sanepar

COMPANHIA DE SANEAMENTO DO PARANÁ
rua engenheiros rebouças, 1376 - CURITIBA

WATER TREATMENT PLANT FOR SMALL COMMUNITIES

Studies in a Pilot Plant

CONTENTS

FUNDAMENTALS OF THE PROJECT

I - FUNDAMENTALS OF THE PROJECT

Carlos A.Richter
C.S. Balkowiski

Nowadays, to get reasonable costs in water supplies for small or rural communities, the use of springs are quite often proposed , in order to become the enterprise viable economically.
But those springs are, however, subject to pollution from transient population and cannot be considered safe without protective measures.

Since to delimitate insurance areas is not easy, the so called protected sources are rarely found and then the alternative consists in digging tubular deep wells.

There is a low sucess probability of getting the quantity of water required from ground water in some types of rock. Many times a great number of non-productive wells have been dug with a lot of financial resources waste.

Taking into consideration that in addition to the safety of a source of water supply it is also very important a fair quantity. The source should be capable of furnishing an adequate quantity of water continously with a minimum danger of interruption. And then we must make use of surface waters, which always need some requeriments for treatment.
For small communities supplies located in rural areas, here we begin some studies and researches in order to develop a water treatment plant of low cost and satisfactory efficiency, which local people can easily operate and afford.
In this work, we demonstrate the possibilities of the realization of this design from its initial outlining being followed by a research on pilot filters.
We describe next the works that include:

(1) The basic design outlining, whose description is the purpose of this section.

(2) The proposition of a theoretical model of flocculation in granular medium included in section II, and possibly the most important one of this report taking into consideration the

open perspectives to the development of future designs.

3) The researches in a pilot plant with the purpose of

a. proving the flocculator, included in section III.

b. proving the hydraulics of the filter backwash system, included in section IV.

The proposed water treatment plant will be useful, mainly, for communities without material facilities and without skilled plant operators. So it shall contain a minimum of equipment and be easy to operate, in a way to need only the essential actions by an operator with rudimentary knowledge on water treatment.

1.1. Technical justification

1.1.1. Water Treatment Plant basic scheme

The water treatment plant will have essentially a granular flocculator and a high rate sedimentation basin followed by a double-layer filter of sand and anthracite. The filter backwash will be made automatically, by siphoning when the head loss reaches a predetermined value.

The simplified scheme of figure 1.1 shows the water treatment plant basic conception. This scheme, in what it concerns to the filter automatism, is very similar to the valveless gravity filter, introduced about 20 years ago by an american industry, however this scheme is very different in the way it makes the backwashing. In the industrial filter, when the siphon is primed, the backwash begins suddenly at a maximum velocity and as the water level diminishes in the washwater reservoir situated over the filter the ascensional velocity diminishes proportionally till it gets to a minimum value when the washwater reservoir is empty.

This action can be demonstrated by the descendant interval and only by it, in the curve of the fig. 1.4. In the sistem here proposed, the backwash begins slowly till it gets a maximum ascensional velocity and, then it diminishes gradually according to the curve shown in fig. 1.4.

1.1.2. Principle of working

The raw water arrives at the water treatment plant in a box with a constant level 1, and then, it goes to the flocculator 3 through a vertical pipe, immediately after the dispersion of the coagulant in the venturi 2.

Following the flocculation, the water enters in a high rate sedimentation basin 4, with parallel plates, next going to filter 5.

When the filtration begins, the effluent firstly fills the washwater reservoir 6, and, only when the maximum water level in it (level 0) is reached, the effluent goes to the filtered water channel 7, to the distribution system. At this moment, the water level in the filter is a level 0 + h_i , being h_i the initial head loss in the filtration, approximately equal to 40 cm . Since at the begining of the filtration the effluent turbidity is higher, diminishing sensibly in the first minutes, the use of this water for the backwashing and the conducting of the following filtered water of better quality to distribution is one of the advantages of this design.

As the filtration runs, the pores of the media diminish their size due to the retained material, and the head loss increases, till it reaches to a maximum value h_f, corresponding to an admissible final head loss, at the end of the filter run, when it must be backwashed.

In this interval, the water level in the filter increased from 0 + h_i to 0 + h_f, being then, primed the main siphon 9 through the taking out of the air from its inside by the auxiliary syphon 8.

Then, the water level in the filter diminishes progres sively and being its area smaller than that of the washwater reservoir, this lowering is very quick in the filter and pratically insensible in the reservoir till the siphon outlet level is reached. From the moment in which the water level in the filter is equal to the water level in the washwater reservoir, a water head arises, starting the backwashing flow from the reservoir to the filter, passing through the waste, by siphonic action.

As the water level in the filter lowers, the backwash velocity increases gradually till the moment when the water level in the filter is equal to that in the siphon outlet, when the washwater reaches its maximum velocity.

The time of this first phase of washing can be defined by the hydraulics of the system, as it is exemplified in figure 1.2. (taken from fig. 4.3. section IV).

A second phase of the washing comes next (fig.1.3.) in which the washwater velocity is slowly and gradually reduced to a minimum value progressively to the decrease of the water level in the washwater reservoir. This value is still enough to keep suspended all the bed and when the washing operation is suddenly interrupted by the entering of air at athmospheric pression in the canalization 10, the filter bed will be perfectly restratified.

In the same way as in the first phase, this second phase time can be determined by the volume and shape of the washing water reservoir and by the other hydraulic conditions of flow.

The evolution of the washwater velocity follows, therefore, a curve as in fig. 1.4, showing that it is possible, theoretically, to obtain a perfect backwashing, without the action of the operator, but only with the hydraulic action of the siphon operation system.

The backwash cicle is finished at the moment when the siphon primer is broken, by venting the backwash siphon by the vent pipe 10, and so the washing time can be established by the depth H of the vent pipe inlet in the washwater reservoir.

1.2. Brief report of each unit

1.2.1. Flocculator

The flocculation in granular medium can be represented by the equation

$$\ln \frac{No}{N} = \eta KGT,$$

or

$$\frac{N}{No} = e^{-\eta KGT}$$

Whose theoretical demonstration is presented in section II and its checking in a pilot plant is subject of section III.
In the above equations

N_o and N - particles concentration that enters and that leaves the flocculation chamber, respectively;
η - efficiency factor in the granular medium flocculation
K - coagulation constant, function of the physical-chemical characteristics of the water and of the kind of coagulant used.
G - velocity gradient
T - flocculation time.

With the analysis of the above it is possible to deduce that it is perfectly possible to get a good flocculation in a shorter time, that it will be demonstrated theoretical and pratically in the following sections.
This represents another conquest in water treatment, where the tecnology trends in the last decades have led to a significant reduction of the settlers and filters size, with an increase in efficiency. However, the same didn't happen with the flocculators, which have kept the same sizes.

With the granular medium floculators one will be able to reduce the necessary volume to the flocculation till 1/5 of the volume of the convenctional floccula - tors, what is the main objective of this work: to deve lop a design of a water treatment plant of low cost and satisfactory efficiency, to be used mainly in ru- ral communities.

1.2.2. Settler

It will be a high rate settler with laminar flow, by parallel plates inclined 60º to the horizontal. To take advantage of the favorable effect of the sludge blanket the flocculated water is admitted at the bottom of the sedimentation basin passing through the previos ly deposited sludge zone, and so increasing the effici ency of the parallel plates settler.

1.2.3. Filter

It will be a gravity dual-media filter of sand and anthracite. Its washing system will be by upward flow in the filter bed induced by the siphon action. The priming of the main siphon will be made by a small auxiliary or priming siphon, as in fig. 1.5.
When the maximum head loss is reached, the water flows initially by the primer siphon of small diameter BC. This piping is designed to support a rate of flow a little smaller than that one which flows throughout the filter, resulting a piezometric line AC. The negati ve head - Z, produced by this flow takes the air off the main siphon, so priming it. From this moment the backwashing runs as it has already been described.

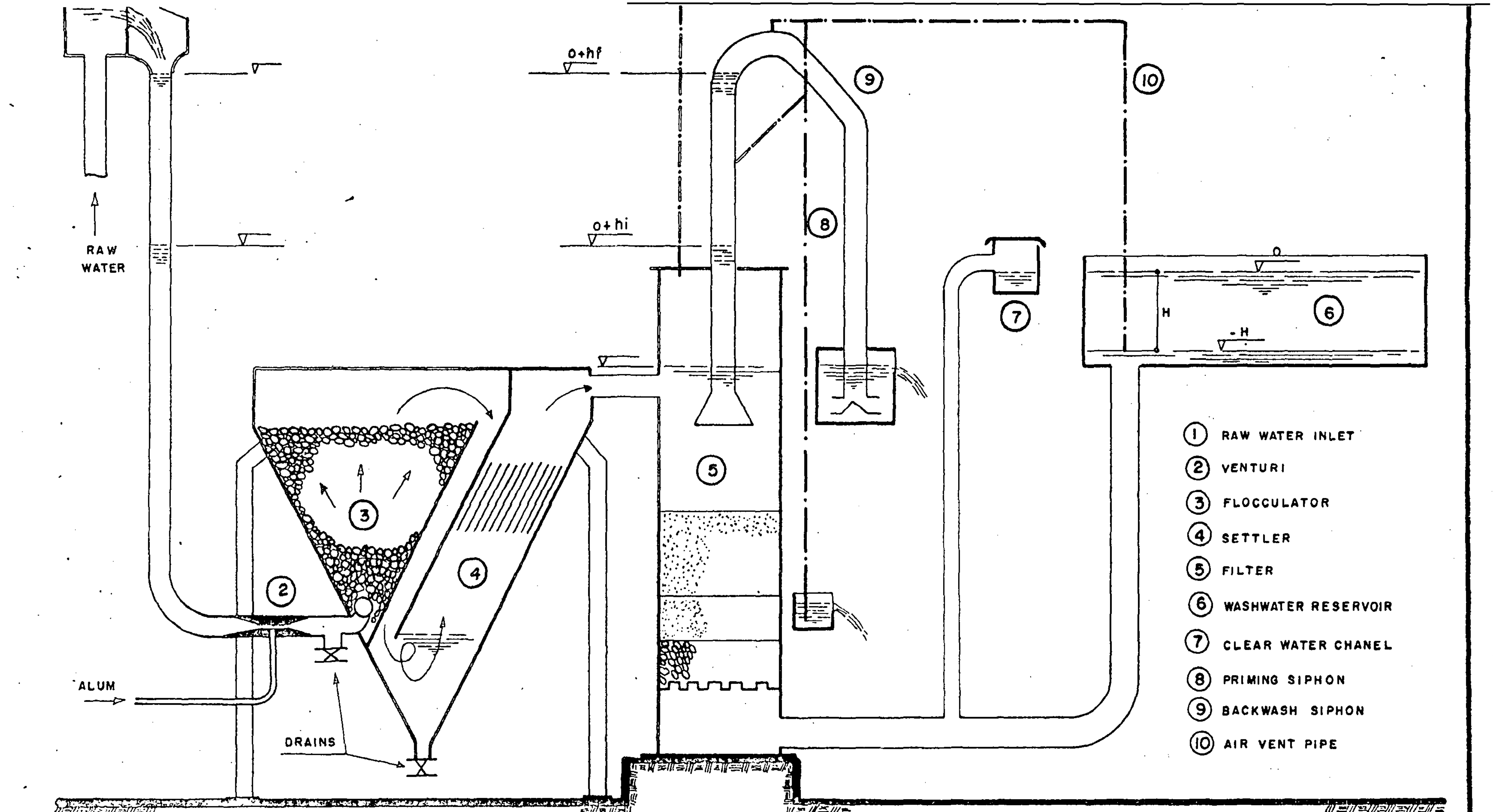


FIG. I.I - SKETCH OF THE WATER TREATMENT. PLANT

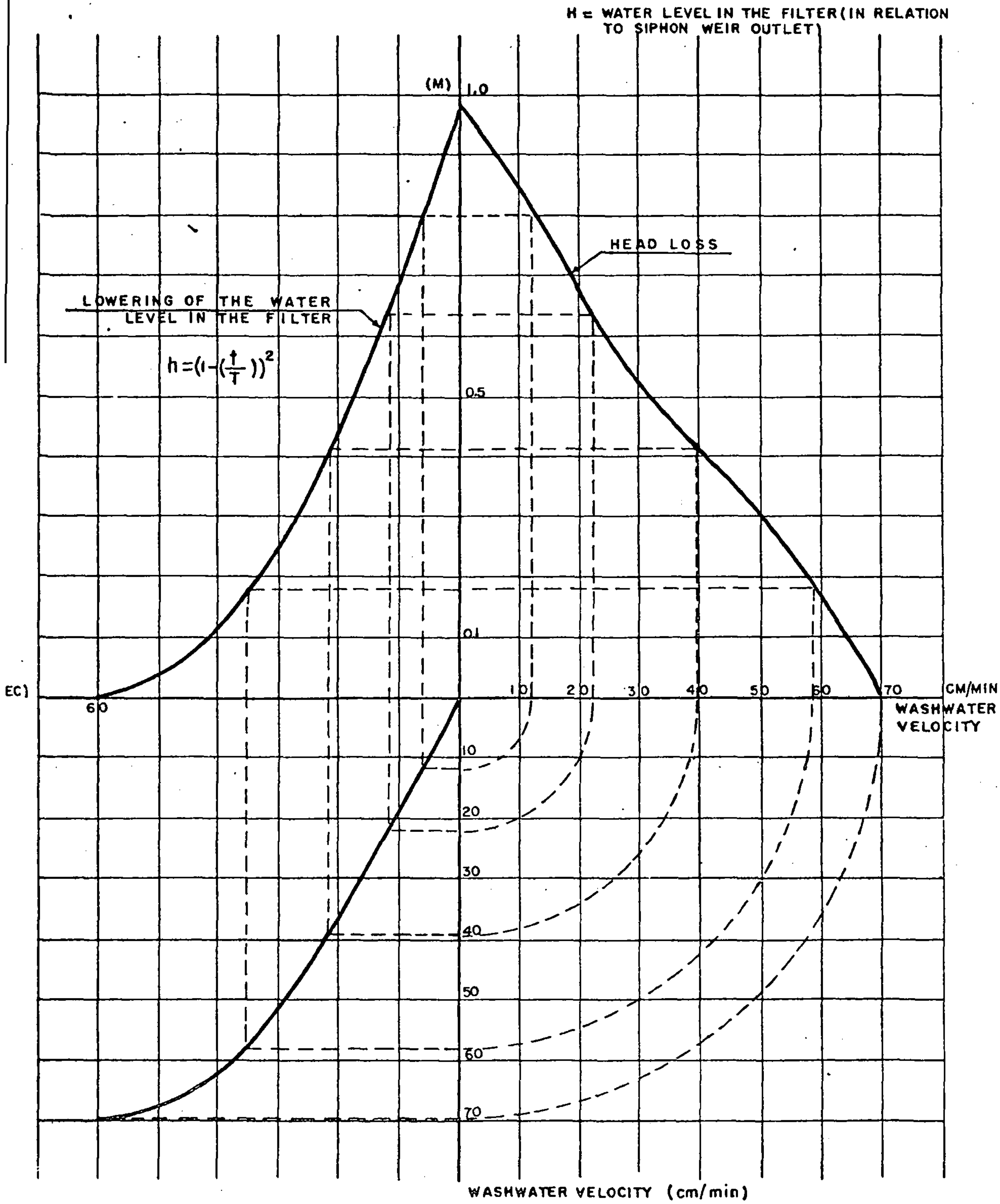


FIG. 1.2 — 1st PHASE OF BACKWASHING

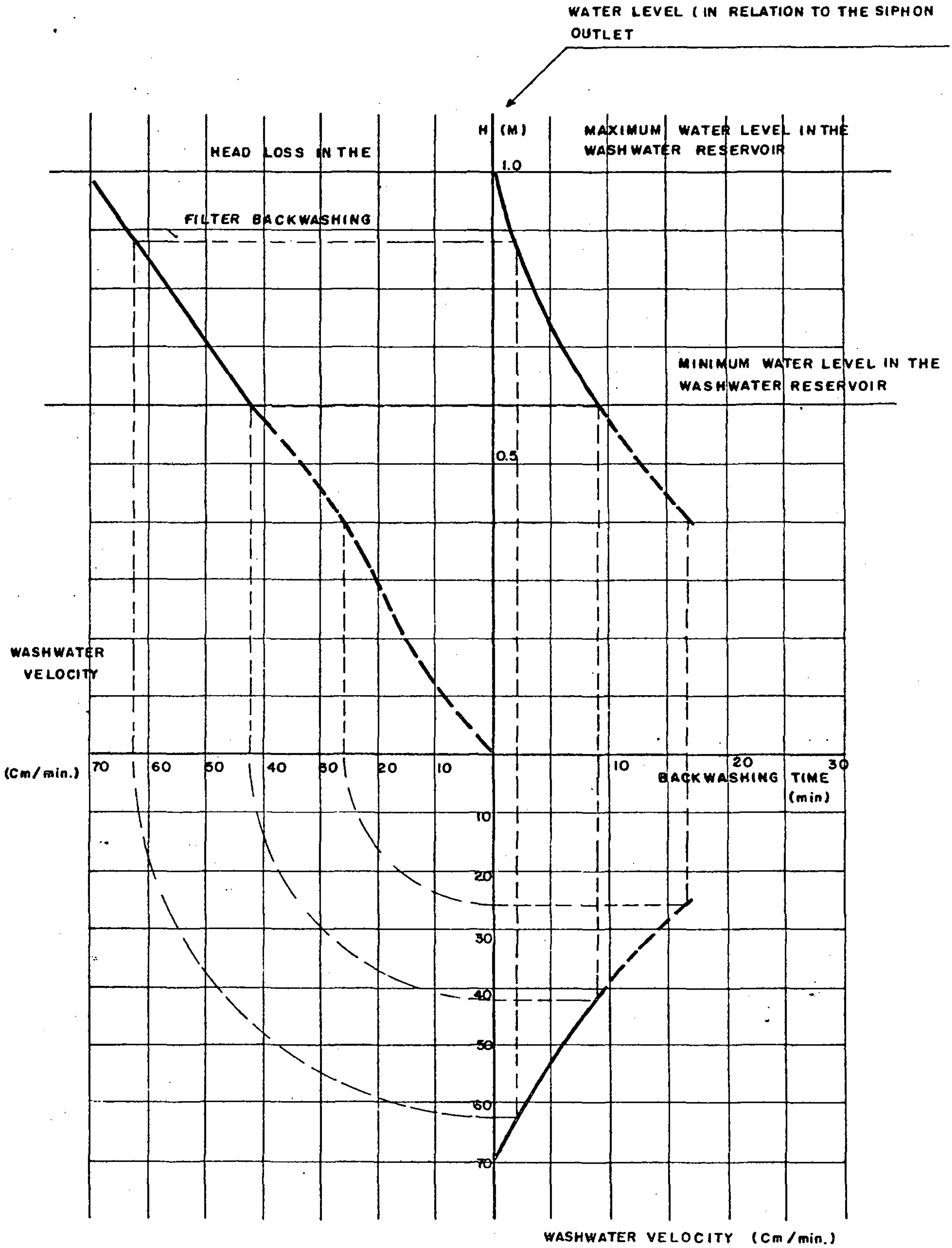
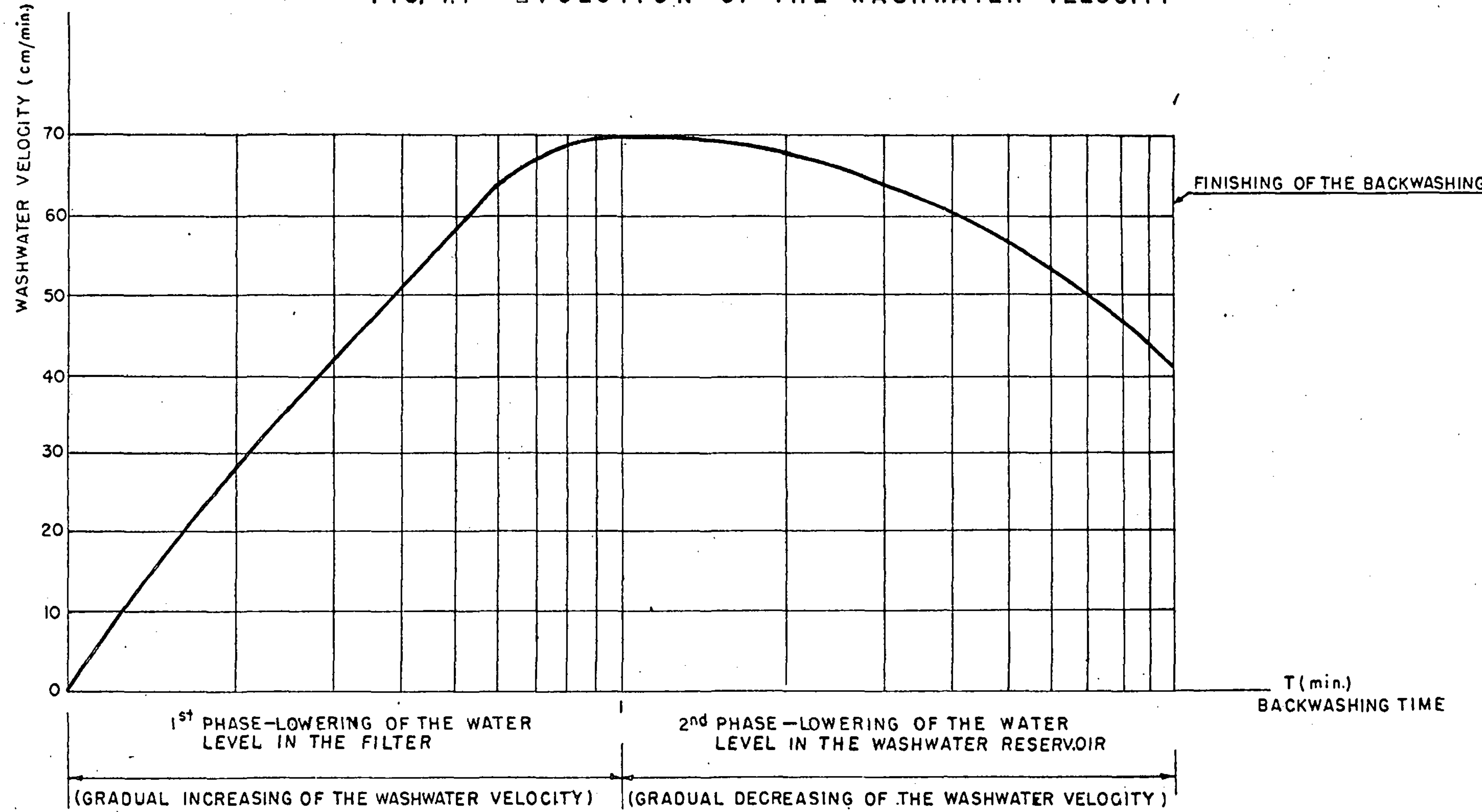
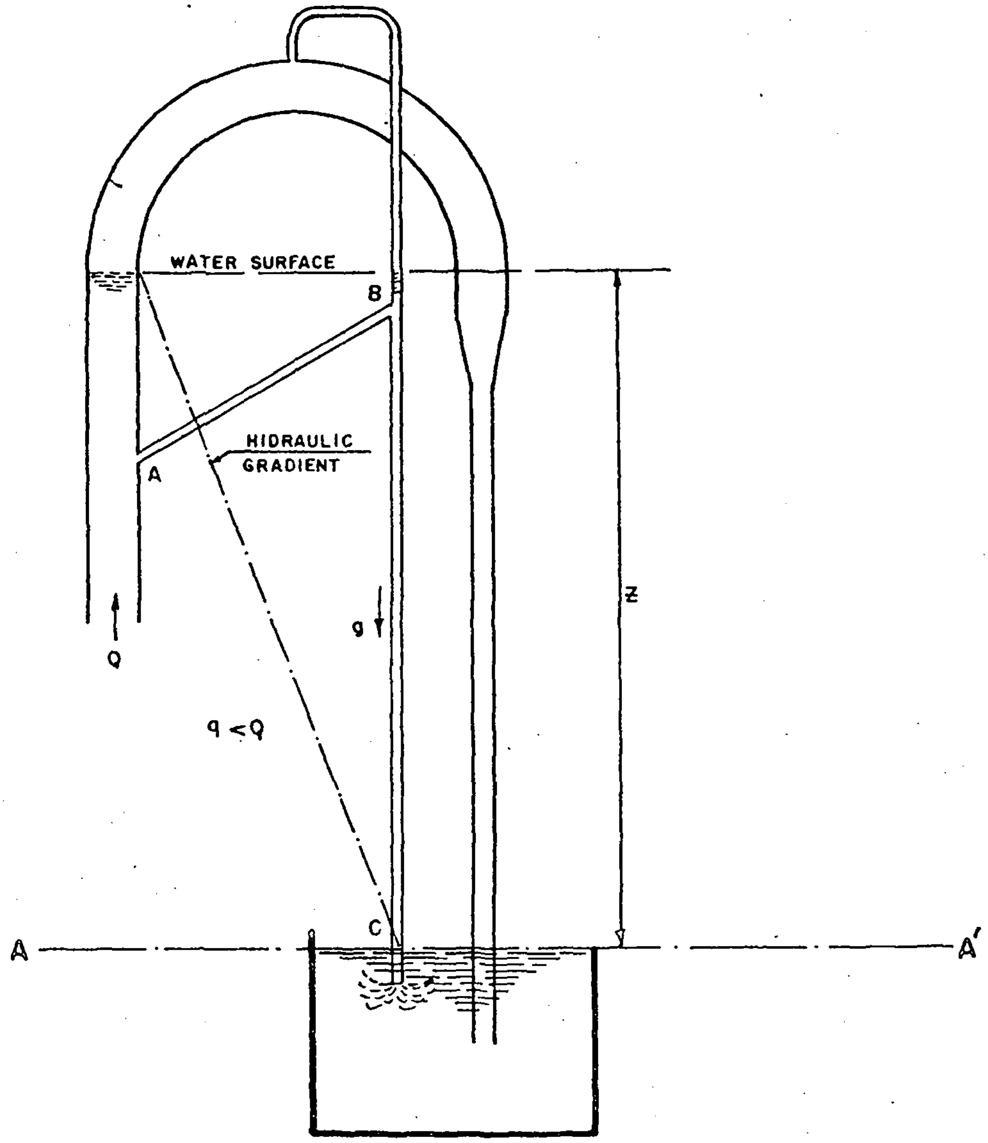


FIG. 1.3.- 2nd PHASE OF THE BACKWASHING

FIG. 1.4 — EVOLUTION OF THE WASHWATER VELOCITY



Formato A4 = 210 x 297 mm



IN RELATION TO DATUM AA', NOT CONSIDERING THE FRICTION AND MINOR LOSSES:

$$\cancel{\frac{V_B^2}{2g}} + \frac{P_B}{\gamma} + Z = \cancel{\frac{V_C^2}{2g}} + 0 + 0$$

$$\frac{P_B}{\gamma} = -Z$$

FIG. 1.5 — THE SIPHON PRIMING

THEORETICAL BASIS OF FLOCCULATION IN GRANULAR MEDIUM

II THEORETICAL BASIS OF FLOCCULATION IN GRANULAR MEDIUM

Carlos Alfredo Richter

2.1. Introduction

In the last years the water treatment technology has undergone an extraordinary development, particularly in the settling and filtering processes. With the use of lamellar settlers and multi-layers filters the settling tanks and the filtering units had their size reduced, with the same or greater efficiency as in the conventional processes.
A considerable amount of theoretical knowledge about coagulation process has been accumulated in the same period;however, the flocculation tanks are still being made with the same detention time, normally from 20 to 40 minutes, and so, producing the same volumes of the conventional processes.

The early theory about flocculation, made by Von Smoluchowski can be summarized in the following equation:

$$\frac{dN}{dt} \equiv \frac{G}{6} n_i n_j (d_i^3 + d_j^3) \qquad (1)$$

where

dN/dt = the collision rate between the particles (i) and the particles (j)

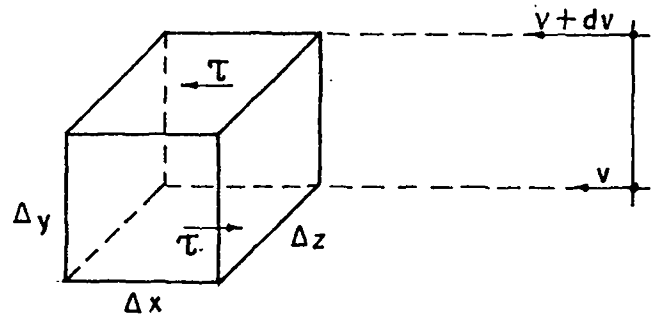
n_i and n_j = the concentration of particles (i) and (j)

d_i and d_j = the diameter of particles (i) and (j)

$G = dv/dy$ = the velocity gradient

The mean gradient velocity calculation is normally made with the use of the equation of Camp and Stein, whose mathematical deduction is presented next.

In considering an element of fluid $\Delta x \Delta y \Delta z$, subjected to mechanical or hydraulic stirring necessary to the flocculation,



The dissipated power is given by

Power P = force x velocity, or

P = shear stress (τ) x area ($\Delta x . \Delta z$) x velocity (dv)

$$P = \tau . \Delta x . \Delta z . \Delta v . \frac{\Delta y}{\Delta y} = \tau . v . \frac{\partial v}{d y}$$

being

$$\Delta x \, \Delta y \, \Delta z = \text{volume } V$$

$$\frac{P}{V} = \tau \frac{dv}{dy} \qquad (3)$$

where P/V is the dissipated power by unit volume.

Under lamellar flow conditions

$$\tau = \mu \, dv/dy$$

where μ is the dynamic viscosity. Substituting, it results

$$\frac{P}{V} = \mu \, (dv/dy)^2 \qquad \text{or}$$

$$dv/dy = G = \sqrt{\frac{P}{\mu V}} \qquad (4)$$

Camp[1], while studying flocculation tanks found satisfactory results when the nondimensional number GT, called Camp number, in which T is the flocculation period, varied between 2×10^4 and 2×10^5, with the values of G varying between

20 and 74 s^{-1}.

Harris, Kaufman and Krone[3], taking the Smoluchowski equation (1) as reference and considering a flocculation tank as a series reactor of continuous flow with m chambers, show that

$$\frac{No}{Nm} = \left(1 + K \, \emptyset \, G \frac{T}{m}\right)^{m} \qquad (5)$$

No e Nm = concentrations of primary particles in the affluent and effluent of chamber m, respectively.

K = constant

Ø = colloid concentration, concerning to the solid total volume including coagulant for each volume of fluid.

T = total flocculation time.

The equation (5) shows clearly that a given efficiency can' be obtained in less and less time with the increasing of the number of flocculation chambers in series.

For pratical and economical reasons, the number of chambers' in series is not very great in the conventional flocculation tanks, as a rule not more than 6 units. The design recommendation give 3 as a minimum number.

Being so, the flocculation time is little or nothing reduced in relation to the minimum of 20 minutes in the convencional designs.

In a subsequent work, Argaman and Kaufman added an erosion ' model and floc breakup to the equation (5), in which the efficiency would be inversely proportional to the square of the' velocity gradient and to the flocculation time:

$$\frac{No}{Nm} = \frac{\left(1 + K_A \, G \frac{T}{m}\right)^{m}}{\left[1 + K_B G^2 \frac{T}{m} \sum_{i=0}^{i=m-1} \left(1 + K_A \, G \frac{T}{m}\right)^{i}\right]} \qquad (6)$$

being K_A = aggregation constant and K_B = erosion and floc ' breakup constant.

For low values of G the equation (6) approaches the equation (5), which is generally valid for the values of G lower than 60 s^{-1}.

Kao and Mason[9] while studying agglomerates of particles not adhesive under the action of hydrodynamic shear stress , $\tau = \mu dv/dy$, deduced a relation next to the radius of the agglomerates of particles, R_t, after a certain period T, as in

$$R_o{}^3 - R_t{}^3 = KGT \qquad (7)$$

From this equation, one can conclude that, at G constant,the radius of the agglomerates of particules is inversely proportional to the time.

2.2. Turbulence and Flocculation

The flocculation tanks design is frequently based on jar - tests, with the determination of the parameters as flocculation time and optimum velocity gradient,and in the application of the known formula of Camp and Stein (equation (4)), in mechanical or hydraulic flocculators.

It is an already observed fact that rarely there is a concordance between the obtained results with the gradients of the jar-tests as well as those applied in the plant and the applied values in different plants. The dissimilarity of the obtained results leads to the conclusion that the velocity gradient parameter determined by the equation (4) is more a qualitative parameter than a quantitative one, so it has a questionable importance in a design.

The first flocculation experiences confirming the theory of Smoluchowski were made in high viscosity liquids and were completed by Stein[1] in rapid sand filters, in which media the flow results lamellar at the experienced conditions. However, Camp admited that the flow in the units of flocculation and flash mixing is always turbulent, even at low values of G and that it can happen conditions of lamellar flow in jar tests.

In fact, at 40 rpm, taking the velocity at 2/3 of the radius of the blade of a "Phipps & Bird" apparatus, it gives a number of Reynolds R $\cong$ 2000. Other kinds of apparatus frequently used in water treatment plants, due to the caracteristics of each one, give better conditions for the establishment of lamellar flow.

In the other hand, the blade of a mechanical flocculator turning at 3,6 rpm from two meters of the axis gives a turbulent flow with Reynolds number equal to $1,5 \times 10^6$.

The analitic comparison of the Argaman and Kaufman equation (6), useful for a turbulent regime, with the Kao and Mason equation (7), found under lamellar regime conditions, allows us to hope a higher efficiency in the lamellar flocculation than in the turbulent flocculation, in equal time and gradient conditions, since the flocs erosion in the turbulent regime is proportional to the square of the velocity gradient, whereas in the lamellar regime it is linear in relation to the same parameter.

The applying of equation (4) suggests the existence of lamellar flow and would be valid only under this condition. This comes out from the definition of the viscosity coefficient itself: $\mu = \tau/(dv/dy)$.

In the same way as in lamellar regime, in which the longitudinal tension due to the friction between two blades, is equal to $\mu \frac{dv}{dy}$, there is another tension due to the turbulent regime, called Reynolds shear stress, defined as $\tau = \varepsilon \frac{dv}{dy}$, where ε is the virtual or turbulent viscosity coeficient.

The total tension is given, then by

$$\tau = (\mu + \varepsilon) \frac{dv}{dy} \qquad (8)$$

and, so, the formula (1) takes the form

$$G = \sqrt{\frac{P}{(\mu + \varepsilon) V}} \qquad (9)$$

This equation becomes equal to (4), when the energy dissipation due to the turbulence is negligible in face of that on due to the viscosity, that is to say, when the flow is la mellar. When the flow is turbulent the turbulent viscosity coefficient increases rapidly when the Reynolds number increases, whereas the dynamic viscosity stays invariable and the turbulence coefficient takes values many times superior to the viscosity coefficient (frequently$>10^3$).

Under this aspect, we can explain, then, the lesser efficiency of turbulent flocculation in relation to the lamellar flocculation, since the erosion and rupture of the flocs is due to the intensity of the hidrodynamical forces, proportional to the sum of the viscosity coefficients whose component due to the turbulence is, in its turn, proportional to the Reynolds number. Due to the large variability of the virtual viscosity coefficient or turbulence coefficient, function of the flow characteristics, and the geometry of the flocculation tanks or channels, and of the stirring equipment characteristics, we can, with the equation (9), explain the dissimilarity of the verified results in pratice, emphasying the character more qualitative than quantitative of the velocity gradient, as originally defined by Camp and Stein (equation 4). We preferred to study the turbulent flocculation through the virtual viscosity concept, to demonstrate its complexity and limitations, keeping the same form of equation (4) whose application is very common among sanitarian engineers.

Argaman and Kaufman[4], when they studied the same problem through the energy spectrum concept, a substitutive form to the virtual viscosity concept to describe the turbulent flow, after an exhaustive theoretical and experimental work, proposed an equation that can in a better way describe and explain the flocculation phenomenon, however they admitted it was not easy to define a quantitative expression by relating the power dissipation function to the stirring mechanism geometry and to the flocculation chamber geometry.

This difficulty can be eliminated if we set up a flocculation in lamellar flow. In practice, this can be achieved, for instance, in granular media.

2.3. Theoretical Model of Flocculation in Granular Media

The efficiency of an hydraulic or mechanical flocculation tank is as high as the number of chambers set in series. The equation (5) can be expressed in the following way:

$$\frac{Nm}{No} = \left[\frac{1}{\frac{KGT}{m} + 1} \right]^{m} \qquad (10)$$

where No is the concentration of colloidal particles in the first chamber, Nm is the concentration of particles (flocs) in the efluent of the last chamber, K depends on the phisical and chemical characteristics of the water and the kind of coagulant used, T is the total time of flocculation, G is the velocity gradient and m is the number of chambers.

Developing T from the equation (10), one gets:

$$T = \frac{m}{KG} \left[\left(\frac{No}{Nm}\right)^{1/m} - 1 \right] \qquad (11)$$

A flocculator in a granular medium, in gravel for instance, is an hydrodinamic flocculator that can be considered as flocculation basin with a very great number of chambers, and the flocculation time needed to obtain a previous established result $\frac{No}{Nm}$ tends to the limit value:

$$T = \lim_{m \to \infty} \frac{m}{KG} \left[\left(\frac{No}{Nm}\right)^{1/m} - 1 \right]$$

or, turning $m = \frac{1}{n}$

$$T = \lim_{n \to o} \frac{1}{KG} \cdot \frac{\left(\frac{No}{Nm}\right)^{n} - 1}{n} = \frac{1}{KG} \lim_{n \to o} \frac{\left(\frac{No}{Nm}\right)^{n} - 1}{n}$$

or

$$T = \frac{1}{KG} \ln \left(\frac{No}{Nm}\right) \qquad (12)$$

or $$\ln\left(\frac{No}{Nm}\right) = KGT \qquad (13)$$

The above equation would be only valid to the total efficiency in the particles collision. Inserting an efficiency factor ($\eta < 1{,}0$), that can be called granular medium flocculation efficiency factor, the equation (13) comes

$$\ln\left(\frac{No}{Nm}\right) = \eta KGT \qquad (14)$$

Developing T, one gets

$$T = \frac{\ln(No/Nm)}{\eta KG} \qquad (15)$$

Under identical conditions, being T the theoretical minimum time in the granular medium flocculation, and θ the flocculation time determined in jar-tests, necessary to achieve the same result, we have:

- granular medium flocculation (equation (12) with $\eta = 1.0$)

$$T = \frac{1}{KG} \ln(No/Nm)$$

- jar-test flocculation

$$\theta = \frac{1}{KG}\left(\frac{No}{Nm} - 1\right)$$

obtained from equation (10) turning m = 1 (only one compartment).

Dividing an equation by the other one, one gets

$$\frac{T}{\theta} = \frac{\ln \frac{No}{Nm}}{\frac{No}{Nm} - 1} \qquad (16)$$

The preceding equation (16) shows that the necessary time for the flocculation in a granular medium will be always a fraction of the flocculation time obtained in the jar test for a given result since $No/Nm \geq 1$ it always results in

$$\ln \frac{No}{Nm} < \frac{No}{Nm} - 1 .$$

The figure 2.1. shows the variation of the flocculation relative time T/θ with No/Nm, or with the correspondent turbidity removal $\rho = 1 - Nm/No$. It means that, if in the jar-test to obtain a turbidity removal of 90% (No/Nm) = 10) it is necessary 20 minutes of flocculation, in the granular medium flocculator the same result will be theoretically obtained with about 5,2 minutes (T/θ) = 5,2 ÷ 20 = 0,26). In the same figure are represented some results obtained in water with 100,300 and 1000 NTU, confirming the theoretical tendency. One must observe that there is a difference between the results because it was of taken into consideration identical values for the efficiency factor in flocculation, taken in both cases, flocculator and jar-test, $\eta = 1,0$.

2.4. Velocity Gradient in Porous Medium

If the flow is lamellar, the velocity gradient G is easily evaluated through the knowledge of the head loss h in the porous medium, by its direct determination, or through known formulas.

The velocity gradient is given by

$$G = \sqrt{\frac{P}{\mu \ V}}$$

however, the dissipated hydraulic power is

$$P = \gamma Qh$$

where γ is the especific weight of the fluid and Q is the rate of flow, and the volume is

$$V = S.L$$

So

$$G = \sqrt{\frac{\gamma Qh}{\mu SL}} = \sqrt{\frac{\gamma \ vh}{\mu \ L}} = \sqrt{\frac{g \ v \ h}{\nu \ L}} \qquad (17)$$

For lamellar movement, in low values of the Reynolds number the head loss can be calculated by the Kozeny formula:

$$h = \frac{5 \nu v (1 - \varepsilon)^2}{g \varepsilon^3} \left(\frac{6}{\phi_s D}\right)^2 \times L \qquad (18)$$

Substituting (18) in (17) one gets

$$G = \sqrt{\frac{5 v (1 - \varepsilon)^2}{\varepsilon^3} \left(\frac{6}{\phi_s D}\right)^2}$$

or

$$G = 13,416 \frac{1 - \varepsilon}{\varepsilon^{1.5}} \cdot \frac{v}{\phi_s D} \qquad (19)$$

where

ε = porosity

V = apparent velocity (= Q/S)

ϕ_s = shape factor

D = particles diameter

The equation (19) shows that in spite of the head loss varies with the temperature ,this doesn't happen with the velocity gradient which for a given porous medium only depends on the velocity of the fluid throughout this porous medium, when the flow obeys the Darcy's law.

In the lamellar flow zone not under the Darcy's law, but where the flow is still lamellar, a gradual increase in inertia makes the pressure gradient deviate from the linearity of Darcy's law, and the equation of the movement can be represented by the Forchheimer[1] equation:

$$\frac{h}{L} = J = av + bv^2 \qquad (20)$$

and the velocity gradient is calculated by the equation

$$G = \sqrt{\frac{g}{\nu}\, v\, J} \qquad (21)$$

with J determined by (20), in which the coefficients a and b can be evaluated by the characteristics of the granular material.

The following table with data took from P. Basak presents values of a and b determined by many researchers for some granular media characteristics:

Table 2.1. Values of a and b for different granular materials

Diamter in mm	porosity %	a, in s/cm	b, in s/cm^2	source
2.86	43.0	0.135	0.072	Rao and Suresh (1970
2.86	42.3	0.225	0.088	
2.86	40.3	0.340	0.40	
4.04	38.4	0.075	0.053	
4.04	36.7	0.105	0.078	
5.5	37.2	0.043	0.043	
5.5	35.6	0.075	0.055	
5.5	34.6	0.105	0.078	
5.5	33.34	0.230	0.380	Rao and Suresh (1972)
4.40	35.11	0.720	0.480	
2.86	39.48	0.520	0.640	
2.0		0.1904	0.2174	Dufgeon (1966)
11.0		0.0115	0.0162	Recalculated
12.0		0.0189	0.0262	by Tyagi and
19.0		0.0082	0.0145	Todd (1970)
40.0		0.0024	0.0051	
84.0		0.00064	0.0015	
19		0.0104	0.0127	Volker (1969)
4.8		0.1514	0.0825	Volker (1975)
3.18	42	0.288	0.093	Niranjan (1973)
	46	0.300	0.101	
	41.2	0.300	0.103	
6.36	43.5	0.06	0.042	
	40.8	0.08	0.048	
	36	0.10	0.067	
11.15	43.0	0.016	0.026	
	38.0	0.076	0.041	
	34.0	0.16	0.054	
17.5	46.5	0.01	0.0102	
	41.5	0.02	0.0105	
	36.0	0.035	0.0173	
23.8	44.7	0.005	0.004	
	40.5	0.005	0.00824	
	35.5	0.007	0.0148	

Diameter in mm	porosity %	a, in s/cm	b, in s/cm^2	source
33.3	50	0.008	0.0021	
	46.6	0.01	0.0029	
	43.0	0.055	0.004	
46.6	50	0.002	0.001	
	46.5	0.004	0.0019	
	41.6	0.028	0.00372	
2.58		0.694	0.165	Ahmed (1967)
5.50		0.00232	0.00055	Sastry (1976)
8.15		0.00450	0.000576	
14.70		0.00500	0.00188	
21.00		0.00393	0.000825	

2.5. Flocculation Constant K

The precise methods to determine the value of K use the counting of the initial particles and after a flocculation time t_m, the number of particles already agglomerated N_m.

However in a water treatment plant operation or design, the instrument used to evaluate the process efficiency is the turbidimeter, being useful to admit a proportionality between the measured turbidity and the number of particles, even though it is cientifically not precise.

Being so, knowing No and Nm in turbidity terms, one can determine K through the jar-tests, once G and T are established and m = 1 in the equation (10) from which one can take

$$K = \frac{1 - (N_m / N_o)}{GT}$$

The constant K is proportional to the fraction in volume of the particles in the water to be treated, included the doses of coagulant necessary to its destabilization. Since the coagulant dosing is a turbidity function, one can conclude that K is not exactly a constant but that it varies with the turbidity of the raw water, according to a law possibly in the same way as the coagulant dosing, that is to say, logarithmic or exponential, as the turbidity and the coagulant dosing are usually related.

In the determination of the parameter K, it was considered' in jar tests $\eta = 1,0$. In this way all the values of η indirectly found in real flocculation tanks, through the previous determination of K, represent relative values, since' one doesn't know exactly how are the values of η in the ' jar tests.

In reality, one can count with few information concerning ' to the magnitude order of η in operation of the real scale. The data obtained from laboratory tests by Hahn and Stumm [8] in the coagulation of silica with alum, resulted in

$$\eta = 0,011, \text{ with } G = 10 \ s^{-1}$$

The efficiency factor at a given value of G, must be inversely proportional to the flocculation time, as in

$$\eta = a + \frac{m}{T}$$

where a and m are constants.

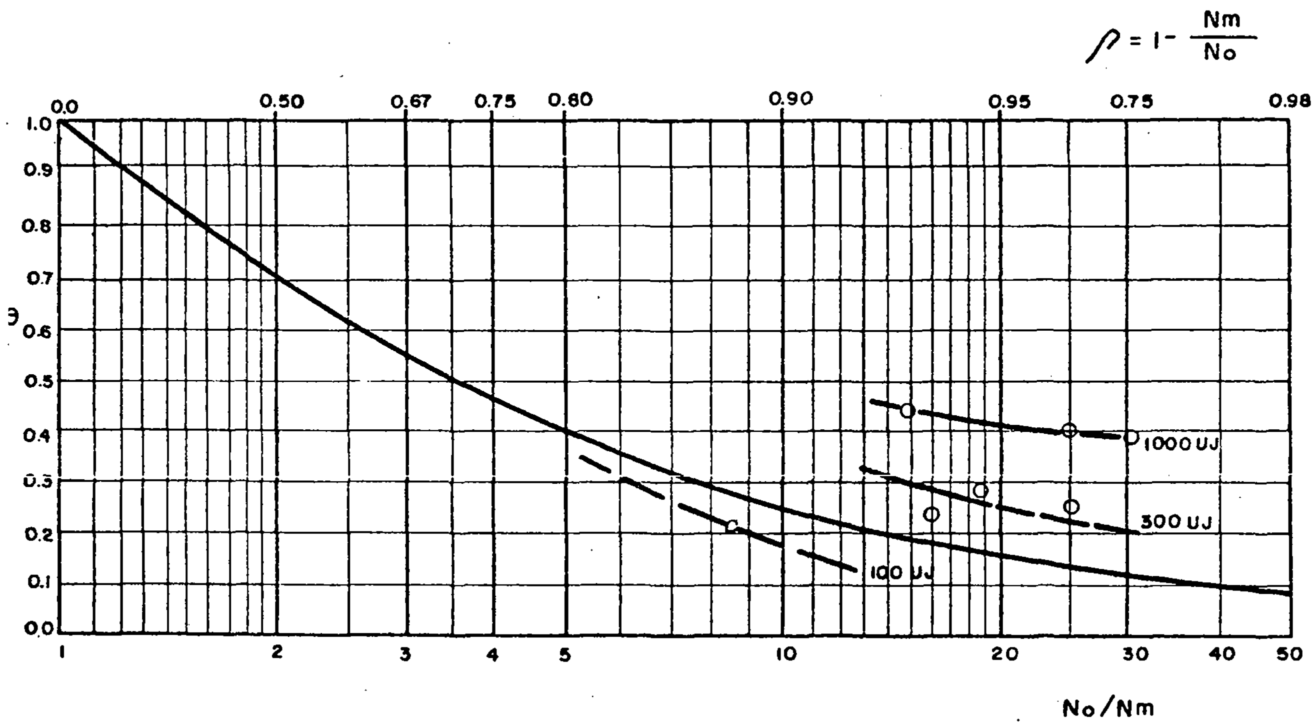


FIG. 2.1 RELATIVE TIME $^T/_\theta$ vs. TURBIDITY REMOVAL

BIBLIOGRAPHY

(1) - Camp,T.R. - "Floculation and Flocculation Basins" - Trans. ASCE, 1953.

(2) - Carlier, M. - "Hydraulique Generale et Appliquee"-Eyrolles Editeus, 1972

(3) - Harris.H.S.W.J. Kaufman and R.B. Krone. "Orthokinetic Flocculation in Water Purification".
J. Environ. Eng. Div.ASCE nº 92 - 1968.

(4) - Argaman, Y e Kaufman, W.J. - "Turbulence in Orthokinetic Flocculation" - SERL Report nº 68-5, University of California.

(5) - Sanches,G. - "Sedimentadores de Sólidos de Contacto de Flujo Ascendente"- Simpósio sobre novos métodos de tratamento de água- Assuncion, Paraguay, 1972.

(6) - Badrinath,S.D.- "An Economical Upflow Clarification for Surface Waters with Low Turbidity". Journal of Indian Water Works Association, set. 1977.

(7) - Riddick, T.M. -"Role of the Zeta Potencial in Coagulation Involving Hydroux Oxides"- Journal of the Technical Association of the Pulp and Paper Industry, jan. 1964.

(8) - Weber.W"-Physicochnical Processes for Water Quality Control Wiley Interscience- 1972.

(9) - Sancks R.L.-"Water Treatment Plant Design for the Praticing Engineer"- An Arboor Science, 1979.

(10) - Basak, P.-" Steady Non-Darcian Seepage Through Embankments Journal of the Irr. and Drain. Division,IR 4, Decembet - 1976.

GRANULAR MEDIUM FLOCCULATORS: EXPERIENCES IN A PILOT PLANT

III . GRANULAR MEDIUM FLOCCULATORS: EXPERIENCES IN A PILOT PLANT

Carlos Alfredo Richter
Rogerio de Barros Moreira

3.1. Antecedents

The tests made in the Iguaçu water treatment plant (Curitiba, Brazil) took place in a pilot installation whose purpose was not only to verify the efficiency of a granular medium flocculator but also to study the behavior, in pilot scale, of a simplified water treatment plant of low-cost for small communities.

The pilot installation has, essentially, two transparent cylindrical pipes (figures 3.1 and 3.2), with 200 mm (8") of diameter, the first one is used as a granular medium flocculator and the second one as a filter with an automatic backwashing systen.

In the flocculator it was put a 2,10m height columm, made of gravel with a 6,7 mm average diameter and 0,33 of porosity , with ascendant vertical flow.

As the figure 3.1 shows, the raw water was taken from the flash mixing chamber of the Iguaçu treatment plant and, in this way, its operation is similar to the operation in a full scale plant and allows a more accurate appreciation of the obtained results. It is, equally, submitted to the imperfections due to the operation or arisen from the physycal characteristics of the rapid mixer from the other plant.

In the first test series made from July to August 1979, when the main purpose was to prove the hidraulics of the filter backwashing some results were obtained that allowed a previous evaluation of the flocculator efficiency in the pilot plant through the comparison with the obtained results in the flocculator of the Iguaçu treatment plant and the correspondent jar-tests, in spite of the flocculation was not, at this phase, controlled in a satisfactory way.

In a second test series, in which the binomial gradient X flocculation time was varied on the purpose of verifying its proportionality with ln (No/Nf) and, so, testing the formula

$$\ln \frac{No}{Nf} = \eta\, KGT$$

previously deduced.

For this purpose the parameters K and G were determined, K through the jar tests and G evaluated through the measured' head loss, and it was also determined the relation between the initial turbidity No and the settled water turbidity' Nf, after a flocculation time T varying from 1,5 to 8 minutes.

3.2. Raw water characteristics

The hydrografic basin of the Iguaçu River, from where it was taken the water for the tests is a region of crystalline rocks. The waters that pass in soils of this type are generally acid and of low alkalinity. The turbidity is generally low, except if they pass through cultivated areas. In this case, there are sudden points of high turbidity in the rains; a fact that happens frequently in the Iguaçu treatment plant.

Turbidity

The turbidity of the raw water of the Iguaçu River varies between a minimum value about 10 NTU and rarely gets values up 300 NTU, with a mean value of 38 NTU. The turbidity value of greater frequency (modal turbidity) is found around 20 NTU. Only about 5% of the time, the turbidity is superior to 90 NTU. In 90% of the time the turbidity is inferior to 60 NTU.

Color

The color of the water "in natura" of the Iguaçu River varies normaly between the limits of 17.5 to 600 units of color. The arithmetical mean is around 100 and the value of greater frequency is 70.

Coagulation Tests

The figure 3.3 represents the doses of alum necessary to coagulate the Iguaçu River waters in various values of the turbidity. This curve, adjusted to the results of more 300 coagulation tests, resulted in the following equation:

$$D = 13.5 \ln No - 23.0$$

Where D is the optimun dosage of alum and No is the raw water turbidity. The coefficient of correlation resulted in r = 0,7 , indicating a reasonable direct correlation for the adjusted curve.

Relations between G and T of the coagulation tests

Andrew-Villegas and Letterman, in a recent study (*), demonstrated, taking the jar tests as reference, that between a given velocity gradient G and a flocculation time T, The best results are obtained when,

$$G^n T = K$$

resulting from their experiences:

$n = 2,8$
$K = 4,9 \times 10^5$, $1,9 \times 10^5$ and $0,7 \times 10^5$ for alum dosis of 10 mg/l, 25 mg/l and 50 mg/l. respectively.

In a series of jar tests in the Iguaçu water treatment plant, with the raw water turbidity 44 NTU and a dosage of alum 20 mg/l, the following results are gotten:

(*) Andrew-Villegas, R, and R.D. Letterman". "Optimizing Flocculator Power Input", J, environ. Eng. Div ASCE 102: 251-264 (1976).

Established Gradient (s^{-1})	Optimum time	
	minutes	seconds
80	10	600
60	12	720
20	60	3600
70	15	900
50	30	1800
30	45	2700

the adjusted curve to these points leads to

$$G^{1.3}T = 2,1 \times 10^5$$

or

$$G^{1.3}T = \frac{58,8 \times 10^5}{D}$$

where D is the alum dosage in mg/l.

Flocculation constant

Taking the Von Smoluchowski equation as reference, Hudson demonstrated that (*)

$$\frac{No}{Nf} = e^{\eta \emptyset GT/\pi}$$

or turning, $\eta \emptyset/\pi = K$:

$$\frac{No}{Nf} = e^{KGT}$$

(*) Hudson,H.E- "Physical Aspects of Flocculation", J.A. Water Works Ass. 57: 885-892 (1965).

where

η = factor of efficiency in the collision between particles

$\emptyset$ = proportion of flocs volume in the water

Nf = particles that do not flocculate after a time T

No = initial concentration of particles

T = flocculation time

The proposed formula to presents the flocculation in granular medium is identical to the Hudson formula, despite its original deducting by Hudson was for conventional flocculators. Its pratical verification involves the knowledge of the factor K, here denominated flocculation constant.

In the pratical experience, it is valid to admit the proportionality between Nf and No and the correspondent values of turbidity, because in the design or operation of a plant one will consider the turbidity measured values according to the traditional methods and not by the particles counting.

So, since the velocity gradient and the time of flocculation in a jar test are established, one can easily evaluate the coefficient K, through the formula

$$K = \frac{1 - \frac{Nf}{No}}{\frac{Nf}{No} \quad GT}$$

where

No = raw water turbidity

Nf = remaining turbidity of the settled water, after a time T of flocculation, in seconds.

G = velocity gradient of the jar test, seg^{-1}.

This can be verified through a jar test under controlled conditions, as in figure 3.4, in which the flocculation time was variable. If the obtained results are coherent, the parameter K can be evaluated through other jar tests already made or to be made.

From figure 3.4:

a) Flocculation period: 10 minutes (600 seconds).

$G = 30\ s^{-1}$
$N_o = 52$
$N = 6.1$

$$K = \frac{1 - \frac{6.1}{52}}{\frac{6.1}{52} \times 30 \times 600} = 4.2 \times 10^{-4}$$

b) $t = 15$ min, $N = 3,3$
$K = 5,5 \times 10^{-4}$

c) $t = 20$ min, $N = 2,7$
$K = 5,1 \times 10^{-4}$

d) $t = 30$ min, $N = 1,8$
$K = 5,2 \times 10^{-4}$

e) $t = 45$ min, $N = 1,8$
$K = 5,2 \times 10^{-4}$

f) $t = 60$ min, $N = 1,25$
$K = 3,8 \times 10^{-4}$

Except for the values (a) and (f), the other values are sufficiently consistent. (a) can be explained because there was not a sufficient flocculation time yet and (f) because it happened erosion and breaking of flocs due to a flocculation time too long.

So, with the results of the routine flocculation tests of the Iguaçu water treatment plant in Curitiba, the values of K were calculated and some curves were adjusted in about 300 pairs of values (K,No).
The curve that showed the greatest correlation coefficient '

was the potencial one, and the following correlation was obtained (figure 3.5):

$$K = 1,92 \times 10^{-5} \quad No^{0,8}$$

where

K = flocculation constant
No = raw water turbidity (NTU).

resulting from it a correlation coefficient equal to 0,8.

3.3. Characteristics of the porous media used:

In the choice of the porous media for the flocculation, the following criteria were considered:

a) That the characteristics of the flow would stay still in lamellar flow conditions.

b) That the volume of pores would be sufficiently great in order to avoid a previous-filtering and turn the cleaning easier.

c) Easy obtaining.

For the porous medium, in the Curitiba pilot flocculator, it was used gravel with an effective size of 6 mm and with an uniformity coefficient of 1,36, both determined in a granulometric analysis using a pattern sieve series.

This meterial presents a non-uniform characteristic shape , angular, tending to an oblate ellipsoid. On the purpose of determinating its main physical and geometrical characteristics, each sample grain was measured in its largest and smallest dimension with which one determinated, previously, the effective size and the uniformity coefficient, in an amount of more than 1400 direct measurings. The following results, were verified:

Extreme values

- maximum dimension : 32 mm, in a 32 mm x 7 mm grain.
- minimum dimension : 2 mm, in a 3 mm x 2 mm grain.

Nominal diameter:

The nominal diameter calculated for each grain by the formula

$$D = 1{,}24 / \left[\frac{1{,}19}{L} + \frac{0{,}35}{l} \right]$$

had a mean geometric value 6,7 ± 0,7 mm

Elongation and eccentricity

The elongation, defined by E = L/D, being L the largest dimension of each grain, had a mean value of 1,50 ± 0,07.

One can calculate the eccentricity with the mean elongation:

$$e = \sqrt{1 - E^{-4}} = \sqrt{1 - (1{,}5)^{-4}} = 0{,}895$$

Factor of form:

The factor of form $\emptyset_S$ can be calculated through the formula

$$\emptyset_s = 4 / \left[2E^2 + \frac{\ln \left[(1+e) / (1-e)\right]}{e\ E^4} \right]$$

resulting in:

$$\emptyset_s = 4 / \left[2\ (1.5)^2 + \frac{\ln\ (1+0{,}895)/(1-0{,}895)}{0.895\ (1.5)^4} \right] = 0{,}78$$

Permeability :

The permeability can be calculated taking the precedent parameters as reference, applying the following formula

$$K = \frac{\varepsilon^2\ \ \emptyset s^2\ \ D^2}{36\ KT\ (1-\varepsilon)^2\ \sigma^{\ln^4 \sigma}}$$

where

ε = porosity (ε = 0,33)

$\emptyset s$ = 0,78

D = 0,67 mm (geometric mean)

σ = 0,7 mm (pattern deviation)

K = non- dimensional constant that depends on the shape of the transverse section to the flow. It varies between 2 and 3. For a porous medium not consolidated K $\cong$ 2,36.

T = Tortuosity approximately egual to 2, for porous medium non-consolidated.

results in

$$K = \frac{(0,33)^3 \times (0,78)^2 \quad (0,67)^2}{36 \times 2,36 \times 2 \times (0,67)^2 \times 0,7^{\ln 0,7}} = 113 \times 10^{-6}$$

3.4 Velocity Gradients

The determination of the velocity gradients in an hydraulic' flocculator and, in this class one includes the porous medium flocculator, involves the knowledge of head losses. In the other hand, these head losses depend on the flow conditions, lamellar turbulent or transitional. In addition, it is important to know "a priori", taking the characteristics of the material used as reference, a general equation of the flow, that would be applicable to any kind of porous medium, for the purpose of future designs.

It is already known that the Darcy's law is valid only for relatively restrict flow conditions, at low values of the Reynolds number, and it has to be substituted by another law, as the following form

$$J = aV + bV^2$$

due to Forchheimer, for higher values of Reynolds numbers , but still in conditions of lamellar flow.

The transition from the lamellar regime of Darcy's law $J = \alpha V$

to the complet turbulence regime correspondent to a law of the form $J = \beta V^2$, is gradual, obeying the Forchheimer equation, with the viscosity force and inertia forces acting simultaneosly.

The coefficients a and b of the Forchheimer equation can be estimated by considering the granulometric characteristics of the material through the following expressions:

$$a = \frac{0.162\ (1-\varepsilon)^2}{\phi^2 D^2\ \varepsilon^3} \cdot \nu$$

$$b = \frac{0.018\ (1-\varepsilon)}{\phi\ D\ \varepsilon^3}$$

with ν = kinematic viscosity

In the following paragraphs one evaluates the various formulas usually used in the head loss calculus in the flow throughout the porous media with the measured results, taking a curve interpolated to the measured values as reference to determinate the velocity gradient.

The aparent velocity $V = Q/A$ varied between 0,19 and 0,83 cm/s; flocculation times corresponding in the 2,10 m high and 0,20 diameter columm, respectively from 6,0 to 1,4 minutes.

In this interval, the Reynolds number defined for porous media with $R = \frac{VK^{1/2}}{\nu}$, where K is the permeability, varies between 0,18 and 0,80 approximately.

In the figure (3.6) are reproduced the various representative points of velocity value pairs and head loss measured to which it has been interpolated a curve by the least square method, resulting in

$$J = 0{,}045\ V + 0{,}224\ V^2.$$

where J is the Hydraulic gradient and V is the aparent velocity in cm/s.

In the figure 3.7 one compares the measured values of the head loss considering the aparent velocity, with the various equations usually used in the calculus of the flow in porous media. The equation of Rose, used in the calculus of head loss in filters, leads to very high results. The equation of Kozeny is very precise till velocities of 0,3 cm/s, but is gives progressively lower values in relation to the observed ones as the velocity passes this value of 0,3 cm/s.

The equation that is more approximated of the values obtained within the interval studied is that one of Forchheimer ; however, in the calculus of the coefficients a and b, one must take the effective size, instead of the mean diameter.

The reason for considering the effective size is based in the fact, confirmed by Hazen, that in the flow in a non-uniform granular medium the small grains that are interposed between the larger ones have more influence, defining, then the effective size as that one in uniform medium that would produce the same head loss of the sample.

Over 0,5 cm/s, the Forchheimer equation begins to move away from the real values, resulting in values progressively lower. However, within the flocculation times and velocities that will be able to be used in the designs, the Forchheimer equation with the coefficients a and b, determined by only taking the grains geometric characteristics of the porous medium and the effetive size of the sample, leads to very approximated values of the observed ones, therefore, being possible to be used in designs within these limitations.

The velocity gradients calculated, then, through the formula

$$G = \sqrt{\frac{\gamma}{\mu} \cdot \frac{vJ}{\varepsilon}}$$

result in the represented values in the figure 3.8

3.5. Obtained results:

The experiences results confirm the proposed theroretical model and can be summarized in an extraordinary efficiency in a very short time.

The figure 3.9 represents the obtained results in the pilot flocculator, in turbidity removal terms, compared to the obtained results in the Iguaçu plant flocculator and the correspondent jar tests. In this test series, the flocculation time in the pilot flocculator varied from 1,5 to about 8,0 minutes, with a mean value around 2,8 minutes (170 seconds) and the mean velocity gradient at 85 s^{-1} resulting in Camp number GT = 14.500.

The jar tests were made in a commercial equipment with the stirres adapted to produce gradients in function of the rotation velocity, according to the Camp calibration curve and the normal procedures (15 minutes of flocculation at 30-40 rpm).

The Iguaçu treatment plant has oscillatory mechanical flocculators of the "Ribbon Flocculator" type, not shared in series, with a flocculation time from 20 to 30 minutes, and a velocity gradient about 15 s^{-1}.

The efficiency of a flocculator in the granular medium is demonstrated by these results: being superior to the mechanical flocculators and to the jar-tests. For instance, in only 2 minutes and 50 seconds of mean period of flocculation the granular medium flocculator obtained turbidity removals as those indicated in the following table, in comparison with the mechanical flocculator at 25 minutes of flocculation time and the jar-test at 15 minutes of flocculation.

Raw Water Turbidity	Turbidity Removal (%)		
	Granular Flocculator	"Jar-tests"	Mechanical Flocculator
20	80	85	70
50	93	92	72
100	96	95	93
200	97	97	96

The color removal in the granular medium flocculator seems to follow the same results of the coagulation tests, obeying to a variation law as that one indicated in figure 3.10, and having its efficiency determinated only in function of the optimum coagulation pH.

The made experiences demonstrate that the efficiency factor in the flocculation, previously defined through

$$\eta = \ln (No/Nf) \ / \ KGT$$

diminishes as the raw water turbidity increases. This fact is confirmed through the obtained results, in $\eta = 3,96/No^{0,58}$, at a flocculation time of about 3,0 min and G = 85 s^{-1}.

It is a common understanding that the efficiency in the flocculation increases as the turbidity increases, a fact that apparently would have been demonstrated in the last table. In a real way the percentage of the turbidity removal and the factor of efficiency in the flocculation are all different things. In the following table, we calculate the removal percentage that the Iguaçu River water would have in a granular flocculator, with a constant efficiency factor equal to 0,68 (for No = 20), with GT = 14.500.

N_o	$K(x10^{-4})$	KGT	ln(No/Nf) / KGT	No/Nf	Theoretical Removal (1-Nf/No)x100	Observed Removal %
20	2.1	3.0	2.07	7.92	88	88
50	4.4	6.4	4.34	76.6	98.7	93
100	7.6	11.0	7.49	1796.5	99.9	97

One can verify, therefore, that the theoretical removal at a constant efficiency factor increases more rapidly than the verified removal, indicating a decrease in the efficiency factor.

In a second test series, one tried to make a relation between the efficiency and the flocculation time and obtained the results demonstrated in the included table.

In this case, it seems that there has been no influence of the flocculation time upon the efficiency, a fact not aparent perhaps due to the relative small variability of the times used in the tests, between 1,4 and 8,5 minutes.

In any way, one shows the importance of the Camp number, GT, as a determinant in the efficiency of the flocculation, as one can see from the made experiences. In fact, one can obtain similar results with a constant GT.

TABLE 3 . 1 EFFICIENCY FACTOR IN THE FLOCCULATION IN GRANULAR MEDIUM

IGUAÇU RIVER RAW WATER - CURITIBA

Flocculation Time		Head Loss	G (s^{-1})	Turbidity (UJT)		K $(x10^{-4})$	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{K G T}$
Minutes	Seconds			In-Natura No	Settled Nf					
1,37	82	43	230	45	6,0	4,0	7,54	7,5	2,015	0,27
1,39	83	40	220	45	3,0	4,0	7,54	1,5	2,708	0,36
1,40	84	39	215	23	3,0	2,4	4,33	7,7	2,037	0,47
1,43	86	38	210	26	4,2	2,6	4,70	6,19	1,823	0,39
1,45	87	35	200	24	4,0	2,4	4,17	6,0	1,792	0,43
1,50	90	34	195	25	7,0	2,5	4,39	3,57	1,273	0,29
1,50	90	33	195	30	4,8	3,0	5,27	6,25	1,833	0,35
1,50	90	37	195	24	4,0	2,4	4,21	6,0	1,79	0,43
1,5	90	34	195	22	5,0	2,3	4,04	4,4	1,482	0,37
1,52	91	35	195	28	6,1	2,4	4,26	4,59	1,524	0,36
1,55	93	33	185	37	6,5	3,5	6,02	5,69	1,74	0,29
1,6	96	31	180	24	3,5	2,4	4,15	6,86	1,925	0,46
1,6	96	31	180	28	6,0	2,8	4,32	4,7	1,540	0,36
1,6	96	31	180	23	5,0	2,4	4,15	4,6	1,526	0,32
1,65	99	30	175	38	8,0	2,9	5,02	4,75	1,558	0,31
1,68	101	29	170	71	5,6	5,8	9,96	12,7	2,540	0,26
1,7	102	30	170	24	4,5	2,4	4,16	5,33	1,674	0,40
1,7	102	30	170	30	5,0	3,0	5,20	6,0	1,792	0,34
1,7	102	29	170	34	6,0	3,3	5,61	5,7	1735	0,31

TABLE 3 . 1 EFFICIENCY FACTOR IN THE FLOCCULATION IN GRANULAR MEDIUM

IGUAÇU RIVER RAW WATER - CURITIBA

Flocculation Time		Head Loss	G (s^{-1})	Turbidity (UJT)		K ($x10^{-4}$)	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{KGT}$
Minutes	Seconds			In-Natura No	Settled Nf					
1,37	82	43	230	45	6,0	4,0	7,54	7,5	2,015	0,27
1,39	83	40	220	45	3,0	4,0	7,54	1,5	2,708	0,36
1,40	84	39	215	23	3,0	2,4	4,33	7,7	2,037	0 47
1,43	86	38	210	26	4,2	2,6	4,70	6,19	1,823	0,39
1,45	87	35	200	24	4,0	2,4	4,17	6,0	1,792	0,43
1,50	90	34	195	25	7,0	2,5	4,39	3,57	1,273	0,29
1,50	90	33	195	30	4,8	3,0	5,27	6,25	1,833	0,35
1,50	90	37	195	24	4,0	2,4	4,21	6,0	1,79	0,43
1,5	90	34	195	22	5,0	2,3	4,04	4,4	1,482	0,37
1,52	91	35	195	28	6,1	2,4	4,26	4,59	1,524	0,36
1,55	93	33	185	37	6,5	3,5	6,02	5,69	1,74	0,29
1,6	96	31	180	24	3,5	2,4	4,15	6,86	1,925	0,46
1,6	96	31	180	28	6,0	2,8	4,32	4,7	1,540	0,36
1,6	96	31	180	23	5,0	2,4	4,15	4,6	1,526	0,32
1,65	99	30	175	33	8,0	2,9	5,02	4,75	1,558	0,31
1,68	101	29	170	71	5,6	5,8	9,96	12,7	2,540	0,26
1,7	102	30	170	24	4,5	2,4	4,16	5,33	1,674	0,40
1,7	102	30	170	30	5,0	3,0	5,20	6,0	1,792	0,34
1,7	102	29	170	34	6,0	3,3	5,61	5,7	1735	0,31

TABLE 3 . 1 CONTINUATION

Flocculation Time		Head Loss	G (s^{-1})	Turbidity UJT		K $(x10^{-4})$	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{KGT}$
Minutes	Secondes			In-Natura No	Settled Nf					
3,3	198	10	70	29	2,7	2,8	3,88	10,7	2,374	0,61
3,45	207	8	63	33	6,0	3,1	4,04	5,5	1,705	0,42
3,74	224	8	58	60	4,8	5,1	6,63	12,5	2,526	0,38
3,90	234	7	56	37	1,8	3,5	4,59	20,6	3,02	0,66
4,0	240	7	53,5	30	4,2	3,0	3,85	7,14	1,966	0,51
4,01	246	7	53	36	2,9	3,4	4,43	12,4	2,519	0,57
4,38	263	6	47	37	2,0	3,5	4,32	18,5	2,92	0,68
5,06	304	6	40	24	6,0	2,4	2,92	4,0	1,39	0,47
5,10	306	6	39	60	4,0	5,1	6,09	15,0	2,708	0,44
5,2	312	6	38	24	5,9	2,4	2,85	4,1	1,4	0,49
6,2	372	3	30	42	1,5	3,8	4,24	28,0	3,332	0,79
6,4	384	4	28	36	5,6	3,4	3,66	6,43	1,86	0,51
6,7	402	4	27	30	6,0	4,0	4,34	5,0	1,61	0,37
8,53	512	3	19	74	4,0	6,0	5,84	18,5	2,918	0,48

TABLE 3 . 1 CONTINUATION

Flocculation Time		Head Loss	G (s^{-1})	Turbidity UJT		K ($x10^{-4}$)	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{KGT}$
Minutes	Secondes			In-Natura No	Settled Nf					
3,3	198	10	70	29	2,7	2,8	3,88	10,7	2,374	0,61
3,45	207	8	63	33	6,0	3,1	4,04	5,5	1,705	0,42
3,74	224	8	58	60	4,8	5,1	6,63	12,5	2,526	0,38
3,90	234	7	56	37	1,8	3,5	4,59	20,6	3,02	0,66
4,0	240	7	53,5	30	4,2	3,0	3,85	7,14	1,966	0,51
4,01	246	7	53	36	2,9	3,4	4,43	12,4	2,519	0,57
4,38	263	6	47	37	2,0	3,5	4,32	18,5	2,92	0,68
5,06	304	6	40	24	6,0	2,4	2,92	4,0	1,39	0,47
5,10	306	6	39	60	4,0	5,1	6,09	15,0	2,708	0,44
5,2	312	6	38	24	5,9	2,4	2,85	4,1	1,4	0,49
6,2	372	3	30	42	1,5	3,8	4,24	28,0	3,332	0,79
6,4	384	4	28	36	5,6	3,4	3,66	6,43	1,86	0,51
6,7	402	4	27	30	6,0	4,0	4,34	5,0	1,61	0,37
8,53	512	3	19	74	4,0	6,0	5,84	18,5	2,918	0,48

TABLE 3 . 1 CONTINUATION

Flocculation Time		Head Loss	G (s^{-1})	Turbidity (UJT)		K ($x10^{-4}$)	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{KGT}$
Minutes	Seconds			In-Natura No	Settled Nf					
1,75	105	32	175	25	2,5	2,5	4,59	10,0	2,303	0,50
1,75	105	27	160	23	5,0	2,4	4,03	4,6	1,526	0,38
1,78	107	26	155	34	2,4	3,3	5,47	14,17	2,651	0,48
1,78	107	26	155	50	7,0	4,4	7,3	7,14	1,966	0,27
1,82	108	24	150	43	5,2	3,8	6,16	8,26	2,113	0,34
1,85	111	25	150	36	5,0	3,4	5,66	7,20	1,97	0,33
1,92	115	23	140	35	5,0	3,3	5,31	7,0	1,946	0,37
2,00	120	22	135	34	6,8	3,3	5,35	5,0	1,609	0,30
2,0	120	22	134	56	5,8	4,8	7,72	9,7	2,267	0,29
2,06	124	21	130	58	4,0	4,8	7,74	13,5	2,603	0,34
2,10	126	20	125	40	6,0	3,7	5,83	6,67	1,90	0,33
2,16	129	20	120	30	6,0	4,0	6,19	5,0	1,61	0,26
2,5	150	15	100	60	5,0	5,1	6,68	1,2	2,485	0,37
2,5	150	15	100	34	5,8	3,3	4,95	5,9	1,769	0,36
2,5	150	15	100	28	3,2	2,8	4,20	8,75	2,169	0,52
2,55	153	15	98	32	5,0	3,1	4,50	6,4	1,856	0,41
2,55	153	14	97	50	5,0	4,3	6,38	10,0	2,303	0,36
2,60	156	14	94	62	3,0	5,2	7,63	20,7	3,029	0,40
3,0	180	11	78	37	1,8	3,4	4,77	20,6	3,023	0,63

TABLE 3 . 1 CONTINUATION

Flocculation Time		Head Loss	G (s^{-1})	Turbidity (UJT)		K $(x10^{-4})$	KGT	$\frac{No}{Nf}$	$Ln(\frac{No}{Nf})$	$\eta = \frac{Ln(\frac{No}{Nf})}{KGT}$
Minutes	Seconds			In-Natura No	Settled Nf					
1,75	105	32	175	25	2,5	2,5	4,59	10,0	2,303	0,50
1,75	105	27	160	23	5,0	2,4	4,03	4,6	1,526	0,38
1,78	107	26	155	34	2,4	3,3	5,47	14,17	2,651	0,48
1,78	107	26	155	50	7,0	4,4	7,3	7,14	1,966	0,27
1,82	108	24	150	43	5,2	3,8	6,16	8,26	2,113	0,34
1,85	111	25	150	36	5,0	3,4	5,66	7,20	1,97	0,33
1,92	115	23	140	35	5,0	3,3	5,31	7,0	1,946	0,37
2,00	120	22	135	34	6,8	3,3	5,35	5,0	1,609	0,30
2,0	120	22	134	56	5,8	4,8	7,72	9,7	2,267	0,29
2,06	124	21	130	58	4,0	4,8	7,74	13,5	2,603	0,34
2,10	126	20	125	40	6,0	3,7	5,83	6,67	1,90	0,33
2,16	129	20	120	30	6,0	4,0	6,19	5,0	1,61	0,26
2,5	150	15	100	60	5,0	5,1	6,68	1,2	2,485	0,37
2,5	150	15	100	34	5,8	3,3	4,95	5,9	1,769	0,36
2,5	150	15	100	28	3,2	2,8	4,20	8,75	2,169	0,52
2,55	153	15	98	32	5,0	3,1	4,50	6,4	1,856	0,41
2,55	153	14	97	50	5,0	4,3	6,38	10,0	2,303	0,36
2,60	156	14	94	62	3,0	5,2	7,63	20,7	3,029	0,40
3,0	180	11	78	37	1,8	3,4	4,77	20,6	3,023	0,63

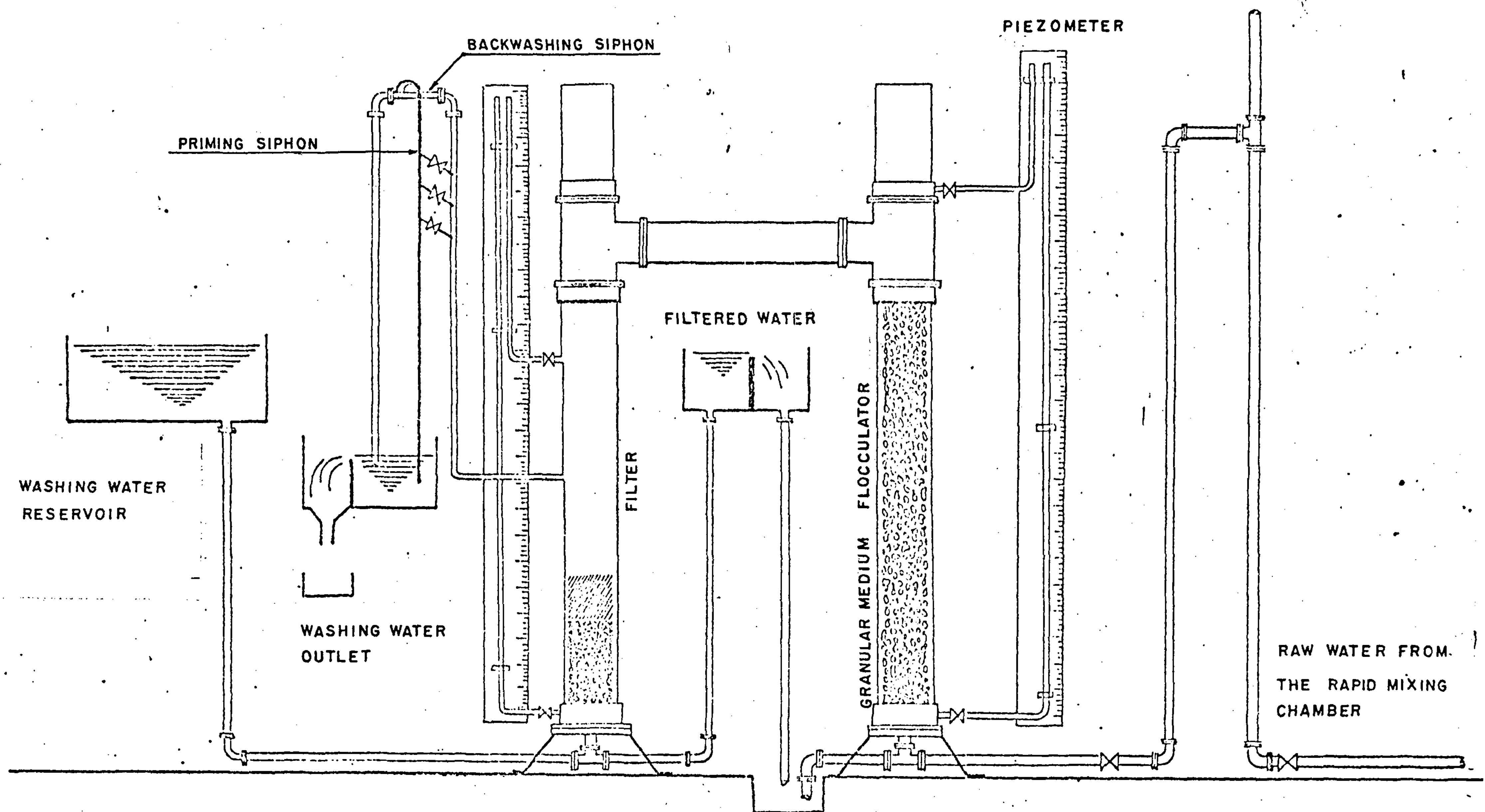


FIG. 3.1 - PILOT PLANT SKETCH

(IN THE IGUAÇU-CURITIBA PLANT)

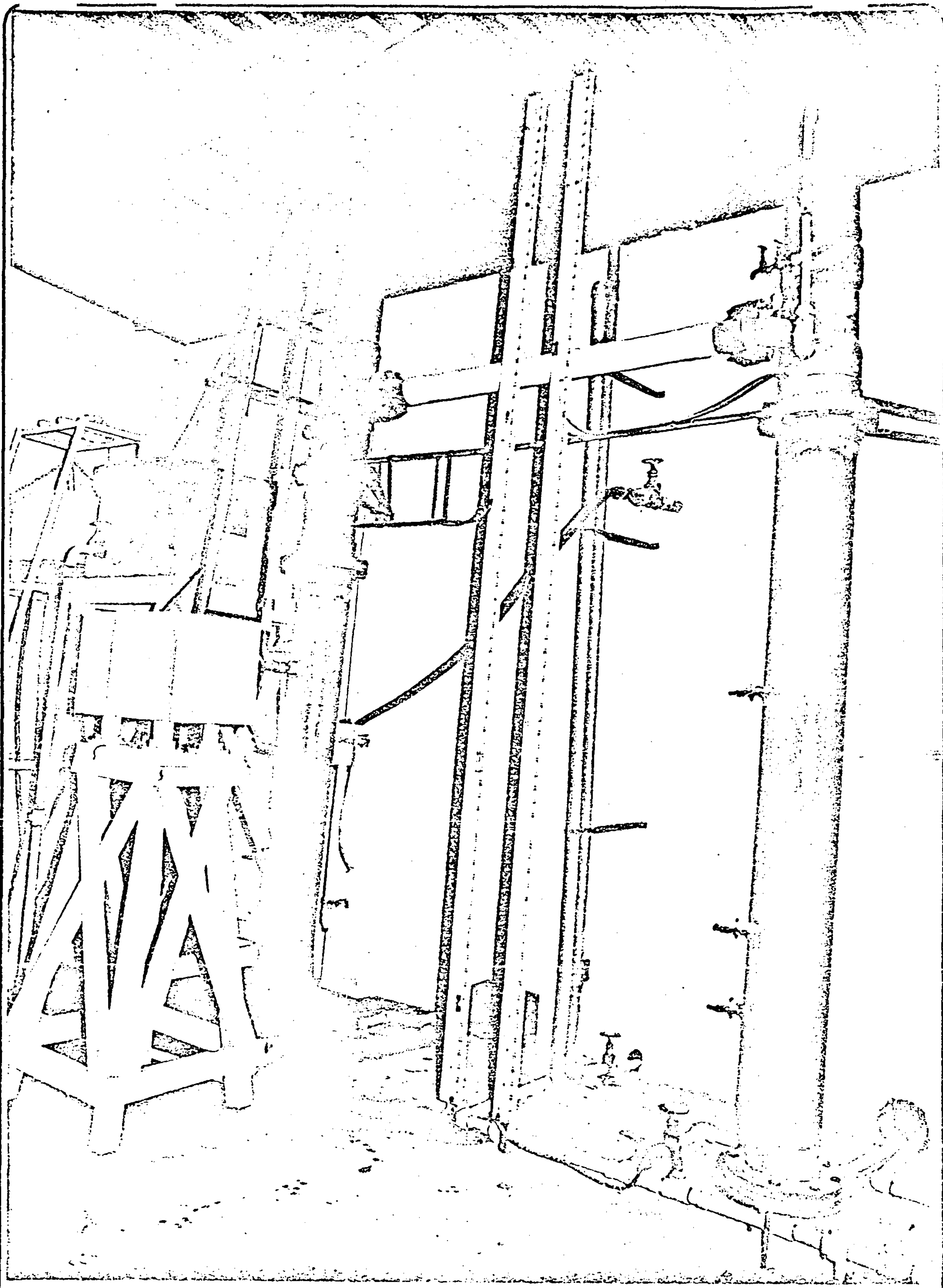


FIG. 3.2 - PILOT PLANT

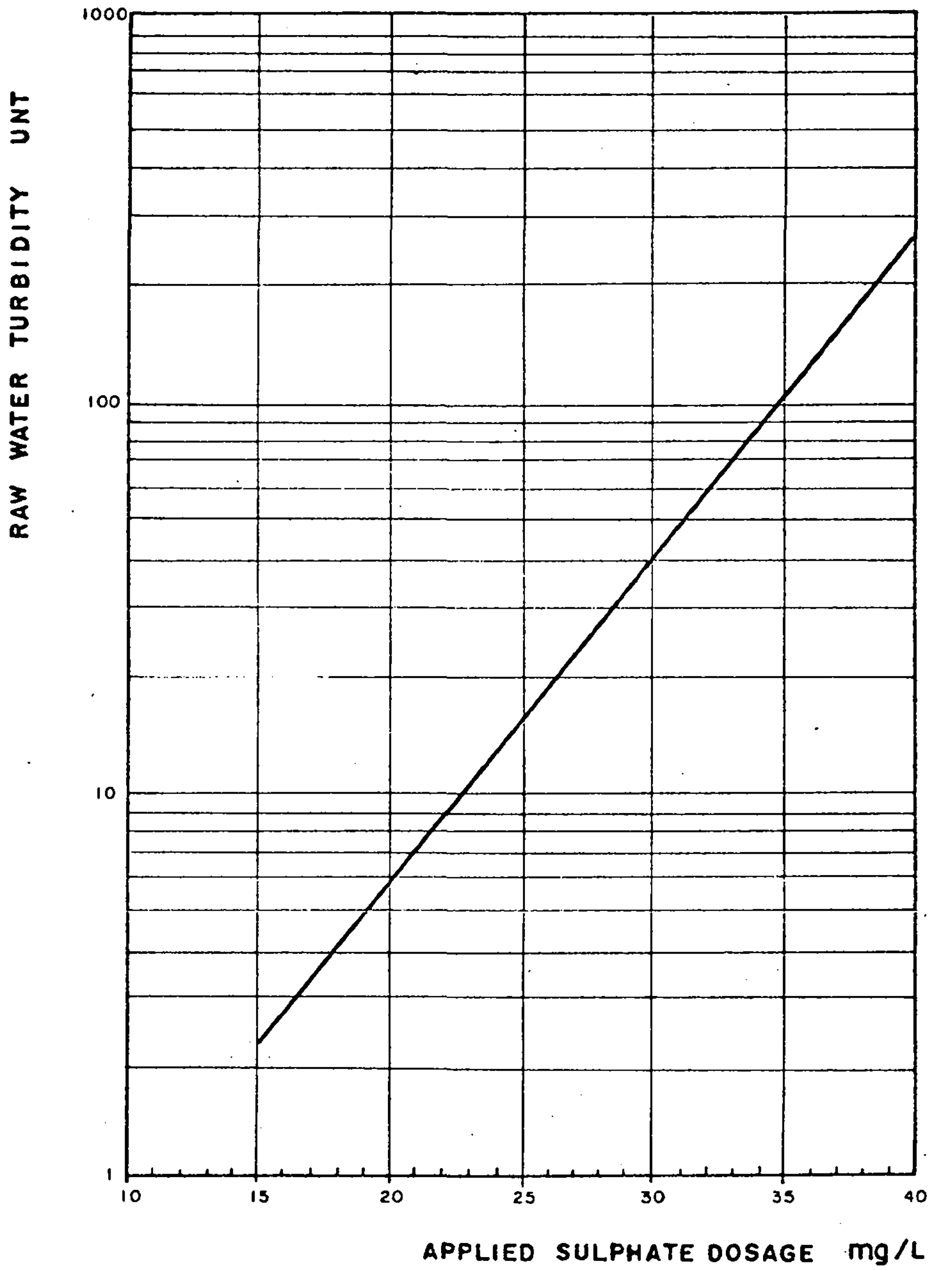
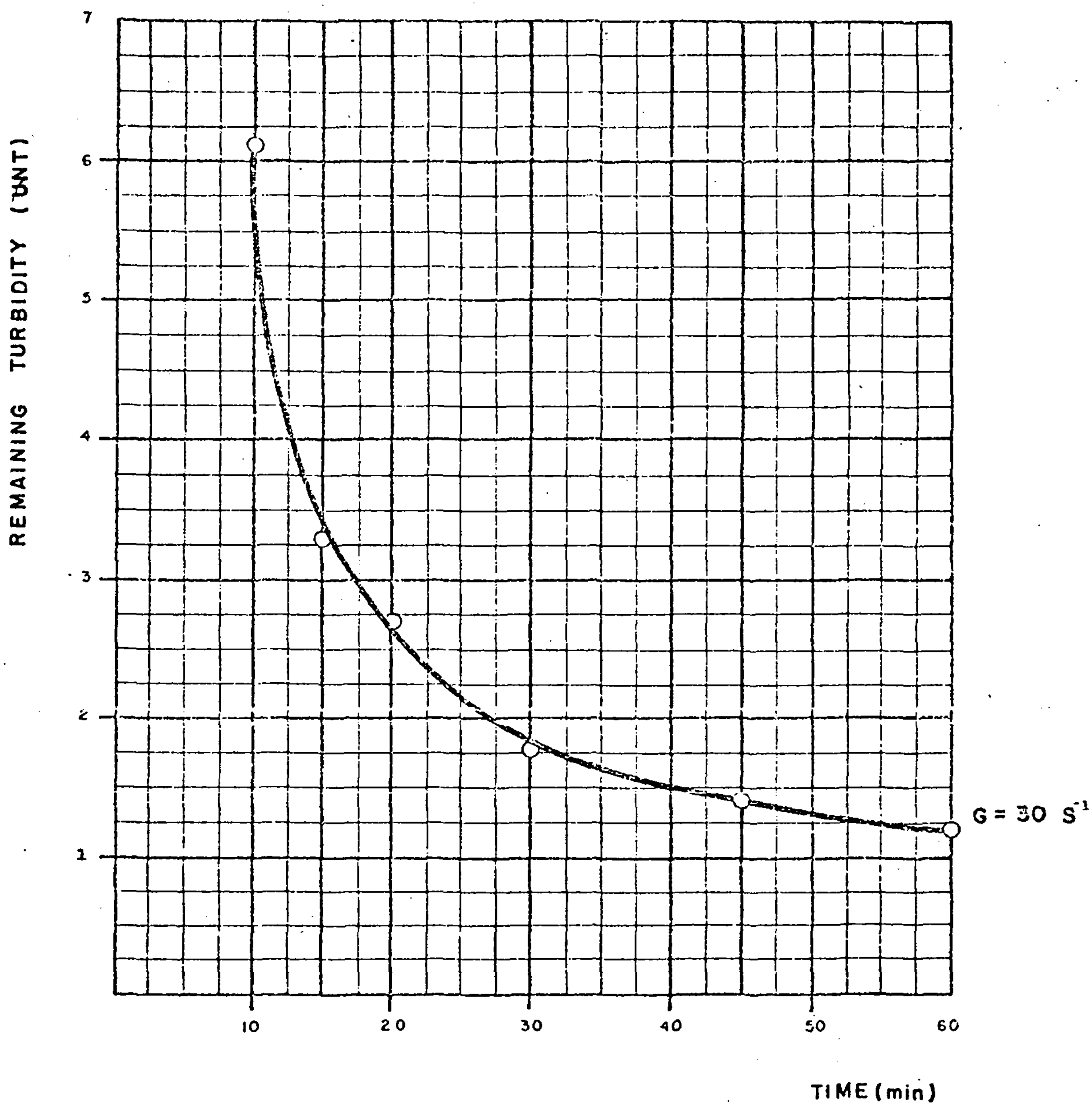


FIG. 3.3 - ALUMIMUN SULPHATE DOSES IN FUNCTION OF THE TURBIDITY VALUES IN THE IGUAÇU TREATMENT PLANT

FIG. 3.4 - IGUAÇU WATER TREATMENT PLANT

JAR TEST



RAW WATER CHARACTERISTIC:

TURBIDITY N_o = 54 UNT

ALKALINITY 12

PH 6,3

COLOR 150

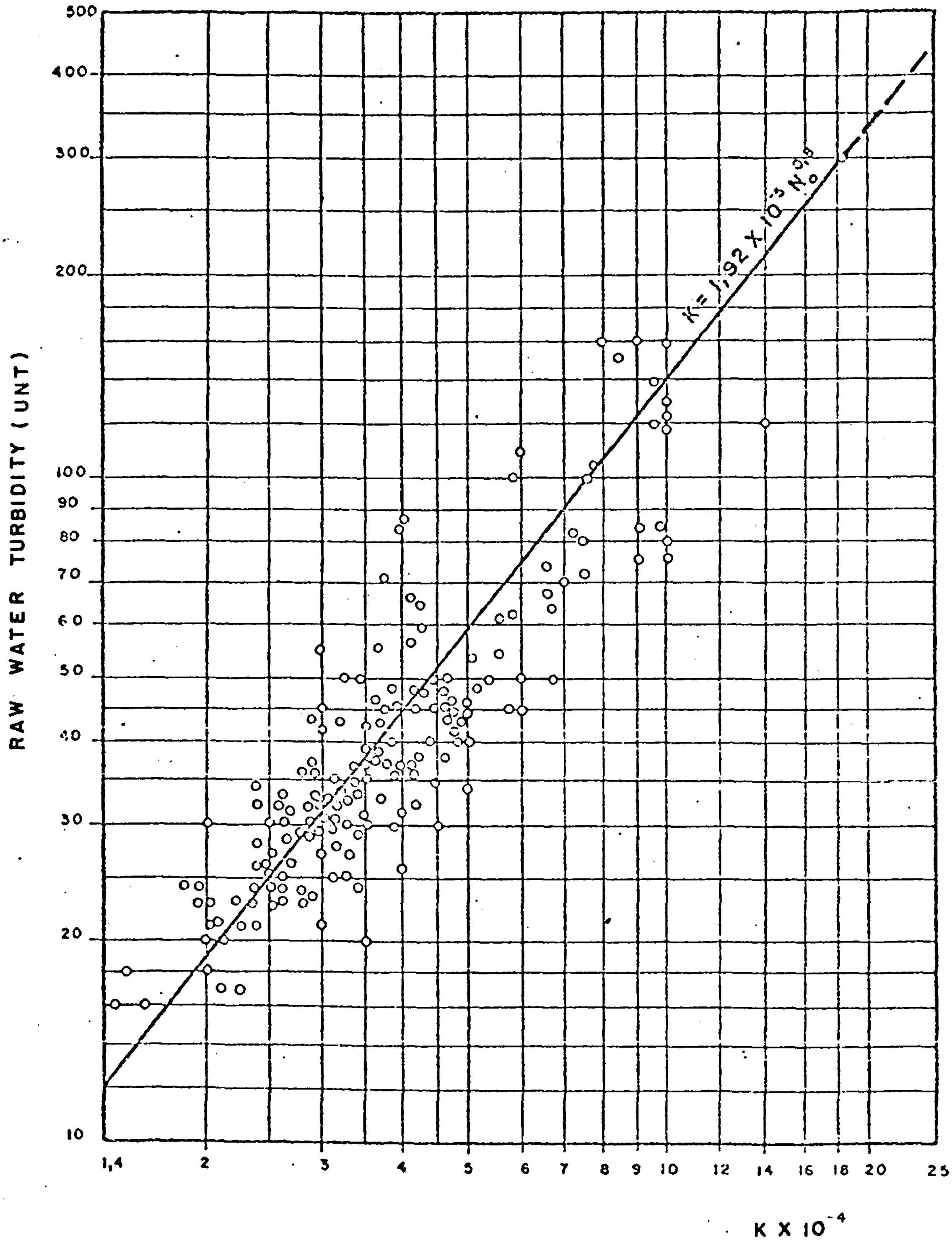


FIG. 3.5 - CORRELATION BETWEEN THE FLOCCULATION CONSTANT AND THE RAW WATER TURBIDITY

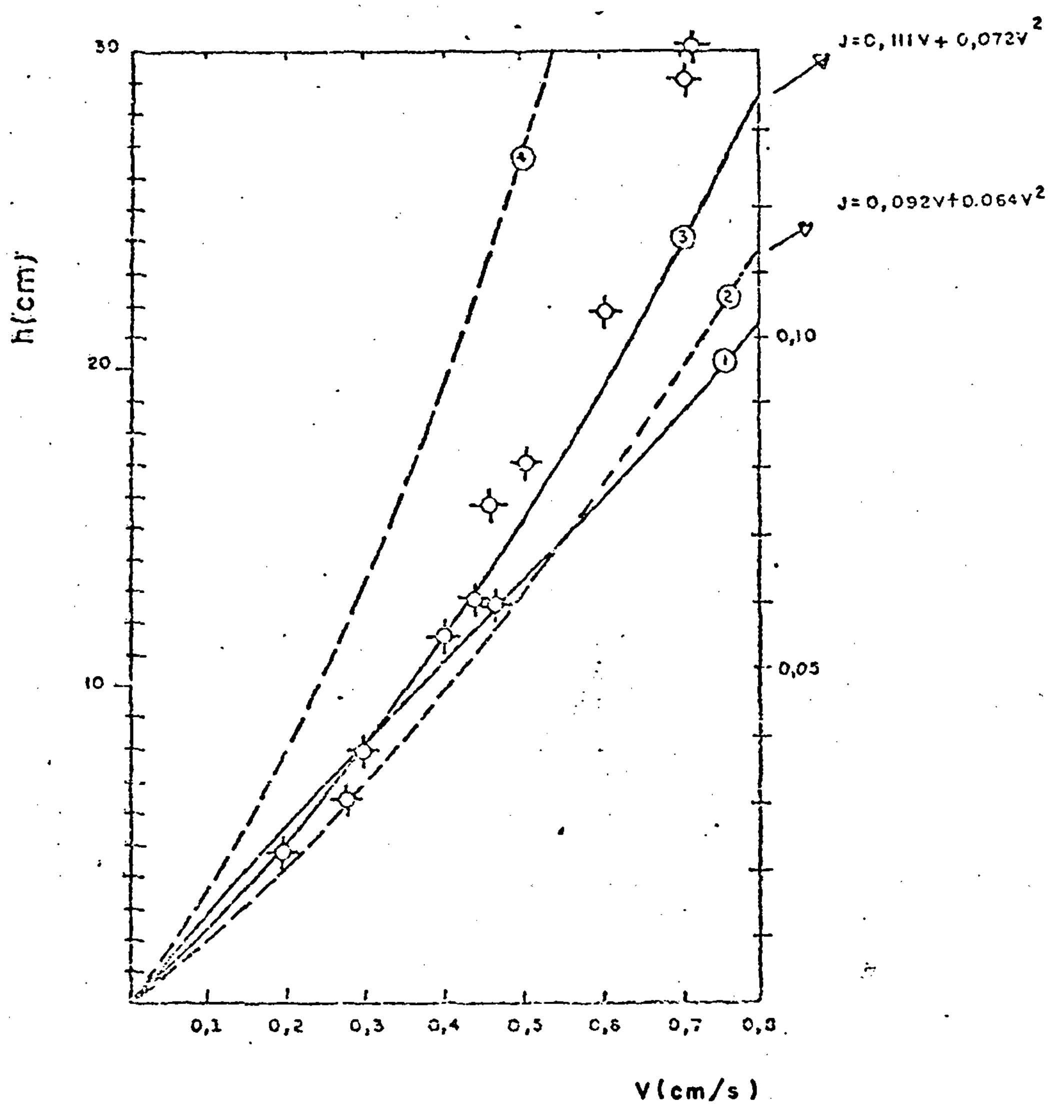
JAR-TESTS:

FLOCCULATION TIME 15 min

VELOCITY GRADIENT $G = 30\ s^{-1}$ (30 RPM)

FIG. 3.6 - HEAD LOSS IN THE POROUS MEDIUM
(PILOT FLOCCULATOR IN THE IGUAÇU PLANT)

COMPARISON BETWEEN THE THEORETICAL FORMULAS AND THE MEASURED VALUES



CURVE 1- KOZENY EQUATION: $J = \frac{5\nu}{g} \cdot \frac{(1-\mathcal{E})^2}{\mathcal{E}^3} \cdot \left(\frac{6}{\phi_s D}\right)^2$, WITH EFFECTIVE SIZE D

CURVE 2- FORCHHEIMER EQUATION $J = av + bv^2$ $a = \frac{0.162\,(1-\mathcal{E})^2}{\phi^2 D^2 \mathcal{E}^3}\nu$ AND $b = \frac{0{,}0018(1-\mathcal{E})}{\phi_s D_3 \mathcal{E}^3}$
WITH D= MEAN NOMINAL DIAMETER

CURVE 3- FORCHHEIMER EQUATION WITH a AND b CALCULATED WITH D EFFECTIVE SIZE (HAZEN)

CURVE 4- ROSE EQUATION $J = 1.067\ C\ \left(\frac{1}{\phi_s D}\right)\left(\frac{1}{\mathcal{E}^4}\right)\left(\frac{V^2}{g}\right)$, D MEAN NOMINAL DIAMETER

-◇- MEASURED VALUES

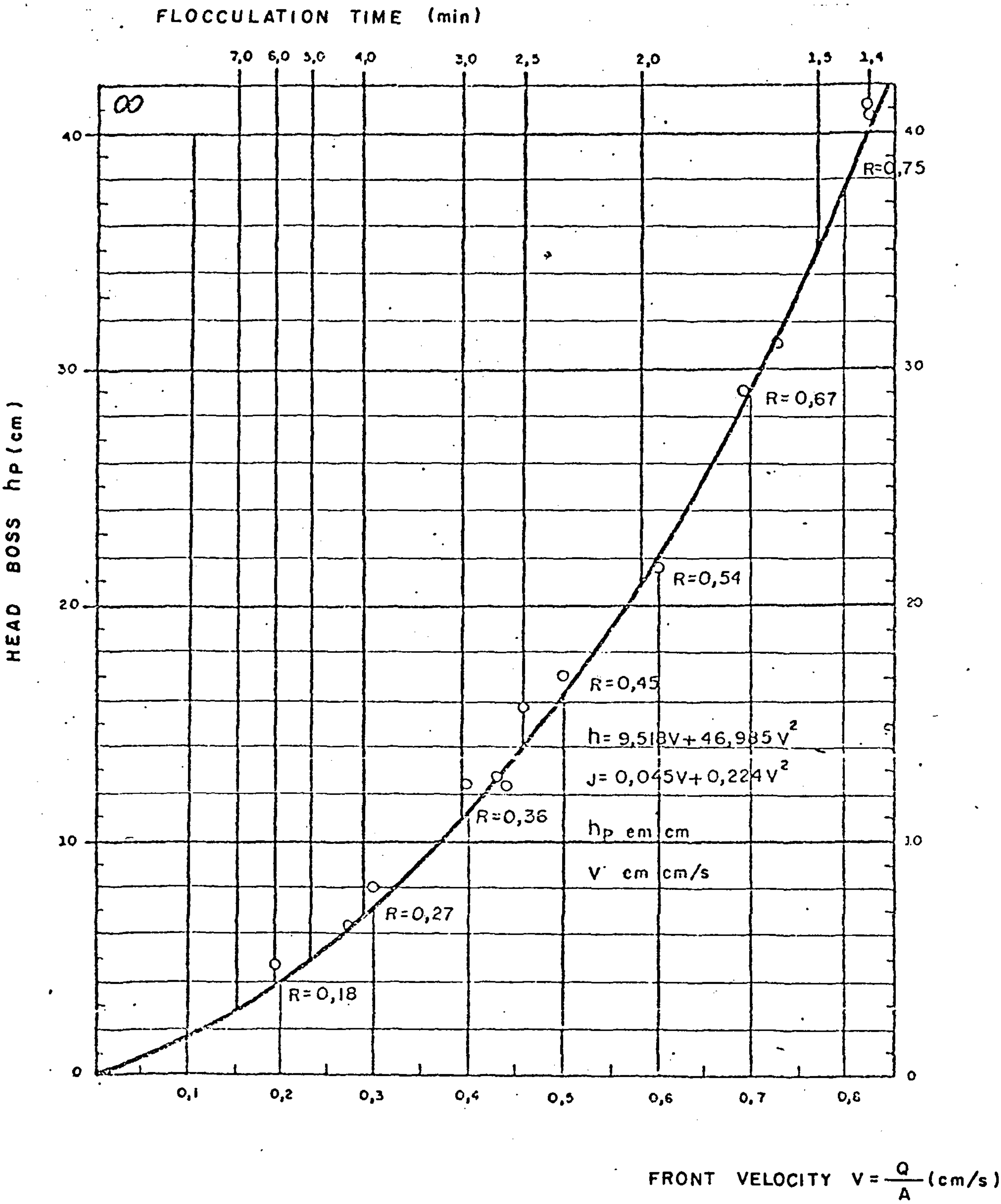


FIG.3.7 – HEAD LOSS IN THE GRANULAR MEDIUM FLOCCULATOR

MATERIAL: GRAVEL DIAMETER 4,76 A 12,7 mm

EFFECTIVE SIZE = 6,0 mm

MEAN SIZE (50%) = 6,7 mm

UNIFORMITY COEFFICIENT = 1,36

POROSITY - P_o = 0,33

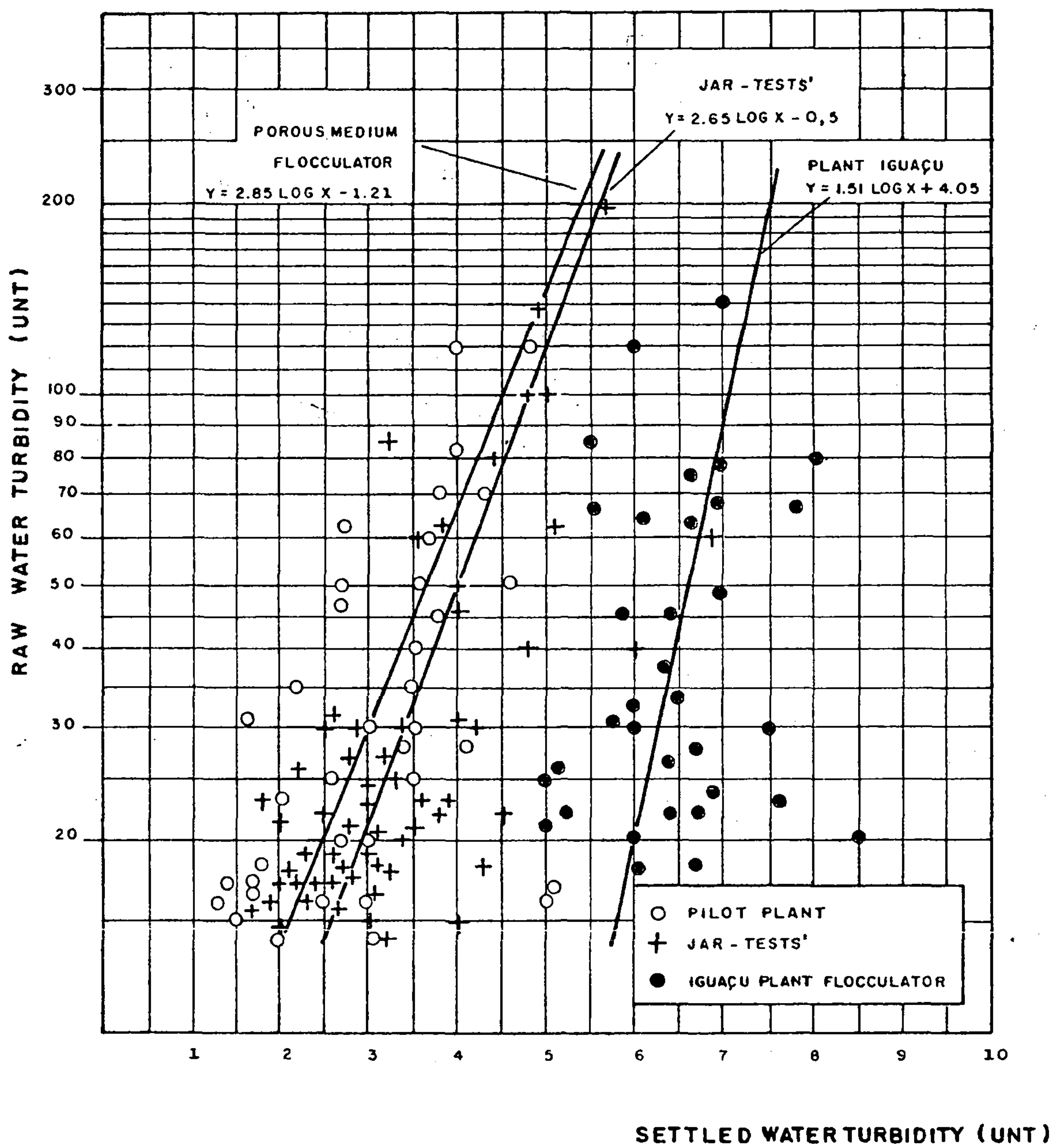


FIG.-3.9 COMPARISON BETWEEN THE RESULTS OF THE FLOCCULATOR IN GRANULAR MEDIUM

PILOT PLANT WITH THE RESULTS OF THE JAR-TESTS' AND

THE FLOCCULATOR OF THE IGUACU WATER TRATMENT PLANT

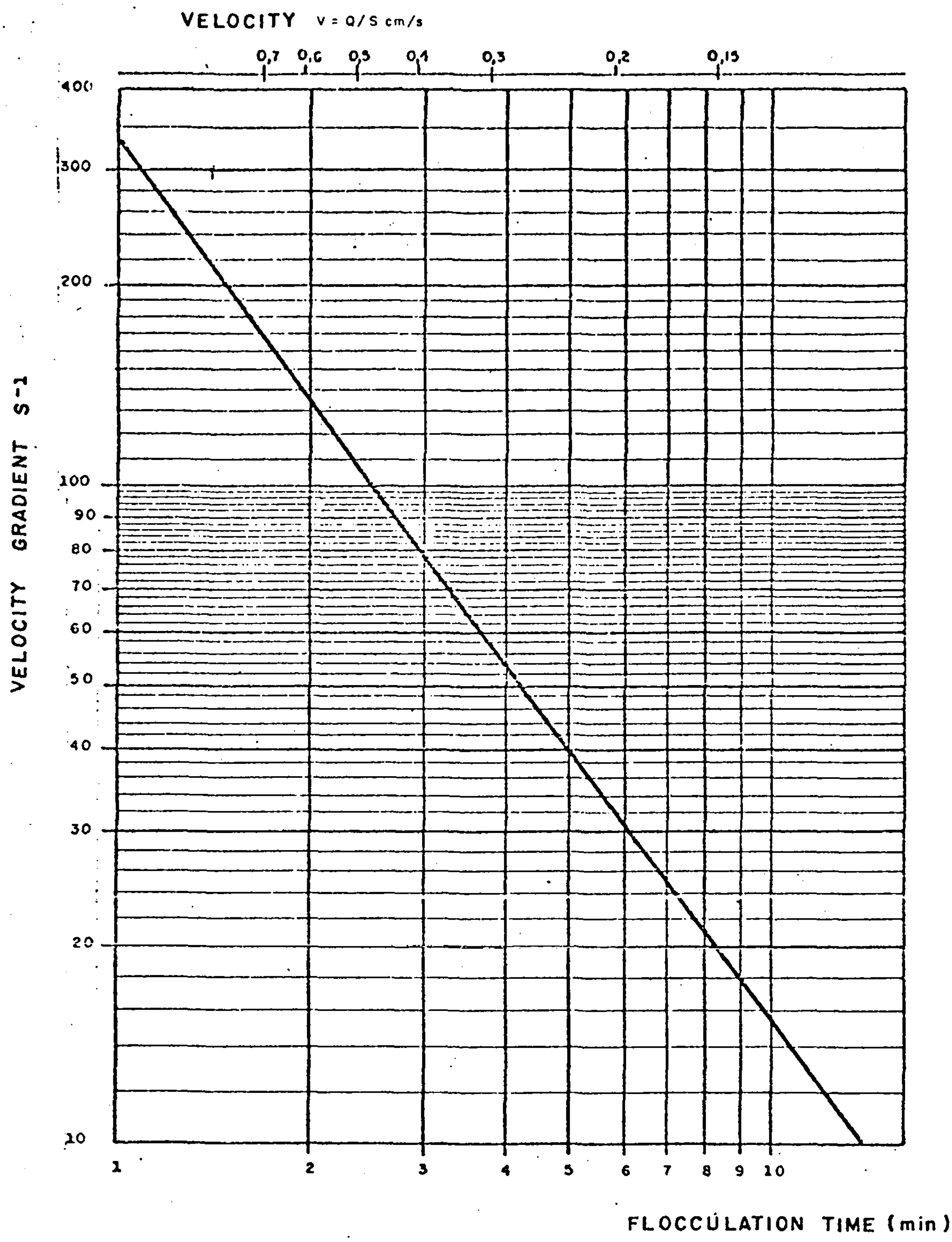


FIG.3.8-POROUS MEDIUM FLOCCULATOR IN THE IGUACU WATER TREATMENT PLANT: VELOCITY GRADIENT VERSUS FLOCCULATION TIME

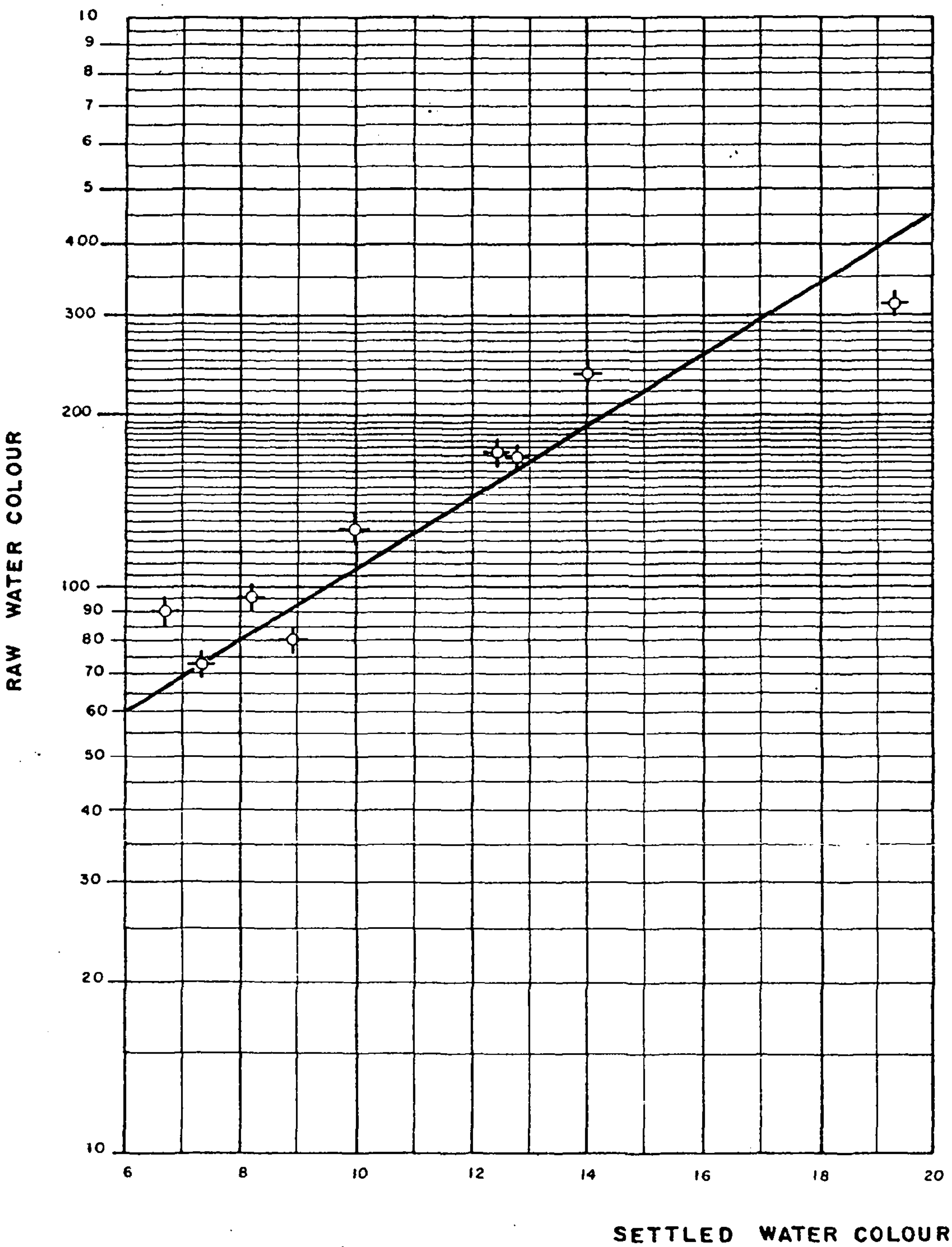


FIG. 3.10 RESULTS IN THE COLOUR REMOVAL IN THE POROUS MEDIUM FLOCCULATOR

HIDRAULICS OF THE FILTER BACKWASHING

0003-400 BLS-100 XI- 05/79-GSU/DVAL

IV - HIDRAULICS OF THE FILTER BACKWASHING

Carlos A. Richter
Clodoaldo S.Balkowiski
Alcione A.P. de Lara

4.1. Pilot Filter

The pilot filter, where have been carryed out the tests to ' prove the hydraulics of the filter backwashing, has essentially a transparente cylindrical tube (fig.4.1) with 200 mm ' diameter. The filter bed has a dual-layer of anthracite and sand with the following characteristics:

. anthracite

thickness of the layer	40 cm
effective size	0,9 to 1,0 mm
coefficient of uniformity	1,6
relative density	1,48

. sand

thickness of the layer	25 cm
effective size	0,6 mm
coefficient of uniformity	1,4
relative density	2,65

The filter bed is supported by a gravel layer with a total ' thickness of 45 cm. The gravel is graded in size between ' 1/8" and 1 1/2" from the upper layer to the bottom. The filter bottom has only one nozzle with a 1/4" orifice.

The flocculated water enters the filter through the piping ' (1) with rates varying from 120 to 700 m^3/m^2 x day, more commonly the superior values according to the rates of flow ' applied to the flocculator.
In this tests we did not have the worry in obtaining an adequate effluent quality, but we tried to reproduce the largest possible series of pilot filter backwashings in order to get an enough quantity of informations. So, we can justify the

high filtration rates and the non-inclusion of a settler between the flocculator and the filter in the pilot plant. It resulted filter runs from 5 to 8 hours.
The filtered water passing by the pipe, firstly (2) will fill the washwater reservoir, after this it will flow through the filtered water outlet in the box set on the left.(fig.4.1).

When the water level reaches the top of the priming siphon the water starts going out through the pipe (3), producing a negative head that takes the air out of the main syphon (pipe 4) making it start rapidly. Next, the washing operation starts as it was described in the section about the Fundamentals of the Project.

A piezometer was used to determine the head loss, and direct measures by the volume taken in the drain pipe (5) at regular intervals were used to determine the washing water rate of flow.

The measurement made were then compared with the theoretical calculus of the washing system, confirming the proposed model. The results were so that they permitted reasonable trust in the system operation as will as in the graphic proccess presented to the foresight of the washing hidraulics behavior.

4.2. Washing system theoretical calculus

4.2.1. Head loss

The first step to the foresight of the hydraulics behavior of the authomatic washing system by siphoning is the determination with enough approach of the head loss curve in function of the rate of flow or washing water velocity.

In the calculus that follow, the formulas and calculus processes, as well as the values of the various coeficients, were taken among the more commonly used and recommended.

(1) Head loss in the filter bottom.

The filter bottom has a 1/4" diameter orifice with the purpose of collecting the filtered water and dispersing the washwater.
At an intermediary value of the washwater velocity, 50 cm/min

the velocity in the orifice will be 829 cm/s and the Reynolds number corresponding to the orifice of 1/4" (0,635 cm), will be:

$$R_e = \frac{829 \times 0,635}{4 \times 0,011} = 1,2 \times 10^4$$

To this value it correspondes:

coefficient of velocity	C_v	= 0,97
coefficient of contraction	C_c	= 0,63
coefficient of discharge	C_d	= 0,61

The head loss is calculated by

$$h_p = (\frac{1}{C_v^2} - 1)\frac{V^2}{2\ g} = (\frac{1}{0,97^2} - 1)\frac{V^2}{2x980} = 3,2 \times 10^{-5}\ V^2$$

resulting the values included in:

Washing water velocity (cm/min)	Rate of flow (cm^3/s)	Velocity (cm/s)	Head loss (cm)
10	52	164	0,9
20	105	332	3,5
30	157	497	7,9
40	209	661	14,0
50	262	829	22,0
60	314	994	31,7
70	367	1161	43,2

(2) Head loss in the piping

The friction head losses were calculated by the Flamant equation and the minor losses were calculated with the fol - lowing coefficients (see fig. 4.1.)

Singularity

	K	Σ K
Inward projecting entrance	0,80	1,60
Valve Ø 1 1/2	0,50	0,50
Bend (90º) (5 units)	0,90	4,50
90º Elbow (3 units)	0,40	1,20
Standard tee (2 units)	0,60	1,20
Outlet piping	1,00	1,00
(Nominal inside diameter of the pipe 1 1/4" = 3,52 cm)		10,00

Minor head losses

$$h_p = 10,00 \frac{V^2}{2 \times 980} \cong 0,005\ V^2 \cong 2 \times 10^{-4} Q^2$$

Result in the following values:

Washing water velocity (cm/min)	Rate of flow (cm^3/s)	Fricton head loss (cm)	Minor head loss (cm)	Head loss (cm)
10	52	0,31	0,43	0,7
20	105	0,62	1,94	2,6
30	157	1,09	4,24	5,3
40	209	1,86	7,54	9,4
50	262	3,10	11,86	15,0
60	314	4,03	17,03	21,1
70	367	5,27	23,21	28,5

(3) Head loss in the filtering bed

The head loss in expanded beds is calculated by

$$h_p = L\ (S-1)\ \ (1-p_o)$$

were

L = thickness of the layer, cm

S = relative density of the medium

P_o = porosity

For the sand

$L = 25$ cm

$S = 2,65$

$P_o = 0,42$

$h_p = 25\ (2,65-1)\ (1-0,48) = 24$ cm

For the anthracite

$L = 40$ cm

$S = 1,45$

$P_o = 0,48$

$h_p = 40(1,45-1)\ (1-0,48) = 10$ cm

Total head loss hp = 24 + 10 = 34 cm

This loss is constant for any value of the washwater veloci - ty,over the fluidification point velocity. Till this value it is aproximately linear. For calculating the fluidification minimum velocity, we applied the formula of Amirtharajah (1970):

$$V_f = \frac{0,00381\ (d_{60})^{1.82}\ Wa\ (Wm-Wa)^{0,94}}{\nu^{0,88}}$$

where

$d_{60} = 0,6 \times 1,4 = 0,84$ mm

$\nu = 1,13$ centipoises

$Wa = 62,4\ lb/ft^3$

$Wm = 2,65 \times 62,4\ lb/ft^3$

Applying the formula above it results:

$V_f = 9\ gpm/ft^2$ or

$V_f = 35$ cm/min.

(4) Head loss in the gravel layer

The head loss is around 3 cm for each 30 cm/min of the washwater velocity.
Summing all these head losses, it results the graphic of fig. 4.2, which, being composed by the lowering-time curves, will permit *the foresight of the washwater velocity* evolution.

4.2.2. Washwater velocity evolution

In the backwashing of the filters by siphoning, one can recognize two different phases, from the moment the main siphon is activated:

(1) The water level lowering in the filter from the level in the washwater reservoir to the level of the siphon outlet weir.

(2) The water level lowering in the washwater reservoir.

In the first phase, one can disregard the parcel of discharge contributed by the washwater reservoir, as the charge for the backwashing is produced by the water level lowering in the filter because this one is much more quick than the correspondent in the washwater reservoir.
In addition to the resistance at the flow, there is the relation between the areas in the reservoir and the filter, much greater in the reservoir.

In both phases the settled water rate of flow that continues entering in the filter, is not considered.

(1) First phase:

Lowering of the water level in the filter from the same level in the washwater reservoir to the level of the siphon outled weir

The time of draining is given by

$$T = \frac{2A}{ca\sqrt{2g}} \sqrt{H}$$

$$A = \text{filter area} = \frac{\pi\ (20)^2}{4} = 314,2\ \text{cm}^2$$

a = siphon outlet area $= \frac{\pi (3,52)^2}{4} = 9,7 \text{ cm}^2$

H = water height = 2.485 - 1.745 = 0,74 m = 74 cm

c = coeff. of discharge = 0,60

$$T = \frac{2 \times 314.2 \sqrt{74}}{0,6 \times 9,7 \times \sqrt{2 \times 980}} \cong 20 \text{ s}$$

The curve of the water level lowering in the filter in function to the time is defined by the equation

$$h = \left(1 - \frac{t}{20}\right)^2 \times H \qquad \text{or}$$

$$h = \left(1 - \frac{t}{20}\right)^2 \times 74$$

We draw this curve, and beside it, the curve of backwash velocity in function to the water level (complement of the head loss). Referring the correspondent points to an auxiliary cartesian system, we determinate the curve that shows the variation of the backwashing in function of the time, resulting the graphic in fig 4.3.

(2) Second phase: Lowering of the water level in the washing water reservoir.

Here, we can not draw a parabola (t,h) as it was made in the first phase, because the head loss is variable with the square of velocity and this one is variable with the water level in the washwater reservoir.

One proceeds then to an arithmetical approximate determination, summarized in the following:

Charge (cm)		Lowering Δl	Volume (cm^3)	Velocity (cm /min)			Rate of flow	Time (min)	
Initial	Final	(cm)		Initial	Final	Average	(cm^3/min)	Δt	Σ
74	70	4	17688	48	45	46,5	14.608	1,21	1,21
70	65	5	22110	45	42	43,5	13.666	1,61	2,82
65	60	5	22110	42	37,5	39,8	12.504	1,77	4,59
60	55	5	22110	37,5	34	35,6	11.184	1,97	6,56
55	50	5	22110	34	31	32,5	10.210	2,17	8,72
50	45	5	22110	31	28	29,5	9.268	2,38	11,10
45	39	6	26532	28	25	26,5	8.325	3,18	14,28

(1) e (2) : Arbitrary values of the water level in the washing water reservoir.
(3) : Water level lowering = (1) - (2)
(4) : Volume spent in the interval = washing reservoir area x lowering = 4422 cm^2 x Δl.
(5) : Washwater velocity, taken in function of the available water height (numerically equal to the head loss) of the graphic 4.2.
(6) : Washwater rate of flow = filter area x average velocity = 314,16 x average velocity.
(7) : Time interval = volume (3) ÷ rate of flow (7)
(8) : Total time spent

Taking this data, we can draw the graphic of fig. 4.4 in the following way:

(1) In the right side, we draw the curve of water level lowering x washing time, with the values previously obtained.
(2) In the left-side, we draws the washwater velocity curve in function of the water height (taken from fig 4.2)
(3) We draws the washwater velocity curve in function to the time, referring the correspondent points to an auxiliary axle system.

The fig 4.5 summarizes the results of the figs.4.3 and 4.4, representing graphically the washwater velocity variation with the time, according to the theoretical model here presented.

4.3 Practical Verification of the backwash system

To verify the backwash system some measurements were taken of the reservoir level lowering and the washing water rate of flow in function of the time; the results are summarized in figs. 4.6 and 4.7. In these measures, we made it sure to close the filter water inlet at the moment that the siphon was activated, in a way that the entering water was not considered in the calculations.

4.3.1. First phase of the backwash

The first phase of the backwash starts when the siphon is activated. From the moment the water level in the filter reaches the auxiliary siphon head and the water starts flowing through it, the air is taken out of the main siphon, which is quickly primed and, in few seconds, it begins to drain the water contained in the filter.

In this phase that corresponds to a gradual increase in the washwater velocity with the water level lowering in the filter, we verified that the time was practically equal to the calculated one, reproducing the phenomenon presented in fig. 4.3.

4.3.2. Second phase of the backwash

The measured values of the water level and of the wash water velocity, represented in figs. 4.6 and 4.7, were respectivelly put in fig 4.8, in which we make the comparison between the observed results and the calculated values. In this figure the calculated values are represented by continuous lines and the observed results by non-continuous lines.

One can observe that the proposed theoretical model for the washwater velocity evolution is confirmed in the realized experiences with an unexpressive difference. This difference is caused by the relative lack of security in the head loss calculation which resulted in an error for excess, from a determined rate of flow.

The total behavior of the washing system by siphoning is shown in fig 4.9 where the obtained results represented by a continuous line are compared with the calculated values represented by a non-continuous line. This fig. shows better the completely adequate backwashing of the filter, thus confirming the proposed model for the hydraulics of the backwashing system.

4.3.3 Head loss in the filter backwashing

The curve of the real head charge loss in the filter backwashing was indirectly determined in fig 4.8. It was de

parted from the lowering-time curve and the washwater velocity-time curve, and then it was transported to fig. 4.10, where it is compared with the curve of the calculated head loss. Initially the real head losses are higher than the calculated ones, till a washwater velocity around 35 cm/min.
From this velocity on, they are lower. This suggests us that there is a greater initial influence due to head losses in the filter bed, which, before the fluidiffication, shall be higher, and there is a lower influence on the other head losses that shall be lower.

In fact, the angular coefficient of the head loss in the filter bed before the fluidiffication, was drawn in an arbitrary way in fig 4.2, connecting the origin of the coordinates to the point corresponding to the minimum velocity of fluidiffication.

In relation to the fluidiffication, we observed a value lower than the calculated one. At velocities so low as 25 and 30 cm/min one could still observe the filtering bed in moviment indicating certain fluidification, despite it as not sensible anymore to its expansion. Hence, the curve of head loss in the filter bed presented in fig 4.2, should have been placed a litle more to the left side.

In the other hand, the coefficients wich were taken for the minor head loss calculus give generally values somewhat higher than the real ones, explaining, in this way, the difference between the obtained results and the calculated ones.

4.3.4 Conclusion

One verified a maximum expansion of about 18% at a washwater velocity around 50 cm/min. The backwashing lasted about 20 minutes, however the time from 12 to 15 minutes is normally enough.
The obtained results permit to come to the conclusion that the filter backwash under the hydraulics point of view is perfect. One obtains a slow and gradual expansion of the filter bed in the first phase of the backwash and only after 20 seconds the maximum expansion is obtained with the maximum washwater velocity. After reaching determined water level in

the washwater reservoir the backwashing operation is interrupted with the air inlet venting the siphon. All this is done automatically and independently from the operator action, based only in hydraulic principles.

The whole model is based in a correct head loss evaluation. In the designs, therefore, one must provide a rate of flow controlling device which can be a plate with variable spacing placed at the siphon outlet pipe.
This plate will have the purpose of bringing in an additional head loss.

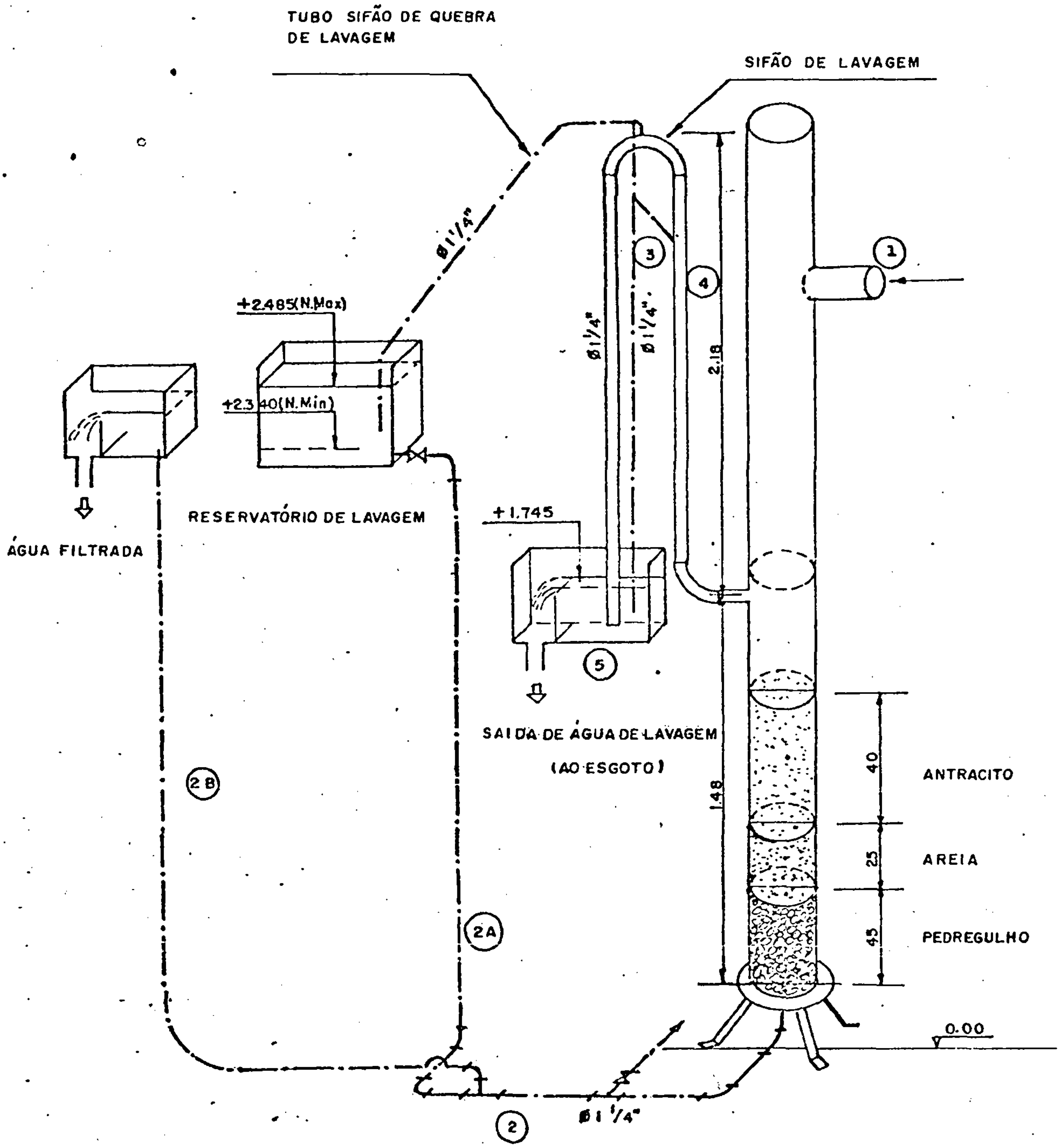


FIG. 4.1 - FILTRO AUTOMÁTICO-PILOTO

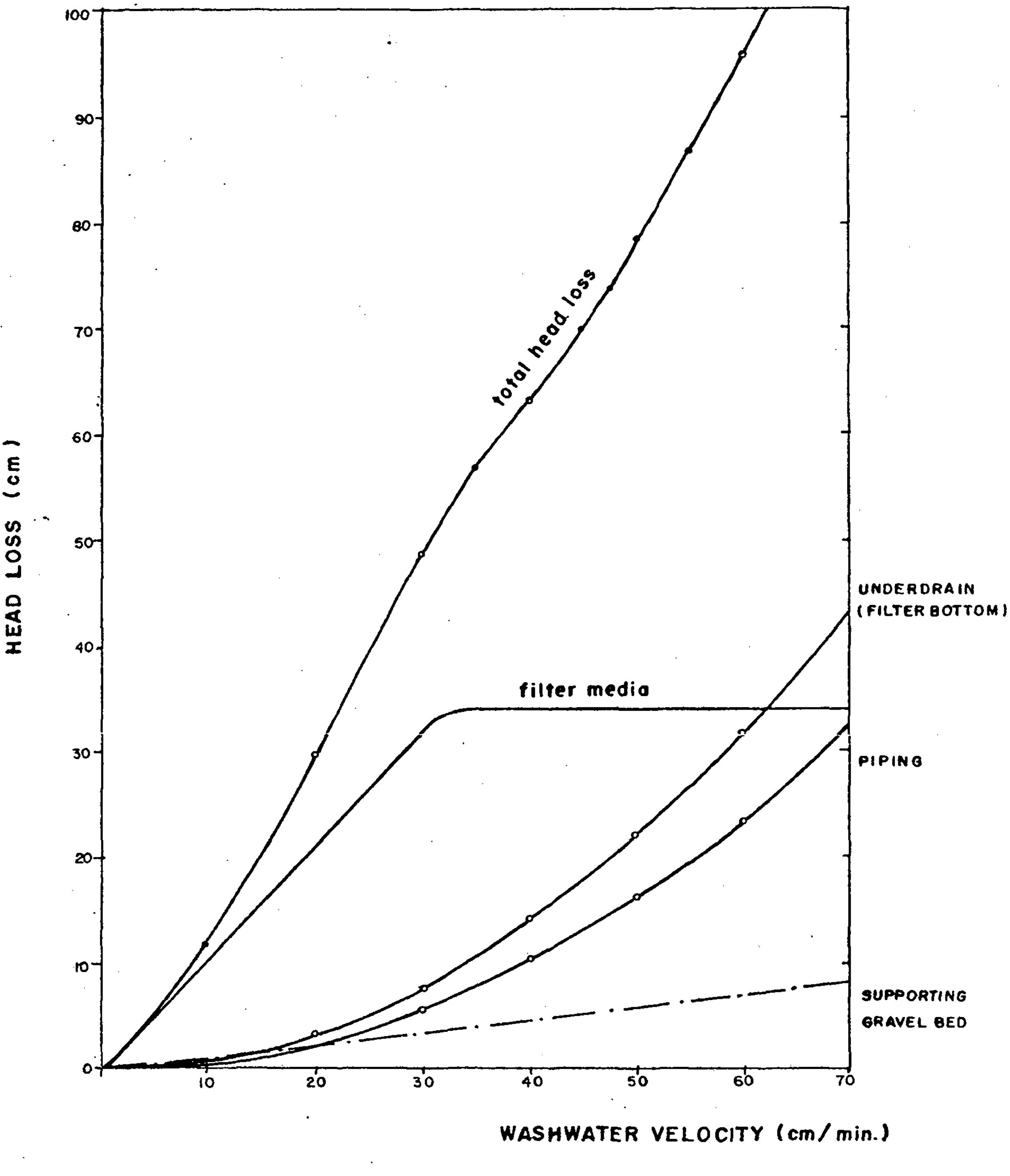


FIG. 4.2 - HEAD LOSS IN THE FILTER

(calculated values)

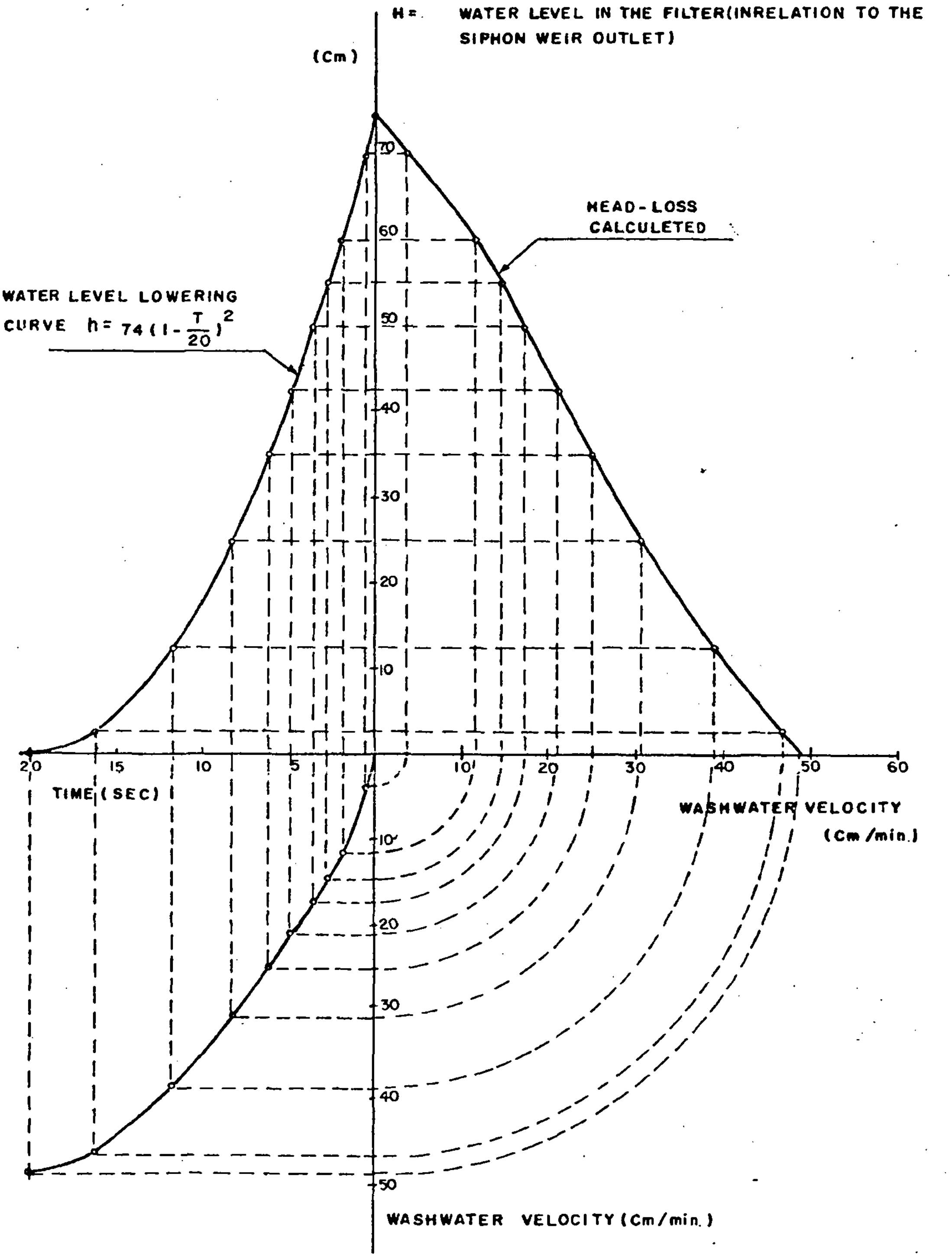


FIG. 4.3.- LOWERING OF WATER LEVEL IN THE FILTER: GRADUAL AND SLOW INCREASE OF WASHWATER VELOCITY

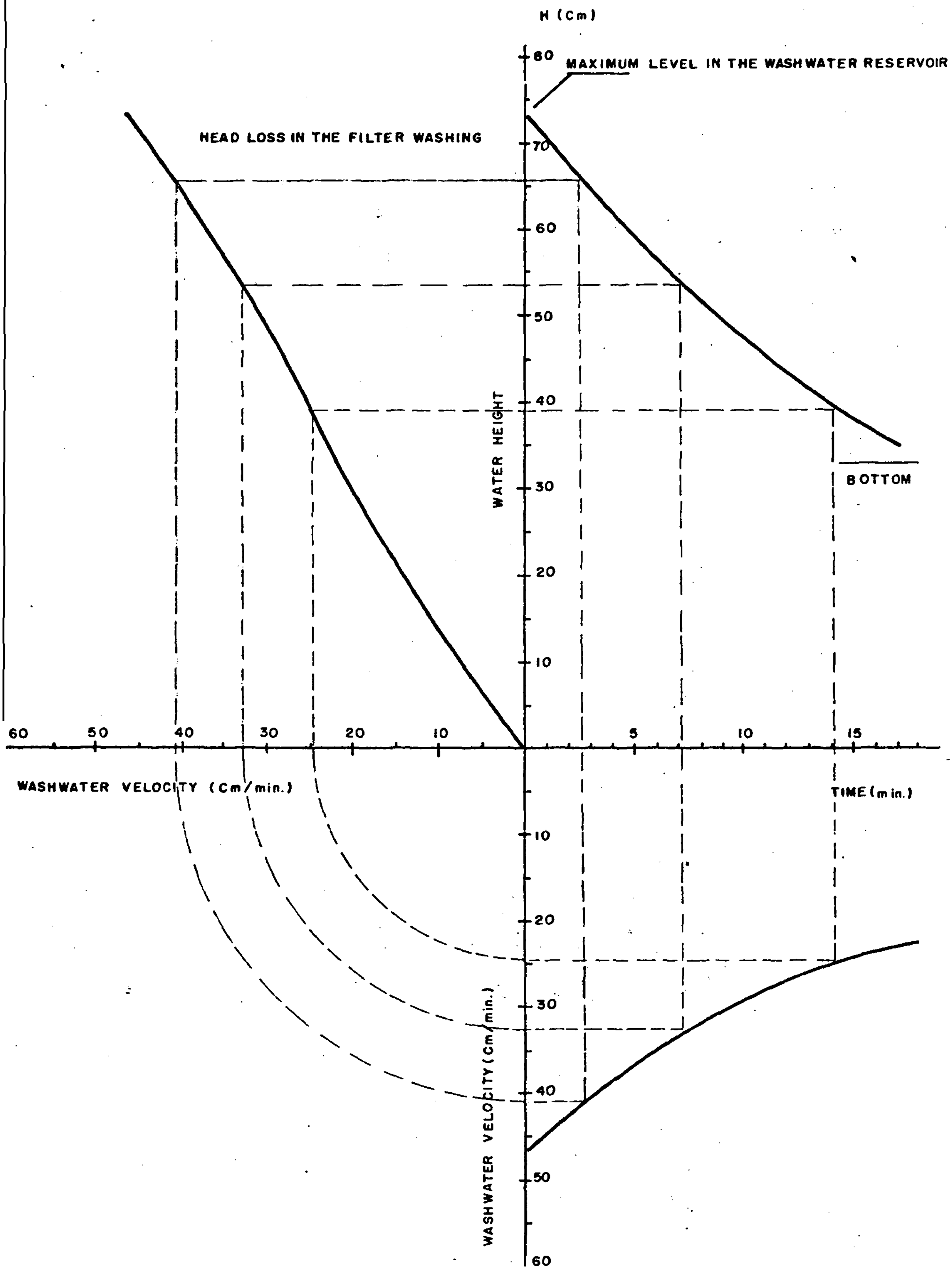


FIG. 4.4.- 2[nd] WASHING PHASE: LOWERING OF WATER LEVEL IN THE WASHWATER RESERVOIR - GRADUAL AND SLOW DECREASE OF WASHWATER VELOCITY

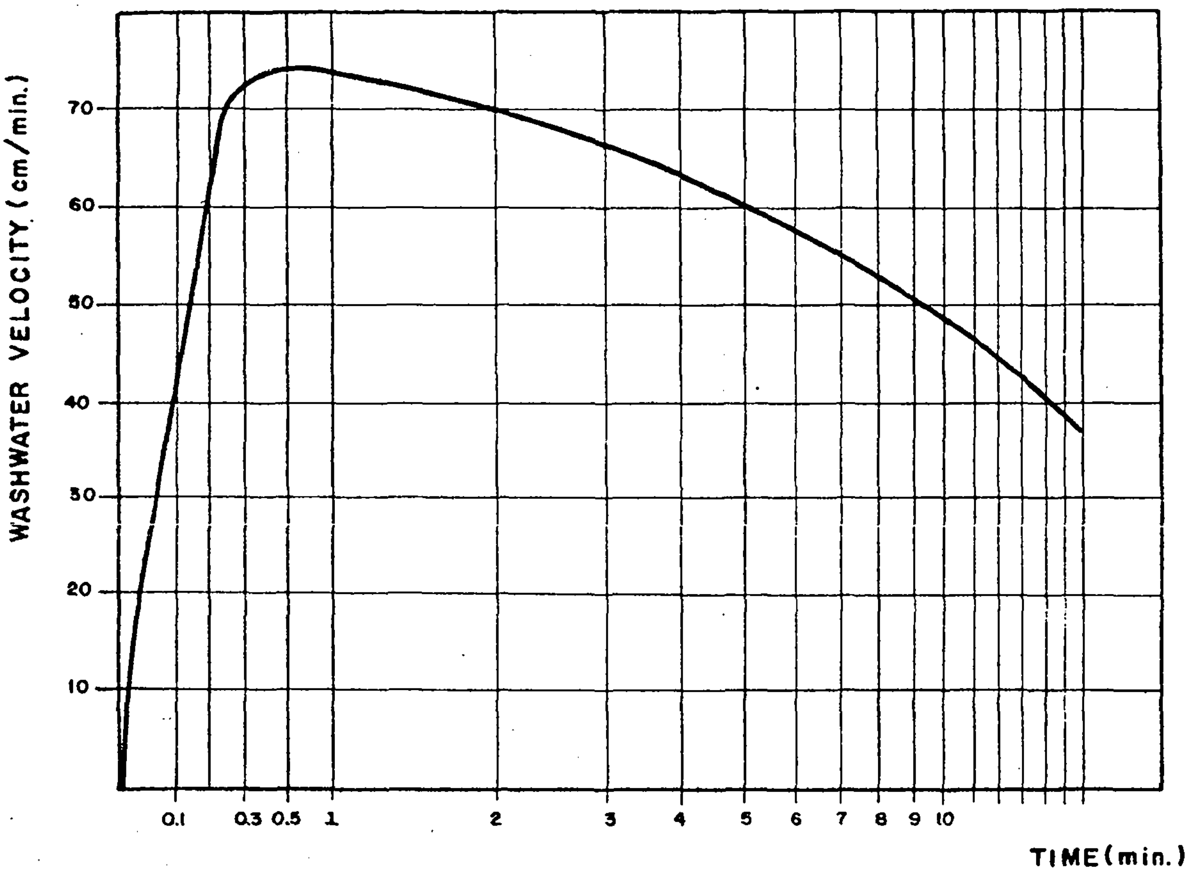


FIG. - 4.5.- THEORETICAL PERFORMANCE OF THE WASHWATER VELOCITY

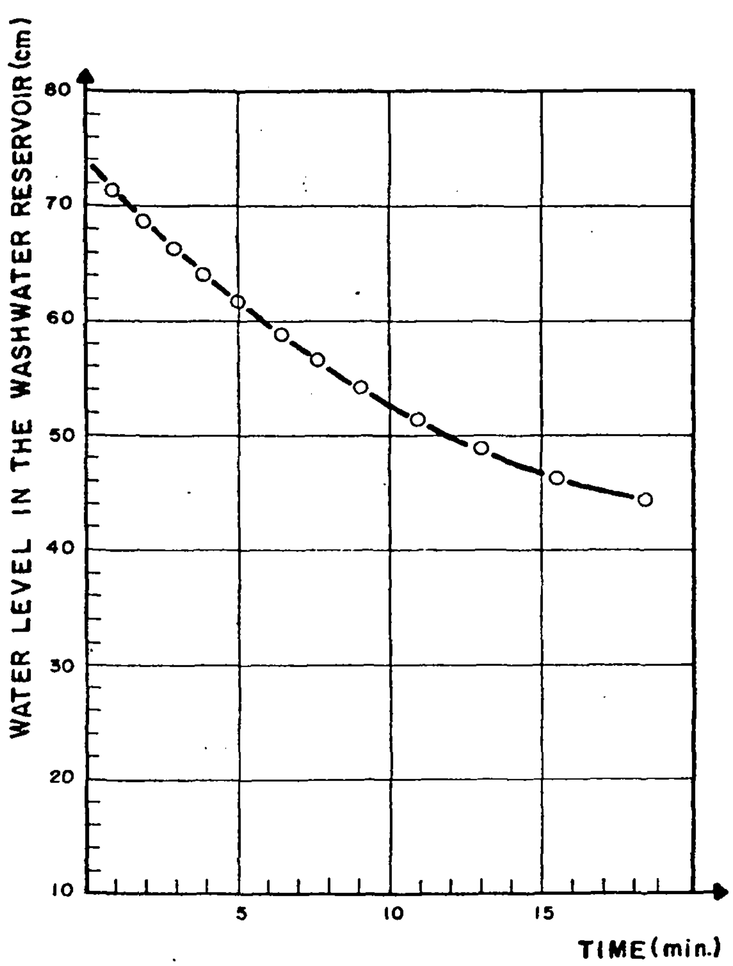


FIG. 4.6 – LOWERING OF WATER LEVEL IN THE WASHWATER RESERVOIR AS A FUNCTION OF TIME

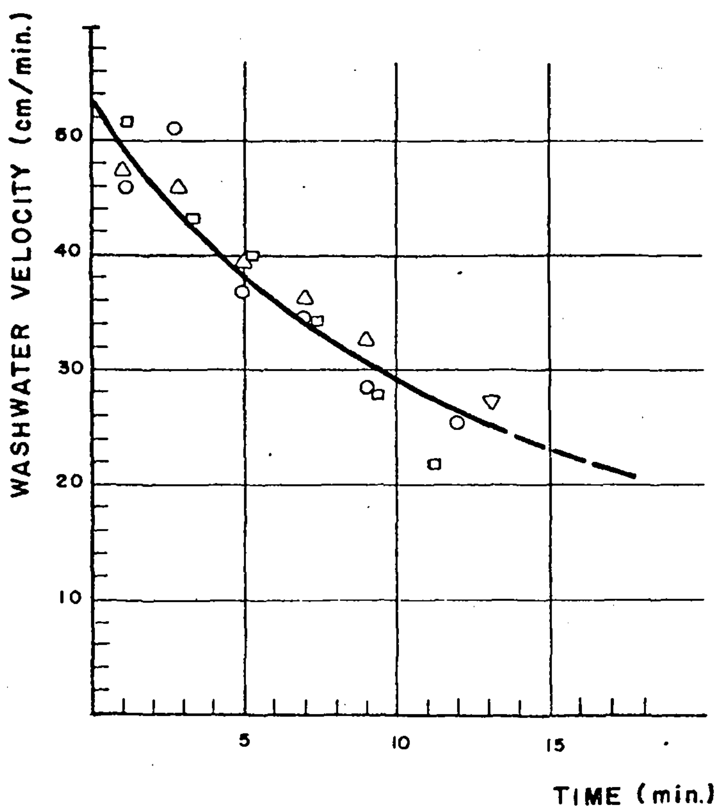


FIG. 4.7 – WASHWATER VELOCITY

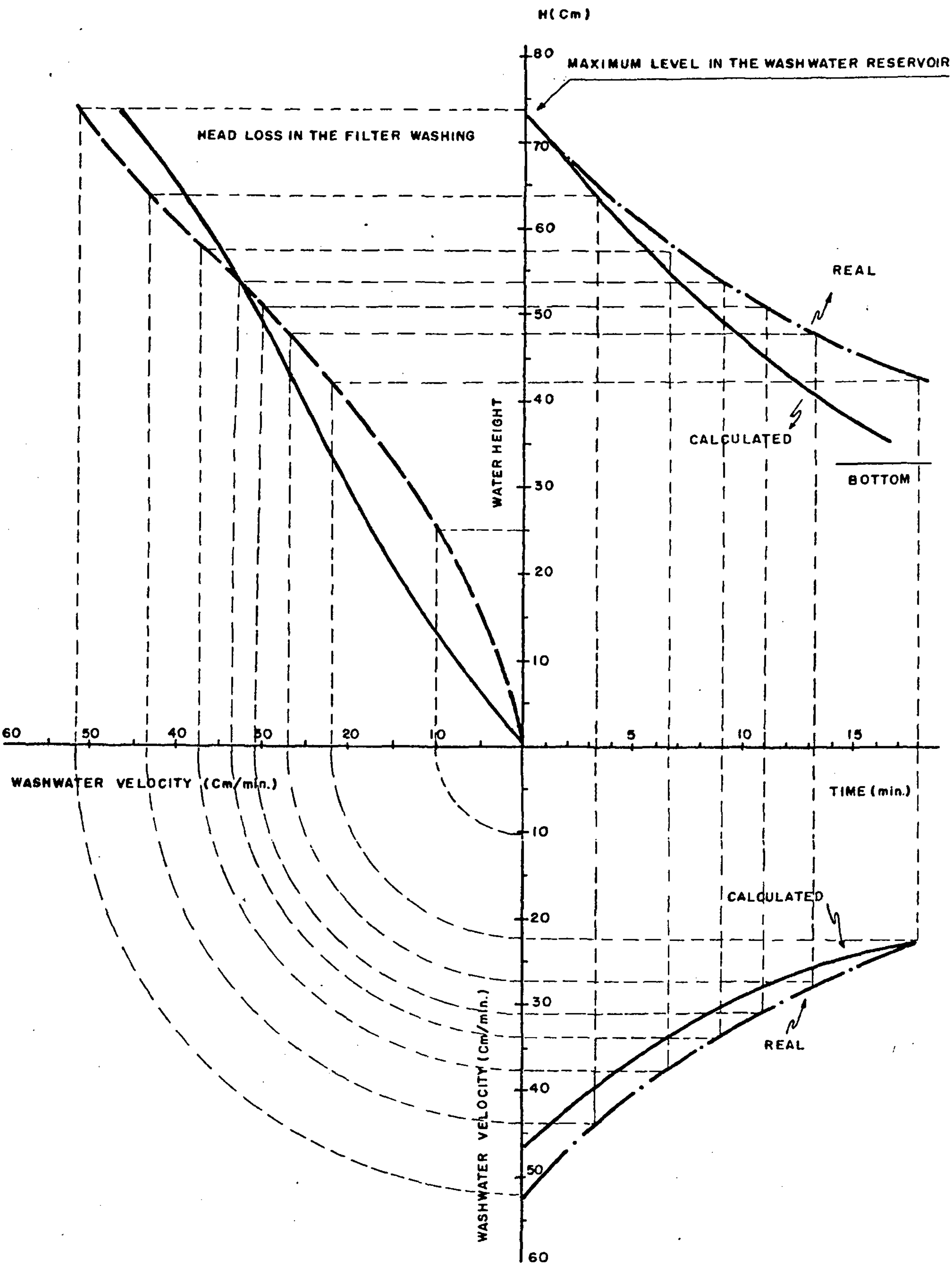


FIG.-4.8. 2nd WASHING PHASE: LOWERING OF WATER LEVEL IN THE WASHWATER RESERVOIR - GRADUAL AND SLOW DECREASE OF WASHWATER VELOCITY

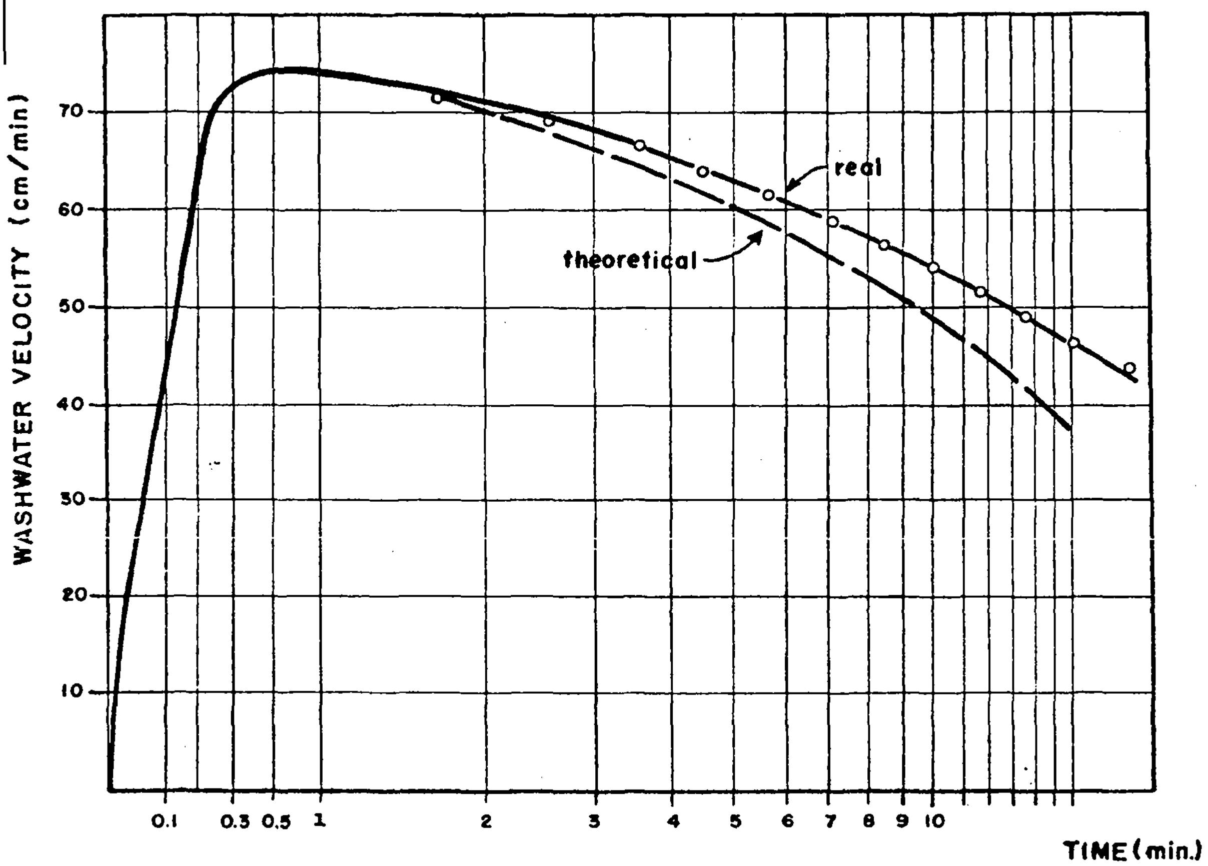


FIG.-4.9.- REAL PERFORMANCE OF THE WASHWATER VELOCITY CONFRONTED WITH THE THEORETICAL ONE

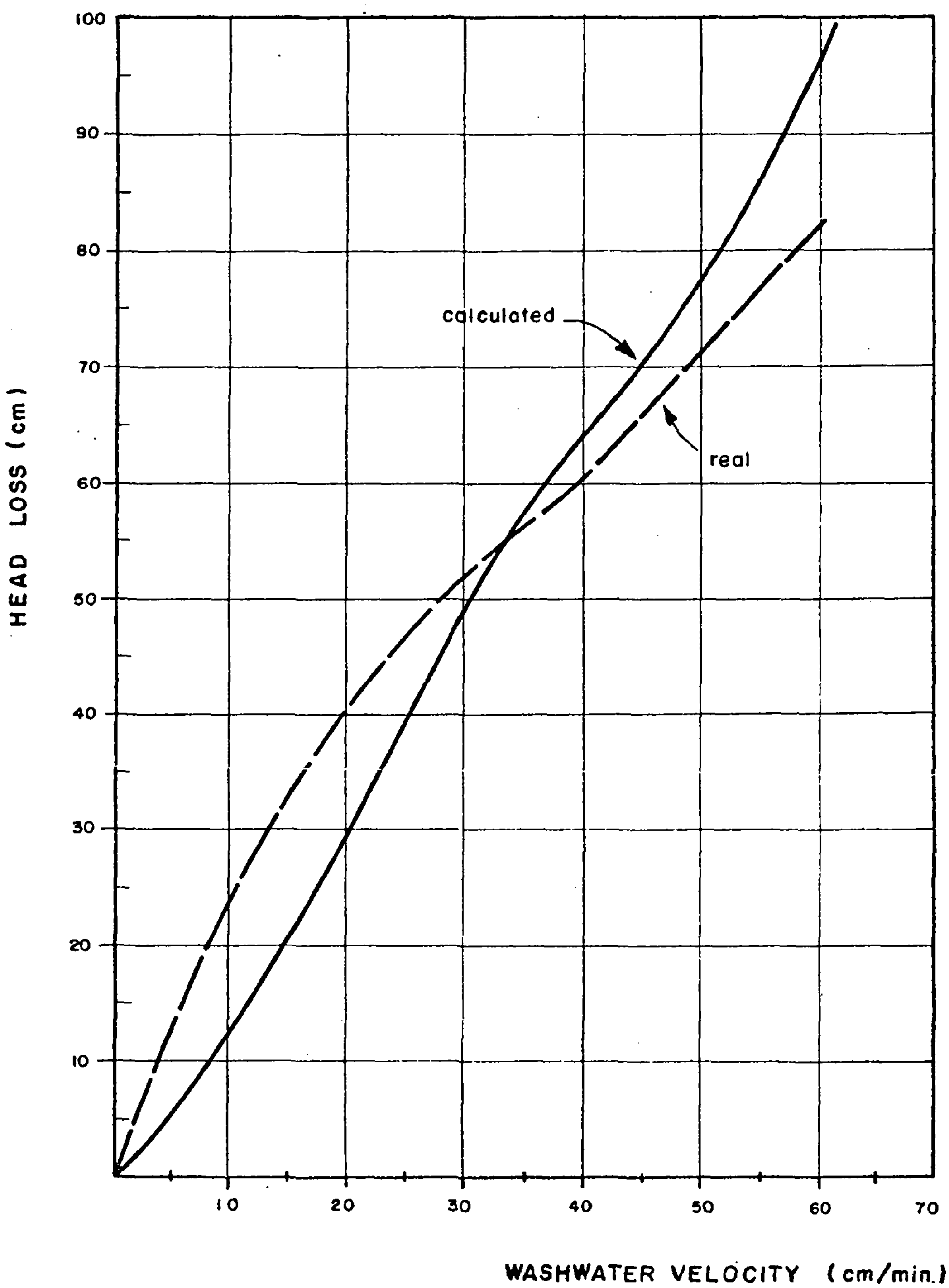


FIG. 4.10 - HEAD LOSS IN THE FILTER BED. REAL VALUES CONFRONTED WITH THE CALCULATED ONES