.5
	E-4	16.16			
	E-5	15.92			
6 p.m.	F-1	15.2			
	F-2	15.4			
	F-3	15.4	15.44	0.55	3.58
	F-4	15.69			
	F-5	15.09			
8 p.m.	G-1	15.2			
	G-2	15.0			
	G-3	15.2	15.16	0.36	2.36
	G-4	15.29			
	G-5	14.92			
10 p.m.	H-1	14.25			
	H-2	14.74			
	H-3	14.86	14.85	0.47	3.2
	H-4	14.90			
	H-5	15.67			
Average for the day			15.60	1.10	7.05

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	A-1	40.88			
	A-2	43.28			
	A-3	42.19	42.44	2.75	6.5
	A-4	41.93			
	A-5	44.80			
10 a.m.	B-1	40.49			
	B-2	39.79			
	B-3	40.29	40.61	0.79	1.95
	B-4	41.42			
	B-5	42.72			
12 noon	C-1	39.15			
	C-2	39.74			
	C-3	39.55	40.21	1.53	3.80
	C-4	41.49			
	C-5	42.06			
2 p.m.	D-1	39.20			
	D-2	37.48			
	D-3	39.51	39.18	2.53	6.45
	D-4	39.79			
	D-5	41.49			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	E-1	39.45			
	E-2	40.33			
	E-3	40.64	40.67	0.35	0.86
	E-4	41.13			
	E-5	41.55			
6 p.m.	F-1	40.40			
	F-2	40.75			
	F-3	40.55	40.45	1.54	3.8
	F-4	39.74			
	F-5	41.46			
8 p.m.	G-1	39.27			
	G-2	38.41			
	G-3	35.17	37.86	6.8	18.0
	G-4	40.73			
	G-5	38.80			
10 p.m.	H-1	41.07			
	H-2	41.89			
	H-3	43.19	42.40	1.28	3.02
	H-4	42.38			
	H-5	41.08			
Average for the day			40.02	3.37	8.4

SAMPLE OBSERVATIONS

Test No. 2 - First Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	A-1-2-3							
8.17	0	0	8.55	0	0	9.36	0	0
8.22	5	10.4	9.00	5	10.4	9.41	5	11.2
8.27	10	29.6	9.05	10	29.6	9.46	10	31.4
8.32	15	48.2	9.10	15	47.2	9.51	15	49.0
8.37	20	65.0	9.15	20	63.0	9.56	20	64.4
8.42	25	76.0	9.20	25	75.2	10.01	25	75.4
8.47	30	81.8	9.25	30	79.6	10.06	30	78.8
8.49	32	83.4	9.28	33	81.0	10.08	32	79.4
8.51	34	84.2	9.30	35	81.6	10.10	34	79.4
SAMPLES	B-1-2-3							
10.15	0	0	10.54	0	0	11.33	0	0
10.20	5	11.0	11.00	6	15.0	11.39	6	15.6
10.25	10	30.4	11.05	11	36.6	11.43	10	31.2
10.30	15	49.2	11.10	16	56.4	11.48	15	50.6
10.35	20	67.0	11.15	21	72.2	11.53	20	68.0
10.40	25	79.4	11.21	27	81.4	11.58	25	79.0
10.45	30	83.4	11.24	30	83.0	12.03	30	83.4
10.47	32	84.2	11.26	32	83.6	12.05	32	83.8
10.49	34	84.2	11.28	34	83.8	12.07	34	84.0

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES C-1-2-3								
12.14	0	0	12.51	0	0	1.27	0	0
12.19	5	11.6	12.56	5	11.2	1.32	5	11.4
12.24	10	33.6	1.01	10	33.0	1.37	10	31.2
12.29	15	55.2	1.06	15	52.0	1.42	15	51.8
12.34	20	72.6	1.11	20	68.8	1.47	20	68.8
12.39	25	82.8	1.17	26	82.0	1.52	25	79.6
12.41	27	84.2	1.21	30	85.0	1.55	28	82.8
12.44	30	85.0	1.23	32	85.2	1.57	30	84.0
12.46	32	85.2	1.25	34	85.4	1.59	32	84.2
SAMPLES D-1-2-3								
2.06	0	0	2.47	0	0	3.25	0	0
2.11	5	11.0	2.52	5	11.0	3.30	5	11.0
2.16	10	31.8	2.57	10	31.6	3.38	13	44.4
2.21	15	51.4	3.02	15	52.8	3.40	15	52.2
2.26	20	68.0	3.07	20	70.0	3.45	20	69.0
2.31	25	79.6	3.12	25	81.2	3.50	25	81.0
2.36	30	83.6	3.14	27	83.6	3.53	28	83.8
2.39	33	84.2	3.16	29	84.8	3.55	30	84.4
2.41	35	84.4	3.20	33	85.2	3.58	33	84.4

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES E-1-2-3								
4.05	0	0	4.42	0	0	5.19	0	0
4.11	6	14.8	4.47	5	11.2	5.24	5	11.0
4.15	10	31.6	4.52	10	33.0	5.29	10	32.8
4.20	15	52.6	4.57	15	53.0	5.34	15	52.0
4.25	20	69.6	5.03	21	73.0	5.39	20	69.4
4.30	25	80.2	5.08	26	83.0	5.45	26	82.6
4.33	28	83.2	5.10	28	84.6	5.47	28	84.0
4.35	30	84.2	5.12	30	85.4	5.49	30	84.6
4.38	33	84.6	5.15	33	85.6	5.51	32	84.6
SAMPLES D-1-2-3								
6.09	0	0	6.48	0	0	7.24	0	0
6.14	5	11.0	6.53	5	11.6	7.29	5	11.0
6.19	10	32.4	6.58	10	32.8	7.34	10	31.6
6.26	17	59.2	7.03	15	53.6	7.39	15	52.8
6.31	22	73.4	7.08	20	71.6	7.44	20	71.6
6.34	25	79.6	7.13	25	83.0	7.49	25	82.0
6.37	28	83.6	7.15	27	84.8	7.51	27	84.0
6.39	30	85.0	7.18	30	85.0	7.54	30	85.0
6.41	32	85.2	7.20	32	85.0	7.57	33	85.0

N.B. Analyses of samples not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 2 September 4th, 1963.

Waste water flow 1.21 m.g.d. Thickener underflow 640 g.p.d.

Sludge pumping into thickener 24 hours Depth of sludge in thickener 6 ft.

Thickener loading 1400 gals./sq.ft/d. Specific gravity 1.046

SOLIDS CONTENT - Second Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	KA-1	16.2			
	KA-2	16.2			
	KA-3	17.7	16.87	0.86	5.10
	KA-4	17.11			
	KA-5	17.22			
10 a.m.	KB-1	15.82			
	KB-2	16.0			
	KB-3	17.00	16.39	1.26	7.70
	KB-4	16.11			
	KB-5	16.00			
12 noon	KC-1	16.4			
	KC-2	14.8			
	KC-3	15.6	15.65	1.48	9.50
	KC-4	16.28			
	KC-5	16.05			
2 p.m.	KD-1	15.20			
	KD-2	14.80			
	KD-3	15.60	15.18	1.12	7.35
	KD-4	15.13			
	KD-5	15.48			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	KE-1	15.4			
	KE-2	15.6			
	KE-3	16.2	16.09	0.39	2.40
	KE-4	16.51			
	KE-5	16.37			
6 p.m.	KF-1	15.6			
	KF-2	15.4			
	KF-3	15.8	15.57	0.79	5.05
	KF-4	15.33			
	KF-5	15.72			
8 p.m.	KG-1	15.8			
	KG-2	15.8			
	KG-3	16.2	15.96	0.64	4.03
	KG-4	15.78			
	KG-5	16.00			
10 p.m.	KH-1	16.17			
	KH-2	15.10			
	KH-3	15.63	15.59	0.96	6.18
	KH-4	15.86			
	KH-5	15.72			
Average for the day			15.69	0.96	6.15

INORGANIC CONTENT - Second Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	KA-1	41.01			
	KA-2	40.18			
	KA-3	42.75	42.16	2.54	6.02
	KA-4	43.54			
	KA-5	42.20			
10 a.m.	KB-1	41.08			
	KB-2	40.14			
	KB-3	40.79	41.04	1.43	3.48
	KB-4	41.53			
	KB-5	44.17			
12 noon	KC-1	40.35			
	KC-2	40.83			
	KC-3	41.00	41.02	0.68	1.65
	KC-4	41.46			
	KC-5	40.83			
2 p.m.	KD-1	39.89			
	KD-2	40.19			
	KD-3	40.43	40.75	0.66	1.61
	KD-4	41.64			
	KD-5	42.26			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	KE-1	40.01			
	KE-2	40.08			
	KE-3	40.73	40.93	0.96	2.36
6 p.m.	KF-1	40.09			
	KF-2	41.01			
	KF-3	41.25	40.98	0.95	2.32
	KF-4	40.67			
	KF-5	41.34			
8 p.m.	KG-1	39.89			
	KG-2	39.74			
	KG-3	40.90	40.77	0.82	2.01
	KG-4	41.52			
	KG-5	42.04			
10 p.m.	KH-1	41.22			
	KH-2	41.28			
	KH-3	41.35	41.16	0.74	1.79
	KH-4	40.72			
	KH-5	41.16			
Average for the day			40.91	0.56	1.38

SAMPLE OBSERVATIONS

Test No. 2 - Second Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES KA-1-2-3								
8.05	0	0	8.48	0	0	9.33	0	0
8.10	5	10.4	8.53	5	10.2	9.38	5	10.2
8.15	10	29.0	8.58	10	29.0	9,43	10	29.4
8.21	16	52.4	9.03	15	49.6	9.48	15	49.8
8.25	20	66.0	9.08	20	62.8	9.53	20	68.0
8.30	25	77.0	9.14	26	77.4	9.58	25	79.6
8.35	30	82.4	9.18	30	81.6	10.01	28	82.2
8.37	32	83.6	9.20	32	83.4	10.06	33	82.8
8.40	35	84.4	9.24	36	85.2	10.10	37	82.8
8.45	40	84.8	9.26	38	86.0	10.12	39	82.8
SAMPLES KC-1-2-3								
12.06	0	0	12.46	0	0	1.23	0	0
12.11	5	10.6	12.51	5	11.0	1.29	6	14.0
12.16	10	30.6	12.56	10	32.0	1.33	10	29.2
12.21	15	50.2	1.01	15	52.8	1.38	15	49.0
12.26	20	65.8	1.06	20	70.6	1.43	20	65.2
12.31	25	77.6	1.11	25	81.6	1.48	25	78.2
12.35	29	82.0	1.14	28	84.6	1.53	30	83.2
12.40	34	83.6	1.17	31	85.2	1.55	32	84.0
12.43	37	83.8	1.20	34	85.2	1.57	34	84.6

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES KD-1-2-3								
2.07	0	0	2.49	0	0	3.26	0	0
2.12	5	10.0	2.54	5	10.4	3.31	5	11.6
2.17	10	29.8	2.59	10	30.6	3.36	10	33.4
2.22	15	47.8	3.04	15	52.6	3.41	15	54.8
2.27	20	64.4	3.11	22	77.2	3.46	20	71.4
2.32	25	77.0	3.17	28	85.0	3.51	25	79.6
2.35	28	81.2	3.18	29	85.2	3.56	30	83.0
2.38	31	83.6	3.21	32	85.2	3.58	32	83.8
3.41	34	84.8	3.23	34	85.2	4.01	35	84.4
SAMPLES KE-1-2-3								
4.07	0	0	4.46	0	0	5.23	0	0
4.12	5	11.2	4.51	5	12.4	5.41	18	60.0
4.17	10	32.0	4.57	11	39.0	5.43	20	67.0
4.22	15	52.4	5.01	15	54.6	5.47	24	77.6
4.27	20	69.0	5.07	21	74.4	5.49	26	80.4
4.32	25	79.4	5.11	25	81.4	5.51	28	82.2
4.37	30	83.8	5.14	28	84.2	5.53	30	83.0
4.39	32	84.6	5.16	30	84.8	5.56	33	84.0
4.42	35	85.0	5.19	33	84.8	5.58	35	84.0

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	KF-1-2-3							
6.06	0	0	6.40	0	0	7.21	0	0
6.11	5	11.4	6.47	7	20.6	7.26	5	11.8
6.16	10	33.0	6.50	10	33.0	7.34	13	46.0
6.22	20	71.4	7.05	25	69.0	7.41	20	70.0
6.32	26	82.4	7.10	30	79.8	7.47	26	79.4
6.36	30	84.4	7.15	35	84.2	7.51	30	82.0
6.38	32	84.4	7.18	38	84.8	7.55	34	84.2
SAMPLES	KG-1-2-3							
8.05	0	0	8.45	0	0	9.24	0	0
8.10	5	12.0	8.50	5	13.0	9.29	5	11.6
8.15	10	33.8	8.55	10	34.2	9.35	11	37.8
8.20	15	55.2	9.00	15	56.2	9.39	15	53.2
8.28	23	77.8	9.05	20	72.0	9.44	20	70.2
8.30	25	79.8	9.10	25	79.6	9.50	26	79.4
8.34	29	82.4	9.15	30	83.4	9.55	31	82.8
8.39	34	84.2	9.19	34	84.4	10.00	36	83.6
8.42	37	84.2	9.20	35	84.4	10.03	39	83.8

N.B. Analyses of samples not shown above were carried out at the Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 3 October 10th, 1963.

Waste water flow 1.13 m.g.d. Thickener underflow 820 g.p.d.

Sludge pumping into thickener 12 hours Depth of sludge in thickener 3 ft.

Thickener loading 700 gals./sq.ft/d. Specific gravity 1.035

SOLIDS CONTENT - First Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations + F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	A-1	12.35			
	A-2	12.60			
	A-3	12.60	12.45	0.40	3.24
	A-4	12.16			
	A-5	12.20			
10 a.m.	B-1	11.20			
	B-2	11.30			
	B-3	11.00	11.03	0.37	3.33
	B-4	10.76			
	B-5	11.01			
12 noon	C-1	10.80			
	C-2	11.00			
	C-3	12.00	11.80	0.60	5.10
	C-4	12.33			
	C-5	12.62			
2 p.m.	D-1	11.60			
	D-2	11.20			
	D-3	11.60	11.51	0.51	4.45
	D-4	11.61			
	D-5	11.64			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	E-1	11.00			
	E-2	11.60			
	E-3	11.00	11.14	0.77	6.90
	E-4	10.93			
	E-5	11.10			
6 p.m.	F-1	11.20			
	F-2	11.00			
	F-3	11.60	11.24	0.92	8.23
	F-4	10.82			
	F-5	11.73			
8 p.m.	G-1	10.80			
	G-2	11.20			
	G-3	11.00	10.96	0.37	3.40
	G-4	10.75			
	G-5	10.73			
10 p.m.	H-1	10.87			
	H-2	10.68			
	H-3	11.02	10.99	0.33	3.02
	H-4	11.18			
	H-5	11.47			
Average for the day			11.36	1.03	9.10

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	A-1	44.95			
	A-2	43.47			
	A-3	41.97	42.71	1.27	2.97
	A-4	42.43			
	A-5	43.81			
10 a.m.	B-1	44.16			
	B-2	41.78			
	B-3	43.31	42.57	4.00	9.40
	B-4	41.44			
	B-5	44.18			
12 noon	C-1	41.80			
	C-2	40.99			
	C-3	42.48	42.27	1.62	3.84
	C-4	42.85			
	C-5	44.34			
2 p.m.	D-1	41.64			
	D-2	42.52			
	D-3	41.54	42.48	2.94	6.94
	D-4	43.93			
	D-5	42.97			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations + F.	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	E-1	43.22			
	E-2	43.10			
	E-3	42.63	42.81	0.57	1.33
	E-4	42.55			
	E-5	43.31			
6 p.m.	F-1	42.02			
	F-2	41.67			
	F-3	42.51	42.50	0.72	1.68
	F-4	43.17			
	F-5	43.54			
8 p.m.	G-1	42.42			
	G-2	43.00			
	G-3	43.19	42.91	1.51	3.51
	G-4	42.31			
	G-5	43.78			
10 p.m.	H-1	42.53			
	H-2	42.76			
	H-3	43.05	42.64	0.84	1.97
	H-4	42.10			
	H-5	41.90			
Average for the day			42.59	0.52	1.21

SAMPLE OBSERVATIONS

Test No. 3 - First Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES A-1-2-3								
8.37	0	0	9.23	0	0	11.21	0	0
8.54	17	44.6	9.28	5	7.8	11.31	10	25.4
9.01	24	65.6	9.33	10	23.6	11.36	15	42.2
9.04	27	73.6	9.38	15	40.4	11.41	20	59.0
9.07	30	79.8	9.43	20	56.2	11.46	25	74.0
9.10	33	83.4	9.48	25	70.8	11.49	28	80.4
9.12	35	85.0	9.53	30	81.4	11.51	30	83.4
9.14	37	86.4	9.57	34	85.8	11.55	34	87.6
9.16	39	87.0	9.59	36	86.8	11.57	36	88.6
9.18	41	87.4	10.03	40	87.4	11.59	38	89.0
SAMPLES C-1-2-3								
12.04	0	0	12.47	0	0	1.25	0	0
12.14	10	24.8	1.12	25	74.6	1.35	10	25.4
12.19	15	41.2	1.17	30	86.0	1.45	20	59.0
12.24	20	60.0	1.19	32	88.2	1.50	25	74.0
12.29	25	75.6	1.21	34	89.0	1.55	30	84.2
12.36	32	86.6	1.23	36	89.0	2.00	35	87.8
12.40	36	88.8	1.25	38	89.0	2.02	37	88.0
12.44	40	89.2				2.04	39	88.0

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	D-1-2-3							
2.06	0	0	2.45	0	0	3.24	0	0
2.16	10	26.4	2.50	5	9.8	3.48	24	79.0
2.21	15	44.2	2.55	10	27.2	3.50	26	82.6
2.26	20	62.8	3.05	20	62.6	3.52	28	85.6
2.31	25	78.0	3.10	25	77.2	3.54	30	87.4
2.36	30	86.2	3.15	30	86.6	3.56	32	88.4
2.39	33	88.2	3.19	34	88.8	3.58	34	88.4
2.42	36	89.0	3.21	36	89.0	4.00	36	88.4
SAMPLES	E-1-2-3							
4.05	0	0	4.41	0	0	5.16	0	0
4.20	15	52.6	4.51	10	31.2	5.21	5	9.6
4.27	22	77.4	4.56	15	51.4	5.31	15	49.8
4.30	25	84.0	5.01	20	70.0	5.36	20	69.2
4.32	27	87.0	5.03	22	76.2	5.39	23	78.2
4.34	29	88.2	5.08	27	86.2	5.43	27	85.6
4.36	31	89.0	5.12	31	88.4	5.47	31	88.6
4.38	33	89.0	5.14	33	88.4	5.49	33	89.0

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES F-1-2-3								
6.05	0	0	6.41	0	0	7.20	0	0
6.10	5	10.0	6.46	5	9.2	7.25	5	10.2
6.34	29	88.0	6.51	10	27.8	7.30	10	30.0
6.35	30	88.4	7.01	20	66.6	7.35	15	48.6
6.36	31	88.6	7.06	25	80.2	7.40	20	66.2
6.37	32	88.8	7.11	30	87.4	7.45	25	80.2
6.38	33	88.8	7.14	33	88.8	7.50	30	86.6
6.40	35	88.8	7.16	35	89.0	7.53	33	88.0
6.42	37	88.8	7.18	37	89.0	7.56	36	88.4
SAMPLES G-1-2								
8.05	0	0	8.44	0	0			
8.20	15	44.2	9.04	25	84.2			
8.25	20	64.0	9.12	28	87.2			
8.30	25	79.6	9.13	29	88.0			
8.33	28	85.8	9.14	30	88.4			
8.35	30	88.2	9.17	33	88.8			
8.39	34	89.4	9.20	36	88.8			

N.B. Analyses of sludge samples not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 3 October 11th, 1963.

Waste water flow 1.15 m.g.d. Thickener underflow 930 g.p.d.

Sludge pumping into thickener 12 hours Depth of sludge in thickener 3 ft.

Thickener loading 700 gals./sq.ft/d. Specific gravity 1.031

SOLIDS CONTENT - Second Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %
8 a.m.	J-1	11.20			
	J-2	11.40			
	J-3	11.60	11.55	0.14	1.21
	J-4	11.70			
	J-5	11.58			
10 a.m.	K-1	11.31			
	K-2	10.53			
	K-3	10.65	10.89	1.04	9.55
	K-4	11.49			
	K-5	10.95			
12 noon	L-1	10.80			
	L-2	11.00			
	L-3	11.20	11.03	0.266	2.42
	L-4	10.98			
	L-5	10.57			
2 p.m.	M-1	11.20			
	M-2	11.20			
	M-3	11.60	11.38	0.646	5.69
	M-4	11.23			
	M-5	11.57			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	N-1	13.2			
	N-2	13.6			
	N-3	13.8	13.90	0.6	4.32
	N-4	14.44			
	N-5	14.16			
6 p.m.	P-1	13.4			
	P-2	13.6			
	P-3	14.8	13.93	2.02	14.50
	P-4	13.19			
	P-5	13.40			
8 p.m.	Q-1	13.85			
	Q-2	14.17			
	Q-3	14.81	14.35	1.13	7.88
	Q-4	13.96			
	Q-5	14.33			
10 p.m.	R-1	14.67			
	R-2	14.09			
	R-3	14.14	14.25	0.92	6.45
	R-4	14.61			
	R-5	13.68			
Average for the day			14.106	0.546	3.87

INORGANIC CONTENT - Second Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	J-1	28.75			
	J-2	28.61			
	J-3	29.43	28.874	1.84	6.35
	J-4	28.23			
	J-5	29.29			
10 a.m.	K-1	30.14			
	K-2	28.42			
	K-3	30.36	29.986	2.60	8.68
	K-4	31.08			
	K-5	29.47			
12 noon	L-1	29.76			
	L-2	29.85			
	L-3	27.76	28.796	2.61	9.06
	L-4	29.12			
	L-5	28.54			
2 p.m.	M-1	28.58			
	M-2	28.62			
	M-3	28.68	28.933	1.68	5.80
	M-4	29.86			
	M-5	28.35			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	N-1	29.71			
	N-2	29.88			
	N-3	30.76	30.468	1.22	5.65
	N-4	31.22			
	N-5	28.82			
6 p.m.	P-1	28.84			
	P-2	28.91			
	P-3	29.04	28.614	1.79	6.26
	P-4	27.64			
	P-5	28.94			
8 p.m.	Q-1	28.46			
	Q-2	28.35			
	Q-3	29.95	29.145	1.74	5.97
	Q-4	29.29			
	Q-5	27.59			
10 p.m.	R-1	30.18			
	R-2	28.56			
	R-3	30.42	29.719	2.49	8.4
	R-4	29.76			
	R-5	29.52			
Average for the day			29.353	2.09	7.12

SAMPLE OBSERVATIONS

Test No. 3 - Second Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	J-1-2-3							
8.05	0	0	8.45	0	0	9.25	0	0
8.15	10	28.8	8.50	5	9.8	9.30	5	9.6
8.20	15	48.4	8.56	11	32.4	9.50	25	78.0
8.25	20	66.8	9.01	16	52.0	9.54	29	85.4
8.30	25	81.0	9.05	20	66.8	9.56	31	86.8
8.32	27	84.2	9.10	25	80.8	10.00	35	88.2
8.35	30	87.6	9.13	28	85.6	10.02	37	88.4
8.37	32	88.6	9.16	31	88.4			
8.39	34	89.0	9.19	34	89.0			
SAMPLES	K-3		SAMPLES	L-1-3				
11.16	0	0	12.04	0	0	1.30	0	0
11.21	5	9.4	12.09	5	9.0	1.40	10	27.8
11.26	10	27.8	12.14	10	27.6	1.51	21	71.8
11.31	15	46.4	12.19	15	47.8	1.53	23	77.6
11.41	25	79.2	12.24	20	67.2	1.55	25	82.2
11.46	30	87.0	12.29	25	83.2	1.57	27	85.4
11.48	32	88.2	12.33	29	87.8	2.00	30	88.2
11.50	34	89.0	12.35	31	89.0	2.02	32	88.8
11.52	36	89.0	12.38	34	89.2	2.04	34	88.8

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	M-1-2-3							
2.09	0	0	2.48	0	0	3.26	0	0
2.14	5	10.4	2.58	10	29.2	3.31	5	10.2
2.19	10	39.6	3.03	15	49.00	3.36	10	30.4
2.24	15	49.6	3.08	20	68.4	3.41	15	50.0
2.29	20	68.6	3.13	25	83.2	3.46	20	67.6
2.35	26	78.8	3.17	39	87.4	3.51	25	80.6
2.38	29	84.8	3.19	31	88.4	3.54	28	85.4
2.43	34	88.8	3.23	35	88.8	3.57	31	87.6
2.45	36	88.8	3.25	37	88.8	3.59	33	88.4
SAMPLES	N-1-2-3							
4.03	0	0	4.39	0	0	5.17	0	0
4.08	5	10.0	4.44	5	10.0	5.22	5	10.2
4.13	10	29.6	4.49	10	30.0	5.27	10	29.4
4.18	15	29.2	4.54	15	50.2	5.28	11	34.6
4.23	20	69.0	4.59	20	70.4	5.38	21	73.6
4.28	25	83.4	5.04	25	84.4	5.42	25	83.4
4.32	29	87.6	5.07	28	87.4	5.46	29	87.6
4.34	31	88.4	5.09	30	88.4	5.49	32	88.8
4.36	33	88.6	5.14	35	89.2	5.51	34	89.4

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES P-1-2-3								
6.05	0	0	6.43	0	0	7.22	0	0
6.10	5	10.2	6.53	10	29.6	7.32	10	31.8
6.15	10	30.6	6.58	15	48.6	7.37	15	51.4
6.20	15	49.6	7.03	20	67.6	7.42	20	69.6
6.25	20	68.6	7.08	25	82.6	7.45	23	78.4
6.30	25	82.6	7.14	31	87.4	7.47	25	83.0
6.33	28	86.0	7.16	33	88.6	7.52	30	88.0
6.35	30	87.6	7.18	35	89.0	7.55	33	89.2
6.38	33	89.2	7.20	37	89.0	7.58	36	89.2
SAMPLES K-1 & L-2								
10.04	0	0	12.45	0	0			
11.16	72	89.0	12.50	5	10.4			
			1.25	40	89.0			

N.B. Analyses of samples not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 4 October 24th, 1963.

Waste water flow 0.91 m.g.d. Thickener underflow 615 g.p.d.

Pumping of sludge into thickener 12 hours Depth of sludge in thickener 6 ft.

Thickener loading 700 gals./sq.ft/d. Specific gravity 1.029

SOLIDS CONTENT - Second Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	J-1	15.4			
	J-2	14.6			
	J-3	15.4	15.29	1.08	7.08
	J-4	15.77			
	J-5	15.26			
10 a.m.	K-1	15.0			
	K-2	15.0			
	K-3	14.2	14.78	1.11	7.52
	K-4	15.12			
	K-5	15.73			
12 noon	L-1	14.0			
	L-2	14.2			
	L-3	14.0	14.04	0.29	2.03
	L-4	13.94			
	L-5	14.14			
2 p.m.	M-1	13.6			
	M-2	13.6			
	M-3	13.4	14.24	3.95	27.8
	M-4	16.48			
	M-5	13.56			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Contents %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	N-1	32.19			
	N-2	32.78			
	N-3	34.14	35.89	5.03	14.0
	N-4	41.14			
	N-5	41.54			
6 p.m.	P-1	32.85			
	P-2	33.69			
	P-3	32.17	35.20	8.22	23.4
	P-4	40.76			
	P-5	39.58			
8 p.m.	Q-1	40.36			
	Q-2	40.71			
	Q-3	41.67	41.33	0.745	1.80
	Q-4	41.75			
	Q-5	41.11			
10 p.m.	R-1	39.84			
	R-2	41.98			
	R-3	40.74	40.98	2.17	5.3
	R-4	40.54			
	R-5	41.63			
Average for the day			37.10	4.32	11.66

INORGANIC CONTENT - Second Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	J-1	41.49			
	J-2	42.66			
	J-3	42.20	42.20	0.96	2.27
	J-4	42.04			
	J-5	41.74			
10 a.m.	K-1	41.60			
	K-2	42.70			
	K-3	43.17	42.41	1.90	4.5
	K-4	41.34			
	K-5	41.78			
12 noon	L-1	42.28			
	L-2	41.44			
	L-3	34.24	38.42	10.5	27.4
	L-4	42.22			
	L-5	42.38			
2 p.m.	M-1	33.16			
	M-2	34.80			
	M-3	33.23	36.49	8.9	23.4
	M-4	42.63			
	M-5	41.63			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	N-1	11.40			
	N-2	10.80			
	N-3	10.60	10.67	0.866	8.12
	N-4	10.63			
	N-5	10.77			
6 p.m.	P-1	10.80			
	P-2	11.00			
	P-3	10.80	10.83	0.283	2.61
	P-4	10.69			
	P-5	10.85			
8 p.m.	Q-1	10.58			
	Q-2	11.55			
	Q-3	10.85	11.00	1.045	9.50
	Q-4	10.85			
	Q-5	10.78			
10 p.m.	R-1	10.465			
	R-2	11.265			
	R-3	10.435	10.57	1.071	10.05
	R-4	10.50			
	R-5	10.50			
Average for the day			10.99	0.685	6.23

SAMPLE OBSERVATIONS

Test No. 4 - Second Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	J-1-2-3							
8.07	0	0	8.50	0	0	9.32	0	0
8.13	6	12.0	8.56	6	12.4	9.37	5	11.0
8.17	10	26.0	9.00	10	27.0	9.42	10	30.2
8.22	15	45.6	9.05	15	46.0	9.47	15	50.0
8.27	20	64.2	9.10	20	64.6	9.52	20	67.8
8.32	25	75.2	9.16	26	78.8	9.57	25	77.8
8.36	29	79.8	9.18	28	81.0	10.02	30	82.8
8.40	33	83.0	9.21	31	83.8	10.05	33	84.4
8.43	36	84.6	9.24	34	85.4	10.07	35	84.6
SAMPLES	K-1-2-3							
10.10	0	0	10.50	0	0	11.30	0	0
10.15	5	10.6	11.0	10	28.6	11.35	5	10.6
10.20	10	31.2	11.05	15	48.4	11.40	10	27.0
10.25	15	51.4	11.10	20	66.6	11.45	15	51.0
10.30	20	69.4	11.15	25	77.4	11.50	20	68.2
10.35	25	79.8	11.17	27	80.2	11.55	25	79.4
10.38	28	83.6	11.20	30	83.0	11.58	28	83.2
10.40	30	84.8	11.22	32	84.2	12.00	30	84.8
10.42	32	85.0	11.26	34	85.0	12.03	33	85.8

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES L-1-2-3								
12.08	0	0	12.50	0	0	1.30	0	0
12.13	5	10.8	12.55	5	10.8	1.35	5	10.6
12.18	10	30.6	1.01	11	34.4	1.40	10	30.8
12.23	15	51.4	1.05	15	49.8	1.45	15	51.0
12.28	20	68.8	1.10	20	67.4	1.50	20	69.4
12.33	25	79.4	1.15	25	80.4	1.56	26	80.4
12.35	27	82.2	1.20	30	85.0	1.58	28	83.0
12.37	39	84.0	1.22	32	85.4	2.00	30	84.4
12.40	32	85.8	1.24	34	85.8	2.03	33	85.8
12.42	34	86.0	1.26	36	85.8	2.06	36	86.0
SAMPLES M-1-2-3								
2.10	0	0	2.50	0	0	3.30	0	0
2.15	5	10.2	2.55	5	10.6	3.35	5	10.8
2.20	10	30.4	3.00	10	31.6	3.40	10	32.4
2.25	15	50.8	3.05	15	52.0	3.45	15	52.8
2.30	20	69.2	3.10	20	70.2	3.50	20	71.2
2.35	25	79.8	3.15	25	81.0	3.55	25	82.4
2.40	30	85.0	3.19	29	85.2	3.59	29	86.0
2.42	32	85.8	3.23	33	86.2	4.01	31	86.4
2.44	34	86.4	3.25	35	86.4	4.04	34	86.6

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES N-1-2-3								
4.06	0	0	4.45	0	0	5.25	0	0
4.11	5	10.8	4.50	5	11.2	5.30	5	11.4
4.16	10	30.8	4.55	10	32.4	5.35	10	32.8
4.21	15	52.0	5.00	15	53.6	5.40	15	53.8
4.26	20	71.8	5.05	20	72.4	5.45	20	72.8
4.31	25	82.2	5.10	25	82.8	5.51	26	81.8
4.36	30	86.2	5.15	30	85.8	5.53	28	83.6
4.38	32	86.6	5.18	33	86.4	5.58	33	86.0
4.40	34	86.8	5.20	35	86.4	6.00	35	86.2
SAMPLES P-1-2-3								
6.06	0	0	6.50	0	0	7.30	0	0
6.11	5	10.2	6.55	5	9.4	7.35	5	11.0
6.16	10	30.2	7.00	10	29.2	7.40	10	33.2
6.21	15	51.0	7.07	17	55.8	7.45	15	54.8
6.26	20	67.6	7.10	20	65.6	7.50	20	76.4
6.31	25	78.4	7.16	26	78.2	7.55	25	79.2
6.36	30	84.2	7.20	30	83.6	8.02	32	83.8
6.39	33	85.8	7.25	35	86.2	8.04	34	84.6
6.41	35	86.6	7.28	38	86.4	8.07	37	85.2

N.B. Analyses not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST NO. 5

September 24th, 1963.

Waste water flow 1.23 m.g.d.

Thickener underflow 835 g.p.d.

Sludge pumping into thickener 24 hours

Depth of sludge in thickener 3 ft.

Thickener loading (with recirculation) 560 gals./sq.ft/d.

Specific gravity 1.024

SOLIDS CONTENT - First Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	DA-1	12.64			
	DA-2	12.13			
	DA-3	12.11	12.15	0.38	3.17
	DA-4	12.19			
	DA-5	11.83			
10 a.m.	DB-1	13.00			
	DB-2	12.00			
	DB-3	12.00	11.92	0.67	5.65
	DB-4	11.55			
	DB-5	11.50			
12 noon	DC-1	11.40			
	DC-2	11.20			
	DC-3	11.20	11.23	0.33	2.92
	DC-4	11.15			
	DC-5	11.59			
2 p.m.	DD-1	11.00			
	DD-2	11.00			
	DD-3	11.20	11.13	0.29	1.57
	DD-4	11.11			
	DD-5	11.37			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	DE-1	10.60			
	DE-2	11.00			
	DE-3	11.40	11.33	0.63	5.58
	DE-4	11.80			
	DE-5	11.10			
6 p.m.	DF-1	11.00			
	DF-2	11.20			
	DF-3	11.40	11.21	0.52	5.62
	DF-4	10.99			
	DF-5	11.24			
8 p.m.	DG-1	11.20			
	DG-2	11.20			
	DG-3	12.02	11.73	0.75	6.40
	DG-4	11.89			
	DG-5	12.10			
10 p.m.	DH-1	10.85			
	DH-2	10.91			
	DH-3	11.80	11.61	0.75	6.5
	DH-4	12.17			
	DH-5	11.68			
Average for the day			11.30	0.51	4.52

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	DA-1	32.07			
	DA-2	31.95			
	DA-3	32.56	32.13	1.06	3.31
	DA-4	31.75			
	DA-5	31.86			
10 a.m.	DB-1	31.65			
	DB-2	31.51			
	DB-3	30.71	31.80	2.66	8.36
	DB-4	33.40			
	DB-5	33.15			
12 noon	DC-1	30.92			
	DC-2	31.20			
	DC-3	31.84	31.81	0.45	1.41
	DC-4	32.33			
	DC-5	33.26			
2 p.m.	DD-1	31.84			
	DD-2	31.61			
	DD-3	32.57	32.35	1.12	3.48
	DD-4	32.54			
	DD-5	33.69			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	DE-1	32.26			
	DE-2	32.92			
	DE-3	32.18	32.96	1.73	5.26
	DE-4	33.90			
	DE-5	34.60			
6 p.m.	DF-1	33.60			
	DF-2	33.80			
	DF-3	32.52	33.50	2.10	6.27
	DF-4	34.31			
	DF-5	34.83			
8 p.m.	DG-1	33.16			
	DG-2	33.73			
	DG-3	35.29	34.51	1.73	5.00
	DG-4	34.38			
	DG-5	34.84			
10 p.m.	DH-1	34.05			
	DH-2	34.54			
	DH-3	35.20	34.41	1.26	3.66
	DH-4	33.69			
	DG-5	32.37			
Average for the day			32.72	0.56	1.70

SAMPLE OBSERVATIONS

Test No. 5 - First Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES DB-1-2-3								
10.04	0	0	10.48	0	0	11.27	0	0
10.09	5	8	10.53	5	7.8	11.32	5	8.0
10.14	10	24.2	10.58	10	24.8	11.38	11	31.2
10.19	15	42.0	11.03	15	42.8	11.42	15	38.4
10.24	20	57.2	11.08	20	59.8	11.47	20	62.4
10.30	26	72.4	11.14	26	77.6	11.52	25	75.8
10.34	30	79.6	11.16	28	81.8	11.55	28	81.2
10.36	32	82.2	11.18	30	84.8	11.57	30	83.6
10.38	34	84.0	11.20	32	86.8	11.59	32	85.4
10.40	36	85.0	11.22	34	88.0	11.01	34	86.4
10.44	40	87.0	11.23	35	88.0	11.03	36	88.0
SAMPLES DC-1-2-3								
12.08	0	0	12.50	0	0	1.32	0	0
12.13	5	9	12.55	5	8.2	1.37	5	8.0
12.19	11	27.4	1.00	10	27.6	1.42	10	26.8
12.23	15	41.0	1.05	15	42.0	1.47	15	40.2
12.28	20	56.8	1.11	21	63.0	1.52	20	60.8
12.33	25	73.0	1.15	25	74.6	1.57	25	76.6
12.38	30	84.2	1.21	31	85.0	2.02	30	84.4
12.40	32	86.2	1.23	33	86.4	2.05	33	87.0
12.42	34	87.4	1.27	37	88.4	2.09	37	88.8
12.46	38	88.6	1.30	40	88.8	2.12	40	88.8

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES DD-1-2-3								
2.15	0	0	2.55	0	0	3.33	0	0
2.21	6	13.0	3.00	5	8.2	3.38	5	8.4
2.25	10	27.6	3.05	10	27.4	3.43	10	39.4
2.30	15	45.0	3.10	15	44.0	3.48	15	46.4
2.35	20	61.4	3.15	20	61.8	3.53	20	62.4
2.40	25	76.0	3.20	25	79.2	3.58	25	76.4
2.45	30	86.2	3.24	29	86.4	4.03	30	84.6
2.47	32	87.8	3.26	31	88.2	4.05	32	86.8
2.49	34	88.6	3.28	33	88.8	4.07	34	88.2
2.53	40	89.0	3.30	35	89.0	4.09	36	88.8
SAMPLES DE-1-2-3								
4.13	0	0	4.50	0	0	5.31	0	0
4.18	5	8.4	4.55	5	8.6	5.36	5	9.4
4.23	10	27.6	5.00	10	27.8	5.41	10	26.8
4.28	15	45.0	5.05	15	45.0	5.46	15	46.0
4.33	20	61.6	5.10	20	60.8	5.52	21	68.4
4.38	25	77.0	5.17	27	81.4	5.56	25	79.8
4.43	30	87.6	5.20	30	85.6	5.59	28	86.0
4.45	32	89.0	5.23	33	87.8	6.01	30	87.4
4.47	34	89.4	5.27	37	89.0	6.05	34	88.4
4.50	37	89.4	5.30	40	89.0	6.09	38	88.6

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES DF-1-2-3								
6.14	0	0	6.53	0	0	7.33	0	0
6.19	5	8.4	6.58	5	8.6	7.38	5	8.6
6.24	10	24.0	7.03	10	24.0	7.43	10	32.8
6.29	15	48.0	7.08	15	48.2	7.48	15	49.8
6.34	20	59.0	7.13	20	66.6	7.53	20	66.4
6.39	25	75.4	7.18	25	80.8	7.58	25	80.0
6.44	30	86.4	7.23	30	88.0	8.03	30	86.4
6.46	32	88.2	7.26	32	88.6	8.05	32	87.6
6.48	34	88.8	7.28	34	88.8	8.07	34	88.4
6.50	36	89.0	7.30	36	88.8	8.10	37	88.6
SAMPLES DG-1-2-3								
8.12	0	0	8.50	0	0	9.35	0	0
8.19	7	17.2	8.55	5	9.4	9.40	5	9.0
8.22	10	28.6	9.00	10	28.0	9.45	10	28.2
8.27	15	46.8	9.05	15	47.4	9.50	15	47.2
8.32	20	65.8	9.10	20	64.8	9.55	20	64.4
8.37	25	81.4	9.15	25	79.0	10.00	25	78.8
8.42	30	88.4	9.20	30	87.0	10.05	30	86.6
8.43	31	88.6	9.22	32	88.2	10.07	32	87.2
8.44	32	88.8	9.26	36	89.0	10.09	34	87.8
8.45	33	88.8	9.29	39	89.0	10.11	36	87.98

N.B. Analyses of sludge samples not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST NO. 6 September 29th, 1963.

Waste water flow 139 m.g.d. Thickener underflow 635 g.p.d.

Sludge pumping into thickener 24 hours Depth of sludge in thickener 6 ft.

Thickener loading (with recirculation) 560 gals./sq.ft/d. Specific gravity 1.059

SOLIDS CONTENT - First Day

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
8 a.m.	KA-1	15.80			
	KA-2	16.65			
	KA-3	15.68	16.13	1.39	8.65
	KA-4	16.26			
	KA-5	16.49			
10 a.m.	KB-1	14.40			
	KB-2	14.60			
	KB-3	15.00	14.94	0.20	1.32
	KB-4	15.18			
	KB-5	15.53			
12 noon	KC-1	14.20			
	KC-2	14.00			
	KC-3	13.80	13.98	0.37	2.66
	KC-4	14.12			
	KC-5	14.18			
2 p.m.	KC-1	12.60			
	KD-2	12.40			
	KD-3	13.80	13.33	1.30	9.7
	KD-4	13.64			
	KD-5	13.68			

OBSERVATIONS (CONT'D.)

Time	Mark	Solids Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	KE-1	12.60			
	KE-2	12.00			
	KE-3	13.40	13.06	1.45	11.10
	KE-4	13.45			
	KE-5	14.18			
6 p.m.	KF-1	13.0			
	KF-2	12.4			
	KF-3	13.20	13.16	0.84	6.38
	KF-4	13.64			
	KF-5	14.21			
8 p.m.	KG-1	12.80			
	KG-2	12.80			
	KG-3	13.09	13.04	0.30	2.29
	KG-4	13.15			
	KG-5	13.57			
10 p.m.	KH-1	13.54			
	KH-2	12.52			
	KH-3	13.17	13.02	1.03	7.86
	KH-4	13.51			
	KH-5	13.34			
Average for the day			13.43	0.285	2.12

INORGANIC CONTENT - First Day

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %
8 a.m.	KA-1	33.86			
	KA-2	33.08			
	KA-3	34.17	33.77	1.56	4.62
	KA-4	33.68			
	KA-5	34.31			
10 a.m.	KB-1	34.14			
	KB-2	33.71			
	KB-3	32.53	32.99	0.91	2.76
	KB-4	32.59			
	KB-5	33.28			
12 noon	KC-1	33.48			
	KC-2	33.20			
	KC-3	32.71	33.23	1.42	4.28
	KC-4	33.92			
	KC-5	33.36			
2 p.m.	KD-1	32.13			
	KD-2	32.31			
	KD-3	32.64	32.43	1.32	4.06
	KD-4	32.04			
	KD-5	33.53			

OBSERVATIONS (CONT'D.)

Time	Mark	Inorganic Content %	Mean M %	Standard Fluctuations ± F	Coefficient of Fluctuations C = ± F/M %.
4 p.m.	KE-1	32.09			
	KE-2	32.31			
	KE-3	32.73	32.70	0.61	1.86
	KE-4	33.20			
	KE-5	32.64			
6 p.m.	KF-1	33.38			
	KF-2	33.01			
	KF-3	33.80	33.48	2.10	6.25
	KF-4	34.11			
	KF-5	31.01			
8 p.m.	KG-1	33.13			
	KG-2	32.80			
	KG-3	33.13	33.19	0.92	2.77
	KG-4	33.33			
	KG-5	33.55			
10 p.m.	KH-1	32.96			
	KH-2	32.09			
	KH-3	32.51	32.68	0.85	2.62
	KH-4	33.28			
	KH-5	33.41			
Average for the day			32.87	1.02	3.10

SAMPLE OBSERVATIONS

Test No. 6 - First Day

Solids Content - Using the Cenco Moisture Balance

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES KA-1-2-3								
8.20	0	0	9.00	0	0	9.38	0	0
8.26	6	11.4	9.05	5	10.0	9.43	5	9.2
8.30	10	26.2	9.10	10	26.4	9.48	10	25.8
8.35	15	44.2	9.15	15	43.8	9.53	15	44.0
8.40	20	61.6	9.20	20	61.8	9.58	20	61.8
8.45	25	74.0	9.25	25	75.0	10.03	25	74.6
8.50	30	81.6	9.30	30	81.8	10.08	30	81.8
8.53	33	83.4	9.33	33	82.8	10.11	33	83.6
8.55	35	84.2	9.35	35	83.35	10.12	34	84.32
SAMPLES KB-1-2-3								
10.14	0	0	10.52	0	0	11.32	0	0
10.20	6	11.4	10.57	5	9.4	11.37	5	8.8
10.24	10	25.2	11.02	10	28.0	11.42	10	27.4
10.29	15	43.6	11.07	15	47.2	11.47	15	46.6
10.34	20	61.6	11.12	20	62.8	11.52	20	64.6
10.39	25	75.0	11.17	25	75.8	11.57	25	77.6
10.44	30	83.0	11.22	30	81.8	12.02	30	82.8
10.47	33	84.8	11.26	34	84.6	12.04	32	84.6
10.49	35	85.6	11.28	36	85.4	12.06	34	85.0

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	KC-1-2-3							
12.08	0	0	12.49	0	0	1.29	0	0
12.13	5	9.0	12.54	5	9.2	1.34	5	9.0
12.18	10	27.6	12.59	10	27.8	1.39	10	25.8
12.23	15	46.4	1.04	15	47.2	1.45	16	46.6
12.28	20	63.6	1.09	20	64.4	1.49	20	61.4
12.33	25	76.4	1.14	25	73.0	1.54	25	75.8
12.38	30	82.6	1.19	30	82.8	2.01	32	85.8
12.42	34	85.6	1.21	32	84.8	2.03	34	86.2
12.44	36	85.8	1.23	34	86.0	2.05	36	86.2
SAMPLES	KD-1-2-3							
2.07	0	0	2.46	0	0	3.26	0	0
2.12	5	9.4	2.51	5	8.8	3.31	5	8.6
2.17	10	28.0	2.56	10	26.2	3.36	10	25.8
2.22	15	47.6	3.01	15	45.6	3.41	15	44.2
2.27	20	64.6	3.07	21	64.4	3.46	20	62.6
2.32	25	78.8	3.11	25	77.0	3.51	25	72.4
2.37	30	85.8	3.16	30	85.6	3.57	31	80.4
2.40	33	87.4	3.20	34	87.6	4.00	34	84.0
2.42	35	87.4	3.22	36	87.6	4.04	38	86.2

OBSERVATIONS (CONT'D.)

Time	Mins.	Moist.%	Time	Mins.	Moist.%	Time	Mins.	Moist.%
SAMPLES	KE-1-2-3							
4.10	0	0	4.47	0	0	5.25	0	0
4.15	5	9.8	4.52	5	8.8	5.30	5	9.8
4.20	10	28.2	4.57	10	27.8	5.35	10	27.4
4.25	15	44.2	5.02	15	43.0	5.40	15	44.4
4.30	20	63.2	5.08	21	63.0	5.45	20	60.6
4.35	25	79.0	5.12	25	76.0	5.50	25	74.8
4.39	29	85.4	5.17	30	85.8	5.55	30	84.2
4.42	32	87.0	5.20	33	88.0	5.57	32	86.2
4.45	35	87.4	5.22	35	88.0	6.00	35	86.6
SAMPLES	KF-1-2-3							
6.04	0	0	6.44	0	0	7.22	0	0
6.09	5	9.6	6.49	5	9	7.27	5	9.2
6.14	10	25.6	6.54	10	26.6	7;32	10	26.8
6.19	15	42.6	6.59	15	45.6	7.37	15	42.0
6.24	20	59.0	7.04	20	64.0	7.42	20	58.8
6.29	25	73.6	7.09	25	78.4	7.47	25	74.2
6.34	30	84.0	7.13	29	86.2	7.52	30	83.6
6.36	32	85.8	7.14	30	87.2	7.54	32	85.6
6.38	34	87.0	7.17	33	87.6	7.56	34	86.6
6.42	38	87.0	7.19	35	87.6	8.00	38	86.8

N.B. Analyses of samples not shown above were carried out at Pointe Claire Laboratories of P.P.R.I.C.

TEST No. 7 May 18th, 1964

Waste water flow	1.28 m.g.d.	Sludge depth in thickener	6 feet
Sludge pumping into thickener	24 hours	Specific gravity	1.030
Thickener loading	745 gals./sq.ft/d.	Plunger pump discharge pressure	19 p.s.i.
Thickener underflow	600 g.p.d.	Temperature of waste water	12°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	560	544	543	553
Volatile	182	164	101	149
Fixed	378	380	442	404
Suspended, total p.p.m.	49	16	19	61
Volatile	37	12	14	42
Fixed	12	4	5	9
Dissolved, total p.p.m.	511	528	524	492
C.O.D. p.p.m.	146	120	75	125
pH	7.45	7.4	7.4	7.3
Settleability ml/l.	4.5	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	100	110	-
Suspended solids, %	0.65	0.53	13.2
Total p.p.m.	6,340	5,120	128,000
Volatile	4,770	4,070	80,100
Fixed	1,570	1,050	47,900
C.O.D., p.p.m.	7,316	3,100	77,128
pH	6.9	7.0	5.6

TEST No. 8 May 31st, 1964

Waste water flow	1.18 m.g.d.	Sludge depth in thickener	4 feet
Sludge pumping into thickener	24 hrs.	Specific gravity	1.028
Thickener loading	745 gals./sq.ft/d.	Plunger pump pressure	18 p.s.i.
Thickener underflow	760 g.p.d.	Temperature of waste water	12°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	570	521	512	569
Volatile	215	168	165	177
Fixed	355	353	347	392
Suspended, total p.p.m.	32	12	11	83
Volatile	22	8	8	59
Fixed	10	4	3	24
Dissolved, total p.p.m.	538	509	501	486
C.O.D., p.p.m.	168	132	90	146
pH	7.35	7.4	7.4	7.2
Settleability, ml/l.	3.5	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	103	90	-
Suspended solids, %	1.94	1.88	11.7
Total, p.p.m.	18,900	18,300	114,000
Volatile	16,060	15,380	76,500
Fixed	2,840	2,920	37,500
C.O.D., p.p.m.	14,570	22,200	96,050
pH	7.0	7.1	5.7

TEST No. 9

June 8th, 1964

Waste water flow	1.09 m.g.d.	Sludge depth in thickener	5 ft.
Sludge pumping to thickener	24 hrs.	Specific gravity	1.046
Thickener loading	745 gals/sq.ft./d.	Plunger pump pressure	23 p.s.i.
Thickener underflow	700 g.p.d.	Temperature of waste water	14°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	581	563	552	635
Volatile	205	170	184	237
Fixed	376	393	368	398
Suspended, total p.p.m.	68	36	39	109
Volatile	48	22	26	71
Fixed	20	14	13	38
Dissolved, total p.p.m.	513	527	513	526
C.O.D., p.p.m.	150	96	53	215
pH	7.35	7.4	7.5	7.0
Settleability, ml/l.	4.0	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	50	55	-
Suspended solids, %	1.28	1.15	12.2
Total p.p.m.	12,250	11,000	116,500
Volatile	10,000	8,430	87,600
Fixed	2,250	2,570	28,900
C.O.D., p.p.m.	10,000	13,150	111,050
pH	6.9	7.0	5.6

TEST No. 10 June 21st, 1964.

Waste water flow 0.967 m.g.d.

Sludge depth in thickener 4 ft.

Sludge pumping into thickener 24 hrs.

Specific gravity 1.024

Thickener loading 745 gals/sq.ft./d.

Plunger pump discharge pressure 17 p.s.i.

Thickener underflow 720 g.p.d.

Temperature of waste water 13°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	569	503	536	591
Volatile	229	170	204	241
Fixed	340	333	332	350
Suspended, total p.p.m.	53	24	31	77
Volatile	39	16	19	61
Fixed	14	8	12	16
Dissolved, total p.p.m.	516	479	505	514
C.O.D., p.p.m.	137	109	98	160
pH	7.4	7.35	7.45	7.1
Settleability, ml/l.	10	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	50	55	-
Suspended solids, %.	1.04	1.78	11.80
Total, p.p.m.	10,200	17,400	115,000
Volatile	7,820	14,450	89,800
Fixed	2,380	2,950	25,200
C.O.D., p.p.m.	13,900	13,415	105,600
pH	7.0	7.1	5.7

TEST No. 11 June 28th, 1964

Waste water flow 0.962 m.g.d.	Sludge depth in thickener 4 feet
Sludge pumping into thickener 24 hrs.	Specific gravity 1.046
Thickener loading 740 gals/sq.ft/d.	Plunger pump pressure 18 p.s.i.
Thickener underflow 900 g.p.d.	Temperature of waste water 13°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	588	548	622	609
Volatile	235	182	242	268
Fixed	353	366	380	341
Suspended, total p.p.m.	90	36	109	99
Volatile	69	25	85	73
Fixed	21	11	24	26
Dissolved, total p.p.m.	498	512	513	510
C.O.D., p.p.m.	170	105	193	206
pH.	7.4	7.3	7.4	7.0
Settleability, ml/l.	10	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	50	55	-
Suspended solids, %	1.14	0.86	11.9
Total, p.p.m.	10,920	8,240	114,000
Volatile	7,860	6,020	84,000
Fixed	3,060	2,220	30,000
C.O.D., p.p.m.	10,595	8,615	126,200
pH	7.1	7.1	5.8

TEST No. 12 July 5, 1964

Waste water flow	1.08 m.g.d.	Sludge depth in thickener	3 feet
Sludge pumping into thickener	24 hrs.	Specific gravity	1.075
Thickener loading	745 g.p.d./sq.ft.	Plunger pump pressure	20 p.s.i.
Thickener underflow	935 g.p.d.	Temperature of waste water	14°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	565.5	534.5	532.0	597.0
Volatile	213.5	178.5	199.0	245.0
Fixed	352.0	356.0	333.0	352.0
Suspended, total p.p.m.	65.4	22.5	24.0	52.0
Volatile	47.3	15.5	14.0	22.0
Fixed	18.1	7.0	10.0	30.0
Dissolved, total p.p.m.	500.1	512.0	508.0	545.0
C.O.D., p.p.m.	149	106	102	141
pH	7.5	7.3	7.5	7.2
Settleability	4.0	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability	95	105	-
Suspended solids, %	1.70	1.45	14.0
Total, p.p.m.	15,800	13,500	130,000
Volatile	11,400	9,730	87,500
Fixed	4,400	3,770	42,500
C.O.D., p.p.m.	19,374	19,123	128,500
pH	7.0	7.1	5.8

TEST No. 13 July 12, 1964

Waste water flow 1.363 m.g.d.	Sludge depth in thickener 3 feet
Sludge pumping into thickener 24 hrs.	Specific gravity 1.025
Thickener loading 745 g.p.d./sq.ft	Plunger pump pressure 19 p.s.i.
Thickener underflow 945 g.p.d.	Temperature of waste water 17°C.

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	617	626.5	578	655
Volatile	230	226.5	191	259
Fixed	387	400	387	396
Suspended, total p.p.m.	78.4	41.9	34	101
Volatile	38.0	28.3	21	49
Fixed	40.4	13.6	13	52
Dissolved, total p.p.m.	538.6	584.6	544	554
C.O.D., p.p.m.	98.4	80.7	68.8	155.6
pH	7.4	7.35	7.5	7.25
Settleability, ml/l.	3.5	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l	85	90	-
Suspended solids, %	1.6	1.5	10.3
Total, p.p.m.	15,600	14,600	100,500
Volatile	9,300	8,200	67,500
Fixed	6,300	6,400	33,000
C.O.D., p.p.m.	21,254	20,346	127,600
pH	7.0	7.1	5.8

TEST No. 14 July 19th, 1964

Waste water flow 1.03 m.g.d.	Sludge depth in thickener 5 feet
Sludge pumping into thickener 24 hrs.	Specific gravity 1.078
Thickener loading 745 g.p.d./sq.ft.	Plunger pump pressure 18 p.s.i.
Thickener underflow 850 g.p.d.	Temperature of waste water 16°C

CHEMICAL ANALYSES

	Sample 1 Influent to Clarifier	Sample 2 Effluent from Clarifier	Sample 3 Chlorinated Effluent	Sample 7 Effluent from Thickener
Solids, total p.p.m.	571	548.5	540.5	617.5
Volatile	239	214.5	205.0	273.0
Fixed	332	334	335.5	344.5
Suspended, total p.p.m.	75.8	30	30.8	77
Volatile	53.0	22	22.3	55
Fixed	22.8	8	8.5	22
Dissolved, total p.p.m.	495.2	518.5	509.7	540.5
C.O.D., p.p.m.	183	120	106	196
pH	7.4	7.3	7.4	7.1
Settleability ml/l.	5	-	-	-

OBSERVATIONS (CONT'D.)

	Sample 4 Clarifier Sludge	Sample 5 Influent to Thickener	Sample 6 Thickened Sludge
Settleability, ml/l.	60	55	-
Suspended solids, %	1.78	1.68	13.0
Total, p.p.m.	16,512	15,632	120,600
Volatile	11,394	11,115	74,614
Fixed	5,118	4,517	45,986
C.O.D., p.p.m.	19,906	18,054	92,503
pH	6.9	7.0	5.6

APPENDIX F

BIBLIOGRAPHY

The following abbreviations have been used:

A.S.C.E. – American Society of Civil Engineers

F.S.I.S.A. – Federation of Sewage and Industrial Wastes Associations

A.P.H.A. – American Public Health Association

W.P.C.F. – Water Pollution Control Federation

A.W.W.A. – American Water Works Association

1.	American Public Health Association, American Water Works Association, Water Pollution Control Federation.	Standard Methods for the Examination of Water, Including Bottom Sedimentation and Sludge.	Prepared and Published jointly by A.P.H.A., A.W.W.A., W.P.C.F., 11th Edition, 1960.
2.	American Society of Civil Engineers, and Federation of Sewage and Industrial Wastes Association, a joint committee.	Sewage Treatment Plant Design	A.S.C.E. Manual of Engineering Practice, No. 36. F.S.I.W.A. Manual of Practice, No. 8, 1959.
3.	Anderson, N.E.	Design of Final Settling Tanks for Activated Sludge.	Sewage Works Journal, 17, 1945.
4.	Behn, V.C. and Liebman, J.C.	Analysis of Thickener Operation	A.S.C.E. Proceeding, Sanitary Engineering Division, Vol. 89, S.A.3, June 1963.
5.	Bolton, R.L. and Klein, L.	Sewage Treatment Basic Principles and Trends.	Butterworth, London, 1961.
6.	Bull, A.W. and Darby, G.M.	The Pre-Sedimentation of Turbid Water Supplies.	Transaction, American Institution of Chemical Engineers, 18, 1926.
7.	Camp, T.R.	Sedimentation and the Design of Settling Tanks.	Transaction, A.S.C.E., Vol. III, 1946, p.895.
8.	Canadian Municipal Utilities Magazine	Canadian Sewerage Works Statistics.	Canadian Municipal Utilities Magazine, Vol. 98, No. 2, February, 1960.
9.	Coe, H.S. and Clevenger, G.H.	Methods of Determining the Capacity of Slime-settling tanks.	Transactions of American Institution of Mining, Met. Engineers, Vol. 55, 1916, pp. 356-384.
10.	Commings, E.W. Pruiss, C.E. and Debord, C.	Continuous Settling and Thickening	Industrial and Engineering Chemistry, Vol. 46, No. 6, June 1954, p. 1164.

11. Dobbins, W.E.	Effect of Turbulence on Sedimentation.	Transactions, A.S.C.E., Vol. 109, 1944, p. 629.
12. Egolf, E.B. and McCabe, W.C.	Rate of Sedimentation of Flocculent Particles.	Transaction, American Institution of Chemical Engineers, 33, 1937.
13. Fair, G.M. and Geyer, J.C.	Water Supply and Waste Water Disposal.	John Wiley & Sons, Inc., New York, 1954.
14. Fitch, E.B.	The Significance of Detention in Sedimentation.	Sewage and Industrial Waste, 28, 1, 1, January 1956.
15. Gauvin, W.H. and Dlouhy, J.	Application of the Atomized Suspension Technique of the Treatment of Sanitary Sludges.	Technical Report No. 109, Pulp and Paper Research Institute of Canada, Montreal, November, 1958.
16. Genter, A.L.	Small Sludge-Elutriation Plants.	Sewage Works Journal, 18, 17, 1946.
17. Gould, R.H.	Operating Experiences in New York City.	Sewage Works Journal, Vol. 14, No. 1, 1942, p. 77.
18. Hazen, A.	On Sedimentation.	Transactions, A.S.C.E., Vol. LIII, December 1904 p. 63.
19. Howard, Sir A.	The Preservation of Domestic Waste for Use on the Land.	Journal, Institution of Sanitary Engineers, England, June 1939, p. 174.
20. Imhoff, K. and Fair, G.M.	Sewage Treatment.	2nd. Edition, John Wiley & Sons, Inc., New York, 1956.
21. Kalinske, A.A.	Settling Rate of Suspensions in Solid Contact Units.	Proceedings, A.S.C.E., Vol. 79, No. 186, April 1953.
22. Kammermeyer, K.	Settling and Thickening of Aqueous Suspension	Industrial and Engineering Chemistry, 33, No. 12, December 1941, pp. 1484-1491.

23.	Kermack, W.O. McKendrick, A.G. and Ponder, E.	The Stability of Suspensions III. The Velocities of Sedimentation and Cataphoresis of Suspensions in a Viscous Fluid.	Proceedings, Royal Society of Edinburgh, Vol. XLIX, 1928-1929, pp. 120-177.
24.	Kiess, F. and Schreckegast, C.	Sludge Dewatering by Vibrating Screens.	Water and Sewage Works, November 1959.
25.	Kynch, G.J.	Settling and Thickening of Aqueous Suspension.	Industrial and Engineering Chemistry, 33, No. 12, December 1941, pp. 1484-1491.
26.	Lea, H.W., Consulting Engineer	Report on the Sanitary Sewerage System, Town of Beaconsfield, Quebec, Canada.	Montreal, Quebec March 1956.
27.	McNown, J.S. and Lim, P.N.	Sediment Concentration and Fall Velocity.	Proceedings, Second Midwestern Conference on Fluid Mechanics, Ohio State University, Columbus, Ohio, 1952, p. 401.
28.	Newton, Sir Isaac	Mathematical Principles of Natural Philosophy.	Vol. 2, 1910, Sections 2 and 7.
29.	Rawn, A.M.	Concerning those Plus Fertility Values of Sewage Sludge.	Waterworks and Sewerage, December 1940, p. 575.
30.	Roberts, E.J.	Thickening, Art or Science.	Mining Engineering, 1, 61, 1949.
31.	Robinson, C.S.	Some Factors Influencing Sedimentation.	Industrial and Engineering Chemistry, 18, No. 8, August 1926.
32.	Rudolfs, W. and Lacy, I.O.	Settling and Compacting of Activated Sludge.	Sewage Work Journal, Vol. VI, No. 4, 1934, p. 647.
33.	Schroeper, G.J.	Economics of Sewage Treatment.	Proceedings, Sanitary Engineering Division, New York, April 1938.

34.	Slade, J.J.	Sedimentation in Quiescent and Turbulent Basins.	Transactions, A.S.C.E., Vol. 102, 1937, p. 306.
35.	Steel, E.W.	Water Supply and Sewerage.	2nd. Edition, McGraw-Hill Book Co. Inc., New York, 1947.
36.	Steinour, H.H.	Rate of Sedimentation.	Industrial and Engineering Chemistry, 36, 1944.
37.	Stokes, Sir G.G.	On the Theories of Internal Friction of Fluids in Motion, and of the Equilibrium and Motion of Elastic Solids.	Transactions, Cambridge Philosophical Society, Vol. VIII, 1845, p. 287.
38.	Talmage, W.D. and Fitch, E.B.	Determining Thickener Unit Areas.	Industrial and Engineering Chemistry, 47, 1, 1955, p. 38.
39.	Van Kleeck, L.W.	Removing Raw Sludge from Primary Settling Tanks.	Waste Engineering, 27, 5, 220, 1956.
40.	White, W.F. and Burns, T.E.	Continuous Centrifugal Treatment of Sewage Sludge.	Water and Sewage Works, October 1962, p. 384.
41.	Zack, S.I.	Sludge Dewatering and Disposal.	Sewage and Industrial Waste, Vol. 22, No. 8, 1952.
42.	Zimmermann, F.J. and Diddams, D.G.	The Zimmermann Process and Its Applications in the Pulp and Paper Industry.	TAPPI, 43, No. 8, 710-715, August 1960.

CURRICULUM VITAE

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