DIRECT ACTIVATED SLUDGE DEWATERING USING HOLLOW FIBERS MEMBRANES

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Abstract

An experimental study for combined stabilization and dewatering of industrial wastewater was carried out by means of a microfiltration unit inside a bioreactor. The module type used was a hollow fiber membrane with an area of 0.3 m^2, and pore size of 0.1 µm. The membrane material was polyethylene. Dewatering was accomplished by applying suction pressure across the membrane.

Four suction pressures in short-term experiments with activated sludge from the Barceloneta Regional Wastewater Treatment Plant were studied. The objective here was to determine the membrane hydraulic characteristics and the best conditions for stable, long-term operation. The experiments showed that the flux through the membrane decreases with vaccum pressure greater than 60 kPa, owing to rapid clogging of the membrane micropores.

Long term experiments of 62 days duration were carried out with a pharmaceutical wastewater at a suction pressure of 20 kPa. Parameters measured included COD, MLSS, and MVLSS. The operation mode was intermittent with cycle time of 20 minutes: 4 minutes of suction, followed by 16 minutes without suction. This operation mode gave better results than

continuous operation, because in the meantime that there was no suction, suspended solids that had adhered to the membrane were released. The MLSS inside the reactor increased throughout the operation, but were always under 40,000 mg/L, the limit for stable operation without sludge wastage, according to previous works. The average COD removal efficiency during the long term experiment was 63% on the average, with a maximum of 86%.

This system studied compared favorably with the conventional steps of stabilization/settling/dewatering, of the activated sludge proces. It can not only advantageously remove COD and maintain the MLSS adequately, but also it can reduce the space requirements of the operation eliminating the settling step, and accomplishing effective dewatering of the activated sludge, usually the most difficult and expensive technical process.

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A	filtration area, m^2
C_b	bulk solution concentration mg/L
BOD	biochemical oxygen demand, mg/L
COD	chemical oxygen demand, mg/L
°C	degrees Celsius
J	flux, m^3/m^2-s
L_m	membrane permeability
MLSS	mixed liquor suspended solids, mg/L
MLVSS	mixed liquor volatile suspended solids, mg/L
ΔP	transmembrane pressure, kPa
Q_i	influent flow rate, m^3/s
R_a	adsorption resistance, kPa/m-s
R_{bl}	boundary layer resistance, kPa/m-s
R_c	clogging resistance, kPa/m-s
R_{ds}	dissolved solids resistance, kPa/m-s
R_g	gel layer resistance, kPa/m-s
R_m	hydraulic resistance of the membrane, kPa/m-s
R_s	membrane surface resistance, kPa/m s
R_{ss}	suspended solids resistance, kPa/m-s
R_T	total resistance, kPa/m-s
r_{bl}	specific resistance of the boundary layer, kPa/m-s-g
S_0	influent flow rate, m^3/s
t	time, s
T_r	retention time, hours or days
V	reactor volume, L
V_a	volume of liquid in aeration stage, m^3
V_p	permeate volume, m^3
X	reactor biomass concentration, mg/L

Greek Symbols

δ	boundary layer thickness
μ	solvent viscosity
σ	reflection coefficient
$\Delta\pi$	osmotic pressure
Ξ	solids fractional retention

1. INTRODUCTION

Management of sludge generated by wastewater treatment plants is a critical concern faced by many communities. This residual is a semisolid, odoriferous, difficult to manage, and dangerous material. Sludge is a mixture of several organic and inorganic compounds, grit, and anything else which may get into the sanitary drainage system. These properties, combined with the quantity of materials which are produced, result in a complex dewatering and disposal problem.

The problems of industrial waste management are at once more complex and yet capable of a more flexible approach than are those of sewage treatment. The essential difference is that the nature, composition, volume, manner, location, and timing of wastes production are all to some extent controllable by the producer, who is, therefore, in a position to optimize the problem itself in terms of production economics.

Industrial waste characteristics reflect the diverse operations of industry and, by definition, can cover the complete spectrum of technology. By modern standards, an industry is composed of a large number of comparatively small

units, but each individual unit is capable of producing a wide range of highly-polluting liquid discharges.

The necessary treatment of these polluting loads creates a further problem to the industry in that the nature of the waste treatment processes available to purify the liquid streams results in the production of large volume of waste solids. The disposal and dewatering of wastes constitutes an additional cost that creates a severe economic burden on the industry, since solids dewatering is usually the most difficult and most expensive technical process. Therefore, alternatives must be examined carefully to ensure that optimum economic solutions to the problem are forthcoming.

During the last seven years, researchers at the College of Engineering of the University of Puerto Rico have concentrated their efforts on the design of treatment alternatives for the sludge generated by the Barceloneta Regional Wastewater Treatment Plant, to reduce adverse effects on the environment. The land disposal scheme now used poses environmental problems and it will have to be abandoned in the near future.

Two of the management alternatives already explored are composting and incineration. Both are attractive, but require previous sludge dewatering to a solids content of, at least,

thirty per cent. This is extremely difficult to accomplish, particularly when the activated sludge generated at the plant is dewatered by conventional techniques. An economic and effective solid-liquid separation method for this purpose must be found if the community at large and industry are not to be submerged in sludge.

This research work studies the arrangement of a biological reactor in which sludge dewatering and activated sludge stabilization occur simultaneously. The expected results would be a useful scaleup strategy to develop a new wastewater treatment process.

2. PREVIOUS WORK

Recently, membrane separation techniques have been applied to municipal and industrial wastewater treatment. Different types of membranes have been used for a variety of industrial wastes such as laundry waste (Bhattacharyya et al. 1974), oily waste (Anderson and Saw, 1987), heavy metals (Bhattacharyya et al., 1979), phenolic compounds (Klemetson and Scharbow, 1979), and protein separation (Le and Atkinson, 1985). Membrane separation applied to anaerobic digestion of organic wastes was investigated by several researches (Hammer and Borchrst, 1969, Grethlein, 1978, Li et al., 1984, and Okuno et al., 1986).

Smith et al. (1969) used filtration membranes for solid-liquid separation in the activated sludge process. The advantages of this process are that it can minimize sludge wastage and reduce the plant size by maintaining high solids concentration in the reactor. High degree of wastewater treatment was obtained in an actual plant using this system (Bemberis et al., 1971). A similar system is used in a building in Japan (Uchida, 1983). The process is not cost effective because its energy requirements are much higher than for conventional activated sludge. The main reason for the high energy requirement is the need for a recirculation

pump which connects the main reactor to a membrane unit and maintains high crossflow velocity on the membrane surface to keep the flux undeclined.

Yamamoto et al. (1989) proposed eliminating the need for a recirculation pump by direct membrane separation in the reactor using hollow fiber modules. They conducted experiments in an activated sludge aeration tank. The substrate used was a synthetic organic waste made of glucose, peptone, yeast extract and various nutrients. Solid-liquid separation was accomplished by suction through hollow fiber modules immersed in the reactor liquor. By intermittent suction, they were able to operate under stable flux conditions for 120 days. COD removal was higher than 95%, and MLSS remained almost constant in the reactor without sludge wastage. Power consumption for pumping was found to be negligibly low.

Yamamoto and Khin (1991) reported the feasibility of a new system for wastewater treatment called Sequencing Batch Membrane Reactor (SMBR), which includes high strength organic matter and heavy metal, chromium, originated from a tannery. The removal efficiency of COD obtained was 93%. Yamamoto, (1991) presented further development and modification of hollow fiber membrane bioreactor, and improved performance of

polymer suspension and activated sludge, if the hollow fibers are kept in well dispersed conditions.

Various research projects have been completed at the Chemical Engineering Department of the University of Puerto Rico related to different aspects of the proposed work. Benítez, (1990) studied the dewatering characteristics of the sludge from Barceloneta using vacuum filtration. Monclova (1989) developed a mathematical model to quantify Starling's effect in a hollow fiber module. This effect is important in biomedical applications of these membrane devices. Bensen (1990) modelled a hollow fiber membrane reactor used for the enzymatic hydrolysis and separation of an isomeric mixture of amino acids.

3. THEORY

3.1 WASTEWATER TREATMENT FUNDAMENTALS

Complete wastewater treatment involves not only the treatment and renovation of the liquid; it also encompasses the processing and disposal of the solids removed or generated during treatment of the liquid. Although the quantity of solids removed during the treatment process represents only about 0.5 percent of the total liquid volume treated, sludge disposal cost usually represents 30 to 50 percent of the total treatment costs, (Benefield et al., 1975).

There are several types of sludge that may be produced at different stages of a conventional wastewater treatment plant. Shown in Figure 1 is a typical flow diagram sheet for an aerobic sludge plant. Solids removed at the aerated grit chamber are mainly inert and can be disposed of directly by land burial with no further treatment required. However, raw primary sludge and waste activated sludge are both putrescible and should be stabilized before further treatment and ultimate disposal.

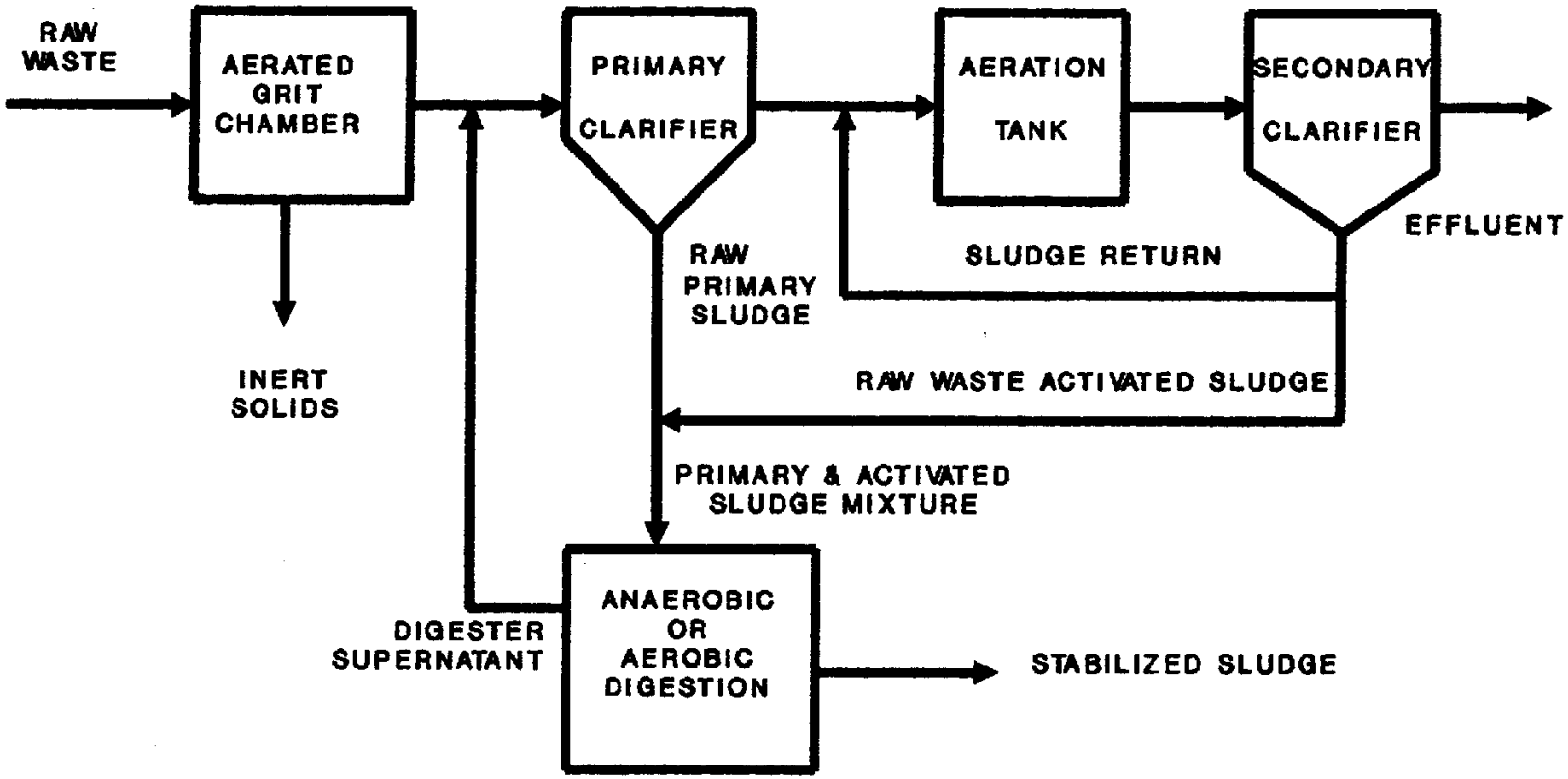

Figure 1. Typical flow diagram for a wastewater biological treatment plant.

3.1.1 PRIMARY TREATMENT PROCESS

The purpose of primary sedimentation is to reduce the velocity of the wastewater sufficiently to permit solids to settle. Sedimentation will remove most of the settleable solids, or about 40 to 60 per cent of the suspended solids. Since approximately 80 per cent of the total BOD is contributed by suspended and colloidal solids, between 30 and 45 percent of the total BOD is removed during this operation. This operation is normally carried out in a series of steps:

1. Screening. The purpose of screens is to remove large objects from the wastewater which tend to clog pumps and pipe lines and to interfere with plant operation. The finer material remains in the water to be removed later in a settling tank.

2. Grit removal. Sand, grit, and small stones are allowed to settle to the bottom of a grit chamber. The grit obtained in this process is disposed of by using it for land fill.

3. Sediment removal. Sewage, even after removal of grit, still contains suspended solids. These will settle out and the solid mass, called raw sludge, is collected, sent to a stabilization process and then disposed of.

The primary treatment is completed when the effluent, from which grit and sludge have been removed, is treated with chlorine gas before discharging into a stream or river, Chlorine gas is added to destroy disease-causing bacteria. Primary treatment removes about one-third of the BOD and suspended solids and a few percent of the persistent organic compounds and plant nutrients.

3.1.2 SECONDARY TREATMENT PROCESS

Two processes are currently available for secondary treatment: the trickling filter and the activated sludge process. An efficiently operating activated sludge system remove up to 90% of the suspended solids and BOD. A good trickling filter system is capable of removing 80-85% but in practice 75% is more common.

A trickling filter is simply a bed of stones and gravel through which the sewage passes slowly. Bacteria gather and multiply on the stones and gravel until they become numerous enough to consume most of organic matter in the sewage. The water, after passing through the activated bed, trickles out through pipes in the bottom of the filter.

In activated sludge treatment plants, the rate of bacterial action is increased by bringing air and bacteria-

laden sludge into very intimate contact with the sewage, which has previously received primary treatment. Sewage, air, and activated sludge remain in contact for several hours in the aeration tank.

The sewage flows from the aeration tank into another sedimentation tank, where solids are removed. Chlorination completes the basic secondary treatment. The sludge, which contains the bacteria, can be used again by returning part of it to the aeration tank.

3.1.3 TERTIARY OR ADVANCED TREATMENT PROCESS

Primary and secondary sewage treatment lower the BOD of the water and eliminate harmful bacteria. They do not, however, effectively remove other dissolved organic and inorganic compounds. Most dissolved unmanageable organic compounds remain in water that has gone through primary and secondary treatment. These persistent compounds resist bacterial action.

There are advanced techniques for tertiary treatment, that are very expensive, ranging from extensions of biological processes capable of removing nitrogen and phosphorous nutrients, to physico-chemical separation

techniques such as adsorption, distillation, ultrafiltration, electrodialysis, and reverse osmosis.

If used water is going to meet the water quality standards of the U.S. federal government (some are now in effect and others are yet to be established), attention must be given to these above techniques.

3.2 THE ACTIVATED SLUDGE PROCESS

Practical experience has demonstrated that the substantial removal of organic matter from municipal and industrial wastewaters can be most economically attained using aerobic biological treatment. This utilizes naturally occurring living microorganisms. These organisms use organic matter as a source of food. In so doing they remove objectionable materials.

Like all living things, microorganisms behavior is strongly influenced by their diet and their environment. Providing an adequate amount of good food under proper environmental conditions makes it possible for man to control, even to train, microorganisms so that they will do almost any desired job of removing unwanted organic matter from wastewaters. Experience has shown that wastes of domestic origin provide a good source of food for

microorganisms. A typical activated sludge process is shown in Figure 2.

3.2.1 PRINCIPLES OF THE PROCESS

The basic principle of the process is that the wastewater is brought into contact with a mixed microbial population in the form of a flocculent suspension in an aerated and agitated system. Suspended and colloidal material is removed rapidly from the wastewater by adsorption and agglomeration on to the microbial flocs. This material and dissolved nutrients are then broken down more slowly by microbial metabolism, a process referred to as **stabilization.** In the stabilization process, part of the nutrient material is oxidized to simple substances such as carbon dioxide (mineralization), and part is converted into new microbial cell material (assimilation). Part of the microbial mass is also broken down in the same way, a process called 'endogenous respiration'. The oxidative process provide the energy needed for the operation for the adsorption and assimilation processes. When the desired degree of treatment has been achieved , the flocculent microbial mass, known as the 'sludge', is separated from the treated wastewater by gravity settling. The separation stage is also referred to as 'clarifying', 'settling' or 'sedimentation'. The supernatant from the separation stage is the treated wastewater, and should be virtually free of solids. Most of the settled

Activated Sludge Process

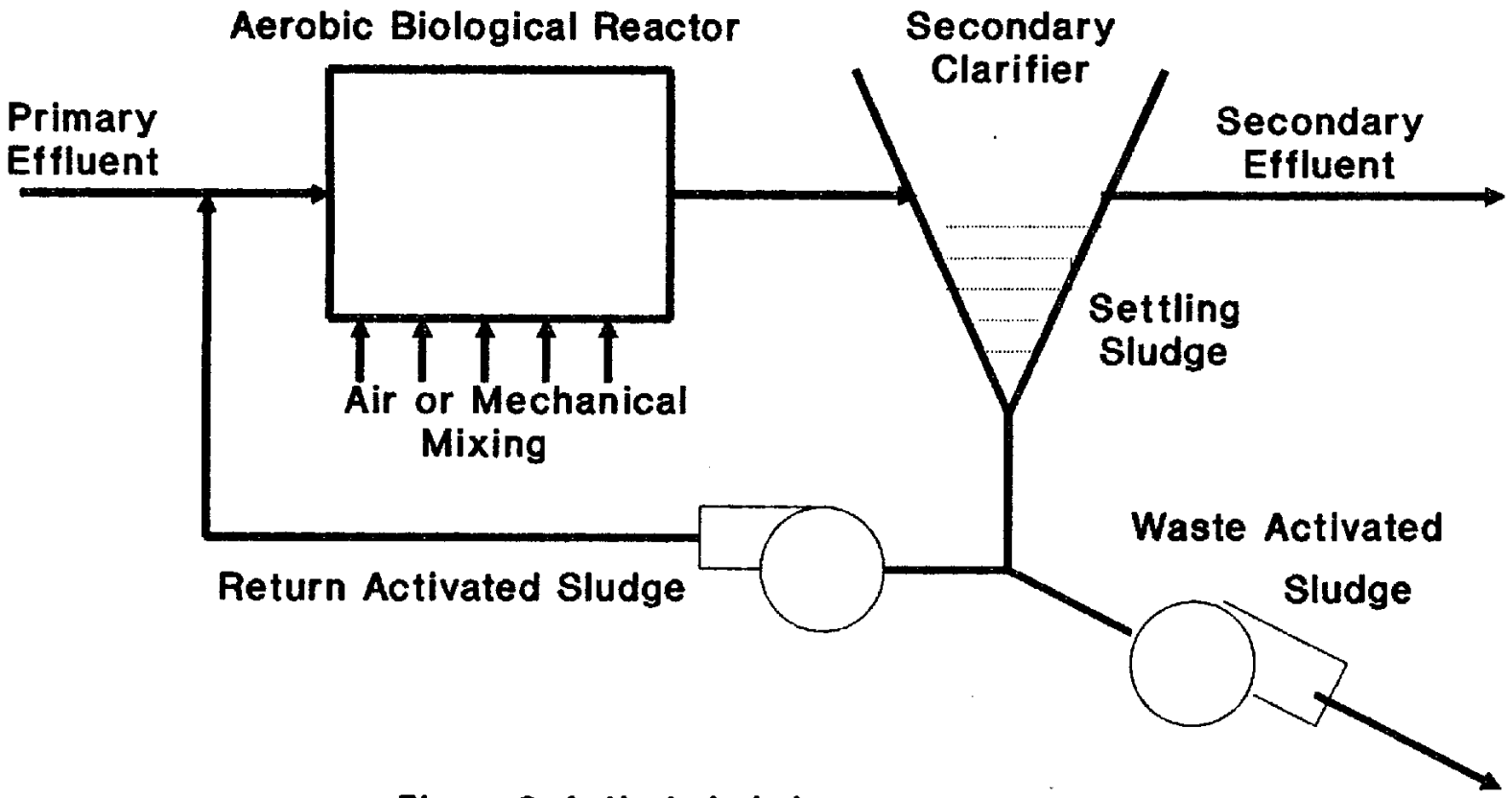

Figure 2. Activated sludge process.

sludge from the separation stage is returned to the aeration stage to maintain the sludge concentration in the aeration stage at the level for effective treatment and to act as a microbial inoculum. Some of the sludge is removed for disposal, and is known as 'waste' or 'surplus' activated sludge. In a balanced system, the waste sludge produced represents the net amount of microbial mass produced by assimilation in the aeration stage, and is effectively the 'pollution concentrate' from the system.

There is a variety of different versions of the activated sludge process, from which arises its versatility in suiting a wide range of treatment requirements. They are made up of different combinations of modes of operation, mixing regime, aeration system an loading level.

3.2.2 CHEMISTRY OF BIOLOGICAL ACTIVITY

Microorganisms are Nature's scavengers. They utilize matter such as that in sewage as a source of food. Part of the organic matter is used for energy and part is used for synthesizing protoplasm, i.e., reproduction. These reactions utilize enzymes. Simplified general equations to illustrate the role of enzymes are shown in Equations 1 and 2, in Table I.

Table I. Chemistry of Biological Activity*

$$\textit{Organic Matter} + \textit{Enzyme} \rightarrow \textit{Organic Matter-Enzyme Complex} \quad (1)$$

$$\textit{Organic Matter-Enzyme Complex} \rightarrow \textit{Enzyme} + \textit{End Products} \quad (2)$$

Organic Matter oxidation:

$$C_xH_YO_Z + (x + \frac{Y}{4} - \frac{z}{2})O_2 \xrightarrow{enzyme} xCO_2 + \frac{Y}{2}H_2O + energy \quad (3)$$

Cell Material Synthesis:

$$n(C_xH_yO_z) + nNH_3 + n(x + \frac{y}{4} - \frac{z}{2} - 5)O_2 \xrightarrow{enzyme} n(C_5H_7)_2N) + n(x - 5)CO_2 + \frac{n}{2}(y - 4)H_2O \pm energy \quad (4)$$

Cell Material Oxidation:

$$n(C_5H_7C_2N) + 5nO_2 \xrightarrow{enzyme} 5nCO_2 + 2nH_2O + nNH_3 \pm energy \quad (5)$$

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*From: Weston, R.F.,Fundamentals of aerobic biological treatment. Public Works, 1963.

If excess molecular oxygen is present the conditions are aerobic. If molecular oxygen is absent the conditions are anaerobic.

Under aerobic conditions, that organic matter which is required for energy may be oxidized to carbon dioxide and water in accordance with Equation 3.

That organic matter which is used in reproduction may be converted to protoplasm [illustrated by the average empirical formula $(C_5H_7NO_2)_n$] in a manner as shown in Equation 4.

It should be noted that a source of nitrogen, such as ammonia, is required for synthesis. Although not indicated, a source of phosphorus must be available also. Since protoplasm is organic matter and may be used as a source of food by other organisms, it may be oxidized as shown in Equation 5.

It should be noted that molecular oxygen is utilized in all of these reactions. The summation of the above reactions is metabolism. The oxygen utilized by microorganisms in the oxidation of organic matter is the biochemical oxygen demand (BOD) of that matter.

Aerobic organic matter removal is relatively rapid and odor free. Anaerobic organic matter removal from dilute solutions is slower and odorous. Therefore, except for special circumstances, biological treatment of wastewaters is most effective, satisfactory and economical under aerobic conditions.

The stoichiometry, kinetics and equilibrium of the above chemical reactions control the behavior of biological treatment systems.

3.3 OPERATING PARAMETERS

The organic nutrient content or concentration is expressed in terms of oxygen demand. Oxygen demand is an important parameter for determining the effect of organic pollutants on receiving waters. As microorganisms in the environment consume these materials, oxygen is depleted from the water. There are two main methods of measuring oxygen demand directly by Biochemical Oxygen Demand (BOD) and COD (Chemical Oxygen Demand).

3.3.1 BIOCHEMICAL OXYGEN DEMAND

This is the amount of oxygen required for the oxidation of a waste by bacteria. It is therefore a measure of the

concentration of organic matter in a waste that can be oxidized by bacteria ('biodegration'). The test loses some of its value, though, because of the length of time (five days) it takes to get results. The BOD test also is inadequate as an indicator of organic pollution when used with industrial wastewater containing heavy metal ions and cyanide. Microorganisms in the waste become poisoned by toxic substance and are unable to oxidize wastes.

3.3.2 CHEMICAL OXYGEN DEMAND

The COD test today has a fairly specific and universal meaning: the oxygen equivalent of the amount of organic matter oxidizable by potassium dichromate in a 50% sulfuric acid solution. Generally, a silver compound is added as a catalyst to promote the oxidation of chloride ions by the dichromate. End products are carbon dioxide, water and various states of chromate ion.

After the oxidation step is completed, the amount of dichromate consumed is determined either by titration or colorimetrically. Either the amount of reduced dichromate (chromic ion) or the amount of unreacted chemical can be measured. If the latter method is chosen, the analyst must know the precise amount of dichromate added.

The present use of dichromate is based on the following advantages (Gibbs, 1981):

1) More complete oxidation than most other methods, when used with a catalyst and a two-four digestion period.
2) Fairly constant and reproducible results.
3) Less time required than most other COD method.
4) More convenient than the iodate or persulfate methods.

However, dichromate also has the following disadvantages:

1) Incomplete oxidation of some organic materials if catalyst are not used.
2) Interference from inorganic pollutants.
3) Relatively long digestion time.
4) Reaction temperature limited by thermal decomposition of the oxidant.
5) Escape of volatile pollutants from conventional apparatus.
6) Cumbersome conventional equipment.
7) Cost of reagents.

The food to microrganism ratio (F/M) is a commonly used parameter to describe the organic loading to an activated sludge process (see Appendix A), Tchobanoglous (1979) suggests that the optimal F/M ratio for wastewater systems is about 0.3 kg COD/kg microorganisms mass-day.

3.4 MEMBRANE SEPARATION PROCESS

The principle of membrane filtration is simple: by applying a driving force across a membrane, such as a concentration difference or pressure difference, a mass transport is forced across the membrane.

The membrane pressure filtration can be divided into :

1) Microfiltration (MF) to separate particles and colloids
2) Ultrafiltration (UF) to separate macromolecules
3) Reverse osmosis (RO) to retain salts and low molecular substances.

These processes are not rigidly separable from each other.

3.4.1 MODULES

The module is the main component of a membrane separation process. It serves to suitably arrange the membrane as well as guiding the flow.

Several partly contradictory requirements are made upon the module design:

1) High packing density and maximum utilization of the membrane area.

2) Simple design and low production costs.
3) Simple handling and cleaning resulting in low operation costs.
4) High separation performance and efficiency.

According to the desired application different module types are available such as tube, capillary, hollow fiber, spiral-wound and plate modules. In crossflow filtration, the most commonly used arrangement, the retentate stream flows along the membrane with high velocity to control the retentate concentration at the far side of the membrane that is increasing continuosly. The concentrated layer close to the membrane surface causes hydraulic resistance thus obstructing more and more the permeate flux.

Hollow fiber modules provide a very high filtration area per unit volume of module and are gaining widespread acceptance where there are space limitations.

3.4.2 MECHANISM OF FILTRATION THROUGH MEMBRANES

Figures 3 and 4 depict the countless micropores found in the wall of the straw shaped hollow fiber membrane. Liquid penetrates the outer wall of the hollow fiber, goes through the micropores, and reaches the surface of the inner wall. The particles are caught on the outer surface and thus cannot enter the inside.

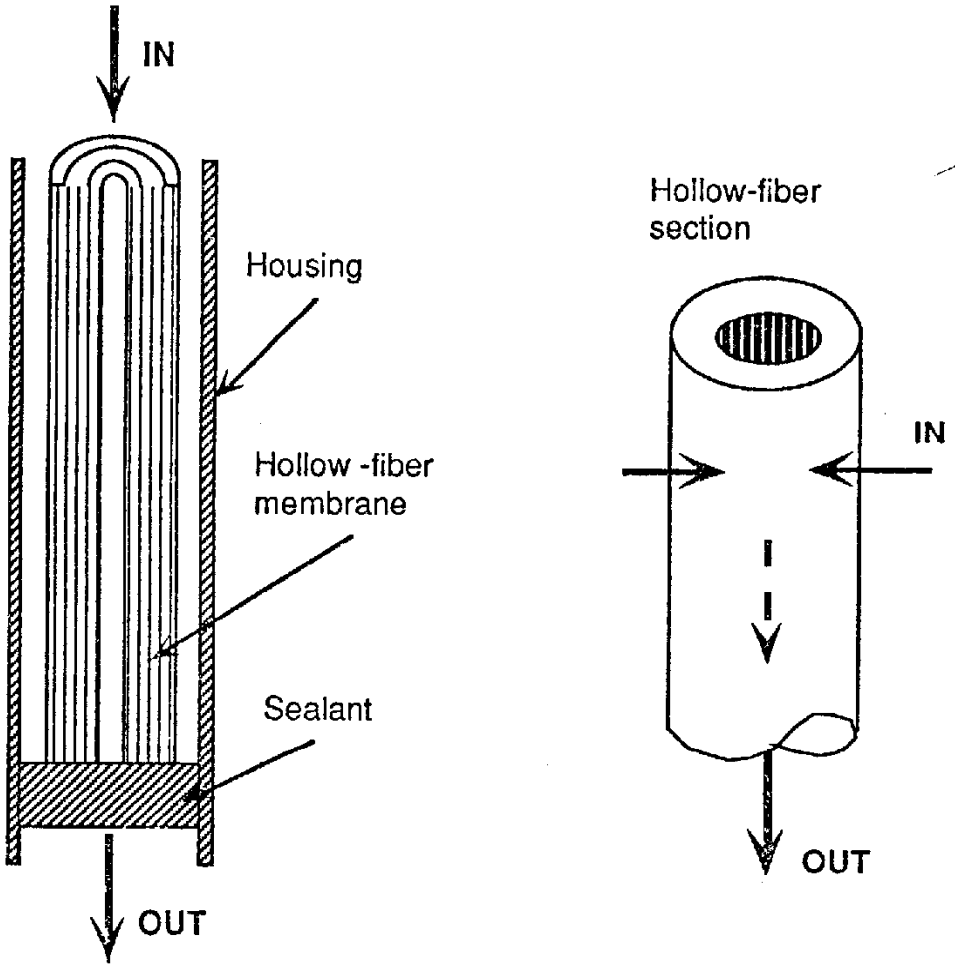

Figure 3. Structure of hollow fiber membrane

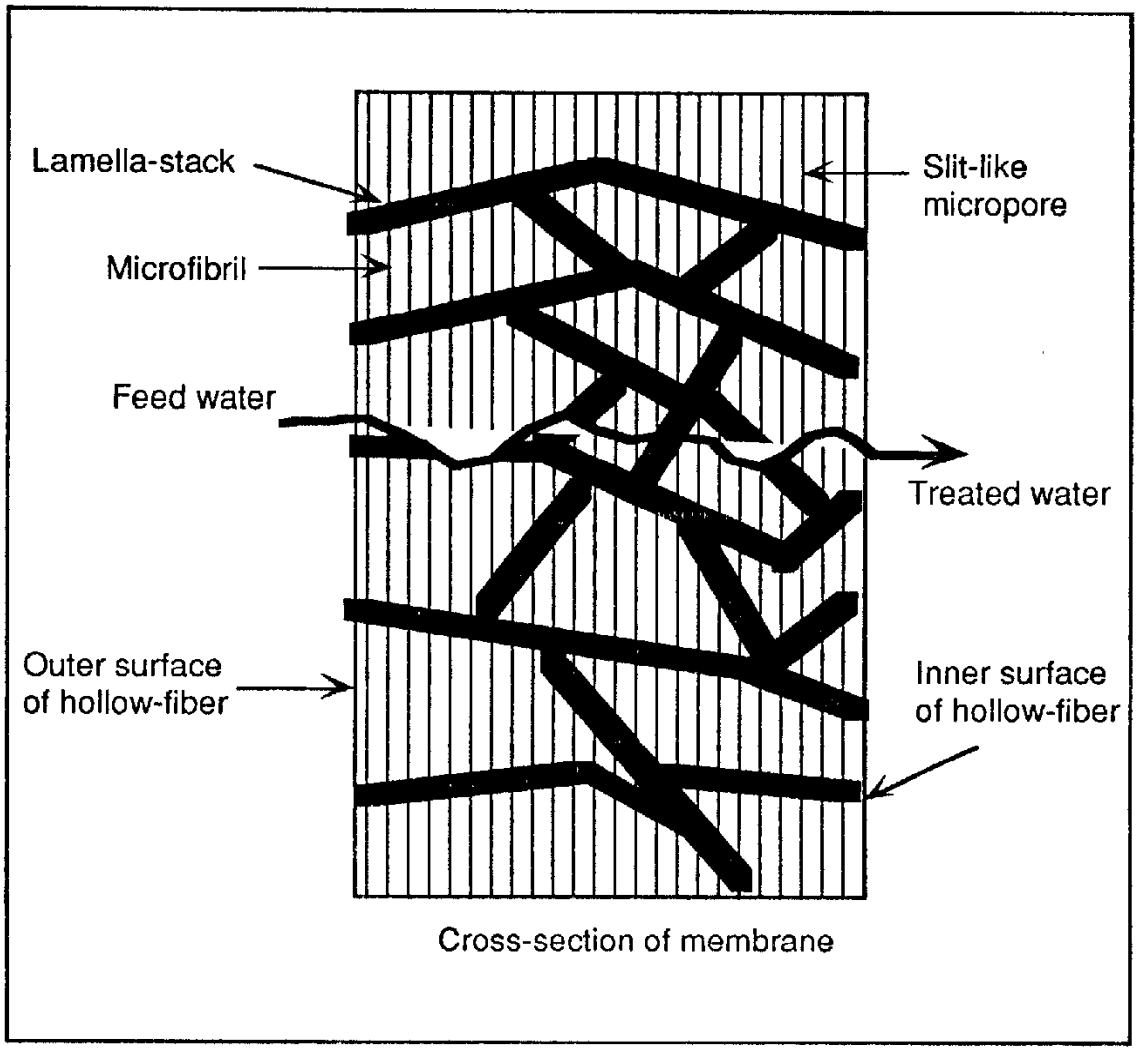

Figure 4. Cross-section of hollow fiber membrane

The filtration of particles is effectively accomplished as the micropores are multilayered. Only a very small number of particles are able to enter into the secondary side of the membrane filter.

3.4.3 RESISTANCE OF THE MEMBRANE

The accumulation of particulate material at the surface of the filtration barrier produces the major resistance to filtration. While tangential movement of the slurry minimizes the accumulation of particles at the filter surface, it does not eliminate it entirely. For unstirred dead-end flitration, particles accumulation at the membrane surface is more severe. In most cases, a layer of solids is formed. Depending on the nature of the filtration barrier and suspension being concentrated , this polarization reduces the filtration rate and alters the apparent rejection characteristics of the barrier.

In the flow of wastewater through membranes the possible causes of flux decline are:

1) Membrane compaction or deterioration
2) Membrane fouling or clogging by macromolecules

3) Changes in the gel layer which increases its resistance to flow

4) Continuosly increasing suspended solids concentration

5) Disolved solids wich causes adsorption in the inner micropores.

These possible causes are shown schematically in Figure 5.

3.4.4 VOLUME FLUX MODEL

In the abscence of solute the volume flux of solvent through microfiltration membranes, (J) is defined by the driving force ΔP and the membrane permeability, L_m, or its reciprocal the membrane resistance, R_m.

$$J = 1/A \frac{dV_p}{dt} = L_m \Delta P = \Delta P/(R_m \mu) \qquad (6)$$

where:

μ is the solvent viscosity

A is the membrane area

V_p is the permeate volume at any time t

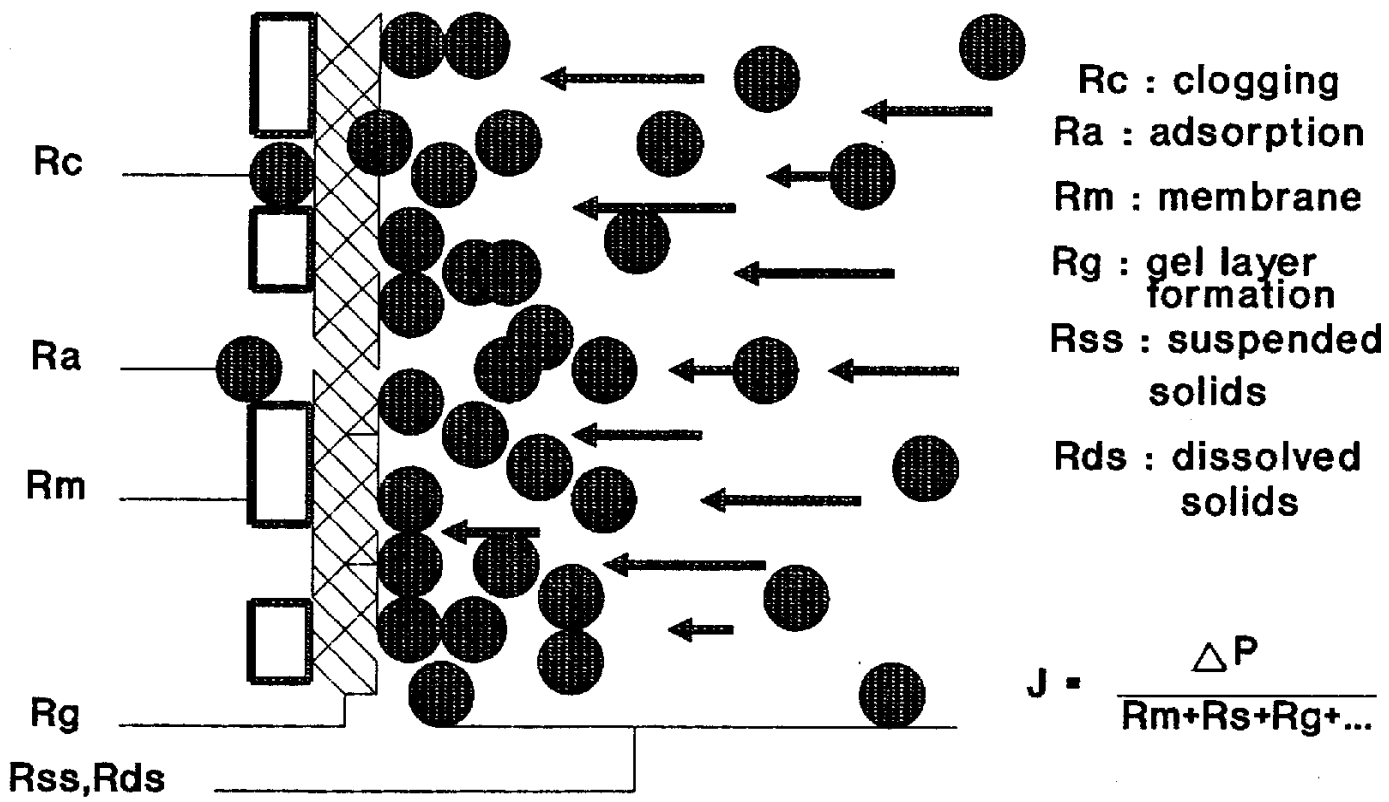

Figure 5. Posible resistances against microfiltration of activated sludge

Under microfiltration conditions the volume flux is lower, by as much as an order of magnitude, due to solute accumulation at the membrane surface. This accumulation or 'concentration polarization', can reduce the flux by presenting an additional resistance and/or osmotic effects according to

$$J = (\Delta P - \sigma\Delta\pi)/(R_m + R_s)\mu \qquad (7)$$

where:

σ is the reflection coefficient

$\Delta\pi$ is the change in osmotic pressure due to the solids

R_s is the resistance due to solids accumulation near the membrane surface.

Equation (7) has been used in various forms to describe microfiltration flux. In some cases $\Delta\pi$ is assumed zero and R_s represents reversible, and sometimes, irreversible (fouling or clogging), deposition of solute. In other cases R_s is neglected and the flux reduction is ascribed solely to osmotic pressure $\Delta\pi$.

If the osmotic term is neglected, equation (7) becomes:

$$J = \Delta P / (R_m + R_s)\mu \qquad (8)$$

The resistance due to the solids can be written as:

$$R_s = \delta r_{bl} \qquad (9)$$

where:

δ is the thickness of the boundary layer

r_{bl} is the specific resistance of the boundary layer.

From a solids material balance:

$$C_b \Sigma V_p = \delta A C_{bl} \qquad (10)$$

where:

C_{bl} is the boundary layer solids concentration

C_b is the bulk solution concentration

Σ is the solids fractional retention.

Substituting Equations 9 and 10 in Equation 8, the following relationship is obtained:

$$J = 1/A\frac{dV_p}{dt} = \frac{\Delta P}{\mu\left(R_m + \frac{C_b \Xi V_p r_{bl}}{A C_{bl}}\right)} \qquad (11)$$

Usually, the membrane resistance is negligible compared to that of the solids boundary layer. Neglecting R_m in Equation 11, integrating and solving we obtain:

$$t = \frac{\mu C_b \Xi r_{bl}}{\Delta P}\frac{}{C_{bl}}(V_p/A)^2 \qquad (12)$$

Equation 12 is the well known relationship for unstirred, dead-end filtration which states that the cumulative permeate volume is proportional to the square-root of the elapsed time (Van den Berg and Smolders, 1990).

4. EXPERIMENTAL SECTION

4.1 Apparatus

A schematic of the experimental set-up used is shown in Figure 6 . The aeration tank is cylindrical , with a diameter of 15 cm, height of 51 cm, with a working volume of 9 liters. The membrane used in the experiment was a hollow fiber module, with fibers made of polyethylene supplied by Mitsubishi Rayon Co. Japan , model STERAPORE™-S. The membranes were completely immersed in the tank and suspended vertically.

The fibers made of polyethylene had pore size of 0.1 μm, with membrane area 0.3 m^2, membrane length of 320 mm (12.6 in), connection 6.3 mm (1/4 in). and normal operating temperature less than 45 °C.

4.2 Materials

Before the module was used for long term operation, it was tested for permeation and separation characteristics. This was checked with both pure deionized water and activated sludge from the Barceloneta Regional Wastewater Treatment

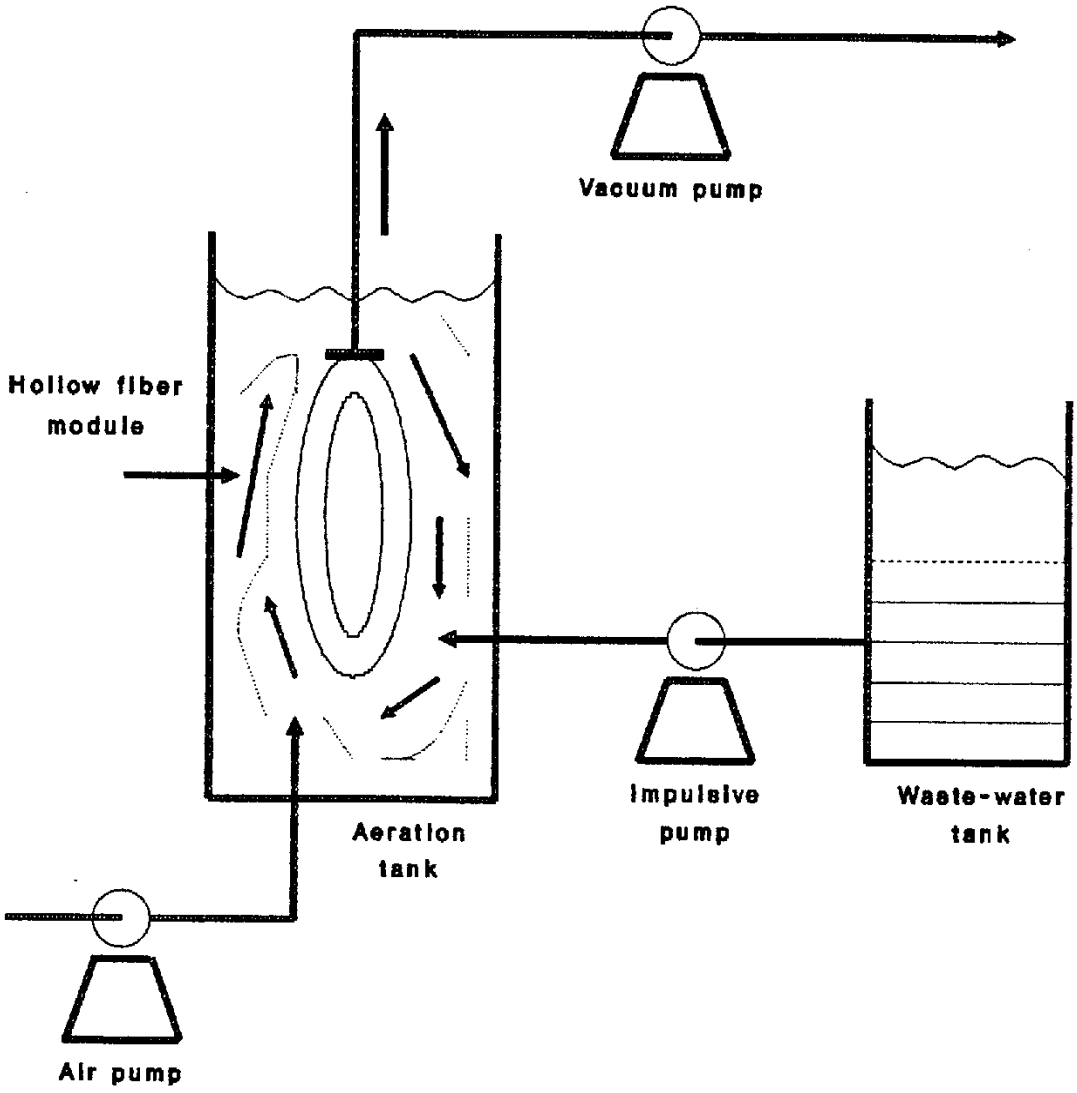

Figure 6. Experimental set-up arrangement.

Plant. The wastewater treated in Barceloneta is composed of 10-30% sanitary and 70-90% industrial effluents.

The long term experiments were done with the wastewater from Eli Lilly, a pharmaceutical plant located in Mayagüez. Activated sludge inoculum was obtained from the same source.

4.3 EXPERIMENTAL PROCEDURE

The first experiments were done with pure deionized water in a cylindrical tank with a working volume of 19 L. the tank was filled, the membrane module was submerged in the water, and a constant suction was applied through the membrane. The time required to filter 200 mL of water was recorded. The experiment was repeated at different suction pressures up to 84 kPa. It was then repeated using activated sludge from Barceloneta.

For the next set of short-term experiments, the tank was filled with activated sludge an a constant continuous suction was applied for 5 hours. The liquid level inside the tank was monitored every 10 min. No fresh material was added to the tank during the experiment. The experiment was repeated at 4 different pressures, namely 20, 40, 60 and 80 kPa. Samples of the activated sludge were analyzed for COD and mixed liquor

suspended solids (MLSS), previous to filtration. Samples of the filtrate were analyzed for COD and MLSS.

For the long term experiments, a 9-L working volume reactor was filled with wastewater from a nearby pharmaceutical plant and inocculated with activated sludge from the same source. The reactor content was mixed by continuous aeration through the bottom with an air flow rate of 2.3 L/min. The microorganisms acclimatized to the medium in a period of approximately 10 days.

During this period different intermittent suction modes were tried. A cycle of 4 minutes of suction at 20 kPa followed by 16 minutes without suction was found to result in a stable flux and a hydraulic retention time of 10.5 hours.

Once stable conditions were achieved, the reactor was operated continuously for 62 days. The influent, supernatant, and effluent were sampled in the morning and in the afternoon of everyday and analyzed for chemical oxygen demand (COD), total mixed liquor suspended solids (MLSS), and mixed liquor volatile suspended solids (MLVSS).

All testing procedures were conducted in accordance with the Standard Methods. Suction pressure was measured by a vacuum gauge located at the outlet of the membrane unit. The

long term experiments were done at a vacuum pressure of 20 kPa.

5. RESULTS

5.1 SHORT TERM EXPERIMENTS

In order to investigate the hydraulic characteristics. of the hollow fiber membrane for further operation, a series of experiments were carried out. The results are shown in Table II to Table VII. Table II shows the variation of wastewater quality in terms of COD and suspended solids at four suction pressures, the SS removed was total and COD removal average was 94.82%. Dewatering of activated sludge of the same treatment plant whitout addition of influent is shown in Table III, the greater deawatering volume is observed at 60 kPa, with decreased volume of 57.89% and the lower at 20 kPa, with 37.89 % decrease. Table IV to VII can be found in Appendix B.

Table IV shows the data of the effect of transmembrane pressure on flux with pure water and activated sludge from the Barceloneta Regional Plant with an initial MLSS of 4470 mg/L. Table V shows the influence of initial pressure on the resistance of the membrane, that increases betwen 40 to 80 kpa of applied suction pressure. Table VI shows the variation of microfiltered volume with time at four applied pressures. The operation time was two hours at 20 kPa, and eight hours at 80 kPa.

Table II. Variation of wastewater quality in terms of COD and SS, in short term microfiltration.

Suction	COD (mg/L)		Removed	SS (mg/L)	
kPa	Influent	Efluent	%	Influen	Efluent
20	1316	137	89.5	4470	0
40	5118	277	94.59	4870	0
60	5897	136	97.69	2200	0
80	5235	135	97.48	3253	0
Removed	average:	COD	94.82%	ss	100%

Sample: Barceloneta Regional Wastewater Treatment Plant.

Table III. Dewatering of activated sludge, at short term, reactor working 19 L, without addition of influent.

Suction, kPa	Volume (L)		Reduction
	Initial	Final	%
20	19	7.25	37.89
40	19	8.02	42.21
60	19	11.0	57.89
80	19	7.6	40.0
Operation time: 5 hrs.			

Sample : Barceloneta Regional Wastewater Treatment Plant.

Table VI shows data of variation of flux versus time when filtering activated sludge, at the same four applied pressures considered in Table V. All these experiments were carried out using activated sludge from the Barceloneta Regional Wastewater Treatment Plant.

5.2 LONG TERM EXPERIMENTS

Continuous experiments were carried out for 62 days to examine the long term process stability. These data, in terms of COD and COD removal, are shown in Table VIII (Appendix B). The reactor used in these experiments was a cylindrical tank with an inner diameter of 15 cm, length 51 cm, and working volume of 9 liters.

The maximum effluent COD observations were 1998 mg/L at the 8th day, and 1875 mg/L at the 52th day . The measured lows were at 28th and 62th days. The MLSS and MLVSS observations are shown in Tables IX and X, respectively.

Table IX shows two measurements of MLSS in the bioreactor, one taken on the morning, at the beginning of the filtration operation, and the other in the afternoon, at the end of the operation. The maximum MLSS of the influent was 5520 mg/L on the 53th day, and a lowest value was observed

the second day (1133 mg/L). The MLSS inside the bioreactor increased continuously, up to 37935 mg/L in the afternoon of the 39th day.

Table X shows the data on MLVSS. It remained almost stable in the influent, the maximum value was 4960 mg/L on the 62th day. The values inside of the bioreactor increased continuously, the same as the MLSS. The organic loading (F/M rate) diminished gradually from 1.184 d^{-1} on the 11th day to 0.6637 d^{-1} on the 62nd day.

6. DISCUSSION

6.1 EFFECT OF TRANSMEMBRANE SUCTION PRESSURE ON VOLUME FLUX

The results of the transmembrane suction applied through the hollow fiber, are shown in Figure 7. The flux is a linear function of pressure when the membrane is still clean, as well as when deionized water is used, since there is no reason for flux decline such as clogging. The flux in the experiments with activated sludge was less than in those with pure water for a given vacuum pressure applied because of the clogging nature of the sludge (the suspended and dissolved solids add to the resistance of the membrane to flow through the micropores).

6.2 INFLUENCE OF SUCTION PRESSURE IN THE RESISTANCE

The resistance of the hollow fiber membrane with initial differential vacuum pressure is shown in Figure 8. The resistence was calculated knowing the time it took to filter 200 mL of liquid, the membrane filtration area, and the applied suction pressure. At first the resistance declined with the applied suction up to 24 kPa, then, the resistance increased continuously with applied vacuum up to 84 kPa.

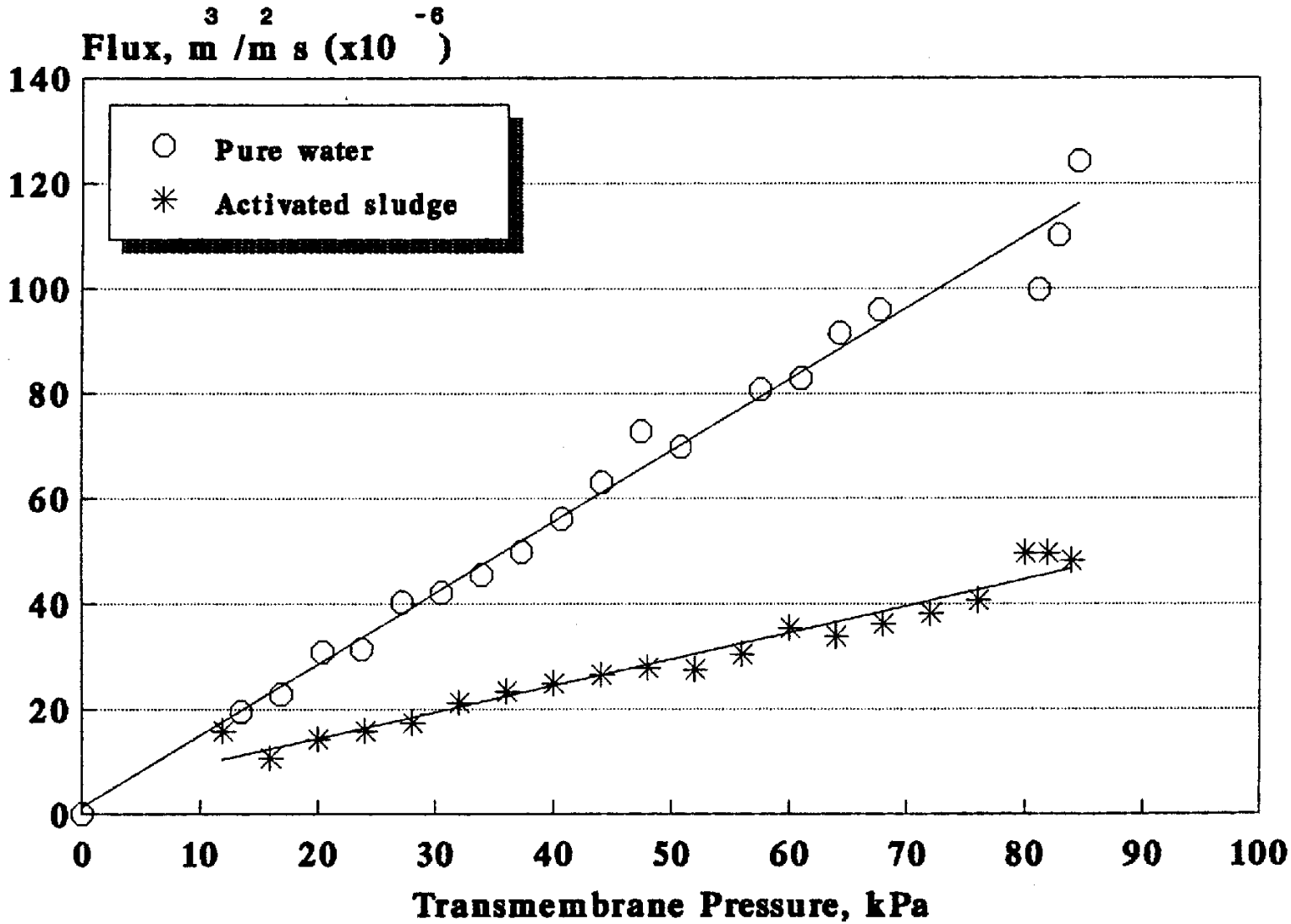

Figure 7. Effect of vaccum pressure on flux, with suspended solids 4470 mg/L.

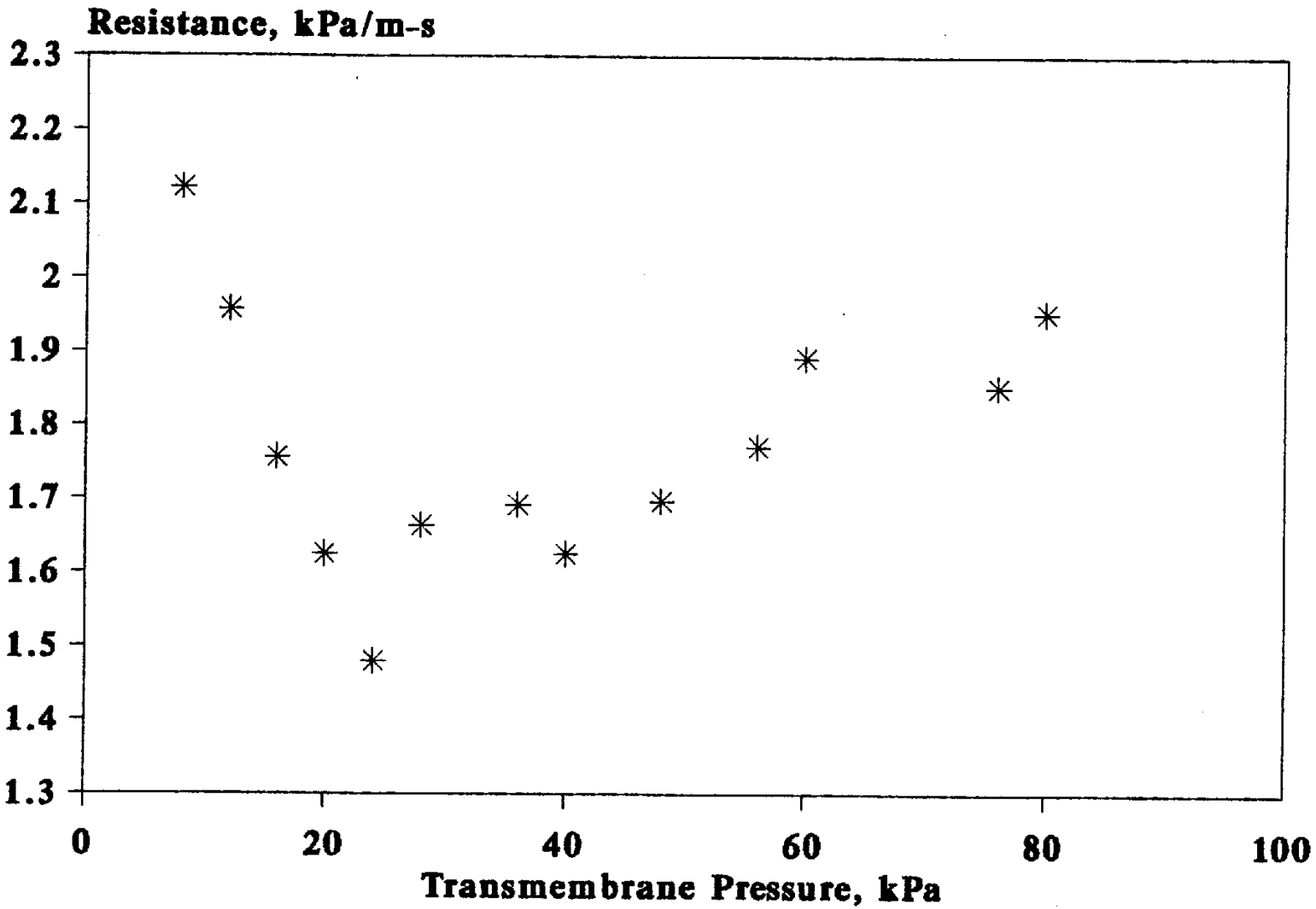

Figure 8. Influence of initial pressure in the membrane resistance

This phenomenon indicates that clogging of the membrane is occurring and becomes worse above 28 kPa. At vacuum pressures superior to this value the performance of the hollow fiber is very poor. This points out that the best condition of operation is between 20-24 kPa.

6.3 VARIATION OF FILTERED VOLUME WITH TIME

The measurements of volume filtered were made under batch conditions at four applied pressures . Typical volume profiles of such filtration are shown in Figure 9. The volume filtered increased rapidly in the first 1 hour, and then it increased slowly. It is important to note that the volume filtered is less at 80 kPa than at 40 kPpa. This indicates that at low pressures the filtration gives better results.

In an operation time of 5 hours, the dewatering of activated sludge achieved a reduction of volume of 57.89% at 60kPa, while at 80 kpa, the reduction of volume of the bioreactor content was 40%. At 40 kpa, 42.21% of reduction was achieved, higher than at 80 kPa. Figure 10 shows the flux observed during the first hour of operation to emphasize the variation of flux at different applied pressures.

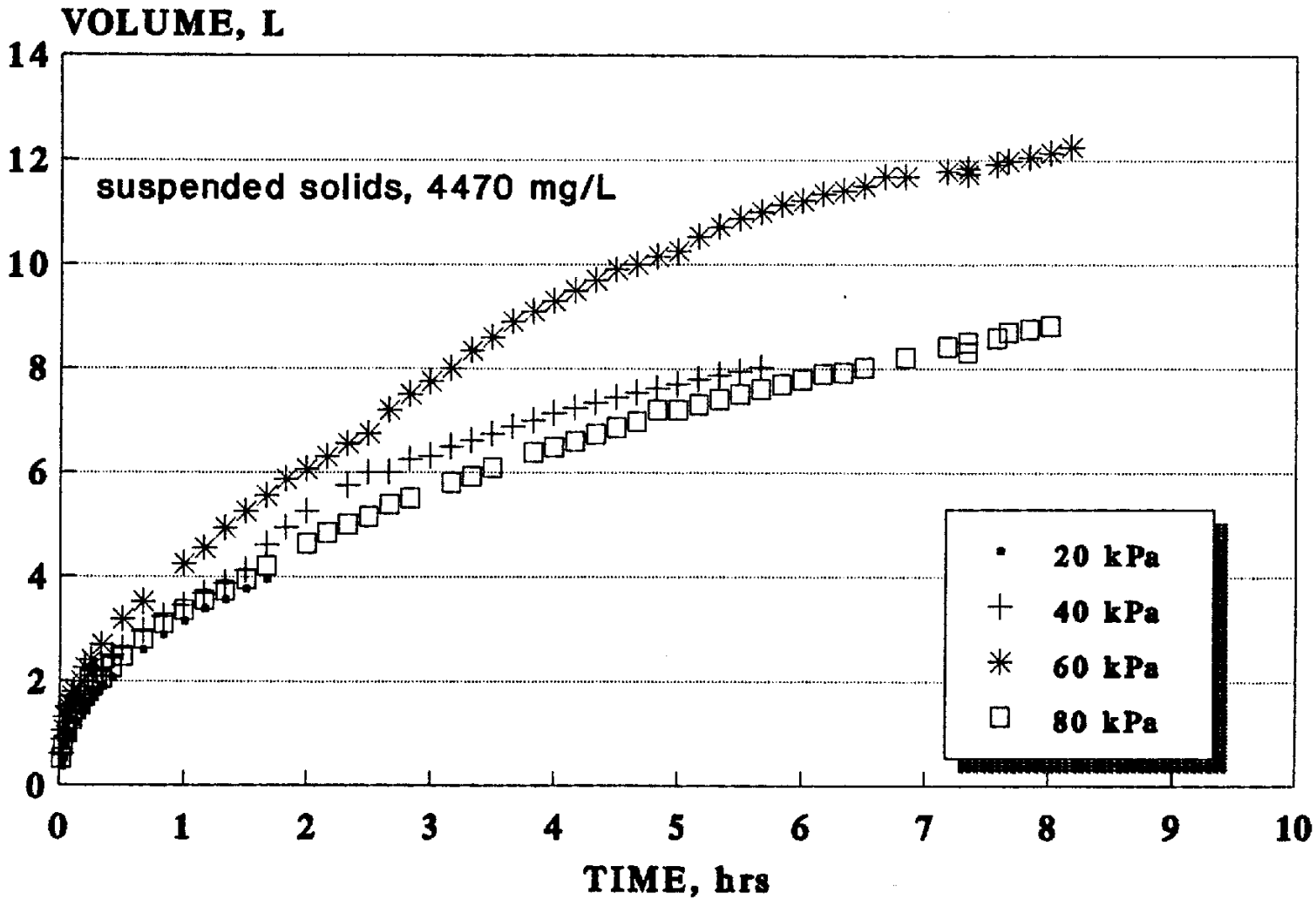

Figure 9. Variation of filtered volume with time.

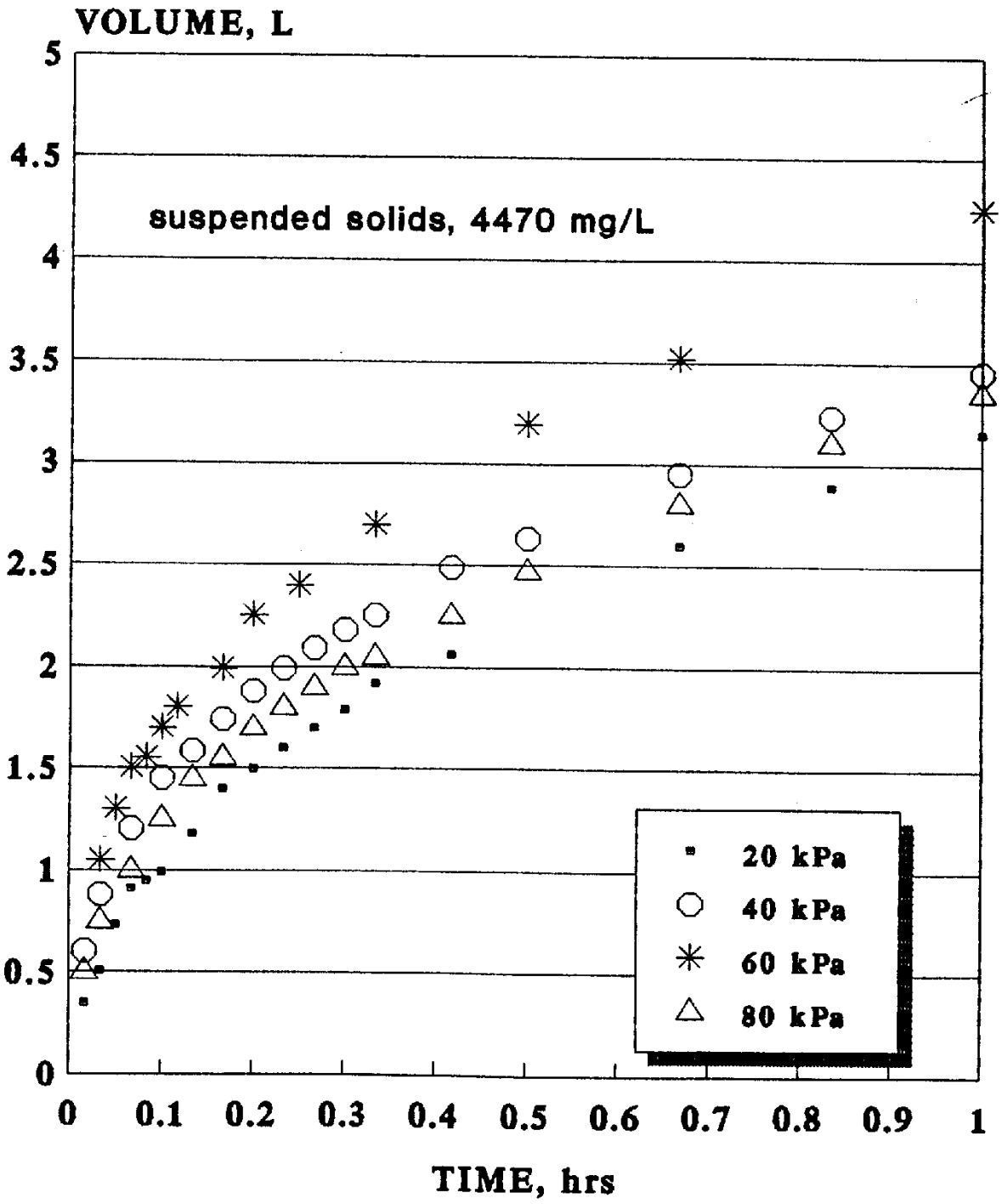

Figure 10. Variation of filtered volume at 1 hour-operation.

6.4 PERMEATION RATE MEASUREMENTS: variation of applied vacuum pressure.

The rate of permeation through the hollow fiber membrane as a function of time at four applied pressures is shown in Figure 11. The initial suspended solids concentration in all the runs was 4470 mg/L. In each case, there is a substantial decline in the permeation rate as the process proceeds due to clogging of the membrane. This phenomenon is more pronunced during the first 0.5 hour, and then the flux stabilizes.

Under the conditions used, a steady state flux was observed after 40 minutes, as shown in Figure 10. A higher flux is observed at 60 kpa than at 80 kpa. This indicate that clogging of the membrane is worse at higher applied suction pressures. Therefore, the increase in resistance to filtration more than offsets the increased driving force.

Figure 12 shows the rate of permeation during the first one hour of operation to better illustrate the effect of membrane clogging. The volume flux observed at 80 kPa, is also less than at 40 kPa.

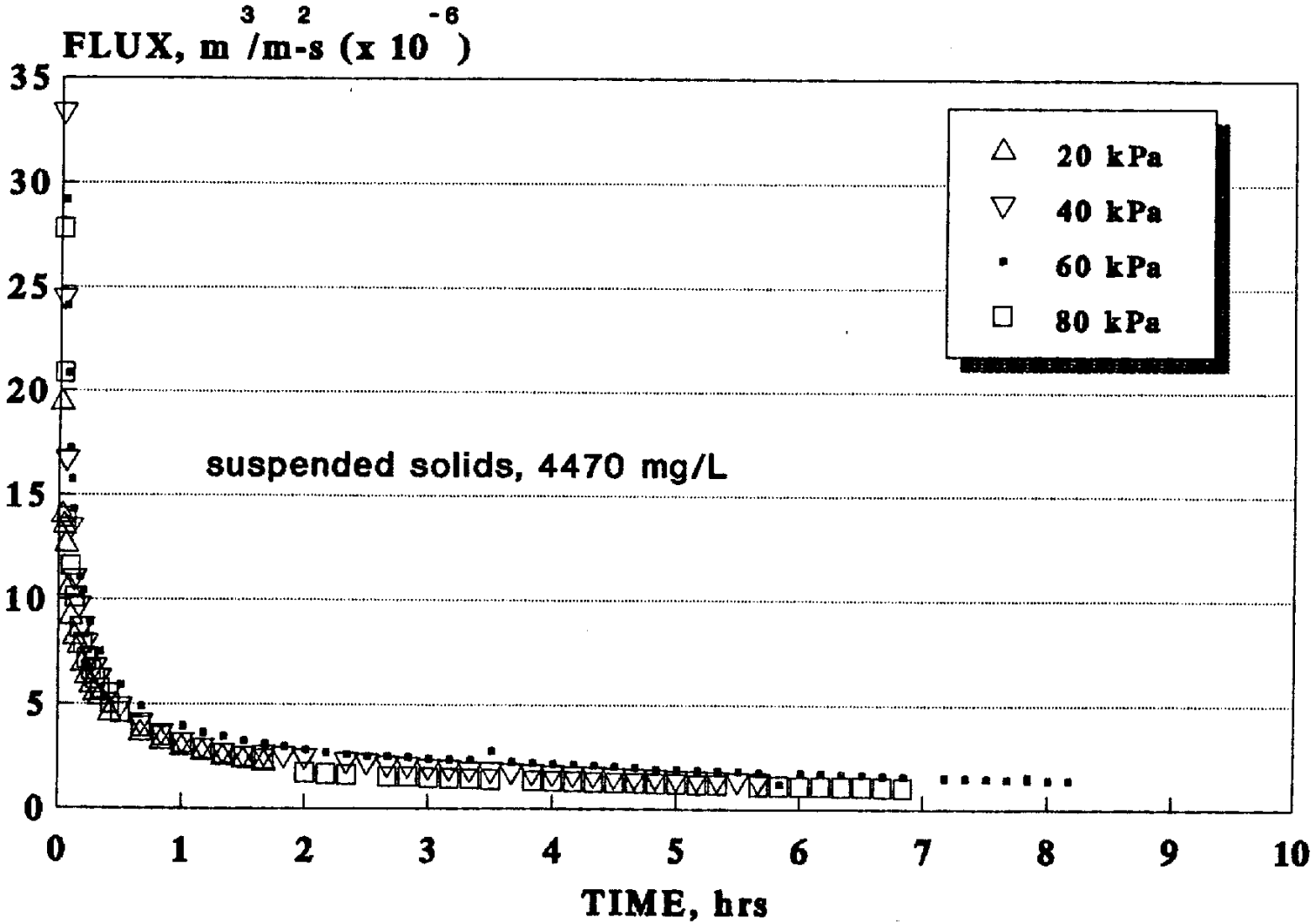

Figure 11. Permeation rate of activated sludge filtration at short term, through 0.1 μm micropore.

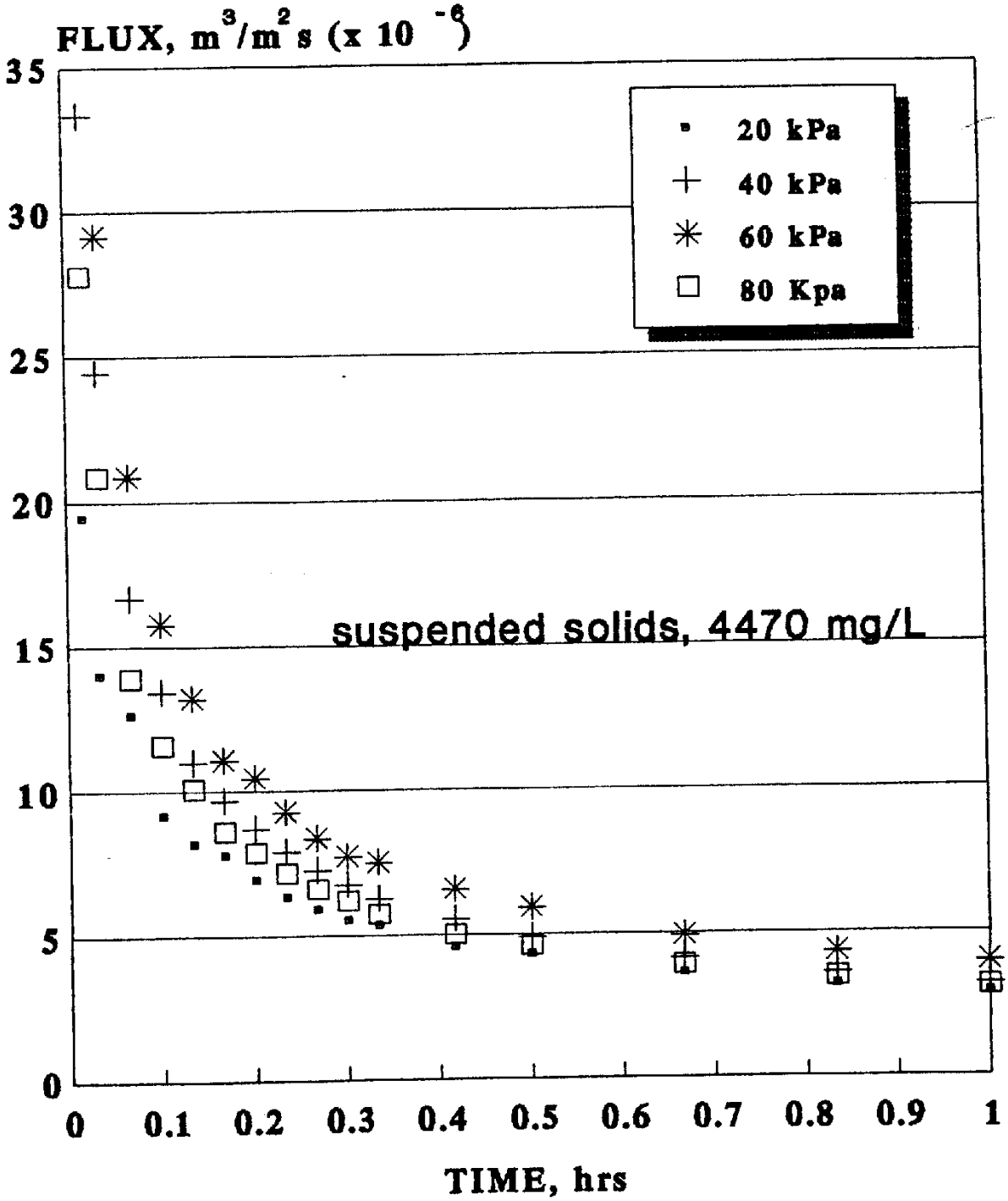

Figure 12. Rate of permeation of activated sludge at 1-hour of microfiltration.

6.5 CHEMICAL OXYGEN DEMAND IN ACTIVATED SLUDGE MICROFILTRATION

The variation of COD in the inlet and oulet to the hollow fiber membrane reactor is shown in Figure 13. The influent COD varied from 1478 to 4472, because, typically, pharmaceutical wastewater are highly concentrated in organic solvents. The effluent COD began high (at the eigth day was 1900 mg/L) due to poor activity of microorganisms, then decreased all the way down to 376 mg/L on the 35th day .

Notice in Figure 13 that betwen the 35th and 52nd day the influent COD increased significantly. The effluent COD followed the same trend. Figure 14 shows that during that period the COD removal efficiency remained relatively constant at about 75%

Figure 14 shows COD percent removal as a function of time. It is clear that COD removal increases as the operation proceeds. It was the lowest, 38% on the first day of filtration, increasing continuosly up to 80% of COD removed on the 61th day. This is the result of the combined action of the microorganisms and the micropore size of the hollow fiber membrane, that rejects all particles bigger than 0.1μm. The average of COD removal along the long term operation was 63%. It is important to point out that, in general, the COD removal increased as the activity of microorganisms advanced.

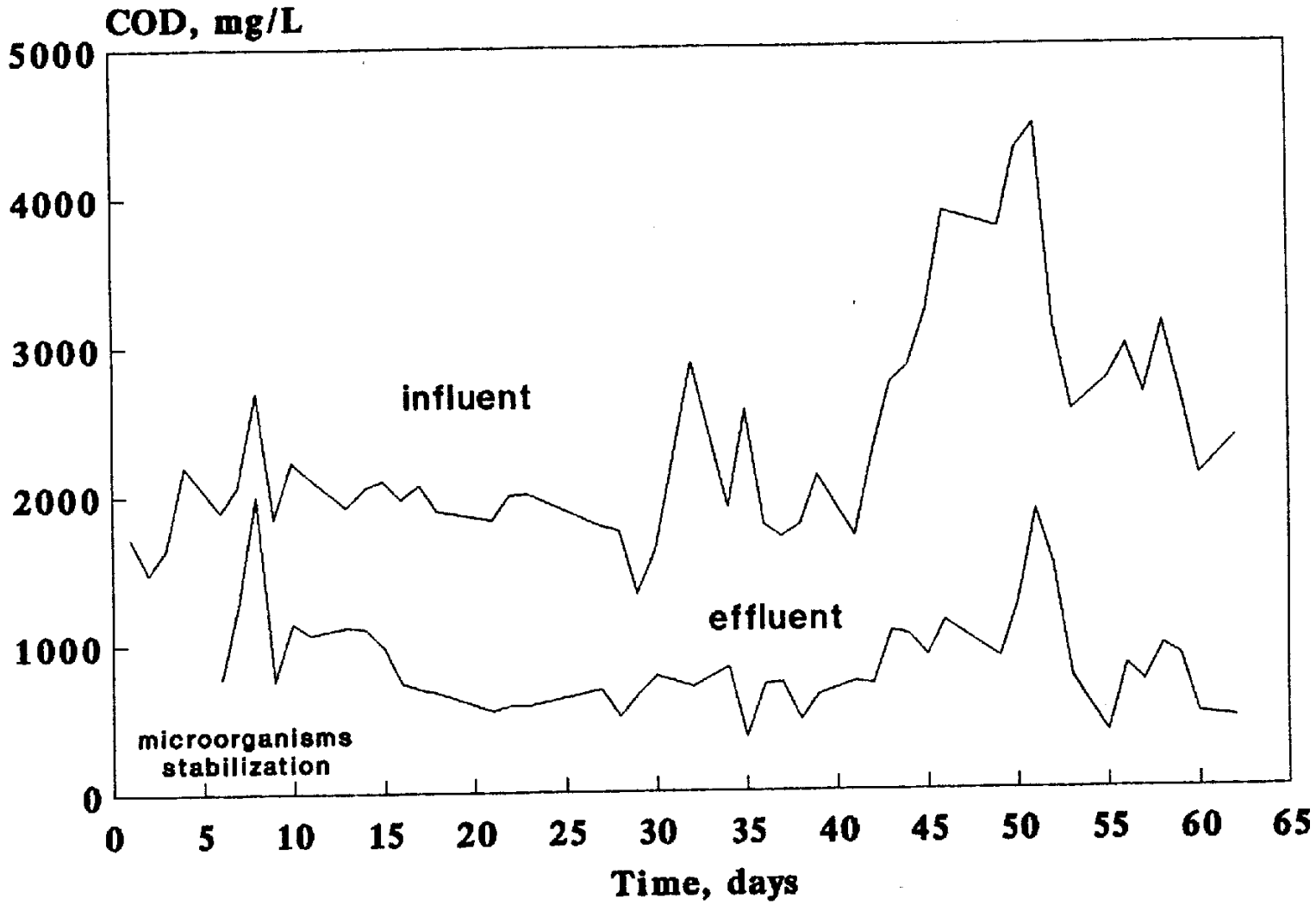

Figure 13. COD variation in the wastewater treatment, through hollow fiber membrane.

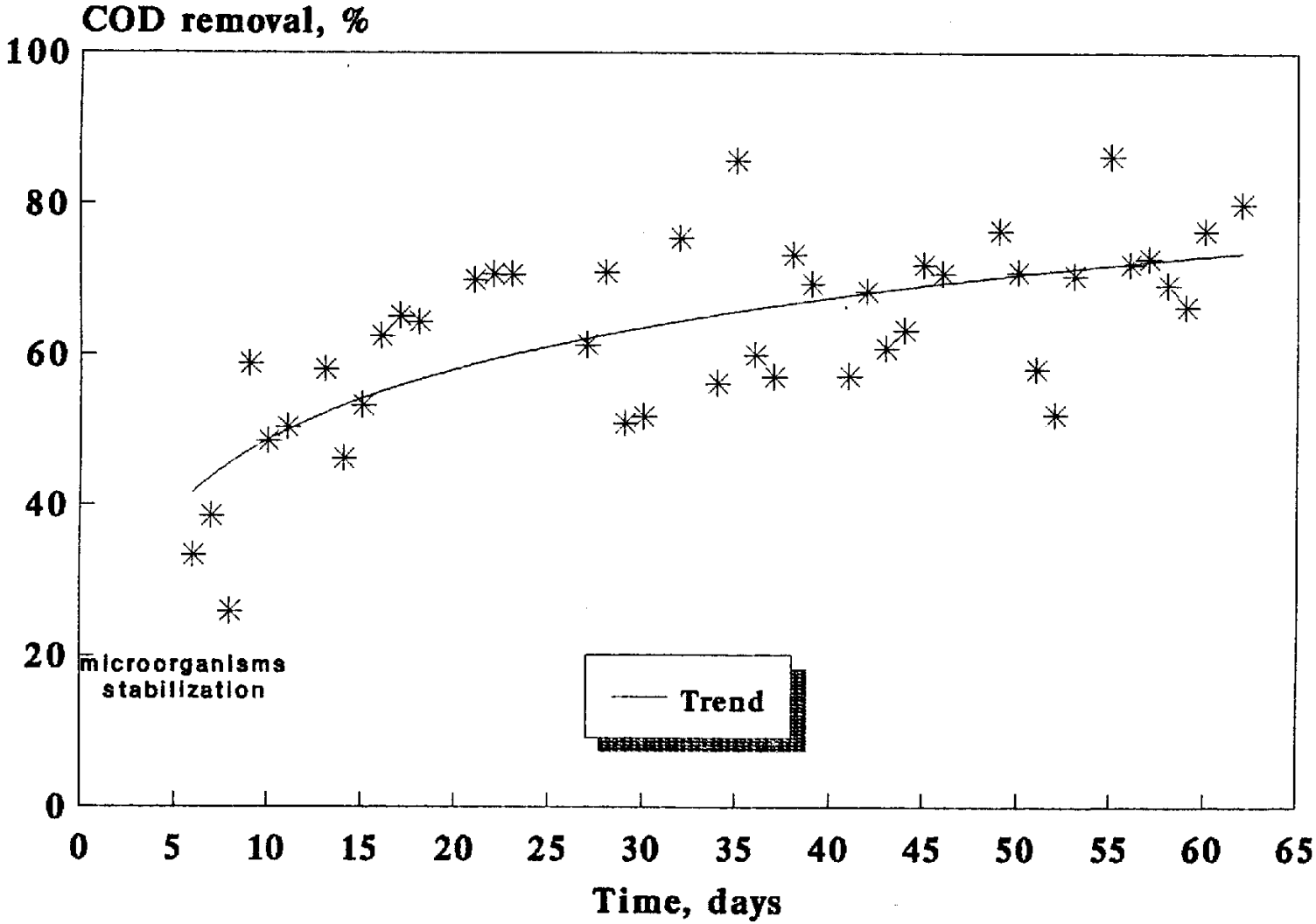

Figure 14. COD removal in activated sludge process of pharmaceutical industry wastewater.

6.6 MIXED LIQUOR SUSPENDED SOLIDS IN THE BIOREACTOR

The variations of MLSS in the influent and inside the bioreactor are shown in Figure 15. The MLSS inside the reactor increased along the operation from an initial value of 6881 mg/L to 37935 mg/L on the afternoon of the 37th day, then decreased slowly, probably due to endogenous respiration.

Is important to notice that the reactor MLSS content shows a significant reduction betwen the afternoon of any day and the morning of the next day as shown in Figure 16. Because of the semi-continuous way in which the experiments were performed, no influent was added during those 14-hr periods. Apparently, endogenous respiration during that period reduced the MLSS inside the reactor until fresh food was added with the influent.

6.7 MIXED LIQUOR VOLATILE SUSPENDED SOLIDS

The MLVSS variation is shown in Figure 17. This followed, in general, the same behaviour exhibited by the MLSS. The influent MLVSS varied between 420 mg/L to 2940 mg/L, and inside the bioreactor between 9091 mg/L and 25920 mg/L. The MLVSS increased coontinuously along the operation. This brought about a decrease in the food to microorganisms factor (F/M ratio), although it always remained above the recommended value of 0.3 kg COD/kg MLVSS-d.

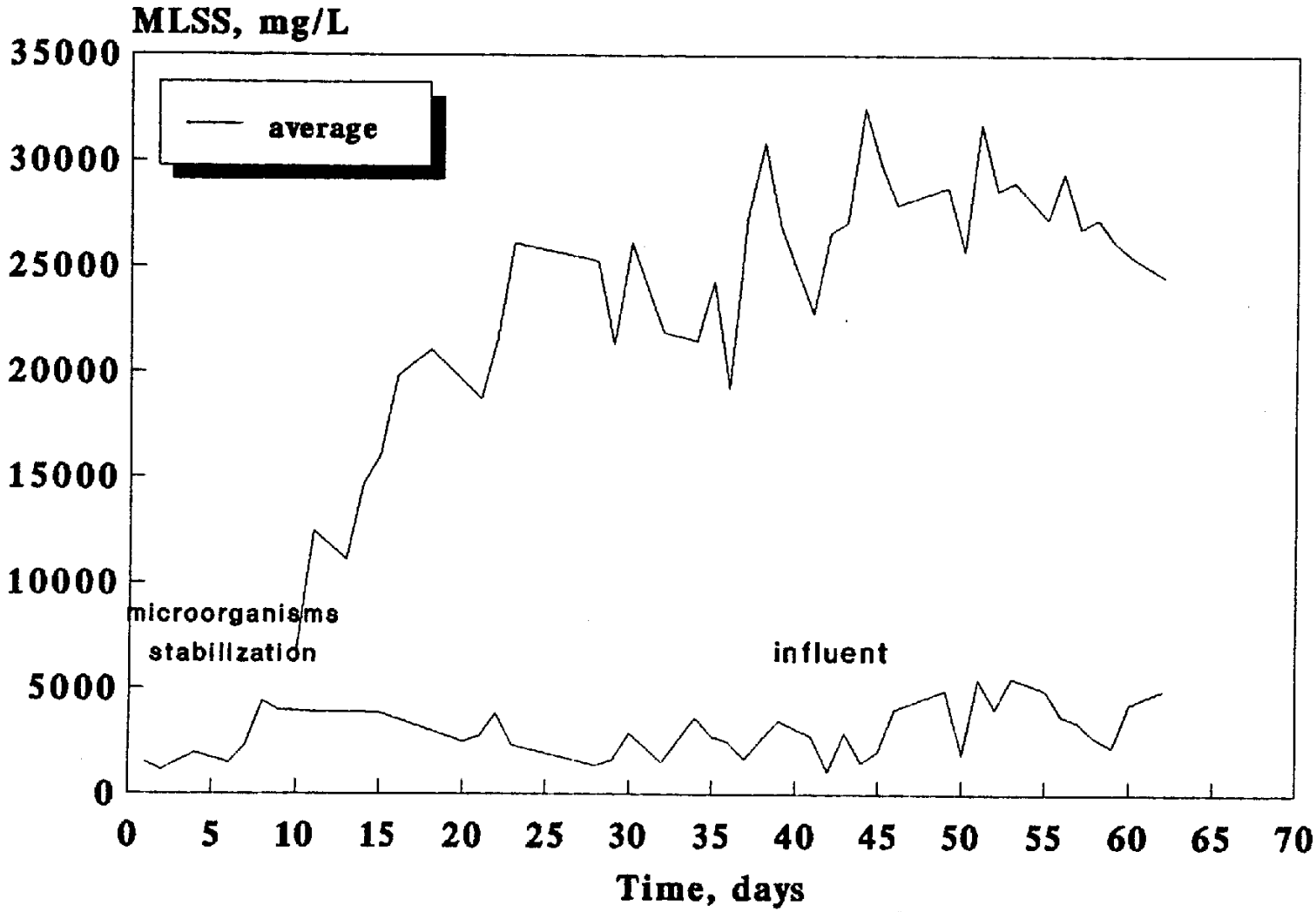

Figure 15. Suspended solids variation in activated sludge microfiltration of industrial pharmaceutical wastewater.

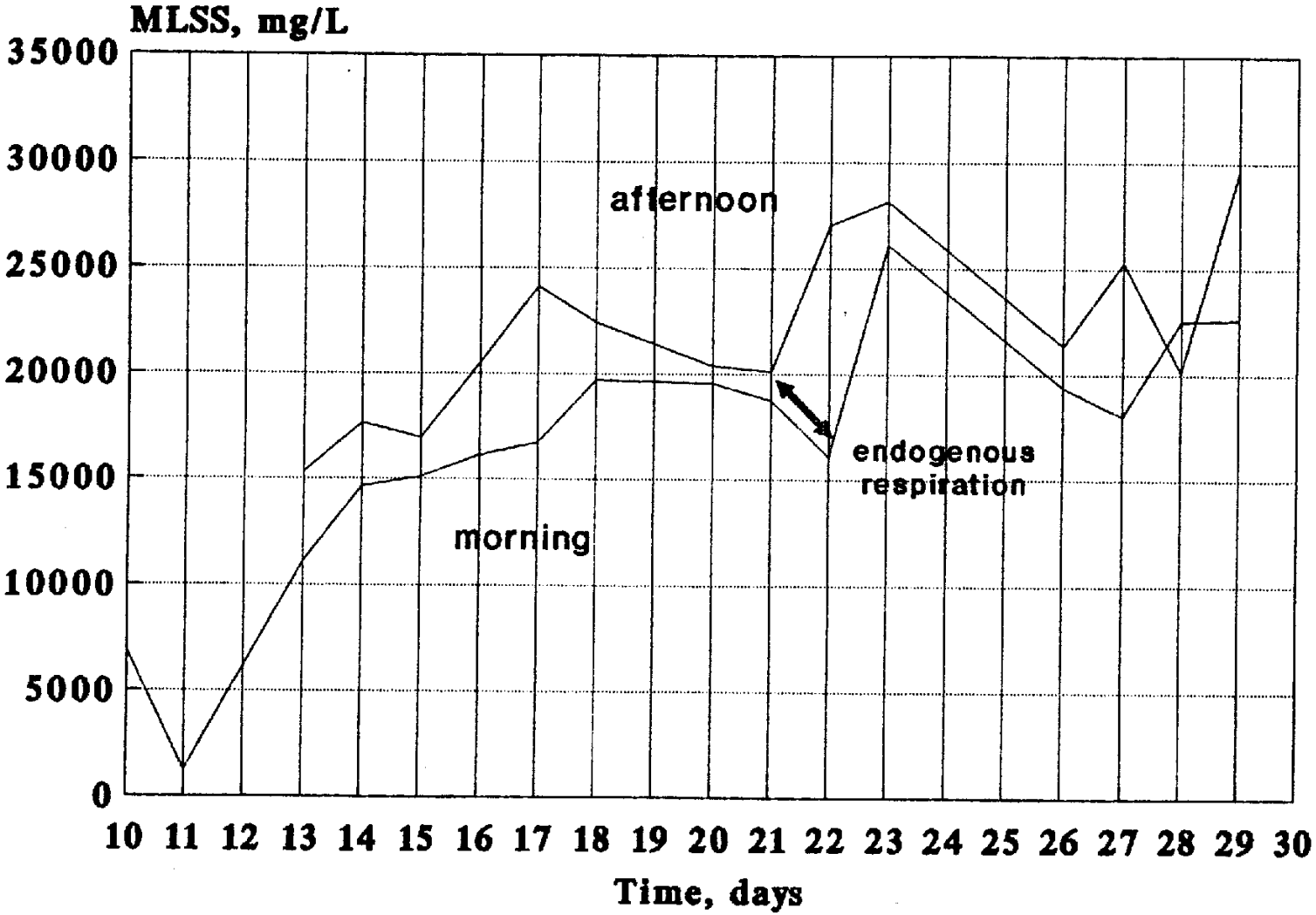

Figure 16. MLSS variation of the afternoon to afterday-morning in the bioreactor.

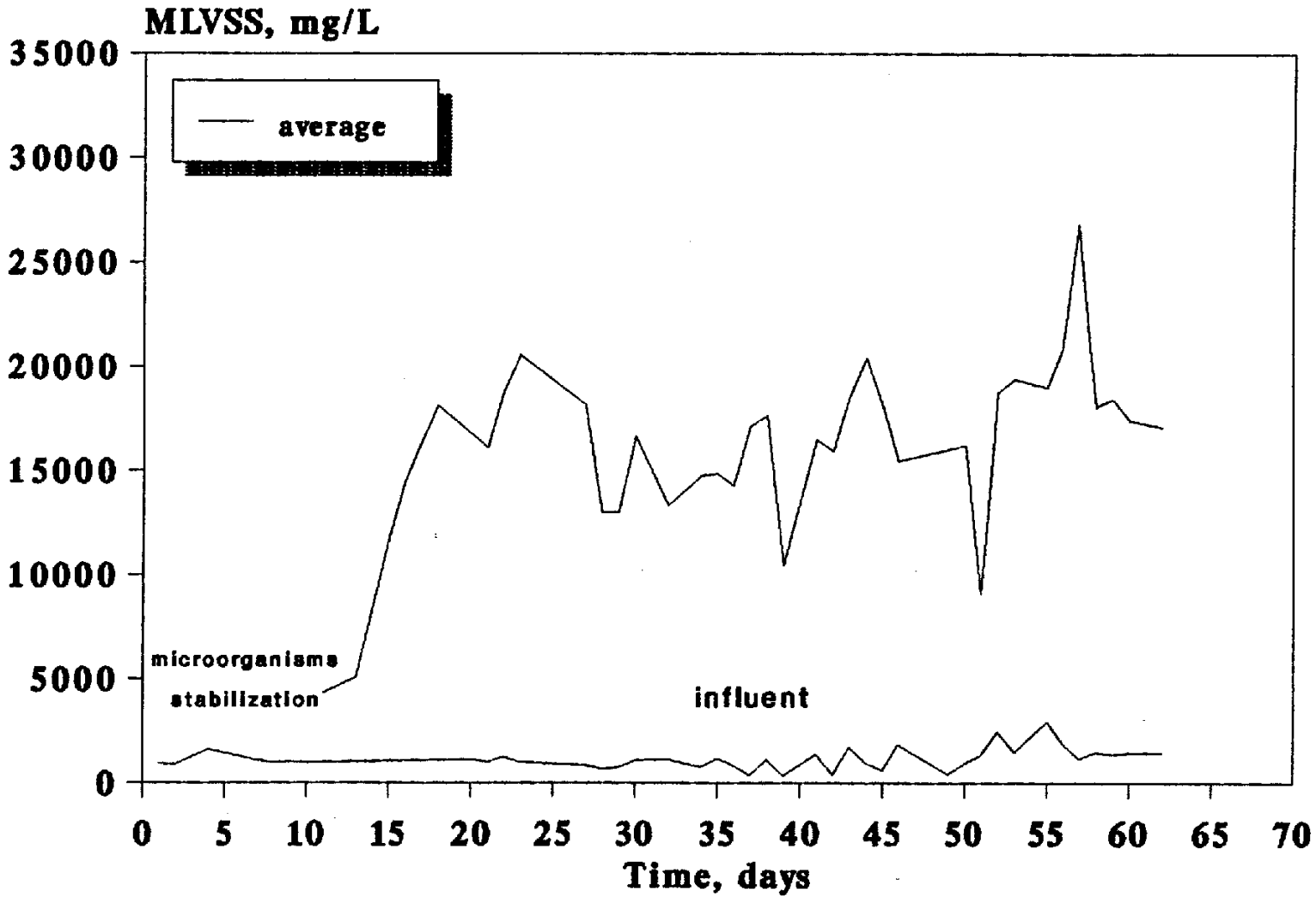

Figure 17. Volatile suspended solids variation in activated sludge microfiltration of industrial wastewater

7. CONCLUSIONS

When microfiltration through hollow fiber membranes is applied to the treatment of industrial wastewater, the efficiency and stability of the dewatering process depends on the mode of suction applied. Intermittent suction at low vaccum pressures shows better performance than continuous suction at pressures higher than 40 kPa. Continuous suction at high pressures makes the unavoidable clogging ocurr faster and irreversibly.

It was observed in preliminary runs that increasing the rate of aeration causes a good agitation, specially during that part of the cycle when no suction is applied (16 minutes). This allows the separation of particles which had adhered to the membrane during suction (some fall down by gravity), and increases the volume flux through the membrane.

In the 62 days of the long term operation, it was not neccesary to waste sludge due to the process stabilization brought about by the activity of microorganisms. Although the MLSS concentration of the bioreactor contents increased considerably during the long term experiment, it never exceded the critical value of 40,000 mg/L. The period of endogenous respiration after 10-hours operation were probably

very helpful in this respect, suggesting that operation in the sequencing batch mode is highly recommended.

8. RECOMMENDATIONS

Additional work is required to optimize the conditions for COD removal. Further research in the effect on the activated sludge process of organic solvents is needed, including the need for pretreatment adequate to the characteristics of the influent wastewater treated. The effect of agitation on flux through the membrane should be studied.

Automatic control of the process will be important in order to experiment continuosly for long periods of time.

Further work should be oriented to the development of a pilot-plant with the final purpose of assessing the true potential of this novel method of wastewater treatment .

Tests should be carried on for long period of time to probe the performance of the membrane, the activity of the microorganisms, and solids disposal needs. The latter represents the most difficult phase of any wastewater treatment process.

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Appendix A. WASTEWATER PROCESS DESIGN PRINCIPLES

The process design for biological wastewater treatment system, uses the same principles for the aerobic and anaerobic contact processes.

1. FOOD TO MICROORGANISMS RATIO

The commonly used parameter, food to microorganism ratio (F/M), since no biomass is present in the effluent, is defined as

$$F/M = \frac{Q_i S_0}{V X} = \frac{S_0}{t X} \qquad (13)$$

where,

Q_i = influent flow rate, volume/time,

S_0 = influent substrate concentration, mass/volume,

V = reactor volume

t = reactor hydraulic retention time

X = reactor biomass concentration, mass/volume.

Volumetric loading (VL), which is defined as

$$VL = \frac{Q_i S_0}{V} = \frac{S_0}{t} \qquad (14)$$

is commonly used in the design of supported growth systems where it is difficult to determine the reactor biomass concentration. F/M can be written as a function of VL by Equation 13 as:

(15)

$$F/M = VL / X$$

X, the biomass concentration, can be expressed in terms of volatile suspended solids (VLSS) in the bioreactor, (Li et al., 1984). Finally

$$F/M = \frac{VL}{(VSS)} \qquad (16)$$

2. RETENTION TIME

The retention time, t_r, is the average time spent by the liquid in the aeration stage and is thus, the time of contact between the sludge and the wastewater .

In a continuos system, this is taken as the volume of the aeration stage divided by the flowrate of the influent liquid, so that

$$t_r = \frac{V_a}{Q_i} \qquad (17)$$

where t_r = retention time, hours or days,

V_a = Volume of liquid in aeration stage, m^3

Q_i = Influent flowrate, m^3/day

The retention time is also called the 'aeration period' , 'detention time', and the 'hydraulic retention time'. The usual retention time in biological wastewater treatment systems is 8-12 hours, at mean flow, (Duncan M., 1976).

Appendix B. Tables of results

Table IV. Effect of transmembrane suction on volume flux

Suction pressure	Flux (m^3/m^2-s x10^{-6})	
(kPa)	Pure water	Activated sludge
10.15	15.72	-
12	-	15.77
13.54	19.43	-
16	-	10.66
16.92	22.76	-
20	-	14.24
20.36	30.88	-
23.69	31.3	-
24	-	15.77
27.07	40.21	-
28	-	17.34
30.46	41.94	-
32	-	21.19
36	45.35	-
37.23	-	23.26
40	49.76	-
40.61	-	24.8
43.99	56.03	-
44	62.92	-
47.38	-	26.40
48	72.74	-
50	-	27.75
76.4	69.77	-
52	-	27.48
56	-	30.33
57.53	80.64	-
60	-	35.37
60.92	82.75	-
64	-	33.75
64.30	91.55	-
67.69	95.83	-
68	-	36.15
72	-	38.12
76	-	40.63
80	-	49.60
81.22	99.68	-
82	-	49.60
82.92	109.99	-
84	-	48.1
84.61	124.19	-

Table V. Influence of initial pressure in the resistance

Suction pressure (kPa)	Resistance (kPa/m-s)
8	2.122
12	1.957
16	1.756
20	1.625
24	1.48
28	1.664
32	-
36	1.692
40	1.625
44	1.911
48	1.697
52	1.914
56	1.771
60	1.892
64	1.709
68	-
72	1.707
76	1.851
80	1.953

Table VI. Variation of filtrated volume with time through membrane

Time	Volume (Liters)			
(hours)	20 kPa	40 kPa	60 kPa	80 kPa
0.017	0.35	0.6	-	0.5
0.033	0.505	0.88	1.05	0.75
	0.73	-	1.3	-
0.067	0.91	1.2	1.5	1.0
0.083	0.95	-	1.55	-
0.1	0.99	1.45	1.7	1.25
0.117	-	-	1.8	-
0.133	1.18	1.58	-	1.45
0.167	1.4	1.74	1.99	1.55
0.2	1.495	1.88	2,25	1.7
0.233	1.6	1.99	-	1.8
0.25	-	-	2.4	-
0.267	1.7	2.09	-	1.9
0.3	1.79	2.18	-	2.0
0.333	1.92	2.25	2.7	2.05
0.417	2.06	2.49	-	2.25
0.5	-	2.63	3.19	2.47
0.667	2.6	2.95	3.52	2.8
0.833	2.89	3.23	-	3.1
1.0	3.15	3.45	4.25	3.35
1.167	3.39	3.68	4.55	3.55
1.333	3.56	3.88	4.94	3.73
1.5	3.77	4.12	5.25	3.95
1.667	3.95	4.62	5.55	4.2
1.833	-	4.95	5.868	-
2.0	-	5.25	6.058	4.63
2.167	-	-	6.295	4.84
2.333	-	5.758	6.55	5.0
2.5	-	5.995	6.75	5.15
2.667	-	6.01	7.2	5.38

Table VI. Variation of filtrated volume with time through membrane (continued)

Time	Volume (Liters)			
(hours)	20 kPa	40 kPa	60 kPa	80 kPa
2.83	-	6.25	7.5	5.5
3.0	-	6.32	7.75	-
3.17	-	6.5	8.0	5.8
3.33	-	6.61	8.035	5.92
4.0	-	6.74	8.06	6.08
4.17	-	6.89	8.09	-
4.33	-	7.0	9.1	6.38
4.5	-	7.14	9.3	6.48
4.67	-	7.25	9.5	6.6
4.83	-	7.35	9.7	6.73
5.0	-	7.45	9.9	6.86
5.17	-	7.54	10.0	6.98
5.33	-	7.62	10.15	7.2
6.0	-	7.7	10.25	7.2
6.17	-	7.79	10.53	7.3
6.33	-	7.87	10.72	7.4
6.5	-	7.94	10.88	7.5
6.67	-	8.02	11.0	7.6
6.83	-	-	11.15	7.69
7.0	-	-	11.22	7.78
7.07	-	-	11.35	7.88
7.33	-	-	11.41	7.92
7.57	-	-	11.5	8.01
7.66	-	-	11.68	-
7.83	-	-	11.68	8.2
8.0	-	-	11.74	8.31
8.17	-	-	11.78	8.41
8.33	-	-	11.84	8.5
8.57	-	-	11.92	8.58
8.66	-	-	11.98	8.69
8.83	-	-	12.05	8.75
9.0	-	-	12.14	8.82

Table VII. Variation of flux with time of microfiltration

Time	Flux (m^3/m^2-s $x10^{-6}$)			
(hours)	20 kPa	40 kPa	60 kPa	80 kPa
0.02	19.44	33.33	-	27.77
	-	24.44	29.17	20.83
	14.03	-	24.07	-
0.07	13.52	16.67	20.83	13.89
0.08	12.64	-	17.22	-
0.1	10.55	13.43	15.74	11.57
0.12	9.17	-	14.29	-
0.13	-	10.97	-	10.07
0.17	8.19	9.67	11.06	8.61
0.2	7.78	8.70	10.42	7.87
0.23	6.92	7.90	-	7.14
0.25	6.35	-	8.89	-
0.27	-	7.26	-	6.60
0.3	5.90	6.73	-	6.17
0.33	5.53	6.25	7.5	5.69
0.42	5.33	5.53	-	5.0
0.5	4.58	4.87	5.91	4.57
0.67	-	4.10	4.89	3.89
0.83	3.61	3.58	-	3.44
1.0	3.21	3.20	3.94	3.10
1.17	2.92	2.92	3.61	2.82
1.33	2.69	2.69	3.43	2.59
1.5	2.47	2.53	3.24	2.44
1.67	2.33	2.57	3.08	2.33
1.83	2.19	2.5	2.96	-
2.0	-	2.43	2.80	1.70
2.17	-	-	2.69	1.64
2.33	-	2.26	2.60	1.61
2.5	-	2.18	2.5	-
2.67	-	2.09	2.5	1.52

Table VII. Variation of flux with time of microfiltration (continued)

Time	Flux (m^3/m^2-s x10^{-6})			
(hours)	20 kPa	40 kPa	60 kPa	80 kPa
2.833	-	2.03	2.45	1.5
3.0	-	1.95	2.39	1.47
3.167	-	1.90	2.34	1.44
3.333	-	1.84	2.32	1.41
3.5	-	1.78	2.75	1.38
3.667	-	1.74	2.25	-
3.833	-	1.69	2.20	1.33
4.0	-	1.65	2.15	1.31
4.167	-	1.61	2.11	1.28
4.333	-	1.57	2.07	1.26
4.5	-	1.53	2.04	1.24
4.667	-	1.50	1.98	1.22
4.833	-	1.46	1.90	1.20
5.0	-	1.43	1.89	1.18
5.167	-	1.40	1.86	1.16
5.333	-	1.37	1.83	1.14
5.5	-	1.34	1.80	-
5.666	-	1.31	1.77	1.11
5.833	-	-	1.20	1.09
6.0	-	-	1.73	1.08
6.167	-	-	1.70	1.07
6.333	-	-	1.67	1.06
6.5	-	-	1.64	1.05
6.667	-	-	1.61	1.03
6.833	-	-	1.58	1.02
7.0	-	-	1.55	-
7.167	-	-	1.52	-
7.333	-	-	1.49	-
7.5	-	-	1.47	-
7.667	-	-	1.45	-
7.833	-	-	1.42	-
8.0	-	-	1.41	-
8.167	-	-	1.38	-

Table VIII. Variation of COD in activated sludge microfiltration

Time	COD(mg/L)		COD removal
(days)	Influent	Effluent	(%)
1	1723	-	-
2	1478	-	-
3	1648	-	-
4	2198	-	-
5	-	-	-
6	1894	779	33.32
7	2067	1270	38.55
8	2698	1998	25.92
9	1848	762	58.82
10	2227	1146	48.52
11	2119	1066	50.32
12	-	-	-
13	1925	1117	58.03
14	2050	1104	46.15
15	2099	981	53.26
16	1975	741	62.48
17	2071	701	65.15
18	1895	675	64.38
19	-	-	-
20	-	-	-

Table VIII. Variation of COD in ativated sludge microfiltration (continued)

Time	COD (mg/L)		COD removal
(days)	Influent	effluent	(%)
21	1830	550	69.95
22	1998	585	70.72
23	2005	588	70.67
24	-	-	-
25	-	-	-
26	-	-	-
27	1786	690	61.37
28	1756	509	71.01
29	1333	654	50.94
30	1626	783	51.84
31	-	-	-
32	2882	706	75.5
33	-	-	-
34	1905	834	56.22
35	2560	365	85.74
36	1786	714	60.02
37	1707	732	57.13
38	1786	476	73.35
39	2118	647	69.45
40	-	-	-

Table VIII. Variation of COD in activated sludge microfiltration (continued)

Time	COD (mg/L)		COD removal
(days)	Influent	effluent	(%)
41	1707	732	57.14
42	2262	714	68.42
43	2738	1071	60.87
44	2849	1046	63.27
45	3220	904	71.93
46	3882	1136	70.73
47	-	-	-
48	-	-	-
49	3777	889	76.46
50	4304	1250	70.96
51	4472	1875	58.07
52	3125	1500	52.0
53	2543	750	70.51
54	-	-	-
55	2750	375	86.36
56	2976	833	72.0
57	2651	723	72.73
58	3133	964	69.23
59	2650	892	66.36
60	2099	494	76.47
61	-	-	-
62	2353	471	80.0

Table IX. Variation of suspended solids

Time	Suspended solids (mg/L)		
(days)	Influent	Effluent	
		morning	afternoon
1	1491	-	-
2	1133	-	-
3	-	-	-
4	1930	-	-
5	-		-
6	1460	-	-
7	2309	-	-
8	4383	-	-
9	3957	-	-
10	3920	6881	-
11	-	12445	-
12	-	-	-
13	-	11118	15320
14	-	14633	17652
15	3855	15100	16967
16	-	19125	20500
17	-	16733	24133
18	-	19700	22433
19	-	-	-
20	2485	19540	20365
21	2756	18740	20125
22	3835	16120	27020
23	2320	26120	28152
24	-	-	-
25	-	-	-
26	-	-	-
27	1545	19384	21354
28	1343	18020	25294
29	1610	22540	20120
30	2900	22600	29660

Table IX. Variation of suspended solids (continued)

Time	Suspended solids (mg/L)		
(days)	Influent	Effluent	
		morning	afternoon
31	-	-	-
32	1500	21882	23154
33	-	-	-
34	3660	19520	21520
35	2750	17220	31400
36	2490	18640	19260
37	1671	27300	28150
38	2620	23780	37935
39	3500	22440	31300
40	-	-	-
41	2760	22581	23040
42	1083	23925	29340
43	2920	26694	27500
44	152	30420	34620
45	2040	29220	30523
46	4040	26300	29580
47	-	-	-
48	-	-	-
49	4940	28220	29320
50	1920	24340	27160
51	5460	34300	29240
52	4080	27680	29540
53	5520	28500	29520
54	-	-	-
55	4960	24400	30080
56	3740	29500	29380
57	3400	25920	27700
58	2660	28500	25980
59	2240	23820	28520
60	4300	25320	25640
61	-	-	-
62	4960	24540	-

Table X. Variation of volatile suspended solids

Time	Volatile suspended solids (mg/L)		
(days)	Influent	Activated sludge	
		morning	afternoon
1	940	-	-
2	890	-	-
3	-	-	-
4	16	-	-
5	-	-	-
6	1260	-	-
7	1090	-	-
8	980	-	-
9	1023	-	-
10	970	-	-
11	-	1010	7627
12	-	-	-
13	-	9091	1023
14	-	-	-
15	1050	10250	13033
16	-	13200	15767
17	-	13800	18800
18	-	15700	20533
19	-	-	-
20	1110	-	-
21	985	14920	17250
22	1240	14240	23300
23	995	19640	21450
24	-	-	-
25	-	-	-
26	-	-	-
27	850	18990	17358
28	687	15380	10580
29	730	15423	10580
30	1070	16200	17040

Table X. Variation of volatile suspended (continued)

Time	Volatile suspended solids (mg/L)		
(days)	Influent	Activated sludge	
		morning	afternoon
31	-	-	-
32	1100	12360	14256
33	-	-	-
34	760	13900	15580
35	1148	10640	18980
36	850	14280	14220
37	380	16080	18120
38	1100	13160	22087
39	340	11380	9500
40	-	-	-
41	1360	15046	17880
42	400	15625	16200
43	1680	19734	17380
44	960	20060	20740
45	600	19280	16920
46	1820	13580	17320
47	-	-	-
48	-	-	-
49	420	-	-
50	940	15080	17260
51	1300	1060	7040
52	2440	17760	19740
53	1458	18500	20300
54	-	-	-
55	2940	16400	21520
56	1860	20860	20840
57	1160	25920	27700
58	1460	18420	17680
59	1360	17080	19800
60	1440	17720	17100
61	-	-	-
62	1440	17080	-