A SOUTH AFRICAN DESIGN GUIDE FOR DISSOLVED AIR FLOTATION

By Johannes Haaroff

WRC Report No TT 60/93

A SOUTH AFRICAN DESIGN GUIDE FOR DISSOLVED AIR FLOTATION

Report for the Water Research Commission

Johannes Haarhoff
GFJ INC., CONSULTING ENGINEERS

Lucas van Vuuren
BS BERGMAN & PARTNERS, CONSULTING ENGINEERS

WRC Project No 332
TT 60/93

1993

Available from:

Water Research Commission
P O Box 824
PRETORIA
0001
Republic of South Africa

ISBN 1 874858 94 2

This publication stems from a research project entitled:

Project No. 332 : *The Development of Guidelines for the Design and Application of Dissolved Air Flotation / Filtration Processes* that was carried out by BS Bergman & Partners and GFJ Inc

FOREWORD

The use and acceptance of dissolved air flotation (DAF) as a process for the separation of floatable suspended material from a liquid are gaining momentum world-wide. Thickening of sludges, industrial effluent, treatment and water purification are applications where increasing numbers of DAF units are commissioned each year.

Over the last 30 years the hesitant steps of a toddler process changed into purposeful strides with the mark of approaching maturity. However, DAF may still be regarded as a relatively young candidate for solids separation when compared to such venerable clarification processes as sedimentation and filtration. Possibly because of its relative youth there is a marked dearth of information on DAF in some current textbooks. The designer lacking access to proprietary DAF design know-how, is consequently left somewhat in the dark regarding an understanding of the underlying principles of the process and detailed design thereof.

This guide, therefore, aims to provide some insight into the DAF process and its design by integrating local experience with both local and international research results. However, as the chapter on research needs points out, a rather tortuous path to process maturity still lies ahead, but with many exciting challenges and rewards for the interested researcher with an inquiring mind.

I trust that this guide will both assist the designer in despair and stimulate the resourceful researcher into contributing toward a better understood, more efficient and cost-effective dissolved air flotation process.

PE Odendaal
EXECUTIVE DIRECTOR
Water Research Commission

EXECUTIVE SUMMARY

This design manual deals with the application of dissolved air flotation to the fields of drinking-water treatment and activated sludge thickening. Two applications are typically encountered, namely:

- The clarification of eutrophic surface water for drinking-water purposes, or of sewage effluent prior to disposal or secondary reuse.

- The primary thickening of sewage sludge, typically waste activated sludge, prior to disposal or secondary dewatering.

Chapter Two starts out with a comprehensive literature review on dissolved air flotation, under the following main headings:

- The reaction zone
- The requirements for chemical pretreatment
- The bubble production system
- The flotation zone
- The removal of the float layer.

Much of the underlying theory is common to both clarification and thickening applications, but the areas where the two applications diverge, are highlighted.

Chapter Three describes the development of the use of dissolved air flotation in Southern Africa from the early days when it was first used in the historical waste-water reclamation in Windhoek during the sixties. The fundamental and applied academic research, coupled with a multitude of pilot projects at numerous locations and applications, led to the contraction of any full-scale plants that are still successfully operating today.

Chapter Four then moves onto a detailed survey of 26 dissolved air flotation plants in Southern Africa, covering both clarification and thickening applications. Good data could be compiled on the design parameters, and what the current operating parameters are. An attempt was also made to assess the efficiency of the process. These data are less complete, but nevertheless yielded useful information. In general it was found that there is relatively little variation in the design parameters amongst the clarification plants, but that much more variation exists amongst the thickening plants. Not surprisingly, there is a similar difference in perception amongst the operational personnel. A positive attitude towards dissolved air flotation was the rule where it was used for clarification, while mixed responses were received where it was used for thickening. A major area of complaints for both thickening and clarification is that of unreliability of, and maintenance problems with, mechanical equipment, particularly the float scrapers.

From the data of the plant survey, as well as from the literature study, a set of empirical design parameters could be formulated in Chapter Five. Not all parameters could be satisfactorily defined; in such cases a guideline value was omitted or tentatively set. In the case of defining reaction zone turbulence, some difficulty exists in finding an appropriate parameter and in defining the boundaries of the reaction zone. The authors adopted the simplistic, tentative parameter of mean flow velocity through the reaction zone.

The practical use of the empirical guidelines is demonstrated in Chapter Six for both a clarification and thickening application. The typical calculation for a fixed injection nozzle is illustrated additionally, as well as a way of analytically handling an adjustable recycle flow rate.

An assessment of the current research needs is finally made in Chapter Seven. The authors concur with recent international views that, while a great deal of research went into initially demonstrating that dissolved air flotation could be a viable alternative to other better known phase separation processes, little research had been subsequently directed at optimising the numerous variables for different applications. A number of specific research needs are pointed out which were encountered during the survey of Southern African dissolved air flotation plants.

A SOUTH AFRICAN DESIGN GUIDE FOR DISSOLVED AIR FLOTATION

LIST OF FIGURES

Chapter 2

Chapter 6

LIST OF TABLES

LIST OF SYMBOLS

a_p	air mass precipitated at atmospheric pressure	$mg.\ell^{-1}$
a_s	air/solids mass ratio	$mg.mg^{-1}$
A_c	crossflow area between reaction and flotation zone	m^2
A_f	plan area of flotation zone	m^2
A_r	plan area of reaction zone	m^2
C	air mass concentration in feed stream	$mg.\ell^{-1}$
C_d	nozzle discharge coefficient	-
C_F	percentage suspended solids in float layer	%
C_s	saturation air concentration at atmospheric pressure	$mg.\ell^{-1}$
d_F	depth of float layer below water level	m
d_p	nominal packing size	m
d_w	depth of float layer above water level	m
D_f	side depth of flotation zone	m
D	inside diameter of flotation zone (circular tank)	m
D_o	outside diameter of flotation zone (circular tank)	m
D_r	depth of reaction zone	m
D_s	diameter of saturator	m
h_n	head loss across injection nozzle	m
H_p	packing height	m
L	overall length of flotation tank (rectangular tank)	m
L_s	length or height of saturator	m
L/W	length/width ratio (rectangular tank)	-
η_s	saturator transfer efficiency	-
n	number of injection nozzles	-
P	saturator pressure	kPa
q	float layer removal rate	$m^3.h^{-1}$
Q	raw water flow rate	$m^3.h^{-1}$
Q.(1 + r)	total flow rate	$m^3.h^{-1}$
Q_s	solids loading rate	$kg.m^{-2}.h^{-1}$
r	recycle ratio	-
r.Q	recycle flow rate	$m^3.h^{-1}$
subscript a	actual	-
subscript d	design	-
subscript f	flotation zone	-
subscript F	float layer	-
subscript r	reaction zone	-

subscript s	saturator	-
subscript T	at temperature T	-
S	solubility constant	$mg.\ell^{-1}.kPa^{-1}$
SS_{in}	suspended solids in raw water	$mg.\ell^{-1}$
SS_{out}	suspended solids in effluent	$mg.\ell^{-1}$
T	temperature	°C
V_c	crossflow velocity	$m.h^{-1}$
V_f	hydraulic loading in flotation zone	$m.h^{-1}$
V_L	limiting hydraulic loading	$m.h^{-1}$
V_r	hydraulic loading in reaction zone	$m.h^{-1}$
V_s	hydraulic loading in saturator	$m.h^{-1}$
W	width of flotation tank (rectangular tank)	m

CHAPTER ONE

INTRODUCTION

1.1 Relevance of dissolved air flotation

The separation of suspended solids from the aqueous phase in water and waste-water engineering is conventionally achieved by sedimentation. The efficiency of sedimentation, however, is severely impaired if the solids have low density, and/or have a high organic content. A decrease in sedimentation efficiency could be highly detrimental; in water treatment it could lead to solids carry-over and rapid clogging in the subsequent filtration step; in waste-water treatment it could lead to either unacceptable solids carry-over or insufficient sludge thickening.

For these reasons, there had been a growing interest in the alternative solids/liquid separation process of dissolved air flotation (DAF) all over the world. In the Scandinavian countries, DAF had been successfully applied for clarification and thickening for many years. About 20 years ago a concerted research programme was launched in the United Kingdom into the clarification of surface waters by DAF. In the USA, during the same time, the emphasis was primarily on DAF as an alternative method to thicken waste-water sludge. A recent publication describes the introduction of DAF at full scale in the Netherlands about ten years ago, and the growing application of the process in Central Europe ever since. Case studies of large DAF applications had also been reported from Australia, Brazil, Canada and others.

In South Africa, with scarce and often highly eutrophic water sources, there had been a keen interest in the development and application of DAF during the past 30 years. A great deal of effort had gone into experimentation with DAF at laboratory, pilot and full scale, on a great number of industrial effluents, eutrophic drinking-water sources, and water and sewage sludges. The results of these efforts led to successful full-scale applications as early as 1969. The DAF process was generally embraced as a viable, robust alternative since the late seventies, as evidenced by the substantial increase in constructed plants since the early eighties.

The need for this guide was born after the realization that a great deal of SA design and operational experience had been accumulated over the years, and that it would be of value to local and overseas designers alike to record this experience together with a critical analysis of the key design parameters.

1 2 Objectives of this guide

The authors and the Water Research Commission recognized the need for a guide which would accomplish the following:

- To record the development and refinement of DAF in Southern Africa during the past three decades.
- To analyse a number of typical SA DAF plants in terms of applications, operational variables and performance
- To scan the literature for fundamental parameters and design criteria.
- To establish appropriate practical design parameters
- To identify critical design areas and related research needs.

1.3 Presentation of information

This guide is organized into four main parts, namely

- A review of the technical literature, with the view to summarize the present state of knowledge (Chapter Two), as well as to record the historical development of the process in SA (Chapter Three);
- A comparative analysis of 26 SA DAF plants which were selected as representative case studies (Chapter Four);
- Recommendations for design (Chapter Five) together with illustrative design examples (Chapter Six), and
- Identification of research needs (Chapter Seven).

CHAPTER TWO

FUNDAMENTALS OF DISSOLVED AIR FLOTATION

2.1 General

A wealth of information on the fundamentals and applications of flotation is currently available in the technical literature. This chapter does not attempt to provide a comprehensive review of all the available sources. It rather aims to provide a concise background on the theory and practice of flotation - information which will prepare the reader for the chapters to follow. For this reason, the published information was rigorously screened for relevance to this guide.

This chapter, therefore, is firstly confined to *dissolved air flotation* (DAF), the process whereby small bubbles are precipitated when water, supersaturated with air under high pressure, is released under atmospheric conditions. It therefore excludes other methods of bubble generation such as *electrolytic flotation* (where gases are electrochemically produced at electrodes immersed in the flotation tank) and *dispersed flotation* where air is directly injected into the recycle stream via a diffuser or on the suction side of the recycle pump. The *vacuum* DAF option (where the air is saturated under atmospheric conditions and the entire flotation tank is then subjected to a partial vacuum) is never used in the water industry (with the exception of a few early Scandinavian plants (Rosen & Morse, 1976)) and is thus also not considered.

This guide is secondly confined to the production of *potable water* and *treatment of municipal waste water*. Flotation also finds many other applications in industry and mining, but here these applications are not of primary importance and are only discussed where they have particular relevance to water and waste-water treatment. Of special interest here is the work done in South Africa on the application of DAF to the paper and pulp industry (Offringa *et al.*, 1987; Offringa, 1986).

There are generally two applications for DAF in the water industry, namely *clarification* (with the emphasis on the quality of the subnatant or effluent) or *thickening* (with the emphasis on the character of the float layer on the surface of the tank). There are, however, a number of areas which are common to all DAF applications, regardless of whether clarification or thickening are most important. With this common ground in mind, the contents of this and the following chapters will be arranged in the following way :

- Reaction zone: Flotation is more complex than other phase separation processes, in the sense that three phases are involved: the solid phase in the form of particles, the liquid phase in the form of water, and the gas phase in the form of air bubbles. The particles, water and bubbles come together for the first time in the reaction zone to form stable particle/bubble agglomerates which are essential for successful flotation.
- Chemical pretreatment: It is often necessary to alter the nature of the particles before successful flotation can be achieved. The pretreatment requirements in terms of treatment chemicals, coagulation and flocculation will be reviewed.
- Bubble production system: Bubble production comprises the abstraction and pumping of treated water (the recycle) through an air saturation system, and injection of the supersaturated water into the reaction zone. This system requires practically all the energy required by flotation and optimisation thereof is thus of economic importance.

- Flotation zone: After formation of stable particle/bubble agglomerates, the sought after phase separation takes place in the flotation zone. The interrelationship between hydraulic loading, solids loading, air requirements, effluent solids concentration, float solids concentration, *et cetera* will be analysed.
- Float layer removal: The various methods of float layer removal each have distinct advantages and disadvantages, for sludge consistency and subnatant quality alike.
- Combined flotation/filtration: This particular DAF application has a number of special considerations, dealing mostly with the filterability of the subnatant from the flotation zone.

2.2 Reaction zone

The first step towards successful flotation is the formation of stable particle/bubble agglomerates which will rise to the top of the flotation zone. These agglomerates are formed in the reaction zone, where the pressurized recycle from the saturation system is released in close proximity to the incoming feed which carries the bulk of the particles to be floated. Practically all published accounts assign great importance to this zone and it is considered to be of crucial importance for the success of subsequent flotation.

Formation models for particle/bubble aggregates

Four conceptual models for the formation of particle/bubble aggregates have been proposed (Kitchener & Gochin, 1981; Schade, 1982; Vrablik, 1959) namely:

- Model A adhesion of bubbles to preformed flocs.
- Model B mechanical enmeshment of bubbles with preformed flocs.
- Model C incorporation of bubbles into growing flocs.
- Model D growth of bubbles on nuclei within flocs.

The first three models are illustrated in Figure 2.1.

Model A (adhesion) presupposes a tendency for the bubbles to adhere to the particles upon collision, which introduces the important concepts of attachment forces and hydrophobicity, which will be shortly discussed. Model A would probably dominate when very small particles (say 20 μm) are successfully floated by bubbles that are comparable in size (say 40 μm).

Model B (enmeshment) does not presuppose a great tendency for the particles to stick to the bubbles, although it has been shown that if attachment is deliberately suppressed under meticulous laboratory conditions, flotation is not possible (Gochin & Solari, 1983). Under natural conditions, there is adequate attachment between particles and bubbles to allow Model B to operate. The bubbles simply get stuck to particles which are much larger than the bubbles (say 40 μm bubbles and 200 μm particles). If the particles have a rough, jagged surface (mineral flotation) or a loose, dendritic structure (flocculent particles) bubble enmeshment is obviously enhanced. With Model B, those bubbles that are not firmly trapped will break loose. Compared to Model A, the collision efficiency will thus be lower.

Model C (incorporation) can operate in one of two ways. It could firstly operate if there is a degree of floc breakup and reflocculation in the reaction zone. During reflocculation, some bubbles could be trapped amongst recombining floc fragments. It could secondly operate if bubbles are introduced before the flocculation process is complete. In this way, some bubbles

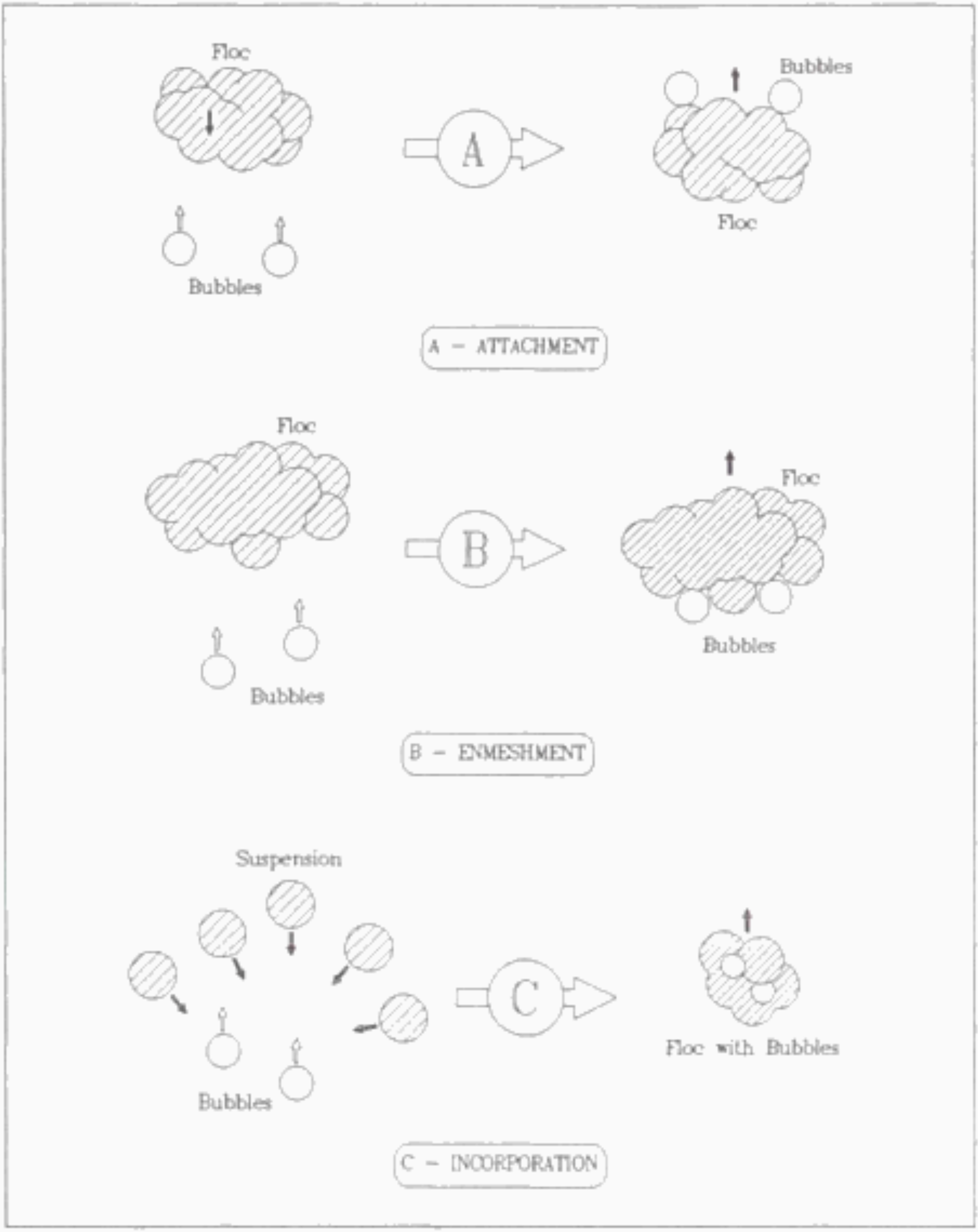


Figure 2.1 : Formation models for air/floc agglomerates (from Schade, 1982)

can be caught up during the flocculation process to end up inside the flocs. Although Model A and Model B are considered to be the dominant ones in practical DAF applications, Model C was demonstrated to be feasible. One commercial system uses a pipe flocculator of which the first 10 m to 20 m is used for coagulation until pinhead floc is formed. Supersaturated recycle and polymer are then added simultaneously and the following 3 m of pipe flocculator is then used for intimate mixing and to form particle/bubble agglomerates by incorporation (Schade, 1982). Another example of Model C at work is with full-stream *microflotation*, where the entire flow is diverted down a deep shaft against a stream of counter-current bubbles, which is introduced at the bottom of the shaft. As the stream rises again in a second bubble-free shaft towards the flotation tank, the hydrostatic pressure is slowly lowered and bubbles are precipitated while flocs are still forming (Hemming & Cottrell, 1976). Model C should work best under conditions of high particle concentration and rapid flocculation.

Model D (growth within flocs) has been proposed as a method to make more efficient use of air. It is based on the principle that supersaturated air will preferentially precipitate on existing nuclei when pressure is gradually released. When supersaturated water was introduced above a layer of settled iron hydroxide flocs, small bubbles started to grow after a few minutes within the flocs (Kitchener & Gochin, 1981). Model D could conceivably work if coagulant was introduced, and the full stream was then pressurized in a saturator, although a high degree of floc breakup can be expected inside a saturator. This model could also be operative to some extent during the process of deep shaft microflotation, described in the previous paragraph (Hemming & Cottrell, 1976).

Bubble size and coalescence

It is intuitively sensed that the size of the bubbles introduced into the reaction zone must be of primary importance. Small bubbles are essential for DAF for the following reasons (Rovel, 1976; Komline, 1976: Conference on, 1977 : 157-165):

- There should be at least one bubble for every particle that needs to be floated - there should therefore be at least as many bubbles as particles. Smaller bubbles imply that more bubbles can be precipitated from the same quantity of air. One bubble of 2 mm occupies the same volume as 64 000 bubbles of 50 μm.
- Smaller bubbles rise slower and will offer more time and bettor opportunity for attachment between particles and bubbles.
- Larger bubbles will rise faster, will be more likely to be detached from the particle/bubble agglomerates and cause more hydraulic disturbance along their rising path.
- One source (Longhurst & Graham, 1987) further recommends that the bubbles must be smaller than the particles to be floated.

Considerable emphasis had therefore been placed on the measurement of bubble sizes actually encountered in DAF. The majority of studies employed photographic techniques for measuring the bubble diameters. In general, the methods consist of photomicrographs being taken of the rising bubbles, the photographic images being enlarged and the bubble diameters being measured and counted under some sort of magnification (Jones & Hall, 1981; Shannon & Buisson, 1980; Kitchener & Gochin, 1981; Cassell *et al.*, 1974). The bubble sizes generally follow a normal distribution, with fairly uniform sizes. The average bubble sizes range from about 40 μm to 110 μm (Jones & Hall, 1981; Shannon & Buisson, 1980; Lovett & Travers, 1986). It was earlier suggested that the size range should fall within 20 μm to 80 μm for DAF to be effective, but later studies showed that a bubble diameter range from 70 μm to 170 μm also gave satisfactory results (Jones & Hall, 1981). Similarly, another study showed that average bubble diameters ranging from 60 μm to 110 μm had no measurable effect on the same experimental system (Lovett & Travers, 1986).

Once the bubble diameter is known, its rise velocity can be calculated by assuming that the upward buoyant force is equal to the hydrodynamic drag (Stoke's Law). Rise rates were measured under a variety of temperatures (and resultant viscosities) and Stoke's Law was verified up to 80°C. Experimentally measured rise rates were almost three times as fast at 80°C than at 20°C - almost in complete accordance with the decrease in viscosity (Shannon & Buisson, 1980). This is, however, only true for bubble diameters up to 150 μm. Above this diameter, the bubbles start to elongate and depart from the rise rate predicted by Stoke's Law (Ramirez, 1979; De Groot, 1987). At about 130 μm there is, in addition, a transition from a laminar to a turbulent flow field around the bubble, which invalidates the use of Stokes' Law above this diameter (Vrablik, 1959).

The bubble diameters, once the air is precipitated, unfortunately do not remain constant. This is due to the *coalescence* phenomenon, where bubbles will collide and merge to form fewer, larger bubbles. If there is a considerable tendency for the bubbles to coalesce, the bubbles will grow as they rise through the water column, leading to a decrease in flotation efficiency (Kitchener & Gochin, 1981). The dominating factor in maintaining stable bubble suspensions appears to be the surface tension (Ramirez, 1979). For stable air suspensions, the surface tension must be low. Three factors favour bubble coalescence (Ramirez, 1979):

- non-uniform bubble diameters, where differential rise rates will increase bubble/bubble collision opportunities.
- violent agitation, where bubbles forcefully collide.
- the presence of a large surface area, which supplies many deposition points where microbubbles will coalesce.

In addition, it was experimentally found that a combination of high temperature and saturation pressure will also increase coalescence. At 80°C, bubble coalescence increased rapidly above a saturation pressure of 350 kPa; at 420 kPa, DAF failed completely (Shannon & Buisson, 1980).

The addition or presence of certain chemical compounds appears to have an influence on bubble size. The addition of carboxylic acids and alcohols, for example, decreased the size of bubbles produced by a diffuser significantly (Zieminski *et al.*, 1967).

Very few direct measurements of bubble coalescence were made. In a Soviet analysis of bubble coalescence in electroflotation (Rulev, 1985), it was shown that coalescence drastically reduced the efficiency of flotation. Efficient flotation was in fact restricted to a small zone in the immediate vicinity of the bubble-producing electrodes. The average bubble size increased rapidly as the bubbles rose. The average bubble size was 45 μm at a level of 20 mm above the electrode, but increased to 60 μm at a level 80 mm above the electrode.

The bubble size, even if coalescence were completely absent, will increase slightly due to the hydrostatic pressure decrease as the bubble rises through the water, but this effect is very small and of little importance.

In the sections that follow, air concentration is simply given as a *mass* concentration or the dimensionless *air/solids* mass ratio a_s. A better intuitive grasp of the reaction zone processes is obtained if one also considers the *volume* and *number* concentration of the bubbles. An air mass concentration of 8 mg.ℓ^{-1} based on feed flow, typical for clarification, is equivalent to about 0,7% by volume. An average bubble diameter of 50 μm now translates into about 100 000 bubbles.cm^{-3}, with the average inter-bubble distance around 200 μm. [For thickening applications with high particle concentrations, proportionately much more air is required].

Particle sizes

The typical particle size is much more variable than the bubbles, and it could be substantially different for every application. When uncoagulated algal waters are floated, the algal sizes typically vary from a minimum of 3 μm for unicellular species up to algal filaments which could be several hundred μm long. When the water is flocculated prior to flotation, the particle or floc size is predominantly determined by the coagulant concentration and the flocculation conditions - these will be discussed further on.

It has been demonstrated very clearly (King, 1982) in mineral separation applications that flotation only works between two critical particle sizes depending upon particle density. If the particle size is above the critical *maximum*, the particles become too heavy to be floated by the air bubbles. If the particle size is below the critical *minimum*, the particles are not sweeped up with the bubbles. The statement was made, without supporting data, that particles below 20 μm will not readily adhere to bubbles in water treatment applications (Conference on, 1977 : 157-165). Theoretically, it was shown (Edzwald *et al.*, 1990) that the ideal particle diameter for water treatment DAF applications should be between 10 and 50 μm, similar to that required for water filtration although in practice DAF can be successful in floating much larger particles (for example flocculated algae).

Transport mechanisms

It stands to reason that bubbles and particles first have to collide before an aggregate can be formed, regardless of which formation model dominates. It is clear that a solid particle must have sufficient momentum to resist the tendency to follow the streamlines around the bubble. Very fine particles tend to skirt around the bubble without making direct contact. It is possible to construct plausible mathematical models for this collision process, which are useful to make general conclusions, despite the uncertainties in the hydrodynamic conditions that surround a bubble.

Such trajectory analyses for a rising bubble had been made to establish the critical parameters for flotation (Edzwald *et al.*, 1990; Reay & Ratcliff, 1973). A number of important general conclusions were drawn.

- The two dominant transport mechanisms are Brownian diffusion and interception. Gravity (settling) is of secondary importance.
- For normal flotation conditions, the critical particle diameter is around 1 μm. Above this size, interception dominates, below this size, Brownian diffusion dominates. At the critical particle diameter, collision efficiency will be lowest. This finding is similar to the results of similar mathematical models constructed for deep bed filtration.

- The most influential parameter is the bubble size. The collision efficiency improves dramatically as the bubble size decreases. For a tenfold increase in bubble diameter, the collision efficiency decreases hundredfold.
- The effects of particle density and temperature on the collision efficiency are negligible.

Attachment and hydrophobicity

Once a particle and bubble collide, adhesion will only follow if the particle penetrates the skin of the bubble. The rupture of the bubble surface is not instantaneous, because the water between bubble and particle must first be displaced by the process of hydrodynamic thinning. The particle continues to be swept around the bubble while hydrodynamic thinning takes place - it may slip over the surface into the water again if the thinning process is not fast enough. It was mathematically shown that the probability of collision and adhesion is proportional to the particle diameter (King, 1982), while the thinning process is slower at lower temperature (Edzwald *et al.*, 1990).

For a bubble to "stick" to a particle, the particle surface must be hydrophobic to allow a finite contact angle between particle and bubble (Ramirez, 1979). A bubble will not adhere to a truly hydrophilic particle. This was demonstrated in a meticulous laboratory experiment where pure kaolin clay could not be floated in organic-free water (Kitchener & Gochin, 1981). Water treatment sludges are distinctly hydrophilic because of a high percentage of iron or aluminium hydroxides (the exception being carbonate sludges which are hydrophobic) (Richard & Dauthuille, 1984). Flotation of water treatment sludges nevertheless takes place. It was explained by the fact that there are organic substances present in any natural aqueous environment and that these compounds will inevitably coat the particles to render them slightly hydrophobic. Naturally occurring suspensions have therefore automatically enough hydrophobicity to allow successful particle/bubble adhesion. When surface-active agents were deliberately added, flotation could be further improved (Kitchener & Gochin, 1981). Attachment is dominantly achieved through Van der Waals forces.

Not all agree that hydrophobicity plays a minor role in conventional water treatment applications. In a field survey of eight flotation plants in the USA (Roberts *et al.*, 1978), a wide variation in performance was found which could not be related to conventional parameters such as hydraulic and solids loading, and air concentration. The variation in performance was finally ascribed to the differing surface properties and hydrophobicity of the particles at the different plants. If this unsupported speculation has any truth, hydrophobicity may play a larger part in flotation than previously thought.

A further factor which will influence attachment between bubbles and particles, is that of electric charge. Air bubbles are reported to be slightly negatively charged, similar to most natural occurring particles in water, but it was not anticipated that it would play a large role (Vrablik, 1959).

Detachment mechanisms

Simultaneous to the attachment and enmeshment of bubbles to the particles, there are other forces at work in the reaction zone to *detach* the bubbles from the particles. A combination of turbulence and gravitational forces will detach a fraction of the particles that were attached upon impact as the bubbles moved through the water. The inertia of a heavy particle would cause it to lag behind an accelerating bubble with a consequent straining of the bubble skin. The detachment probability gets higher as the particle size increases (King, 1982).

Reaction zone configuration

A number of general prerequisites for the ideal reactor zone can be based on fundamental considerations such as those described above:

- Particles and bubbles should be in their respective size ranges that will allow successful flotation.
- Enough turbulence to provide adequate collision opportunities between bubbles and particles. All particles should be kept in suspension to sweep up smaller particles (Kitchener & Gochin, 1981).
- Regions of high shear, such as edges of blades, should be avoided because of damage to flocs (Kitchener & Gochin, 1981; Vrablik, 1959) or detachment of bubbles from particle/bubble agglomerates - specially important for clarification applications (Vosloo *et al.*, 1985).
- The hydraulics through the reaction zone should approximate plug flow (Kitchener & Gochin, 1981).
- Sufficient time to establish particle/bubble agglomerates which will float successfully (Vrablik, 1959).
- The reaction zone should be confined to a separate compartment ahead of the flotation zone (Kitchener & Gochin, 1981).
- Air must be released very close to the point where the recycled water is mixed with the flocculated water to minimize the loss of air bubbles resulting from coalescence (Zabel, 1985).

It is not a simple matter to design a reaction zone to comply with all the above requirements. A number of different arrangements have been recorded in the literature, of which a few are illustrated in Figure 2.2.

One of the earliest practical recommendations is to introduce the recycle with a pipe running concentrically inside the feed pipe. This recommendation states that the recycle pipe must be stopped short to allow complete mixing before the reaction zone. The velocity in the recycle pipe was two to three times higher than the velocity of the feed to allow for rapid dispersion of the recycle into the feed stream. The mixture was introduced tangentially into the reaction zone (Vrablik, 1959). A commercial system (Anon, 1972) makes use of the same principle where feed and recycle are injected tangentially in the bottom of a truncated cone.

The geometry of the reaction zone is of undisputed importance. Six different reaction zone configurations were experimentally tested (Ettelt, 1964) and it was demonstrated that seemingly small modifications had a significant effect on the properties of the float layer.

An important factor often overlooked during the design of a reaction zone is that of scale. A prominent designer of sludge thickening plants in the USA, for example, typically uses a single inlet for feed and recycle at the head of the flotation tank, which works entirely satisfactorily for smaller flotation tanks. For higher flows, more violent mixing is required to ensure intimate contact between particles and bubbles. The single-inlet arrangement cannot be scaled up indefinitely - for larger tanks, it becomes necessary to use two or more inlets to keep the turbulence within bounds (Conference on, 1977 : 166-178). In another example, a full-scale

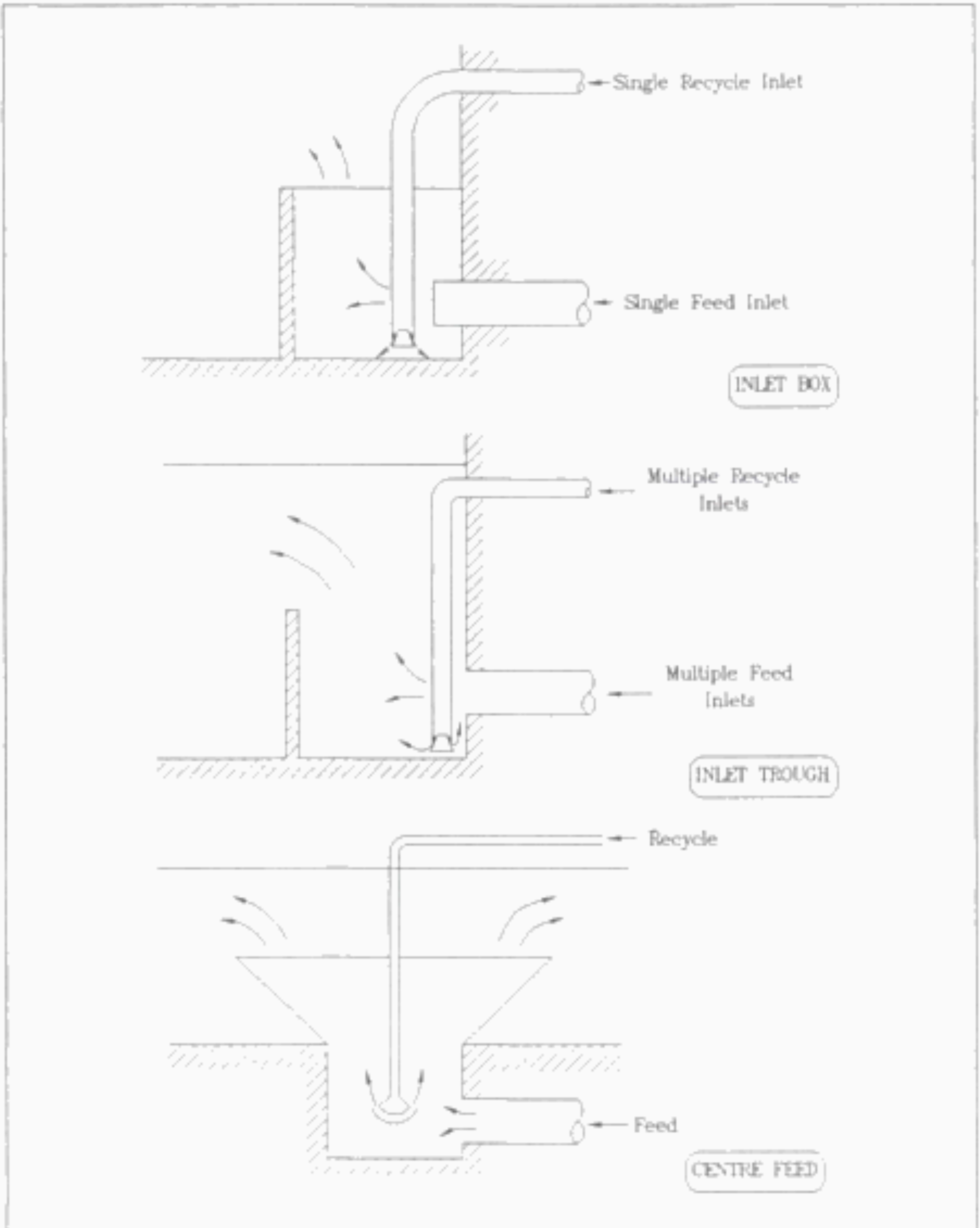


Figure 2.2 Typical reaction zone configurations

sludge thickening plant did not perform nearly as good ($SS_{out} > 100$ mg.ℓ^{-1}) as a much smaller pilot unit ($SS_{out} < 50$ mg.ℓ^{-1}) from which the full-scale design parameters were derived. This discrepancy was retrospectively ascribed to "inadequate design of the inlet arrangement in the large unit where a high degree of turbulence was evident with an excessive number of large unattached bubbles breaking through the accumulated float layer" (Bratby, 1978).

One principal difficulty lies with quantification of the turbulence within the reaction zone. One method is to calculate a rise rate within the reaction zone itself. For the flotation of algal waters, for example, a rise rate of 60 m.h^{-1} had been suggested (De Wet, 1980). Similarly, a crossflow rate from the reaction zone to the flotation zone had also been suggested. For algal waters, this was recommended to be 80 m.h^{-1} (De Wet, 1980).

It can be concluded that there are very few definite, quantitative guidelines for the design of an optimal reaction zone. A related problem lies with the exact demarcation of where the reaction zone stops and where the flotation zone begins. In most cases, it is possible to distinguish a general area for reaction and another for flotation, but in some cases, where the reaction zone is continuous with the flotation zone, there is no such possibility.

2.3 Chemical pretreatment

Advantages of chemical addition

Flotation, as practiced in the mining industry for the beneficiation of certain ores and minerals, is always accompanied by the addition of a number of treatment chemicals to manipulate the process for selective flotation and the highest possible recovery. The water and waste-water industry occasionally borrows some of the following terms used in mineral flotation (Daniels, 1977):

- Collectors - for contact improvement and better adhesion.
- Frothers - for the stabilization of bubble suspensions.
- Modifiers - for pH change and electrical charge alteration.
- Coagulants - for floc destabilization and precipitation.
- Flocculants - for agglomeration of smaller flocs.

The first two chemical types (collectors and frothers) are especially important in mineral flotation because air bubbles and particles are large (typically an order of magnitude larger than in water and waste-water treatment). In fact, where these chemicals have been tested for water and waste-water treatment, they were found to act as deflocculants which actually impeded the DAF process (Conference on, 1977 : 166-178).

In water and waste-water treatment, ferric chloride, aluminium sulphate and synthetic polymers are typically used as coagulants, with polymers as flocculants. It was shown (Folkard *et al.*, 1986) that the naturally occurring polymers in *Moringa* seed suspensions also could be successfully used for flotation. The pH is typically adjusted by lime or acid (as modifiers) to get to the range where the coagulants operate best, and to render the water stable in terms of its corrosivity.

Where natural particles are present in raw drinking water supplies, it is usually necessary to use coagulants to sweep up the very small particles into pin flocs that could be further flocculated or directly floated. When treating raw waste water or thickening activated sludge, the primary particles are much larger and naturally flocculent. In this case, only flocculants are necessary to form larger flocs, or the particles may be floated directly. The requirements for chemical dosage, coagulation and flocculation are thus substantially different for thickening and they are discussed separately in the following sections.

Chemical dosage determination

For conventional *clarification* processes, chemical dosage is normally determined with a standard jar test, in which the processes of rapid mixing, flocculation and settling are simulated. In principle it may be disputed whether the jar test would also be a good predictor for the three-phase process of flotation. It was however shown (Luthy *et al.*, 1978; Zabel & Hyde, 1976; Edzwald & Wingler, 1990; Packham & Richards, 1975b) that the jar test does indeed predict the same optimum chemical dosage as that empirically determined at an actual pilot-scale DAF plant, or that the optimum dosage for DAF is very slightly higher than the jar test prediction (Rees *et al.*, 1980b). Other commercial bench-scale flotation cells are available for the same purpose which would also indicate the optimum chemical dosage. A detailed systematic procedure of conducting a bench-scale flotation test is given in the literature (Zabel & Melbourne, 1980). Another method was described (Blommaert *et al.*, 1990) whereby a constant sampling flow rate from the bottom of a laboratory flocculation/flotation reactor is maintained, and the turbidity is tracked with time. From this, the rise rate of the particle/bubble agglomerates can be calculated. The optimum chemical dosage then corresponds to the maximum rise rate.

The dosage determination for *thickening* applications is best determined by pilot- or full-scale plant experimentation. When the dosage is applied solely for the purpose of subnatant clarification, the jar test could be used, but when the focus is also on the sludge properties, there is no option but to use a larger unit.

Compared to sedimentation, flotation seems to be more vulnerable to overdosing of coagulant. It was found (Zabel & Hyde, 1976; Longhurst & Graham, 1987) that higher than optimum aluminium sulphate dosing leads to a weaker floc which was broken up by air injection. In other studies with a ferric coagulant (Childs & Rees, 1976; Stock, 1976), it was also found that overdosing led to poorer quality, especially at higher turbidity. This is in contrast to a finding that the rise rate of particle/bubble agglomerates in raw surface water increases with higher dosage of ferric chloride (Blommaert *et al.*, 1990).

The claim had been made that flotation requires less coagulant than sedimentation (Rosen & Morse, 1976) on the basis that "...the aim is not to produce heavy flocs, only stable flocs are required...", but no supporting experimental evidence was submitted.

Pretreatment for clarification

When dilute suspensions are *clarified*, coagulation and a degree of flocculation are indispensible to gather the very small particles together as floatable flocs; particles which would not have been removed on their own, or would have required a much higher air concentration (Edzwald & Wingler, 1990). Without a coagulant, for example, a minimum a_r of 0.2 was required for 80% flotation of an artificial iron suspension; with the addition of polymer the same results could be achieved at an a_r of 0.05 (Shannon & Buisson, 1980).

The rapid dispersal of chemicals into the water was shown to be important (Sandbank, 1983), and an in-line mixer was eminently suitable for clarification of a range of raw waters (Sandbank, 1983; Rees *et al.*, 1980b), without the need for a separate coagulation vessel. An in-line mixer was found to be more effective than a standard flash mixer (Zabel, 1978; Rees *et al.*, 1979).

There are conflicting suggestions on the need and extent of flocculation prior to flotation. The following findings are reported on a number of flocculation aspects:

- Flocculation time. For algal-laden oxidation pond water, a flocculation chamber was found to be unnecessary (Bratby & Marais, 1973); with raw waste water (Bratby, 1982) a very short flocculation period of 30 seconds was more than adequate. This finding was supported by recent work on algal-spiked surface water (Edzwald & Wingler, 1990; Edzwald *et al.*, 1990) which indicated that very little flocculation is required for successful flotation. At 21°C, there was practically no difference between 0 and 5 minutes flocculation (99,2% versus 99,7% algal removal). Only at a low temperature of 4°C, the advantage of 5 minutes flocculation over no flocculation became apparent (99,2% versus 74,8% algal removal). There was no advantage, at any temperature, in flocculation periods longer than 5 minutes. Contrary to these findings, flocculation was definitely required for a number of surface waters, where extensive testing during a number of pilot programs showed that a separate flocculation step is indispensable (Rees *et al.*, 1980b; Hyde, 1975; Packham, Richards, 1975b). Full flocculation worked better than partial flocculation, which worked better than no flocculation, which worked better than adding the coagulant after the air. A flocculation time of 12 minutes was typically required for algal water, but in the case of a soft, coloured water, a significant improvement was found at a flocculation time of 16 minutes. A study specifically directed at three different types of algal water (De Wet, 1980) recommended a minimum flocculation period of 6 minutes - longer periods did no harm. In Sweden, flocculation periods of 20 minutes or longer are commonly provided (Rosen & Morse, 1976). A flocculation time of 30 minutes is needed for chemical coagulation of raw waste water (Maddock, 1976).
- Flocculation intensity. A study on a wide variety of waters (Rees *et al.*, 1980b; Hyde, 1975; Packham & Richards, 1975b) found that a velocity gradient (G) of 75 s^{-1} was typically required. A minimum value of 75 s^{-1} was also found to be required for the flotation of algal water (De Wet, 1980). This corresponds closely to an optimum range of 60 to 80 s^{-1} (ferric chloride) and 80 to 100 s^{-1} (polyaluminium chloride) for raw surface water (Blommaert *et al*, 1990). For maturation pond effluent, the best results were obtained at velocity gradients between 130 s^{-1} and 180 s^{-1} (De Wet, Van Vuuren, 1980). For clarification of paper mill effluent, in contrast, velocity gradients of 40 s^{-1} to 60 s^{-1} were sufficient (De Wet & Van Vuuren, 1982). An optimal G = 60-80 s^{-1} was found for chemically treated raw waste water (Anon, 1986).
- Gt-product. Very few studies have suggestions on the traditional dimensionless product of flocculation time and velocity gradient. For algal water, a minimum value of 1.10^4 was suggested (De Wet, 1980) and for a maturation pond effluent, a minimum value of 3.10^4 (De Wet & Van Vuuren, 1980). For a paper mill effluent, the recommended product was 5.10^3 (De Wet & Van Vuuren, 1982).
- Tapered flocculation. No benefit was found from tapered flocculation after aluminium sulphate addition, except for a marginal improvement in the case of a turbid river water (Bratby, 1982). In another study with ferric coagulant, no benefit was found from tapered flocculation (Childs & Rees, 1976). No benefit from tapering for chemically precipitated raw waste water could be demonstrated (Anon, 1986). Others have found

(Longhurst & Graham, 1987) that severely tapered flocculation was poor, but that moderately tapered flocculation worked better than untapered flocculation. A prominent designer of flotation plants in Sweden, however, stated unequivocally that tapered flocculation was the most successful method, where a flocculation time of 20 minutes or more was commonly adopted (Conference on, 1977 : 1957-165).

- Type of flocculation. There is no agreement on the best type of flocculation device. Baffled hydraulic channels and pipe flocculators are favoured because of their simplicity and because they closely approximate plug flow. It was stated (Conference on, 1977 : 407-416), that three minutes of hydraulic flocculation is equal to 10 minutes of mechanical flocculation. On the other hand, hydraulic channels are not favoured because they cannot offer the same degree of control (Longhurst & Graham, 1987). With mechanical flocculation, it is also not certain how many compartments should be provided. For surface water, quality is not improved by multiple compartments, while for groundwater two compartments worked better than one (Stock, 1976). Compartmentalization did improve flotation of chemical precipitated raw waste water (Anon, 1986).

Some years ago, it was stated that flocs should ideally be in the size range of 500 to 1000 μm for successful flotation (Longhurst & Graham, 1987). It was recently, however, suggested (Edzwald *et al.*, 1990) that flocculation should only be continued until the average particle diameter is in the region of 50 μm. Earlier work on the flocculation of kaolin suspensions with an iron coagulant (Kitchener & Gochin, 1981) gave some indication of the floc size that can typically be formed during flocculation. For short flocculation periods at low mixing intensity, weak and highly branched flocs are formed with particle diameters ranging from 180 to 1 000 μm. If the flocculation period is extended and the mixing intensity increased, the flocs become nodular in shape and much more compact; the diameter ranging from 300 to 800 μm. Extrapolating from these results, it will appear that only a limited amount of flocculation would be required to get the floc into the required size range.

A good example of the benefits of aluminium sulphate dosing (Bratby, 1982) is reproduced as Figure 2.3.

Pretreatment for thickening

When *thickening* concentrated suspensions of flocculent solids, such as waste activated sludge, chemical addition may not be indispensible for a reasonable degree of thickening, but the subnatant clarity and float layer stability are greatly improved if polymers are added. Raw sewage demanded no polymer, but is vital for activated sludge mixed liquor to obtain good effluent quality (Gehr & Henry, 1980). According to practical experience with a wide range of operating DAF plants in the USA (Komline, 1978), polymer addition typically

- increased the float solids concentration from 3% to 4%;
- allowed solids loadings of 11 to 15 $kg.m^{-2}.h^{-1}$; and
- improved solids capture to 95% or higher.

In one of the many examples in the literature, suspended solids out (SS_{out}) improved from an average of 37 $mg.\ell^{-1}$ to an average of 6,7 $mg.\ell^{-1}$ upon polymer addition, while the float solids concentration improved by a factor 1,35 (Bratby, 1978). Figure 2.4 shows typical results from another pilot study where activated sludge was thickened with and without polymer (Bratby, 1978).

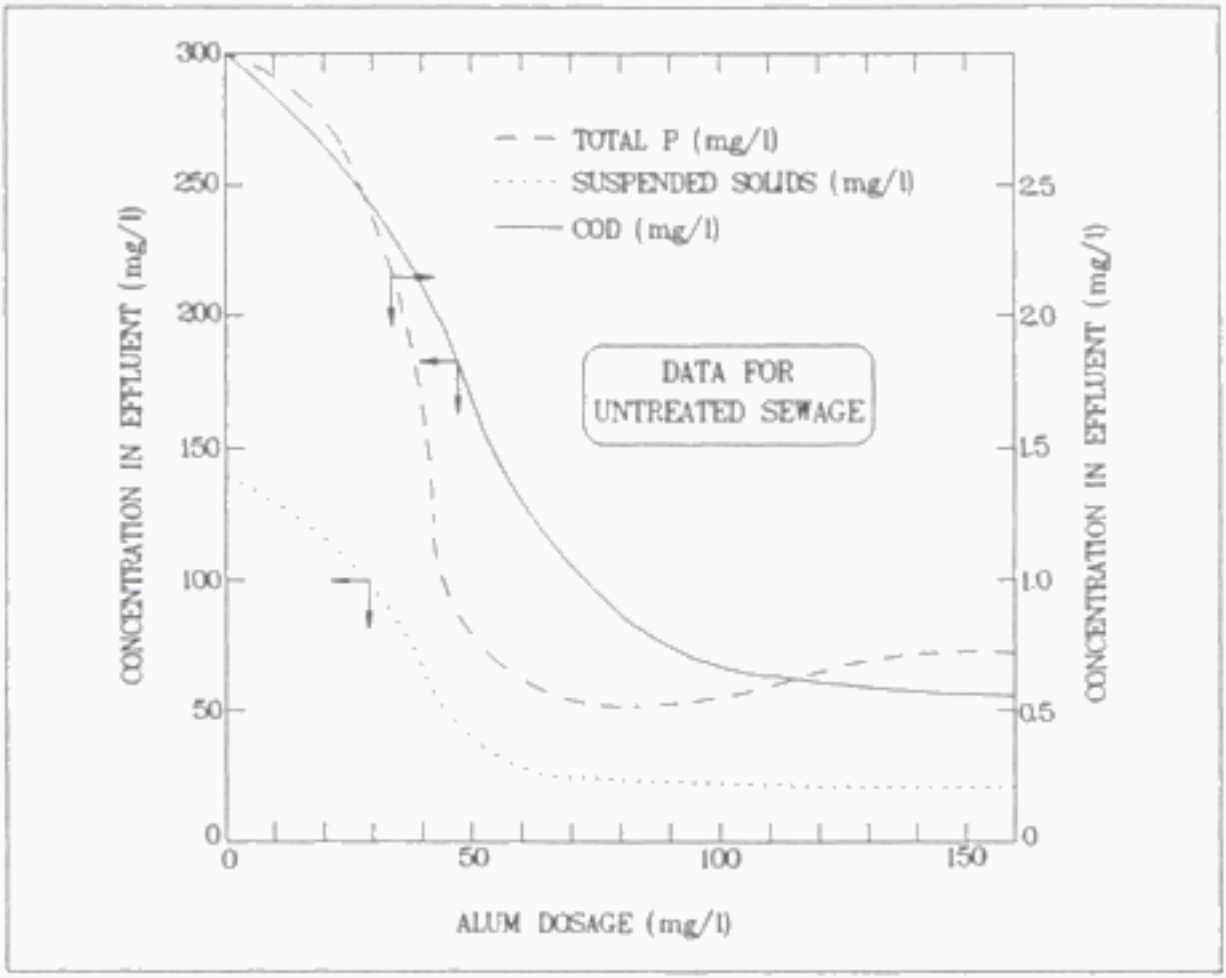


Figure 2.3 : Improvement in flotation performance by addition of aluminium sulphate (from Bratby, 1982)

The beneficial effects of polymer were explained by its ability to

- flocculate the particles smaller than 20 μm, which will not adhere to bubbles,
- reduce the total number of particles, to increase the number of bubbles available per particle, and
- improve bubble attachment by changing the nature of the particle surface

The benefits of polymer addition must be balanced against the additional cost and complexity of operation. One of the prominent European suppliers, for example, generally does include polymer dosing for their DAF thickeners. There are also some instances in the literature where the benefits of polymer are not supported. In one case (Roberts *et al.*, 1978) the benefits of polymer were haphazard and not reproducible - in others (Maddock & Tomlinson, 1980, Langenegger & Viviers, 1978 (discussion)) no improvement was observed during the clarification of activated sludge. An explanation was offered to reconcile these seemingly contradictory findings, namely that in some instances the activated sludge has excellent

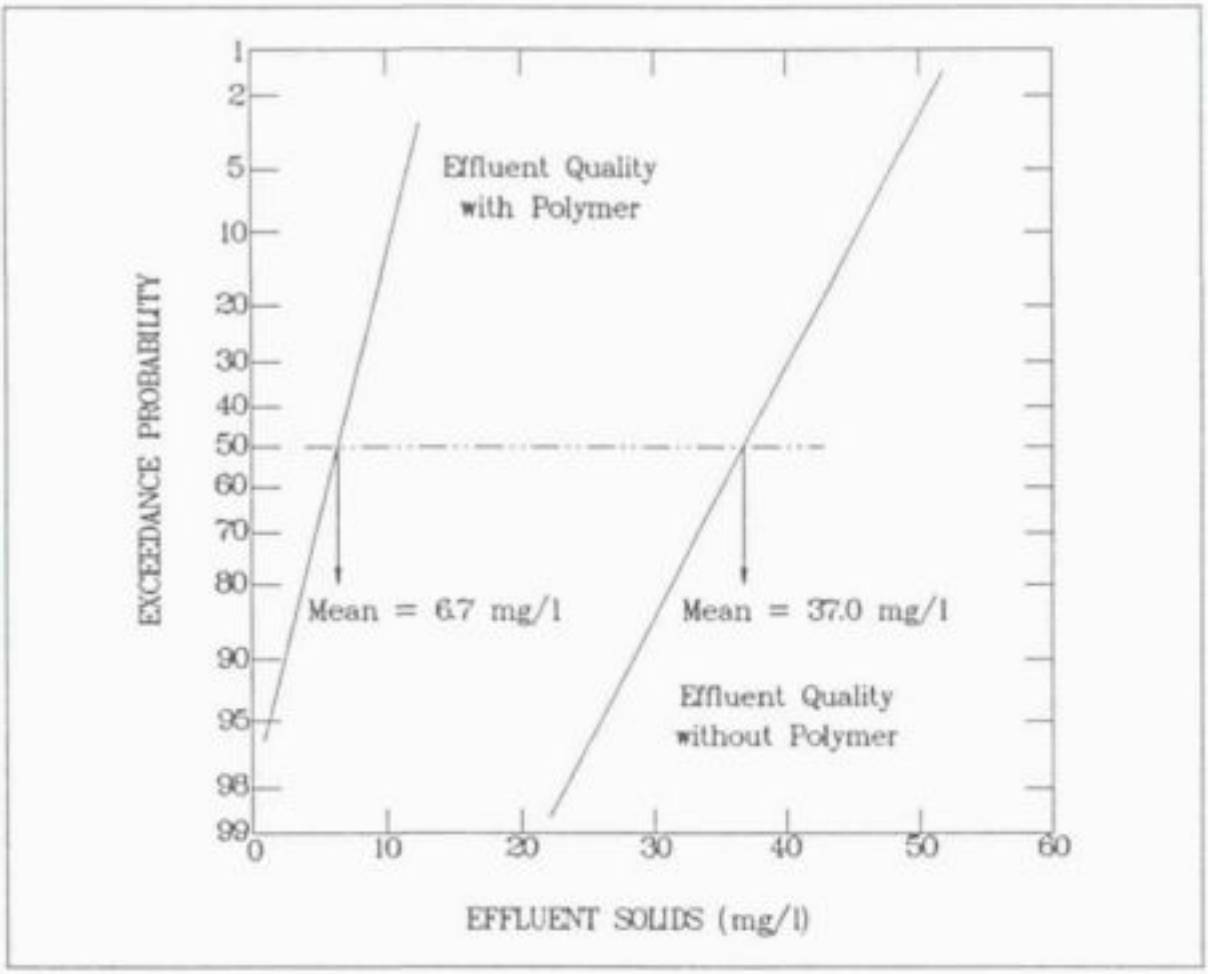


Figure 2.4 : Improvement in effluent quality by polymer dosage (from Bratby, 1978)

bioflocculating properties in which case additional polymer would make little or no difference (Henry & Gehr, 1981; Gehr & Henry, 1980).

No flocculation of the sludge prior to the reaction zone appears to be necessary. One prominent supplier in the USA adds the polymer to the recycle from the saturator before it enters the reaction zone (Komline, 1978). The flocculant thus reaches the particles at the same time as the precipitating air bubbles, which might cause Model C to play a significant role (see paragraph 2.2).

2.4 Bubble production system

The bubble production system comprises of a few elements. Firstly, a *pumping system* is required to pump a controlled amount of water through the recycle system. Secondly, the water needs to pass through the *saturation system*, where enough contact with air under pressure is allowed for the recycle to become nearly saturated with air under the elevated pressure. Thirdly, the water must be *conveyed* to the flotation tank(s) and *distributed* amongst flotation tanks where necessary. Fourthly, the pressurized recycle must be released into the tank through a *injection device* which lowers the pressure and which causes the now supersaturated air to precipitate as small, evenly sized bubbles.

Saturation systems

Numerous cost comparisons between DAF and the more conventional settling have been published. In all these comparisons, it is clearly shown that DAF has lower capital cost, but higher operating costs. About half of the high operating cost is caused by the energy required to dissolve air into water at high pressure (Rees *et al.*, 1980a). The optimal design of air saturation systems has thus been the subject of a great number of published studies. Practical designs apparently leave much to be desired in terms of efficiency. A survey of six DAF systems showed saturator efficiencies of 77%, 62%, 48%, 45%, 31% and 30% (Roberts *et al.*, 1978) while it appears from other studies that an efficiency of at least 50% should be attainable regardless of the saturation system used.

A number of systems have been devised for getting air dissolved into pressurized water. The two most general systems used are packed and unpacked saturators, which are discussed further on in greater detail. Four other systems have also been reported in the literature which will be briefly mentioned here:

- In earlier references, much emphasis was placed on bleeding air into the suction side of the recycle pump. It takes very little energy to get the air into the low pressure side of the pump. On the high pressure side, the air is then dissolved in the water. The air concentration introduced in this manner is limited to about 4% by volume (approximately 50 $mg.\ell^{-1}$) before cavitation problems are encountered (Roberts *et al.*, 1978). At 690 kPa, for example, only 50% of the air required for saturation can be injected. This system was later additionally shown to have very low transfer efficiency (Bratby & Marais, 1975b) and has subsequently mostly been replaced with more efficient systems.
- Air sparging is accomplished by bubbling air through a pressurized vessel. This method is hampered by the fact that that it takes a long time to reach saturation concentration. Even at a high pressure of 760 kPa and at the highest air flow rate investigated, it took at least 40 minutes to reach equilibrium (Bratby & Marais, 1975b).
- Deep shaft microflotation is a method occasionally used for domestic waste water and domestic effluents in Scandinavia. The entire flow is diverted down a descending shaft of about 10 m deep, and then up a rising shaft into the flotation tank at ground level. Air is introduced at the bottom of the descending shaft, which dissolves as it rises countercurrently against the descending flow. As the flow rises in the ascending shaft, the hydrostatic pressure is gradually relieved and excess air slowly precipitates. The efficiency of this system is about 60 to 70% (Hemming & Cottrell, 1976).
- A novel method, called the LEPGT (low energy pressurized gas transfer) system was proposed (Speece *et al.*, 1975), as a means of decreasing the energy demand. Air and

water are drawn into a reactor at low pressure. The reactor is then pressurized, with an internal recirculation pump to create the turbulence inside the reactor to dissolve the air. This pump requires very little energy, because it has the reactor pressure on both the suction and delivery sides. Once the air is dissolved, the reactor is emptied into the reaction zone before the cycle starts again. Two reactors are proposed, with each reactor operating in turn to get reasonably continuous recycle. Results from a small LEPGT prototype showed that the system is comparable with the recycle produced by a conventional saturator, and the energy demand was only about a third of conventional saturation. As far as known, this system has not been implemented on full scale.

Air solubility

Atmospheric air consists, for practical purposes, of only two major gases: nitrogen and oxygen. All the other gases together make up only about 1% of air and can be safely neglected for calculation purposes.

The solubility of air is mainly determined by three factors. It firstly depends on the *temperature* of the water: oxygen and nitrogen become less soluble with increasing temperature. At 0°C, for example, oxygen is 84% more soluble than at 40°C. Air solubility secondly depends on the *air pressure*. If the air pressure in the saturator, for example, is increased to three times that of atmospheric pressure, the air solubility will also increase threefold. The solubility of air thirdly depends on the *absolute air composition*. Oxygen is about two times more soluble than nitrogen; as the oxygen fraction in the air mixture increases, the overall air solubility will also increase. The above arguments are embodied into Henry's Law, which relates the partial pressure of the gas in the atmosphere above the water to the quantity of gas dissolved in the water. Henry's Law was experimentally validated for temperatures up to 80°C (Shannon & Buisson, 1980), pressures up to 500 kPa, for pure water and for waste waters containing suspended solids up to 1 000 $mg.\ell^{-1}$ (Lovett & Travers, 1986).

Water temperature and saturator pressure are readily measured, but the air composition inside a saturator is slightly more complex. The composition of atmospheric air (excluding the small quantity of other gases) is about 79% nitrogen and 21% oxygen. When this mixture is pumped into a saturator, the oxygen fraction dissolves more rapidly in the water than nitrogen due to its higher solubility. The oxygen fraction in the saturator air will therefore decrease while the nitrogen fraction will increase. A mass balance for oxygen and nitrogen across the saturator shows that this trend will continue until the relative concentration of oxygen and nitrogen in the outflowing water is equal to that of the incoming air. When this equilibrium is reached, the composition of saturator air will be about 88% nitrogen and 12% oxygen. Saturator air will therefore be about 9% less soluble in water than atmospheric air, as is indicated on Figure 2.5.

The above point has an important bearing on the expression of the air transfer efficiency of saturators. If saturation concentration is defined in terms of atmospheric air, it will be impossible to attain 100% transfer efficiency, because saturator air is less soluble than atmospheric air. In such a case a totally efficient saturator will only be about 91% "efficient". It is therefore more appropriate to rather define saturation concentration in terms of saturator air, in which case a totally efficient saturator will in fact show up as 100% efficient (Zabel & Melbourne, 1980). When air is directly introduced from the atmosphere, such as air injection into the suction side of a pump, efficiency has to be expressed in terms of atmospheric air.

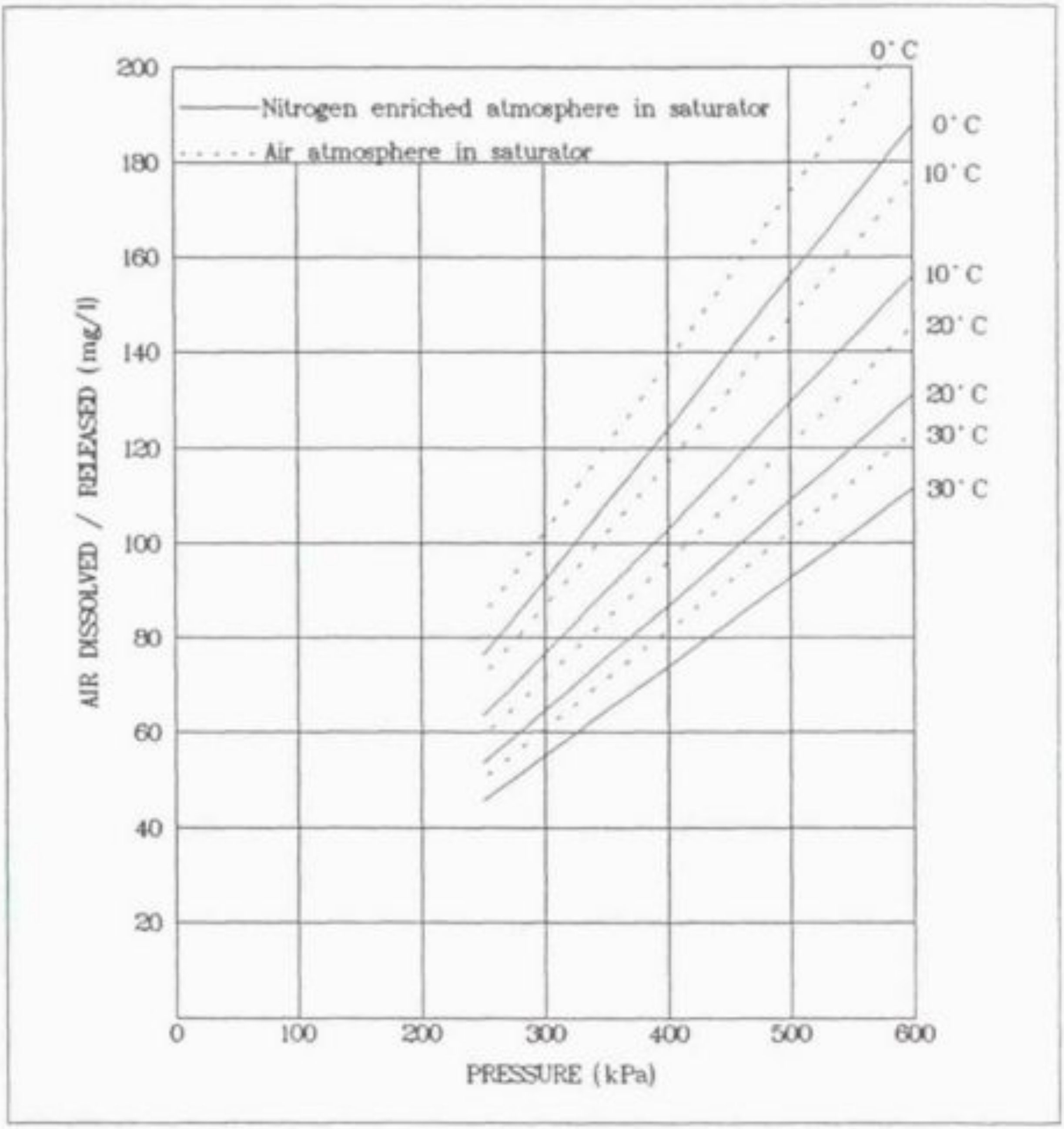


Figure 2.5 : Maximum air dissolution in water as a function of pressure and temperature (from Rykaart, 1991)

A second point that has a bearing on the expression of saturator efficiency, is whether the absolute air concentration is considered, or only the air concentration beyond the atmospheric equilibrium. Once the water from the saturator is released in the reaction zone under atmospheric conditions, not all the dissolved air will be precipitated, but only that in excess of atmospheric solubility. For DAF applications, the interest is only in the fraction of the air that will precipitate as bubbles, which makes it more appropriate to work with the air in excess of atmospheric solubility. For this reason, Figure 2.5 is expressed in terms of the *gauge pressure*, which gives the excess air concentration directly.

A third factor in calculating air solubility is the role of water vapour. At lower temperatures up to say 20°C, the vapour pressure is very low and normally ignored. At higher temperatures, the vapour pressure increases exponentially. If the water vapour is added to air dissolved in the water to arrive at the gas volume available for bubble formation, the available gas increases sharply above 65°C (Shannon & Buisson, 1980) and must be taken into account. At the normal temperature range encountered in water treatment, water vapour is negligible and the available air decreases with rising temperature.

In practice, practically *all* the excess air beyond saturation will be precipitated after the pressure is released. It was experimentally verified that neither the presence of particles (which provides additional nucleation sites) nor the downstream turbulence increased the amount of air precipitated. The precipitation is complete and in accordance with theory (Bratby & Marais, 1975b). However, local experience indicated that saturation at lower pressure, for example full-stream aeration, did not lead to complete precipitation (Offringa *et al.*, 1986). This is possibly due to a lower driving force and/or the type of device used for depressurisation.

Measurement of air concentration

Different air measurement techniques have been described in the literature:

- Batch measurement of air precipitated from solution. A sophisticated method has been described (Packman & Richards, 1975a) which samples the pressurized water before pressure release. The sample is then transferred to a Van Slyke apparatus, the pressure is released and the precipitated air is accurately measured. By first chemically absorbing the carbon dioxide and then the oxygen, the composition of the saturator air can also be determined. More commonly, a sample is taken after pressure release while the sample volume is measured as well as the volume of precipitated air. This method is described in detail by a number of sources (Henry & Gehr, 1981; Van Vuuren *et al.*, 1985; Leininger & Wall, 1974).
- Continuous measurement of air precipitated from solution. If an air precipitation vessel is connected to a saturator and steady state is achieved, the precipitated air will vent off at a fixed rate. An air flow meter connected to the air vent, in conjunction with a flow meter measuring the water flow into the precipitation vessel, will give the air concentration in the saturated solution (Bratby & Marais, 1975b).
- Measurement of oxygen concentration by electrode. Direct measurement of the oxygen concentration will enable calculation of the air concentration if the oxygen concentration in air is assumed. Care should be taken that temperature and pressure compensation is allowed for in the probe used (Henry & Gehr, 1981). This method provides a simple quick method to evaluate the efficiency of dissolved air releasing devices.

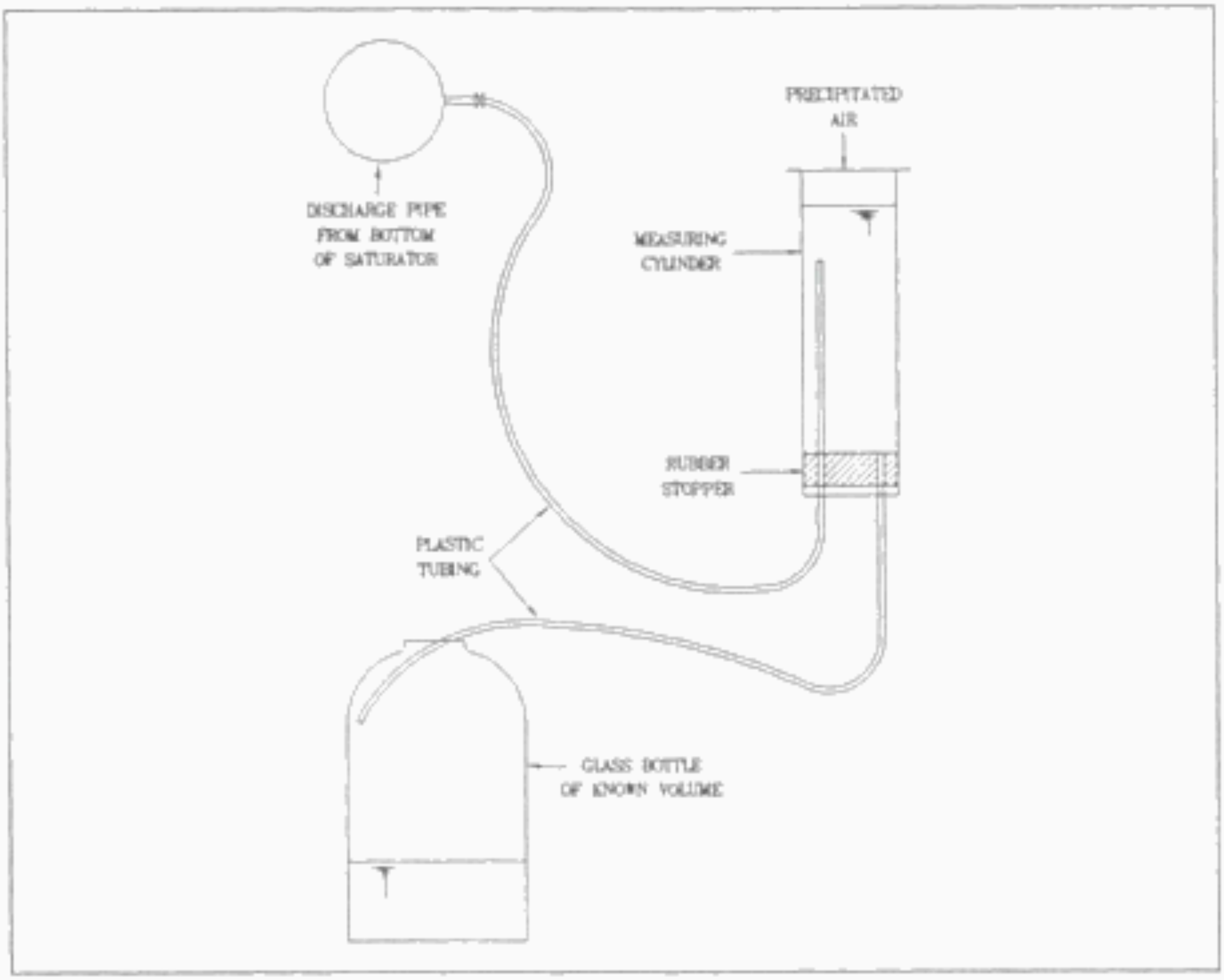


Figure 2.6 : Apparatus for measuring air released by pressurized recycle (from Vosloo *et al.* 1986)

Calculation of theoretical solubility

The theoretical solubility of air can be expressed in the form:

$$a_p = S \cdot P \qquad (2.1)$$

with a_p the excess air concentration in mg.ℓ⁻¹, P the gauge pressure in kPa and S the solubility constant in mg.ℓ⁻¹ kPa⁻¹. The following values for S had been reported at 20°C (adapted from Rykaart, 1991) :

S_{20} = 0,242 for atmospheric air

S_{20} = 0,219 for saturator air

A commonly reported relationship is C = 0,195 P at 25°C (Bratby & Marais, 1975b). This is equivalent to C = 0,234 P at 20°C, which is within 6,4 % of the above value. Where the water to be treated is not fully saturated, aeration requirements for supersaturation will obviously be marginally higher.

For temperatures other than 20°C, the solubility constant in Equation 2.1 can be adapted with the following relation (adapted from Rykaart, 1991):

$$S_T = S_{20} \cdot \left(\frac{293}{273+T}\right) \cdot 10^{-500\left(\frac{1}{293}-\frac{1}{273+T}\right)} \qquad (2.2)$$

with T the water temperature in °C. The solubility constant is also affected by the total dissolved solids (TDS) in the water. This effect only plays a role at TDS levels appreciably higher than those encountered in water and sewage treatment, and is therefore not further considered.

Recycle percentage

There are two pressurization options, namely full-stream or part-stream. For full-stream DAF, the entire stream is pressurized beforehand and depressurized in the flotation tank. This has the advantage of low saturator pressure, but requires a much larger saturator. The full-stream option is seldomly used.

The usual option is part-stream DAF, where a fraction of the treated water is recycled to a saturator which is operated under much higher pressure. Enough air is then dissolved into the small volume of recycle to sustain flotation for the entire flow after the recycle is mixed with the feed in the reaction zone. The air requirements for DAF can be met by either recycling a small percentage under very high pressure, or by recycling a larger percentage under reduced pressure.

The choice between full-stream and part-stream pressurization is normally based on practical and cost considerations. In a rare experimental comparison between the two options for the thickening of water treatment sludge (Richard & Dauthuille, 1984), it was found that the underflow from full-stream pressurization had a higher SS concentration (between 100 and 300 $mg.\ell^{-1}$) than the underflow from part-stream pressurization (between 50 and 150 $mg.\ell^{-1}$).

The recycle ratio r can be calculated in a rational manner for any given combination of variables. For clarification applications, where the air requirement is expressed as a mass concentration per volume, the relationship is derived as:

$$r = \frac{C}{S_T . P . \eta_s} \qquad (2.3)$$

For thickening applications, the air requirement is expressed as the dimensionless air/solids ratio a_s. The relationship is then expressed as:

$$r = \frac{a_s . SS_i}{S_T . P . \eta_s} \qquad (2.4)$$

Equation 2.4 is often reported (e.g. Gulas, Lindsay, Benefield, Randall, 1978) in another form, namely:

$$r = \frac{a_s \, . \, SS_r}{C_s \left[\left(\frac{101{,}3 + P}{101{,}3}\right)\eta_s - 1\right]}$$

Note that this expression assumes saturator efficiency in terms of *atmospheric* air. If the saturator efficiency is known in terms of *saturator* air, it has to be converted before this expression can be used. Equation 2.4, which is flexible enough to allow the use of the solubility constant for either *atmospheric* or *saturator* air, is therefore recommended for practical use.

In the above equations, the variables are:

r	=	recycle ratio dimensionless
S_T	=	air solubility constant, $mg.\ell^{-1}.kPa^{-1}$
P	=	saturator gauge pressure, kPa
η_s	=	saturator air transfer efficiency, dimensionless
C	=	required air concentration, $mg.\ell^{-1}$
a_s	=	required air/solids ratio, $mg.mg^{-1}$
SS_r	=	suspended solids in raw water, $mg.\ell^{-1}$
C_s	=	saturation air concentration at atmospheric pressure, $mg.\ell^{-1}$

The recycle ratio for clarification usually varies from 6% to 12% of the raw water flow rate. In Sweden, the recycle percentage varies between 8 and 12% and the saturator pressure between 450 to 600 kPa (Rosen & Morse, 1976). In a survey of British plants, the recycle percentage varied between 6 and 10% and the saturator pressure between 310 and 830 kPa (Longhurst & Graham, 1987). Some designers add a safety factor to the minimum required recycle rate. A typical example is where the pilot studies indicated a recycle rate of 6%, but where the full-scale plant was eventually designed for 9% (Williams *et al.*, 1985). Experimental studies have shown that not much is gained by increasing the recycle percentage beyond the required minimum for clarification. Increasing the recycle rate from 7% to 10% did not improve the clarification of activated sludge at all (Maddock & Tomlinson, 1980). A similar observation was made when increasing the recycle rate above 6% when treating an eutrophic surface water (Bernstein *et al.*, 1985). In one study the treated water quality actually deteriorated slightly when the recycle rate was increased beyond the required 8% (Zabel, 1978).

For thickening, which required substantially more air than clarification, the recycle rate becomes of the same order as the raw water flow rate and thus has more significance. Not only will the recycle ratio have a significant influence on the hydraulic loading in the reaction, crossflow and flotation zones, but the designer will also have to provide an auxiliary water supply for the recycle during plant start-up if the recycle is taken directly from the subnatant stream.

Saturator pressure

Practical saturator pressures for part-stream recycle are typically between 350 kPa and 600 kPa gauge pressure. Two considerations come into play when the actual saturation pressure has to be fixed. The first consideration deals with the efficiency of the saturator. It was shown (Bratby & Marais, 1975b) that the saturation efficiency of packed saturators drops off sharply below a pressure of approximately 300 kPa, allegedly because of the reduced driving force. This does not pose a significant restriction on saturator design or operation, as the vast majority of saturators are operated at pressures higher than 300 kPa.

A second, more nebulous relationship is claimed between the saturator pressure and the diameter of the bubbles produced in the reaction zone. There is no consistency in the reports from the literature. Some claim that higher pressure produces larger microbubbles. In the pressure range 350 kPa to 420 kPa, larger bubbles were measured at higher pressure (Ramirez, 1979). In another study, the average diameter was 80 μm at 200 kPa, but 110 μm at 500 kPa (Lovett & Travers, 1986). Others claim that higher pressure produces smaller bubbles. At 210 kPa, the average diameter was 66 μm, while the average diameter decreased to 42 μm at 350 kPa (Shannon & Buisson, 1980). Some claim that there is no difference in bubble diameter, regardless of saturator pressure. Bubbles generated at 270 kPa and 340 kPa showed the same size distribution (Jones & Hall, 1981). A prominent research group came to the conclusion that DAF performance for clarification is only dependent on the total air quantity, regardless of recycle rate or saturator pressure (Rees *et al.*, 1980b). Most of the reports quoted here admit that although there may be differences at different pressures, the differences are too small to be of great practical significance.

Some studies have not measured the bubble sizes directly, but have evaluated DAF performance as a function of saturator pressure. For secondary clarification of waste water, SS_{out} showed a clear trend. From 200 kPa to 300 kPa, there was a sharp drop in SS_{out}, but from 300 kPa upwards SS_{out} gradually increased again (Maddock & Tomlinson, 1980). This does not only indicate an optimum bubble size at 300 kPa, but also refutes the claim that saturator pressure effects are too small to be of practical significance. On the other hand, the flotation of raw waste water was shown to be unaffected over a very wide range of saturator pressures, namely at 350 kPa, 550 kPa and 900 kPa (Bratby, 1982).

The data presented above have to be treated with circumspection. It is difficult to believe that saturator pressure was the only independent variable in the studies quoted above. It is very likely that the saturation efficiency and especially the injection nozzles also play a significant part towards the final bubble diameter in the reaction zone. This nevertheless remains a grey area in an important aspect of DAF and should be clarified before practical designs can be optimised with confidence.

Packed saturators

Packed saturators are vertically mounted cylindrical pressure vessels which are partially filled with packing material. The packing material consists of proprietary plastic packing pieces about 25 to 50 mm in diameter, and are randomly dumped during installation. The flow enters the saturator from the top through a flow distributor to ensure that the incoming flow is evenly spread over the area of the saturator. The flow is collected below or just above the packing support plate.

There are four saturator design parameters of importance, besides the saturator pressure which was already discussed:

- The *liquid loading rate* should not be too high, otherwise the packing will "flood" and the interfacial area between air and water will be reduced. An empty bed downflow velocity of 100 $m.h^{-1}$ was found to be a practical maximum in one case (Bratby &

Marais, 1975b), but in another, flooding was not evident even at 200 $m.h^{-1}$ (Rees *et al.*, 1980a). It was later shown that the flooding point is related to the void fraction of the packing (Casey & Naoum, 1986). For spheres with a void fraction of 44%, flooding was found at 90 $m.h^{-1}$, but for other types with void fractions of more than 75%, no flooding was found up to 120 $m.h^{-1}$. In terms of the air transfer efficiency, the hydraulic loading had a limited effect in the ranges which were experimentally investigated (Casey & Naoum, 1986; Rees *et al.*, 1980a; Zabel & Hyde, 1976).

- The *packing depth* should be above a certain minimum to ensure that the residence time of the water on the packing is long enough for adequate mass transfer. A minimum packing depth of 450 mm had been determined experimentally (Bratby & Marais, 1975b). Another study had suggested a packing depth of 800 mm for practical design. The packing depth has a significant effect on the efficiency of the saturator (Casey & Naoum, 1986; Rees *et al.*, 1980a).
- The *packing size* should not be more than one-eighth of the saturator diameter to limit short-circuiting down the wall of the saturator (Bratby & Marais, 1975b).
- The *packing type* was found to have just as important an effect on the air transfer rate as packing depth (Casey & Naoum, 1986). Another study, however, found that the packing type had little effect (Rees *et al.*, 1980a). Each study used a different range of packing types and sizes, so these conclusions are not necessarily contradictory.

Under conditions of appropriate liquid loading rate and packing depth, packed saturators had been found to be highly efficient. Values of 90% and higher are commonly reported. Packed saturator efficiency was also verified at higher temperatures. At 50°C, the measured efficiency varied between 70% and 90%; at 80°C the efficiency ranged from 90% to 95% (Shannon & Buisson, 1980).

One criticism against packed saturators is that the openings and crevices between the packing pieces are prone to blockage by solids that may be present in the recycle stream. One manufacturer of unpacked saturators harshly criticized packed saturators on the grounds that it will become a "biofilter" after a while and that it will start out at a high level of efficiency and will gradually deteriorate (Komline, 1976). This is an exaggerated claim because the high hydraulic loading in the saturator will be largely self-cleaning. It should anyway be a priority to keep the recycle stream as clean as possible, also to prevent blockage of injection nozzles. At a water treatment plant it is therefore advisable to use filtered water for recycling.

Unpacked saturators

Unpacked saturators are cylindrical pressure vessels, mounted vertically or horizontally, which are partially filled with water. Air and water are pumped into the saturator, and saturated recycle is withdrawn from the bottom.

Unlike packed saturators, where it is assumed that practically all mass transfer takes place in the packing, three zones of mass transfer can be distinguished in an unpacked saturator. Firstly, mass transfer takes place while the incoming water drops through the air cushion until it reaches the pool of water at the bottom. This part of the process is enhanced in practice by deflecting the incoming water with a splash plate (to increase the contact time) or by spraying the water into the saturator as fine droplets (to increase the interfacial area). Secondly, mass transfer takes place below the surface of the water due to the entrainment of air bubbles by the falling water. This part of the process is considerably enhanced by adding *internal recycling*, which continuously draws air below the water surface with an air eductor, as shown in Figure 2.7.

Thirdly, mass transfer takes place at the interface separating the top of the water pool and the bottom of the air cushion. This part of the process could be enhanced by setting up a convection pattern by asymmetric air injection below the water surface. This air-lifted circulation pattern will increase the surface renewal rate, as was demonstrated in China (Zhang, 1985).

The authors are not aware of any study which systematically investigated the relative importance of these three mass transfer zones. Until such information is available, the reported efficiencies reviewed below must therefore be treated with circumspection, as subtle differences in inlet arrangement or water level, for example, could perhaps cause widely differing efficiency.

The spray saturator is not very efficient, as the droplets exist only for the short period while they fall through the depth of the saturator. A thorough evaluation of this type of saturator showed it to be 60% to 70% as effective as packed saturators (Zabel, 1978). A single claim had been made that 85% efficiency had been achieved with a spray saturator, but without supporting data (Everett, 1983). Another study reported saturator efficiency varying widely between 58% and 100% (Bernstein *et al.*, 1985). To equal the air transfer rate of packed saturators, unpacked saturators have to be operated at pressures 100 kPa to 200 kPa higher than packed saturators (Zabel, 1978).

The efficiency of the saturator with internal recirculation depends on the internal recirculation rate. A single reported data point indicated efficiency of more than 100% for this type of saturator, but without the internal recycling rate being given (Komline, 1976). In general, these saturators were claimed to be between 75% and 95% efficient, depending on the internal recycle system design (Komline, 1978). Although internal recirculation may enhance the air transfer efficiency, it is wasteful of power. In one reported example, it was shown that the recirculation pump had to operate at 950 kPa when the saturator pressure was only 500 kPa (Vosloo & Langenegger, 1979), although this pressure differential is obviously affected by the way the system is designed.

Saturator control

Saturator control is a practical aspect on which little is reported in the literature, and which perhaps has a limited effect on the flotation process itself. There are, however, a number of systems in operation and a few general remarks are warranted to limit confusion.

There are three external connections to any saturator, namely the recycle inlet, the recycle outlet and the air inlet. It is also required to maintain an air cushion inside the saturator; in other words, the water level in the saturator must be controlled. This can be done in a number of ways:

- The recycle inlet and outlet are uncontrolled, which means that a constant flow rate is maintained through the saturator. The water level is then controlled between a maximum and minimum level by simply closing the air inlet when the minimum level is reached, and opening it when the maximum level is reached. This is the simplest method, but does not allow the air injection rate to be directly measured as the air flow into the saturator is intermittent. The air injection rate has to be calculated on the basis of saturator pressure, saturator efficiency and recycle flow rate.
- The air is supplied by a constant pressure source, but otherwise uncontrolled. The water level is then maintained by controlling the recycle *outlet*. If the water level drops too low, the outlet is throttled until the level rises again, etc. This has the disadvantage

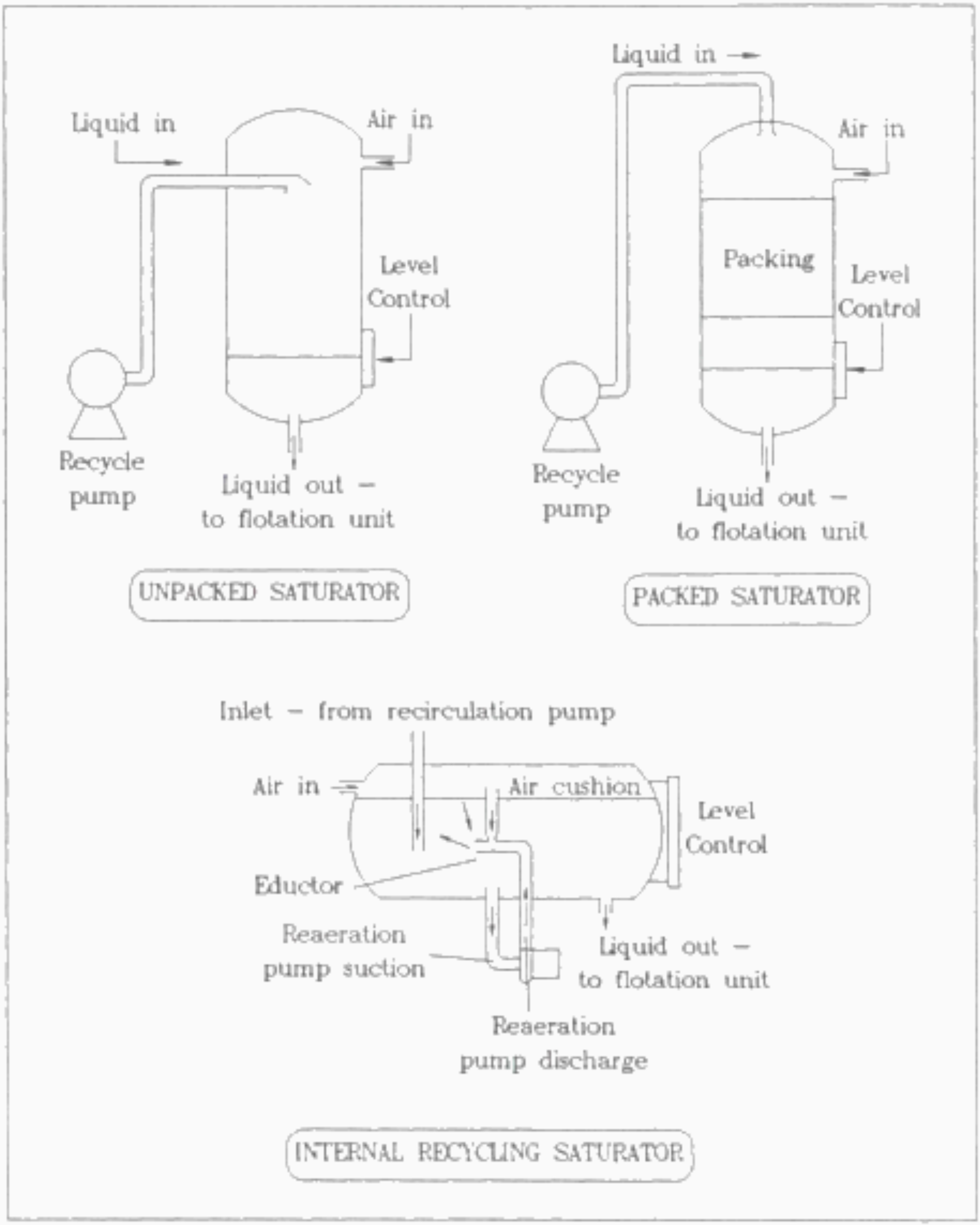


Figure 2.7 Different air saturator types

of not having a constant recycle rate, but a recycle rate fluctuating according to the water level in the saturator. The air flow rate is also not constant, but is interrupted when the saturator pressure exceeds the air supply pressure. Perhaps most importantly, the throttling valve causes a pressure loss after saturation which may lead to premature bubble formation.

- The air is supplied at a constant rate, and the water level is maintained by modulating the recycle flow rate *into* the saturator. If the level drops too low, more water is directed towards the saturator, and *vice versa*. This system has the advantage of being able to set a predetermined air injection rate at any time, but also suffers from a fluctuating recycle rate.

The ideal situation for efficient flotation is to maintain a constant recycle flow into the reaction zone, with very little or no premature air precipitation. The first of the above control options is therefore recommended for practical implementation. In all cases, it is recommended to provide an external sight glass on the saturator to enable the operator to see where the water level in the saturator is. One source (Childs & Rees, 1976) states that this requirement is indispensable for efficient operation.

Injection nozzles

The role of injection nozzles is considered to be of vital importance for DAF (Zabel, 1978). The aim of the injection nozzle(s) is to release the recycle stream into the reaction zone, while at the same time reducing the recycle pressure in such a way to generate microbubbles of uniform size. There are no hard-and-fast rules on how to achieve this. In fact, a prominent reference of about ten years ago stated that "the orifices used in DAF seem to be incapable of yielding only microbubbles. No method has yet been devised for generating in potable water a dense micro-bubble cloud, free from undesirably large bubbles." (Kitchener & Gochin, 1981). Many new nozzles have been developed and patented since, which probably have partly overcome this weakness.

A fundamental analysis of the physics of bubble nucleation shows that energy must be imparted to a liquid supersaturated with air before bubbles will form. This energy is usually provided by liquid turbulence. The energy is proportional to the cube of the surface tension and inversely proportional to the square of the applied pressure. The formation of many, small bubbles will be favoured if the bubble formation energy can be minimized. According to this reasoning, bubble size should be directly related to surface tension and inversely related to pressure (Lovett & Travers, 1986).

A few practical suggestions appear in the literature:

- The pressure must be reduced as close as possible to the point of dissolved air injection into the process stream. If this is not done, serious bubble coalescence can be expected (Zabel, 1978).
- The pressure reduction must occur instantaneously to avoid any premature partial reduction of pressure (Van Vuuren & Offringa, 1985; Williams & Van Vuuren, 1984). Contrary to this suggestion, a prominent European manufacturer argues that it is better to break the pressure in two distinct stages. Their two-stage pressure relief system is able to produce "almost 100% small bubbles, particularly if the temperature is maintained below 35°C" (Rovel, 1976). During the discussion of this paper, it turned out that the the two-stage relief system only worked if the saturation efficiency was below

60% to 70%. At 100% saturation, the bubbles formed too rapidly after the first stage. It was nevertheless claimed that the two-stage system offered more operational flexibility, because the recycle rate could be throttled between 10% and 25% (Conference on, 1977 : 68-76).

- The bubble formation can be improved by releasing the pressure through a number of small holes, rather than through a single injection point (Vosloo *et al.*, 1985).
- The bubble size distribution can be improved by letting the high-velocity jets impinge on a solid surface (Vosloo *et al.*, 1985).
- A shroud around the jet results in more satisfactory bubble formation, but even then larger bubbles are prevalent. This is ascribed to a recirculating region immediately downstream of the orifice in the low-pressure zone. Recirculation leads to bubble coalescence. The performance is improved by drilling small holes in the shroud to eliminate the low-pressure zone (Vosloo & Langenegger, 1979). Another theory explained the beneficial effect of a shroud by the fact that bubble formation is enhanced when the pressure reduction is done in a zone where the water is already saturated with air. The shroud effectively contains such a saturated volume close to the orifice (Williams & Van Vuuren, 1984).

Two nozzle designs were patented during the 1970s which have a number of features in common. The WRc nozzle was patented by the Water Research Centre in the UK as part of a systematic research programme into the use of DAF for water treatment clarification. The exact particulars were not published, but a general description of the underlying principles was given (Zabel, 1978; Rees *et al.*, 1980b; Zabel & Melbourne, 1980). It has two orifice plates through which the pressure is instantaneously reduced, as well as an orifice cover onto which the jets issuing from the orifices are directed. The orifice cover is needed to prevent the turbulence of the issuing jets from breaking the flocs. The entire nozzle is further directed onto a plate or the floor to further minimize floc breakup.

Parallel to these efforts, the NIWR nozzle was developed by the South African Inventors Development Corporation after research by the SA National Institute of Water Research. The NIWR nozzle has a ring of orifices at the end of a blanked-off pipe, with a cup-shaped orifice cover directly outside the ring of orifices. Water is forced through the orifices at high velocity and impinges perpendicularly onto the walls of the cup. Its success is ascribed to the very rapid reduction of the pressure within the cup. This time is estimated to be less then 0,01 seconds. The velocity through the orifices varies from 18 $m.h^{-1}$ to 25 $m.h^{-1}$ (Van Vuuren & Offringa, 1985; Williams *et al.*, 1985). A typical NIWR-type nozzle is detailed by Kegel (1987).

Both the WRc and NIWR nozzles have found extensive application in practice. The WRc nozzle was licensed to commercial contractors which have incorporated these nozzles in a number of plants. The NIWR nozzle has also been used in a number of variations as one large single nozzle or as multiple small nozzles, with the idea of an enclosing shroud or cup central to all the NIWR applications.

Numerous other nozzles had since been patented, for example the AKA-nozzle from Sweden, the RIKTOR- and VERKO-nozzles from Finland, and the DWL-nozzle from the Netherlands.

The use of ordinary diaphragm and needle valves is frequently discussed in the literature. Diaphragm valves are reportedly not very efficient, because of a relatively wide seat which causes a gradual pressure drop with many large bubbles. Needle valves are more successful, provided that the seat is narrow - not wider than 3 mm (De Wet, 1980). In some instances, the

NIWR could not achieve success with a needle valve, but found that it formed large bubbles which disturbed the float layer (Bernstein *et al.*, 1985). The WRc, however, reported that an ordinary needle valve gave very satisfactory results, except that such a valve would be subject to erosion due to the high velocity (Zabel, 1978). Some reports claim that needle valves are just as efficient as nozzles (Longhurst & Graham, 1987), while other prefer needle valves because they can be briefly opened at regular intervals to flush out any blockages (Vosloo, Langenegger, 1979). At any rate, needle valves were found to be superior to other methods such as capillary jets, perforated tubes and sintered metal plates (Packham & Richards, 1975b).

Limited work was done on the spacing and number of nozzles. The WRc nozzles were typically spaced 300 mm apart to keep the flow per nozzle as low as possible. In one experiment, eight WRc nozzles at 300 mm centres were compared to twelve WRc nozzles spaced 200 mm apart, but no difference in DAF performance could be measured (Rees *et al.*, 1980b). In another study, one large needle valve in the inlet pipe performed just as good as a number of smaller needle valves at 300 mm centres (De Wet, 1980).

An injection nozzle passing turbulent flow must obey the fundamental hydraulic relationship :

$$\frac{r.Q}{n} = C_d \cdot A_n \cdot \sqrt{2.g.h_n} \qquad (2.5)$$

with
A_n = nozzle opening(s) (m^2)
C_d = nozzle discharge coefficient (-)
h_n = head loss across injection nozzle (m)
n = number of nozzles
$r.Q$ = recycle flow rate ($m^3.s^{-1}$)

With adjustable nozzles, C_d and A_n are variable and the operator can thus manipulate the ratio between flow rate and head loss. With fixed nozzles, this is not possible and the design must be precisely done - this is illustrated in Chapter 6.

2.5 Flotation zone

Particle buoyancy and air requirements

Many discussions on the fundamentals of flotation start off by considering how much air is required to counteract the gravitational force on the solids in the water. This is the underlying reason for customarily expressing air requirements in terms of a_s, the dimensionless mass ratio between air and solids. Once the total mass and the specific gravity of the suspended solids are known, it is a trivial matter to calculate how much air has to be attached to the solids to obtain neutral density. The implicit assumption is thus made that every single air bubble is attached to a solid particle with 100% efficiency. Common sense dictates that perfect air efficiency will never be achieved, for any of the following reasons:

- bubbles not colliding with the particles at all
- bubble detachment from the particle/bubble agglomerates
- reduction of the number of bubbles through coalescence

- particle/bubble agglomerates must have a lower than neutral density before they will float.

A number of studies reported widely varying air efficiencies. The highest efficiency is obtained with waste-water sludge thickening, with the efficiency reportedly between 20% and 50% (Conference on, 1977 : 157-165). When clarifying activated sludge, the air efficiency is between 2.5% and 25% while it is as low as 1% when clarifying slightly turbid river water (Kitchener & Gochin, 1981). In terms of a_r, widely fluctuating air requirements are also reported. In a flotation study of metal hydroxides (Jones & Hall, 1981), required a_r values were 0.03 (for 680 mg $Zn.\ell^{-1}$), 0.08 (for 75 mg $Zn.\ell^{-1}$) and 0,30 (for 10 mg $Cu.\ell^{-1}$). For the thickening of waste activated sludge, a minimum ratio of 0,02 is prescribed, while the thickening of the lighter brown water sludges requires a ratio of 0,04 (Bratby & Marais, 1976b). For the clarification of raw waste water, a_r rises to 0.09 (Bratby, 1982), while a very dilute artificial iron suspension required a ratio of 0,20 (Shannon & Buisson, 1980).

A clear trend is evident from the data presented, namely that air is less efficient in dilute suspensions than concentrated suspensions. Relatively higher values of a_r are therefore required for more dilute suspensions. This phenomenon can be explained by considering the collision opportunities between bubbles and particles. At a high concentration of particles, the separation between particles and bubbles is necessarily small and the chances that every bubble will collide with a particle is very good. In other words, only a few bubbles will make it to the top of the flotation zone without colliding with a particle. In such a case, the use of a_r is entirely justified, because the air quantity is proportional to the mass of solids to be floated. In thickening applications, a_r is therefore critical.

For suspensions of low particle concentration, the situation is entirely different. Here the average interparticle distance is much larger. Waste activated sludge, for example, has a solids concentration of 1 000 times more than a typical eutrophic surface water. If the air quantity is reduced in accordance with the particle concentration, the chances for collisions between bubbles and particles reduce dramatically. The only option is to increase the air quantity to the point where the entire water volume is "covered" with the presence of air bubbles to ensure that particles will collide with the bubbles, regardless of how few particles there are. In this case, it becomes obvious that the air requirement is no longer a function of the particle concentration, but of the water volume. In clarification applications, it is thus the air mass per water volume that is critical. For example, there was no difference found in the air requirements of two dilute suspensions with suspended solids concentrations of 5 $mg.\ell^{-1}$ and 130 $mg.\ell^{-1}$ respectively (Packham & Richards, 1975b). It should be noted however that although the a_r is lower for thickening, the air requirement *per volume of water to be treated* is much higher than for clarification of dilute suspensions.

Clarification versus thickening

DAF is used for either clarification, or thickening. In the case of clarification, the concern centers primarily around the clarity of the effluent and less emphasis is placed on the float layer concentration. In the case of thickening, the focus is reversed. When clarification is the primary objective, the influent normally already has a low solids concentration. Conversely, in thickening applications the influent normally has a high solids concentrations to start out with.

In the sections that follow, it will be shown that the effluent quality is mainly determined by the *hydraulic* loading, whereas the float layer concentration is mainly determined by the *solids* loading. *For clarification of an effluent already low in solids concentration, the design is thus mainly based on the hydraulic loading, whereas designs for thickening are mainly based on the solids loading.* It is good practice, especially where the specific application is neither clear-cut clarification nor thickening, to check the design for effluent quality as well as float layer concentration. One study suggested that the transition between clarification and thickening occurs in the region of SS_{in} = 800 mg.ℓ^{-1} (Maddock & Tomlinson, 1980).

Limiting hydraulic loading

The designer of a flotation tank is firstly interested in the hydraulic loading (also termed the downflow rate) as this parameter determines the area and cost of the flotation zone. The design hydraulic loading has to be set lower than the *limiting* hydraulic loading v_L, which is the highest flow rate that can be tolerated before the quality of the effluent starts to deteriorate.

There is a very definite relationship between v_L and the mass of air added. With more air added to the suspension, the particle/bubble agglomerates will rise faster and it will require a higher hydraulic loading to prevent these agglomerates from rising to the surface. Numerous studies have verified this phenomenon by fixing a_s and then varying the hydraulic loading. The corresponding quality of the effluent shows a characteristic pattern as shown in Figure 2.8 (Williams *et al.*, 1985).

After finding v_L for each a_s, a plot can be produced of v_L against a_s for the specific application. It was shown (Bratby & Marais, 1975a) that the relationship between v_L and a_s follows the form:

$$v_L = K_1 \cdot a_s^{K_2} - K_3 \quad \ldots\ldots (2.6)$$

where v_L has the units m.h^{-1}

The second right-hand term K_3 relates to the settling velocity which the particles have on their own without the addition of air. With particles that have very little tendency to settle, such as algae, this term can be safely ignored. When the particles do have a definite settling tendency, K_3 has to be taken into account to produce a linear plot. This is illustrated in Figure 2.9.

A number of reported values of the constants in Equation 2.6 are summarized in Table 2.1.

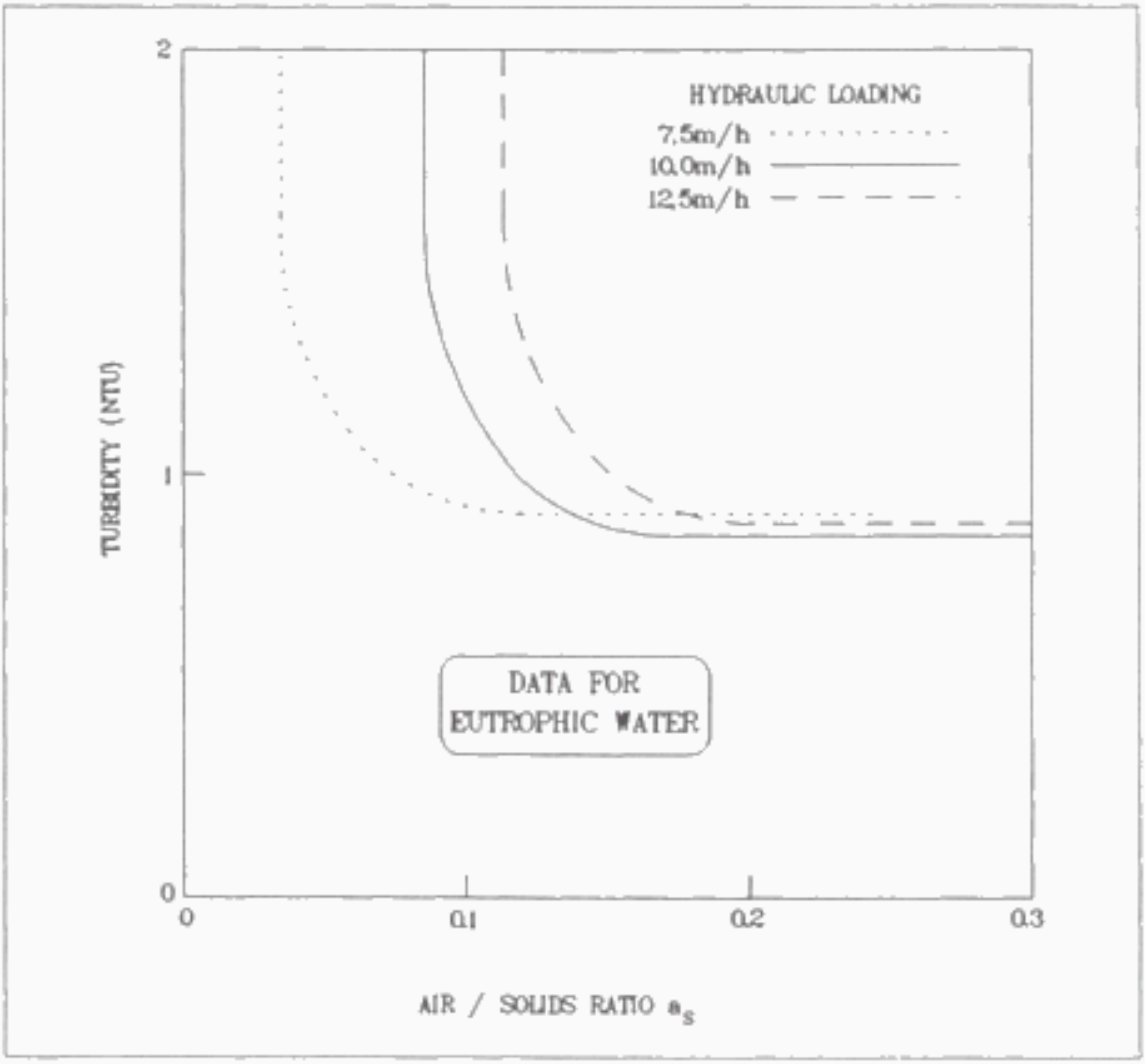


Figure 2.8 . Determination of minimum air/solids ratio a_s for different hydraulic loadings V (from Williams *et al*. 1985)

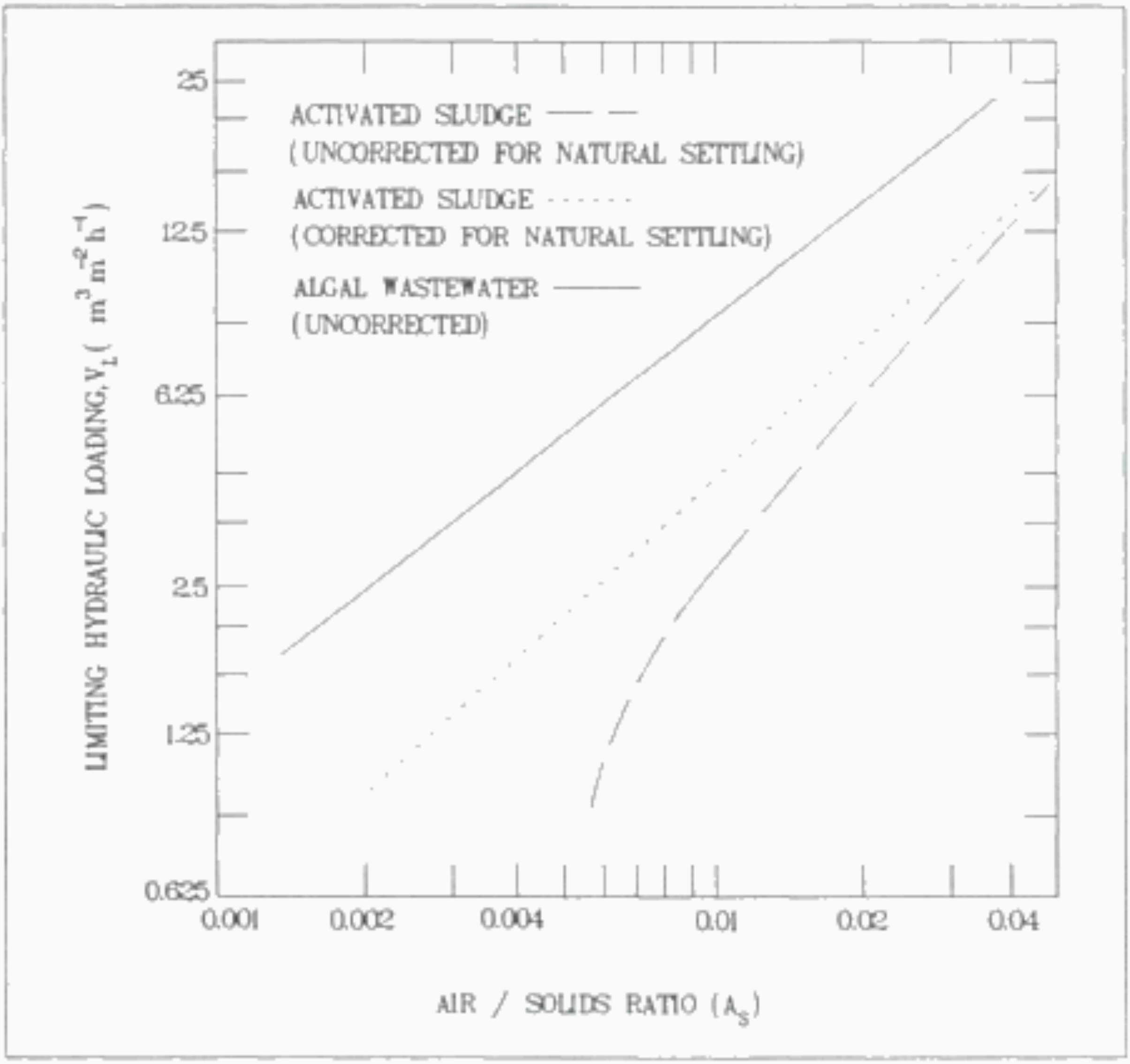


Figure 2.9 Relationship between air . solids ratio a and limiting hydraulic loading V (from Bratby & Marais, 1975a)

TABLE 2.1 EXPERIMENTALLY DETERMINED CONSTANTS FROM THE LITERATURE FOR CALCULATION OF THE LIMITING HYDRAULIC LOADING

Reference	Type of suspension	K_1	K_2	K_3
	Clarification			
#43	Raw municipal waste water	700	1,78	1,0
#56	Maturation pond effluent	3 690	1,92	0,0
#94	Eutrophic impoundment	68	0,91	0,0
#136	Algal-laden oxidation pond water	330	0,77	0,0
	Thickening			
#45	Activated sludge with extended aeration	133	0,92	1,3
#141	Activated sludge with primary settling	92	0,87	1,5
#141	Brown water sludge	108	0,72	0,5

An analysis of Table 2.1 shows that the relationship holds good for thickening, but that wide scatter occurs with clarification. It therefore seems as if the practical use of the relationship should be limited to thickening only, and then with caution.

The above procedure does not dictate a specific hydraulic loading to the designer. It only gives v_h for any given a_s. The designer still has to choose between a large flotation tank with a low air requirement, or a small tank with a high air requirement. A detailed economic analysis by Bratby & Marais (1975a) found the least-cost combination of v_h and a_s when a_s had the value

$$a_s = \frac{0,15}{SS_{in}^{0,36}} \qquad (2.7)$$

This relationship holds for both clarification and thickening.

Clarification

For drinking water clarification, where the designer usually has to contend with wide fluctuations in raw water quality and chemical dosage levels, a short-term pilot study will seldom encompass the full spectrum of raw water variability. However, from numerous pilot studies and full-scale operational experience, a few general design criteria have emerged that may be used for practical design.

It was earlier pointed out that the concept of a_s becomes artificial at very low solids concentration because of air inefficiency and that it becomes more realistic to work with the air mass concentration rather than a_s. For most clarification applications, the air requirement seems to be in the range from 5 mg.ℓ⁻¹ to 10 mg.ℓ⁻¹, based on the influent flow (Q). A typical plot of effluent turbidity as a function of air dosing, as determined from pilot work on raw water from the River Thames, is reproduced as Figure 2.10 (Zabel, 1978). At a successful DAF application at a hypertrophic source, the air dosing was set at 7,2 mg.ℓ⁻¹, 50% higher than determined from a pilot study along the lines of that described earlier (Williams *et al.*, 1985). Practical experimentation elsewhere had shown that a minimum air concentration of about 8 mg.ℓ⁻¹ is required for clarification of dilute suspensions (Rees *et al.*, 1979). In other examples, an air concentration of 5 to 6 mg.ℓ⁻¹ was sufficient (Vosloo *et al.*, 1985). A recommendation was made that the air dosage should be between 4 and 8 mg/ℓ (Van Puffelen, 1990).

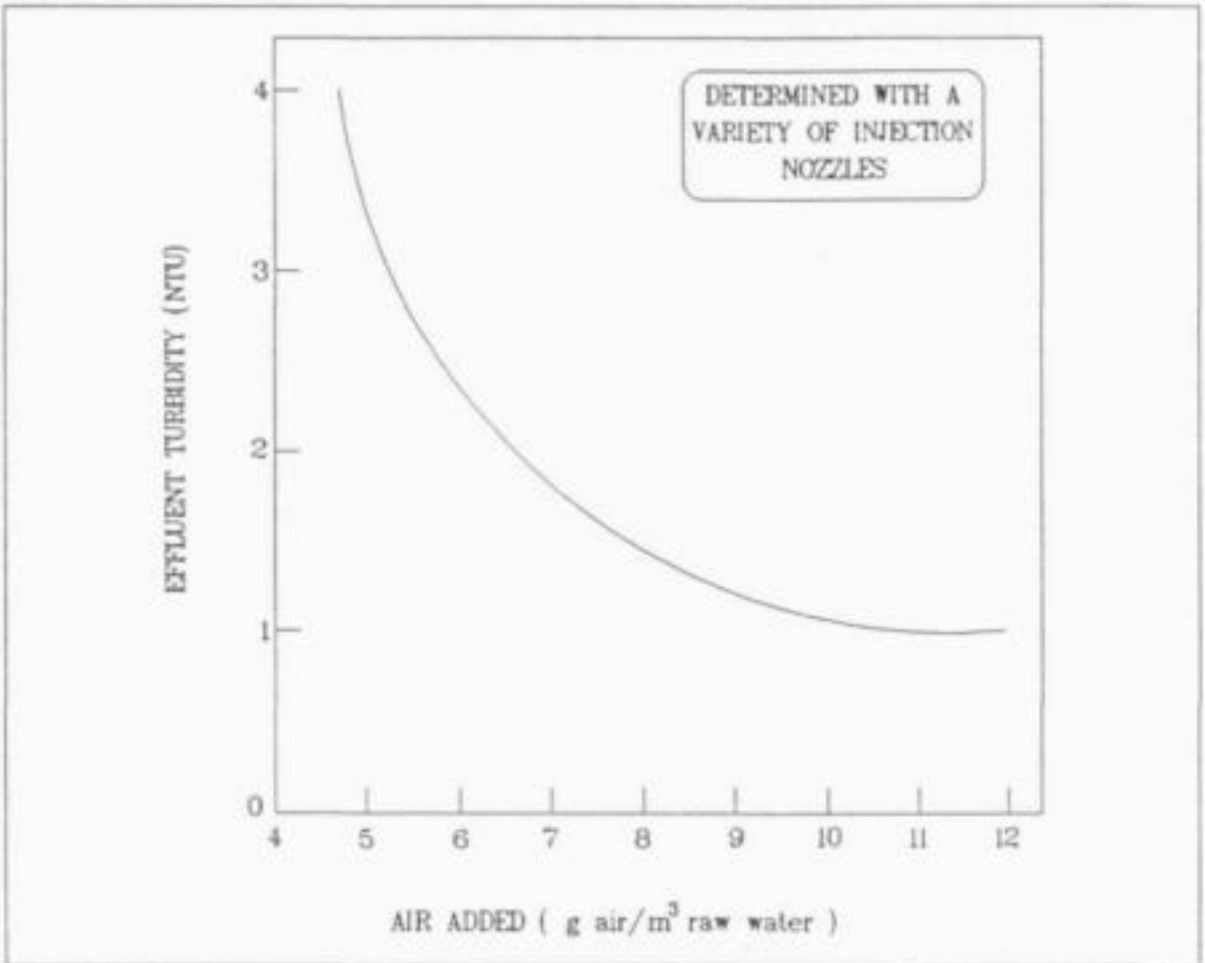


Figure 2.10 : Relationship between air dosage and effluent turbidity in a typical clarification application (from Zabel, 1978)

For clarification purposes, the hydraulic loading practically varies between about 5 m.h⁻¹ and 15 m.h⁻¹. In one study, an upper limit of 12 m.h⁻¹ was reported above which performance deteriorated, regardless of how much air was applied. Only at a solids concentration above 800 mg.ℓ⁻¹, was it necessary to decrease the hydraulic loading below 12 m.h⁻¹ to maintain performance (Maddock & Tomlinson, 1980). Other reported findings include: no significant change in treated water quality between 2,5 and 11,4 $m.h^{-1}$ (Zabel & Hyde, 1976); a range of 6 to 10 $m.h^{-1}$ in Sweden for cold water applications (Rosen & Morse, 1976); a range of 6 to 9 $m.h^{-1}$ for 12 operating plants in the United Kingdom (Longhurst & Graham, 1987); no difference in algal removal between 5 and 10 $m.h^{-1}$, and no difference in turbidity removal between 7 and 15 $m.h^{-1}$ (De Wet, 1980); satisfactory flotation of chemically treated raw waste water between 4 $m.h^{-1}$ and 10 $m.h^{-1}$ (Anon, 1986); and good performance with maturation pond effluent up to 15 $m.h^{-1}$ (De Wet & Van Vuuren, 1980) probably due to photosynthetic oxygen supersaturation.

The hydraulic loading range mentioned applies only to clarification in municipal water and waste-water treatment. For other industrial applications, the limiting hydraulic loading could be drastically different. For the clarification of seawater prior to membrane desalination, for example, the limiting hydraulic loading was experimentally found to be only 0,8 $m.h^{-1}$ (Hebden & Botha, 1978).

The hydraulic loading can be increased if lamellae are inserted in the top of the flotation tank. If these plates are shaped with longitudinal corrugations, the float layer collects in the top section of the corrugations as the water flows in a downward direction through the plates. The trapped float layer flows counter-currently out of the tank in an upward direction and leaves the tank through a conduit. The operation of the lamella-assisted flotation tank is illustrated in Figure 2.11. In one example, the hydraulic loading could be increased from 4 $m.h^{-1}$ (maximum without lamella) to 10 $m.h^{-1}$ if lamellae were inserted (Schade, 1982). Two other examples from the paper and pulp industry showed that the loading rate could be increased 30% without affecting the solids removal, or that the flow rate could even be increased two to three times (Henry & Gehr, 1981). One report, which also reported that lamellae enabled the designer to double the flow rate, ascribed the beneficial effect of lamellae to the fact that it directed the flow uniformly to the float layer and that it also prevented the turbulence from large-scale circulation patterns from disrupting the float layer. Very little accumulation of solids on the lamellae was experienced when paper mill effluent was clarified (Lundgren, 1970). Little design information is available and lamella-assisted flotation has not nearly gained as much ground as lamella-assisted sedimentation.

With a certain pattern emerging from the various design parameters, the question arises whether it is still necessary to precede each DAF design for clarification with a pilot testing program. Up to a number of years ago, no such guideline values existed and these tests were indispensable (and also gathered invaluable scientific data). For clarification, the float layer characteristics are of secondary importance, and the main emphasis is on the quality of the subnatant. The quality of the subnatant, again, is largely a function of the coagulant used and the chemical dosage selected. These parameters are fully adjustable during normal operation and does not really impinge on the design of the plant. It would, therefore, appear as if DAF design for clarification applications could now go ahead without necessarily doing pilot work first, provided that optimum flocculation conditions have been defined. It stands to reason that any water with unusual characteristics would require pilot work of some sort, and that a reasonable degree of operational flexibility should be allowed. From the authors' experience sufficient guidelines for design is currently available for clarification of eutrophic water and thickening of waste sewage sludges. If necessary simple batch or continuous laboratory tests at most need to be considered.

This section on clarification is closed with a discussion of cases where DAF may *not* be a feasible process. In cases where high levels of inorganic turbidity are experienced, DAF

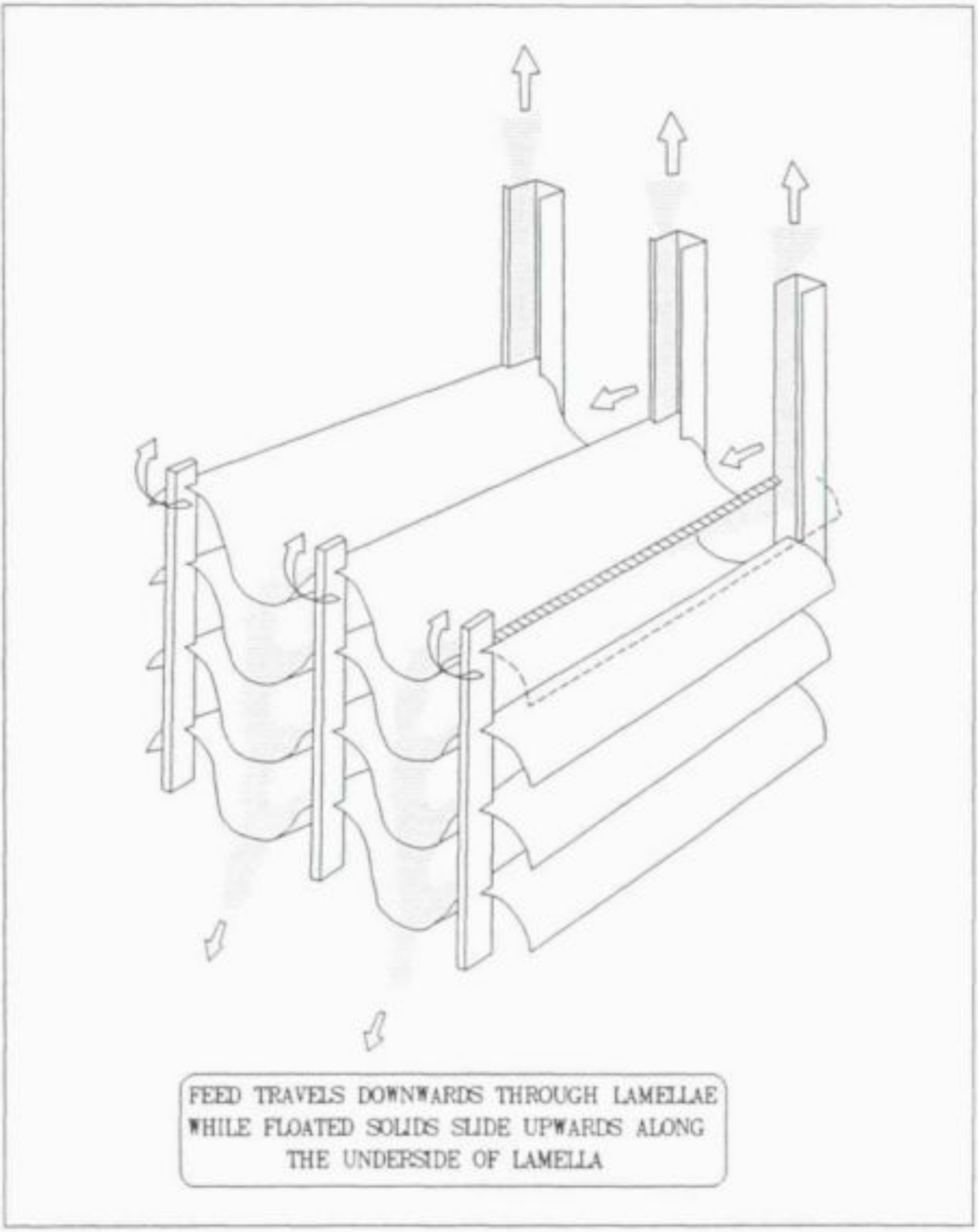


Figure 2.11 : Example of lamella - assisted flotation (from Schade, 1982)

performance will deteriorate during high turbidity (Conference on, 1977 . 407-416). In such cases, it will become necessary to employ presedimentation ahead of flotation. The benefits of presedimentation at 4 $m.h^{-1}$ for subsequent flotation was clearly illustrated at a full-scale plant treating eutrophic surface water (Botes & Van Vuuren, 1990). In this case, the critical turbidity level, above which effluent deteriorates was found to be around 80 NTU. Presedimentation is also required for combined flotation/filtration (to be discussed in a later section) when the turbidity is high. A detailed cost analysis indicated that, on economic grounds alone, flotation was not justified for clarification when the suspended solids in the raw water exceeded 1 000 $mg.\ell^{-1}$ (Bratby & Marais, 1975a).

Thickening

In thickening applications the emphasis shifts towards the character of the float layer. The designer primarily wishes to know the effects of the process parameters on the concentration of the float layer C_F, expressed here as a percentage.

The solids loading rate Q_s has an undisputed effect on the solids concentration of the float layer C_F. With one exception (Gulas & Lindsey, 1978), a higher solids loading rate causes a lower C_F, and *vice versa*. With more solid particles to float, the rate of float layer removal must increase which leaves less time for the particles within the float layer to compact or consolidate to higher density. This phenomenon is widely reported (Bratby & Marais, 1975a; Vrablik, 1959) and a typical experimental plot is shown in Figure 2.12.

The air/solids ratio a_s also appears to have an effect, although this is not an unanimous conclusion. The very significant work of Bratby & Marais (for example Bratby & Marais, 1975a) conclusively shows that the a_s ratio has no direct effect on C_F. Other workers (Gulas & Lindsey, 1978; Vrablik, 1959; Langenegger & Viviers, 1978) show that a higher a_s ratio does cause a higher C_F, but it is evident that the effect on C_F does decrease at higher a_s ratios. These conclusions are not really contradictory, for Bratby & Marais show how a lower a_s does indirectly affect C_F by causing a thicker overall float layer - a subtle distinction which the other reports did not draw. Figure 2.13 illustrates some of the results.

From a survey of a large number of plants in the USA (Komline, 1976), an a_s of 0,02 is considered sufficient but a value of 0,04 is provided in the design as a safety factor. Calculated a_s values for activated sludge by Bratby & Marais (1975a) in a design example are of the order 0,01 but a "safer" value of 0.02 is recommended because of d_F/d_A sensitivity at low a_s values. For the thickening of brown water sludge (when highly coloured humic water is chemically precipitated) they recommend an a_s of 0,04. Further statements made by Bratby & Marais are briefly the following :

- From an economic viewpoint the saturation pressure (>300 kPa) is immaterial provided that the recycle ratio is such that in combination with saturation pressure the optimum a_s is achieved
- a_s values are based on the amount of air precipitated and not on actual quantity attached to the solids
- Optimum a_s for lowest cost is independent of Q and solely dependent on SS_i.
- a_s has little effect on C_F but has a significant effect on the ratio d_F/d_A.
- The principal variable influencing clarification is a_s because it affects v_s.
- a_s values required for a given v_s for activated sludge are higher than for algal waste waters

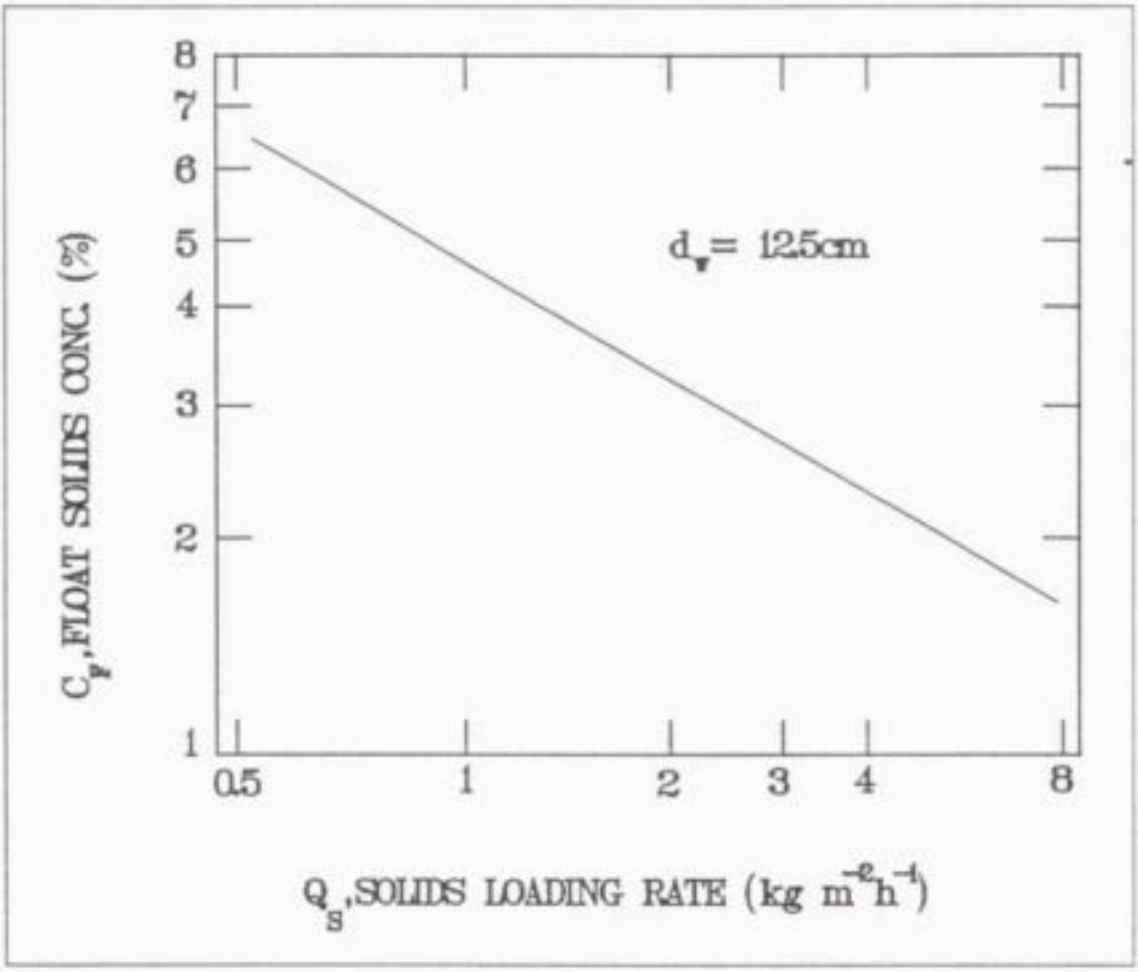


Figure 2.12 : Relationship between solids loading rate Q_s and float layer concentration C_F (from Bratby & Marais, 1975a)

The depth of the float layer has a very definite effect on C_F. A deeper float layer causes higher C_F. Most workers report their results in terms of *total* float depth, but the very plausible distinction had been made between those portions of the float layer that lie *above* and *below* the water level (Bratby & Marais, 1975a). Bratby & Marais argued that only the layer above the water level affords the interstitial water the opportunity to drain out of the float layer and that this portion is therefore more important. They demonstrate experimentally that this is indeed the case and build a comprehensive design theory around this principle. A linear relationship exists between the depth above the water level and C_F. Another report (Langenegger & Viviers, 1978) shows the concentration gradient through the float layer, where the layer above the water level has a significantly higher concentration than below the water level. In a series of pilot trials in Johannesburg, the depth above the water level was also shown to be the critical factor towards successful thickening (Langenegger & Viviers, 1978, (discussion)).

The character of the sludge has also been investigated for its effect on the degree of thickening by dissolved air flotation. The most generally used indicator of sludge settleability is the sludge

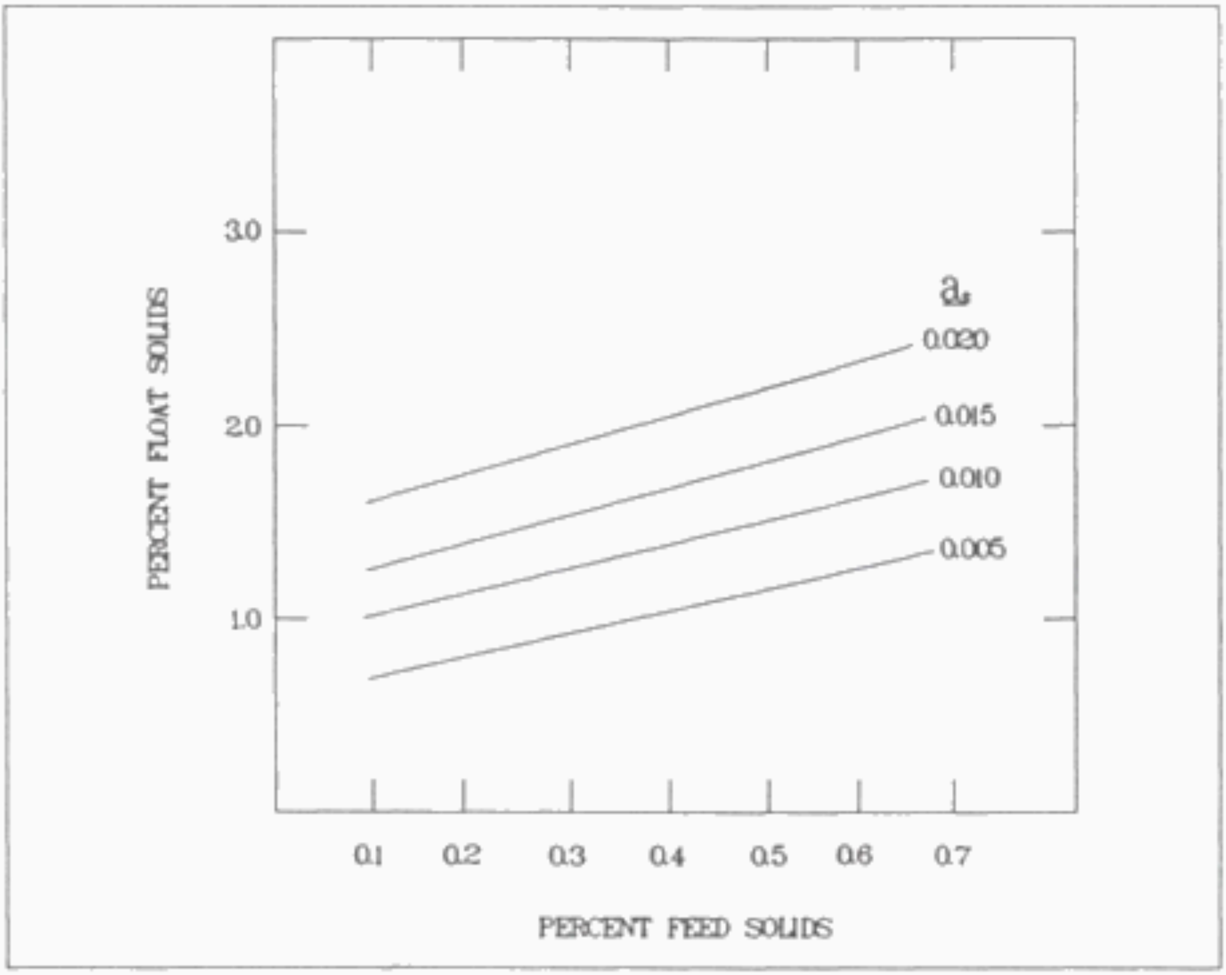


Figure 2.13 : Relationship between feed solids concentration and float layer concentration C_F (from Gulas & Lindsey, 1978)

volume index (SVI), and this has understandably also been used as a potential indicator of how well a sludge will thicken during flotation. Bratby and Marais (1975a) found very little effect on C_s due to the SVI and have ignored this effect in the development of their design model. Other workers (Gulas, Lindsey, 1978; Ettelt, 1964; Langenegger & Viviers, 1978) have found a very definite effect, as demonstrated in Figure 2.14.

The value of the SVI was confirmed during a discussion (Conference on, 1977 : 166-178) when it was suggested that the SVI is essentially an indicator of sludge *compactability* and therefore applies equally well to sedimentation as well as flotation. Others (Langenegger & Viviers, 1978, (discussion)) acknowledge the fact that sludge properties affect thickening, but consider the SVI as having too many shortcomings. It appears however to be an indisputable fact that the SVI, at least in some cases, has a definite, quantifiable effect on C_s.

After these remarks on the effect of *individual* parameters on C_s, the interest turns towards the knotty problem of the *interrelationship* between these process variables. The work of Bratby

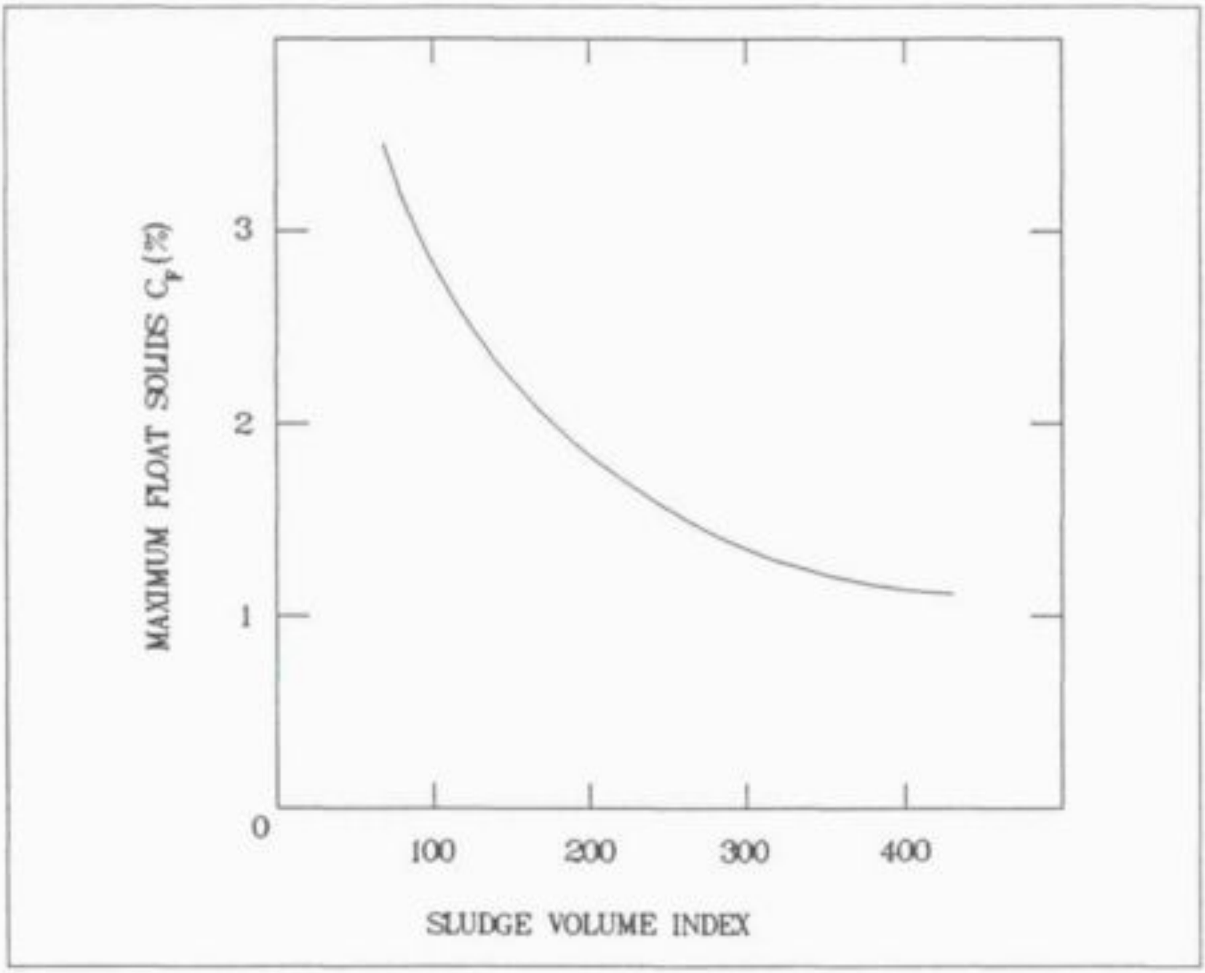


Figure 2.14 : Relationship between sludge volume index SVI and float layer concentration C_F (from Gulas & Lindsey, 1978)

& Marais (1975a) includes a lucid illustration of just how the different process parameters are intertwined. In a previous section it was shown that there is an interrelationship between a_r and the limiting hydraulic loading V_L: the higher a_r, the higher V_L. Two more such interrelationships exist. If a_r is decreased, the solids loading rate must also be decreased for the same C_F (i.e. a larger flotation area). Further, a lower a_r will give rise to a deeper float layer, thus requiring a deeper tank. Each of the interrelationships represents a balance between conflicting costs which the designer has to resolve. A comprehensive cost analysis of all the flotation elements was conducted to find the least-cost combination of process variables.

The following optimum process parameters emerged from this analysis. The optimum a_r is firstly determined from Equation 2.7 previously given. A target value for C_F is arbitrarily set, and the depth d_w above water level can then be calculated:

$$d_A = 0{,}025 \,.\, C_F + 0.19 \qquad \ldots\ldots\ldots\ldots \qquad (2.8)$$

with d_A in metres. With d_A known, the total depth of the float layer is given by:

$$d_A + d_S = d_A \,.\, (1 + 0{,}76 \,.\, a_r^{-0.4}) \qquad \ldots\ldots\ldots\ldots \qquad (2.9)$$

Finally, the solids loading rate Qs can be determined from.

$$Q_S = \left(\frac{31.7 \quad d_A}{C_F}\right) \qquad \ldots \qquad (2.10)$$

Langenegger & Viviers (1978) conducted pilot studies to relate design criteria to the physical characteristics of sludge. Some of the results are as follows

- Mixed liquors with higher SVI values yield lower solids float concentrations under similar conditions. The physical characteristics of a sludge thus affect the design criteria of the plant.
- Higher a_r ratios result in more compact sludge layers and increased rate of compaction. This effect tapers off beyond $a_s = 0{,}026$ and there is an optimum a_s for each sludge
- Depending on sludge properties it would be possible to thicken waste activated sludges to C_F values between 2 and 6 per cent.
- For design purposes it is essential to conduct tests to determine some of the physical properties and its amenability to flotation.
- It would be advisable to determine the average SVI of the sludge to predict its effect on the flotation process.
- A solids loading rate of up to 10 $kg.m^{-2}h^{-1}$ proved to be workable, provided the air solution system could maintain a_r values of up to 0.03.
- An adjustable weir should be installed at the effluent outlet, in order to control the water level over about 300 mm. The total float layer thickness should be limited to approximately 600 mm with provision for operational control

Results from laboratory studies have to be extrapolated to full-scale designs with caution. Not only is there the obvious scale difference, but laboratory results are often obtained from batch tests while large units operate continuously. For the thickening of waste activated sludge, almost no scale effects were evident (Bratby, 1978; Maddock, 1976; Henry & Gehr, 1981). In the continuous case, however, a slightly better effluent was obtained (Henry & Gehr, 1981).

Flotation zone configuration

Once the hydraulic loading is fixed, the flotation tank area is determined by the design flow rate (including recycle) and the number of flotation units. The first choice then lies between rectangular or circular units. The advantages of both types are summarized in Table 2.2 (Everett, 1983; Bennet, 1988).

TABLE 2.2 ADVANTAGES OF FLOTATION TANK SHAPES

CIRCULAR	RECTANGULAR
Decreasing velocity towards outer edge Simpler rake arm mechanism Cheaper - more area per unit wall length Lower outlet weir loading Bottom scrapers are cheap Top mounted drive eliminates wet bearings	Easier housed inside buildings Multiple units cheaper due to shared walls Hoppered bottom eliminates bottom scraper Prefabricated units can be transported Easier introduction of water

For thickening of activated sludge, circular tanks are preferred to rectangular tanks. Where rectangular tanks are used, it is better to use broad, short than long, narrow tanks (Bratby, 1978). Both these suggestions are based on minimizing the scouring effect on the underside of the float layer. Although the average horizontal velocity under the float layer may be low, considerable streamlining was measured in this zone which resulted in local velocities three times higher than the average (Bratby, 1978). For clarification of algal water, it was suggested that circular tanks are used for smaller applications, and rectangular tanks with an L/W ratio of 2,5 for larger applications (De Wet, 1980). Two other reports also suggest that rectangular tanks are better than circular tanks (Anon, 1984; Zabel & Melbourne, 1980). Of practical interest is the fact that single scraper arms in rectangular tanks have a maximum span length of about 3 m, which does restrict the choice somewhat as far as tank width is concerned (Zabel & Melbourne, 1980).

Two recommendations on the maximum design flow for a single flotation tank were found, namely 420 (De Wet, 1980) and 750 $m^3.h^{-1}$ (Zabel & Melbourne, 1980). Above these rates, multiple tanks are unavoidable. A tank area of 50 to 60 m^2 was proposed as the upper limit of economical design (Conference on, 1977 : 401-406).

The minimum side depth required for a flotation tank is an important cost parameter. For the thickening of activated sludge, the float layer can reach a depth of up to 1 m. In a practical case where the total depth was 2 m, the remaining 1 m under the float layer was found to be insufficient for proper clarification. The average horizontal velocity in this case was 51 $m.h^{-1}$ (Bratby, 1978). No suggestion for minimum flow depth under the float layer or the horizontal velocity under the float layer was forwarded. At pilot trials with an eutrophic surface water, where clarification was the main concern, the turbidity was vertically tracked from the top (Bernstein *et al.*, 1985). The turbidity decreased in a downward direction up to 900 mm from the top. Below this level, the turbidity stayed constant. From a clarification point of view, it would thus seem that about 1 m is required for satisfactory phase separation. This conclusion is supported by a study on turbid river water clarification where no difference in performance was measured between side depths of 1,2 m and 1,8 m (Rees *et al.*, 1980b). In a survey of 12 operating flotation plants for clarification in the United Kingdom, the side depth varied between 1,0 and 3,2 m, with an average of 2,4 m (Longhurst & Graham, 1987). For algal waters, a much deeper side depth of 3,5 m was recommended (De Wet, 1980), but without supporting evidence.

The design of the outlet was shown to have a marked effect on the performance of the flotation zone (De Wet, 1980). With a pipe collector system, which provided an array of draw-off points near the tank floor, there was a steady downflow pattern. The algal removal in this case was 95% and the minimum retention time 7 minutes. When the pipes were removed and the water was withdrawn through a single horizontal slot near the bottom at the far end, a recirculation pattern was set up and the algal removal dropped to 90% and the minimum retention time to 5 minutes. The flow patterns are demonstrated in Figure 2.15.

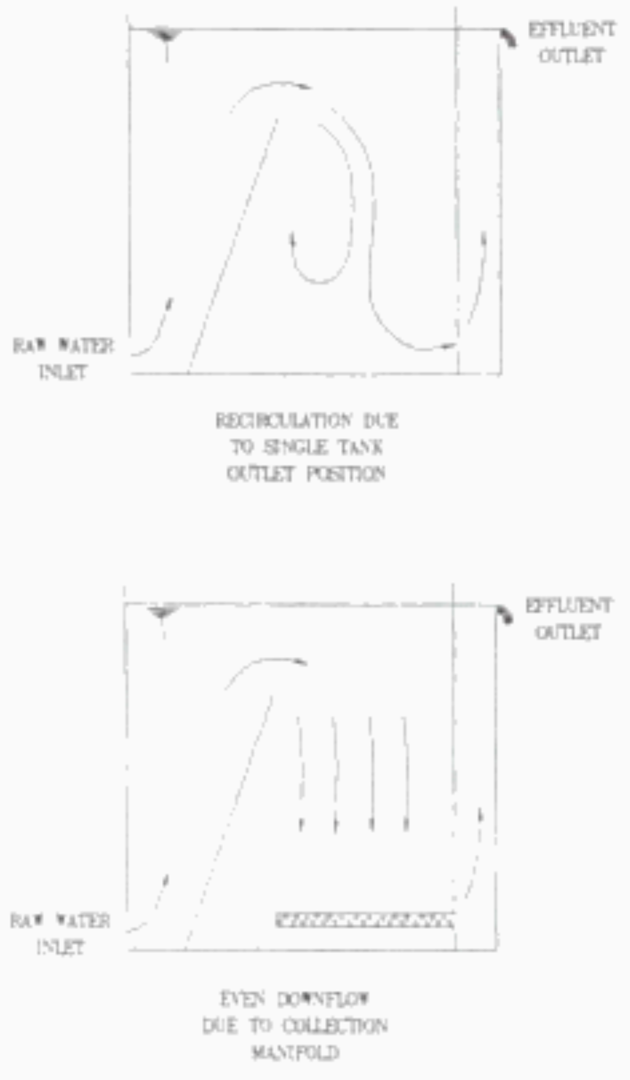


Figure 2.15 : Flow patterns induced by different collection systems (after De Wet, 1980).

The use of a roof over the flotation zone is generally recommended. A roof will eliminate float layer disruption by rain and will also diminish float layer disruption by wind, but it is especially important in colder climates where it may prevent freezing of the water and ancillary equipment. In a recent survey of flotation plants in the United Kingdom, it was found that roofs were provided without exception (Longhurst & Graham, 1987).

2.6 Float layer removal

Float layer stability

Two different phenomena are understood under the general term of float stability. The first describes the stability of the float layer under normal operating conditions, it is how long the float will remain undisturbed on the surface before it starts to disintegrate and sink to the bottom of the flotation tank. This stability of the float layer depends on the nature of the suspension being floated. In the case of water with low turbidity and high colour, the float layer became unstable after 30 minutes, while a turbid river water allowed float layer accumulation of up to 48 hours without a deterioration in effluent quality (Zabel, 1978). An experience with algal water was reported where the float layer remained stable for up to three days until the float layer thickness reached 150 mm (Williams *et al.*, 1985), and another where the float layer remained stable for five days (Van Vuuren & Van der Merwe, 1989).

The second type of stability pertains to the stability of the float layer when the feed and recycle are interrupted, i.e. that no more air and particles are added to the float layer. This is not so much a design as an operational consideration, for it determines the DAF startup and shutdown procedures. In the case of an algal water, where the float layer was shown in the previous paragraph to have excellent stability under normal operating conditions, the float layer became unstable within two hours of the DAF process being interrupted, and the operation of the plant consequently had to be changed to keep the recirculation pumps running all the time, even when the plant was stopped (Williams *et al.*, 1985).

Removal options

The float layer cannot, even if it remains stable under normal operating conditions, build up indefinitely and has to be removed periodically. The simplest option is to drain the tank from time to time, thereby scouring out the float layer with the tank contents. In the case of combined flotation/filtration, where the tank is drained anyway when the filter is backwashed, this option is indeed utilized and no float removal measures are taken at all. Some combined flotation/filtration applications make use of scrapers as well.

Most DAF applications, however, have special provision for the continous or intermittent removal of the float layer. The two options are flushing, where the float layer is drained off the top by lifting the water level relative to an overflow weir, or by scraping the top of the float layer into a sludge collection trough. Flushing produces a very dilute sludge with fairly high water loss, while scraping achieves the opposite.

The choice between these options is largely determined by the eventual fate of the sludge. If DAF serves as pre-thickening step before vacuum or belt pressing, or the sludge has to be landfilled, the float layer would obviously be scraped. If, on the other hand, the primary purpose of DAF is clarification and the sludge is blended with filter washwater and the recovered water returned to the head of the plant, flushing of the float layer could also be considered. For thickening purposes continuous or intermittent scraping are the only options.

Float layer flushing

Intermittent float layer flushing can be achieved in one of two ways. The float layer can be flushed off the top by closing the outlet of the flotation tank, which causes the water level to rise up to an overflow weir. The actual discharge into the overflow trough will start off slow, and eventually reach a maximum rate equal to the inflow rate into the flotation zone. The float layer can also be flushed by lowering one side of the tank adjacent to the overflow channel. In this case, the initial discharge into the sludge trough will start out high, but will eventually stabilize at the inflow rate into the flotation zone. It was reported that flushing is the usual way of removing the float layer in Swedish water treatment plants (Rosen & Morse, 1976).

In one of the rare reports on this subject, the float layer was flushed every 12 hours by closing the outlet. It took 9 minutes to desludge the tank completely and the water loss associated with this method of operation was about 1,4% (Zabel, 1978).

The float layer can also be flushed continuously by keeping the water level very slightly above the overflow weir. This method was evaluated with a low turbidity water with high colour. The water loss was 1,6% while the average sludge concentration was very low at 0,11% (Zabel, 1978).

Float layer scraping

A simple mass balance on the solids across the flotation zone yields the following expression:

$$C_F = \frac{Q}{q} \cdot (SS_{in} - SS_{out}) \cdot 10^{-4} \qquad (2.11)$$

with Q the flow rate into the flotation zone and q the rate at which sludge is being removed.

Equation 2.11 shows that the float solids concentration C_F is inversely proportional to the sludge removal rate q. As long as the actual float solids concentration is equal to or higher than the concentration calculated by the above equation, steady state will be achieved. If, however, the actual float solids concentration is lower than the calculated concentration, steady state will not be achieved and there will be a gradual buildup of solids in the tank. This increase of float layer thickness cannot continue indefinitely, as the horizontal flow velocity between the float layer underside and the tank floor will increase to the point where fragments of the float layer are scoured into the effluent. A practical example is reported in the literature (Ettelt, 1964) where variable speed scrapers were used to maintain a constant C_F. At optimum conditions, the float layer depth was 600 mm and C_F was 4,0%. If the scrapers were set too fast, C_F would decrease to 2,5% within 24 hours; too slow a setting would lead to buildup of the float layer depth. In another similar design (Martin & Bhattarai, 1991) the scraper step speed was the control variable selected to meet the desired thickness and float layer concentration range. A variable-speed drive controlled the thickness. A float layer thickness of less than 300 mm resulted in poor dewatering and a low value of C_F.

The most important drawback of float layer scraping is the deterioration of the effluent while the scraping is being done. Previously separated solids break loose from the float layer while it is squeezed and disturbed by the scraper blades which necessarily have to dig into the float layer.

A well documented study on the scraping of alum sludge showed a striking difference in effluent quality between a scraped and an unscraped tank, which is reproduced as Figure 2.16 (Maddock & Tomlinson, 1980).

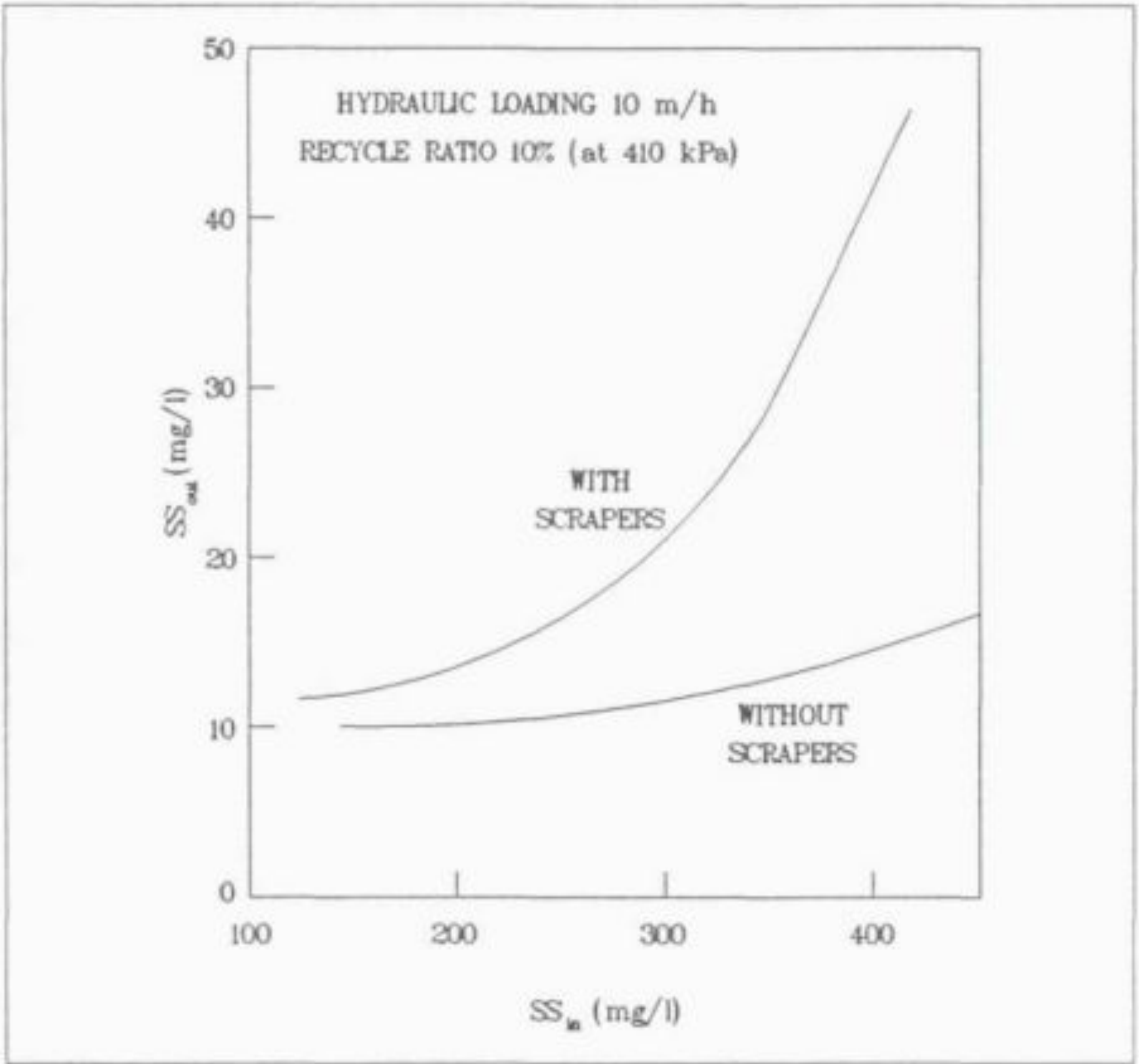


Figure 2.16 : Effect of float layer scraping on effluent quality (from Maddock & Tomlinson, 1980).

If a tank has to be scraped, there is a choice between continuous and intermittent scraping. In general, intermittent scraping causes greater deterioration of effluent quality, but produces a thicker sludge (Hyde, 1975). In the case of a turbid river water, treated water turbidity was shown to deteriorate during scraping, but recovered to normal 15 minutes after the scraper was stopped. In the case of algal water, scraping was only done every 2 hours - in this case, the quality recovered within 5 minutes after scraping ceased (Zabel, 1978). The temporal effect of scraping is shown in Figure 2.17 (Zabel & Rees, 1976).

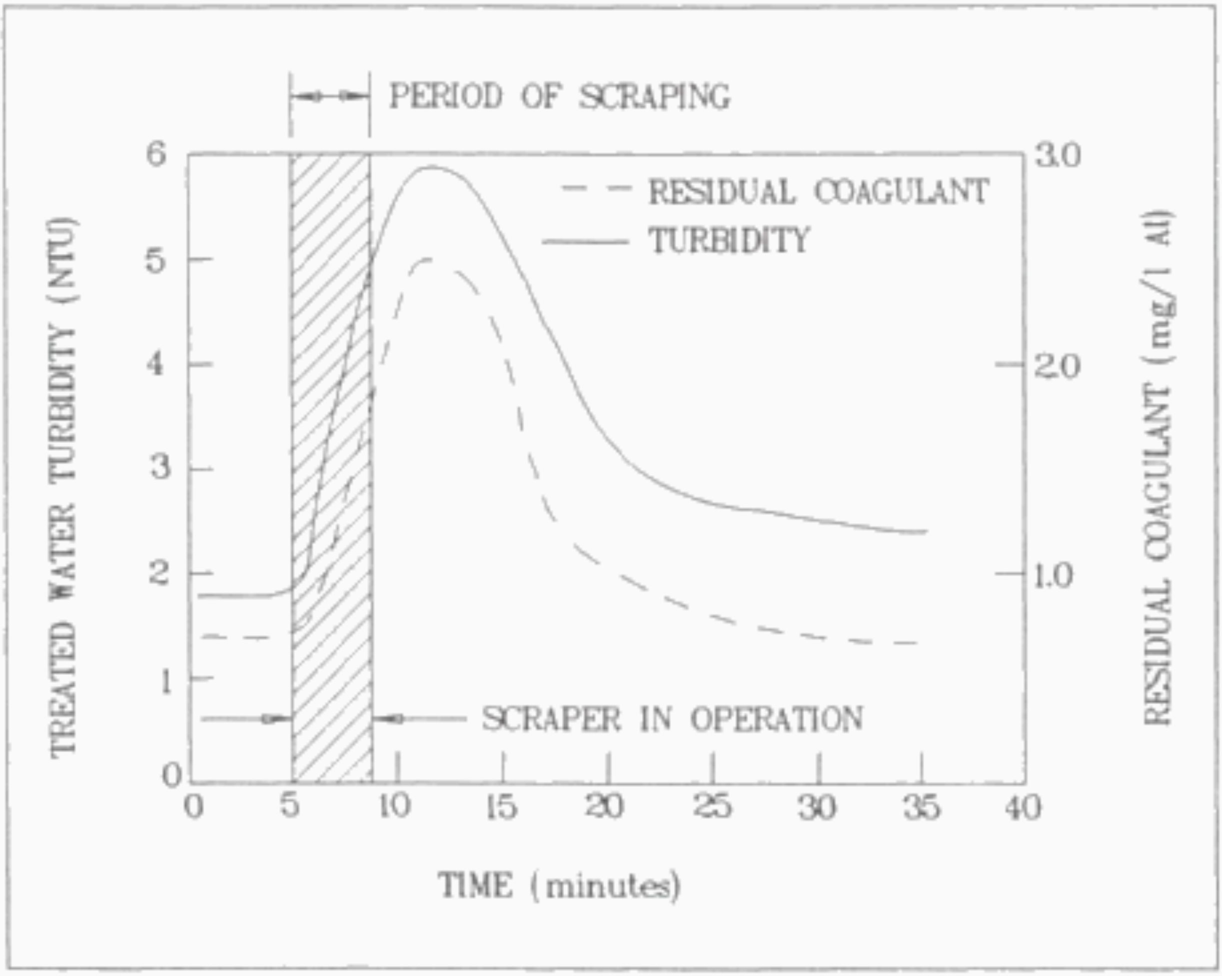


Figure 2.17 : Effect of desludging by intermittent scraping on treated water quality (from Zabel & Rees, 1976)

There is a danger with intermittent scraping if the float layer is left too long between scrapings. The water quality may deteriorate as the accumulation time increased. In one case, 1 hour was already too long (Stock, 1976). In the case of combined flotation/filtration, no effect on filtered water quality was found even after 40 hours (Rosen & Morse, 1976).

One way of diminishing the effect of scraping is to minimize the length of travel of the scraper blades. This is done with partial scraping, where the scraper blades do not travel the full length of the tank, but only a short length of the tank in the vicinity of the sludge trough. In one of the few comparisons between full versus partial scraping, it was shown that the success of partial scraping is largely dependent on the character of the float layer. In the case of turbid or algal waters, the float layer will remain stable for long periods and partial scraping will work very well. In the case of a soft, coloured water where the float layer starts to break up after 30 minutes, scraping has to be carried out over the full length to be successful (Zabel, 1978; Zabel, 1985).

Some references discuss the direction of scraping. The most common method is to scrape the float layer away from the inlet side of the tank, but some thickening cases are reported where the float is successfully scraped towards the inlet end. The motivation is that the float layer is always thickest nearest to the inlet end, and that the mean scraping distance will be least when the scraping direction is towards the inlet (Tavery, 1979; Conference on, 1977 : 157-165). The shorter the distance, the less chance of sludge break-off, settling and effluent deterioration. In one reported case (Ettelt, 1964), scraping towards the inlet end caused a SS_{out} of 1 000 $mg.\ell^{-1}$. When the direction of scraping was reversed, SS_{out} increased to 1 400 $mg.\ell^{-1}$.

The speed at which the scraper blades travel, has an effect on the float layer removal. For water of low turbidity and high colour, continuous scraping at low speed resulted in a float solids concentration of 1% and losses of 0.2%. At higher speed, the losses were higher and the float solids concentration lower (Zabel, 1978). A speed of 60 $m.h^{-1}$ and scraping depth of 5 mm caused no obvious turbulence (Langenegger & Viviers, 1978). For algal water, a scraping speed of 60 $m.h^{-1}$ is recommended (De Wet, 1980).

Float layer scraping typically produces sludge with concentration between 1% and 5%. Although higher levels have been reported, it is admitted that sludge with a concentration higher than about 5% becomes very difficult to remove and to handle (Grant & Williams, 1976).

Besides the general considerations discussed up to now, it is apparent that a great deal of the success of float removal hinges on the practical design details of the system. Practical recommendations include:

- The scraper should be rigidly fixed in a vertical position and should not be allowed to swing back (Komline, 1978).
- The blades should be specially shaped and spaced (no details given) and should cut 100 mm deep into the float layer (Schade, 1982).
- Float layer knockdown can be reduced by using nylon brushes in the place of rubber blades, which additionally produces a drier sludge through filtering (Longhurst & Graham, 1987).
- The water level in the tank should be controlled as low as possible at the bottom of the beach to attain the highest float solids concentration (Zabel, 1978).
- A horizontal section on the upstream end should form part of the sludge beach to trap the solids in front of the scraper, to prevent it being pushed down under the blade (Komline, 1978). This is demonstrated in Figure 2.18.

In summary, the designer of a successful float layer scraping system has to find a compromise between (Rovel, 1976):

- the largest possible accumulation of sludge at the surface to achieve a high float solids concentration;
- the removal of the float layer before a significant fraction of the entrapped air escapes, otherwise the partially de-aerated sludge will settle as large pieces as soon as they are sheared by the scraper blades; and
- a minimum cost solution, as the float layer scraping equipment typically contributes 10% to 20 % of the total capital cost of a DAF system (Zabel & Melbourne, 1980).

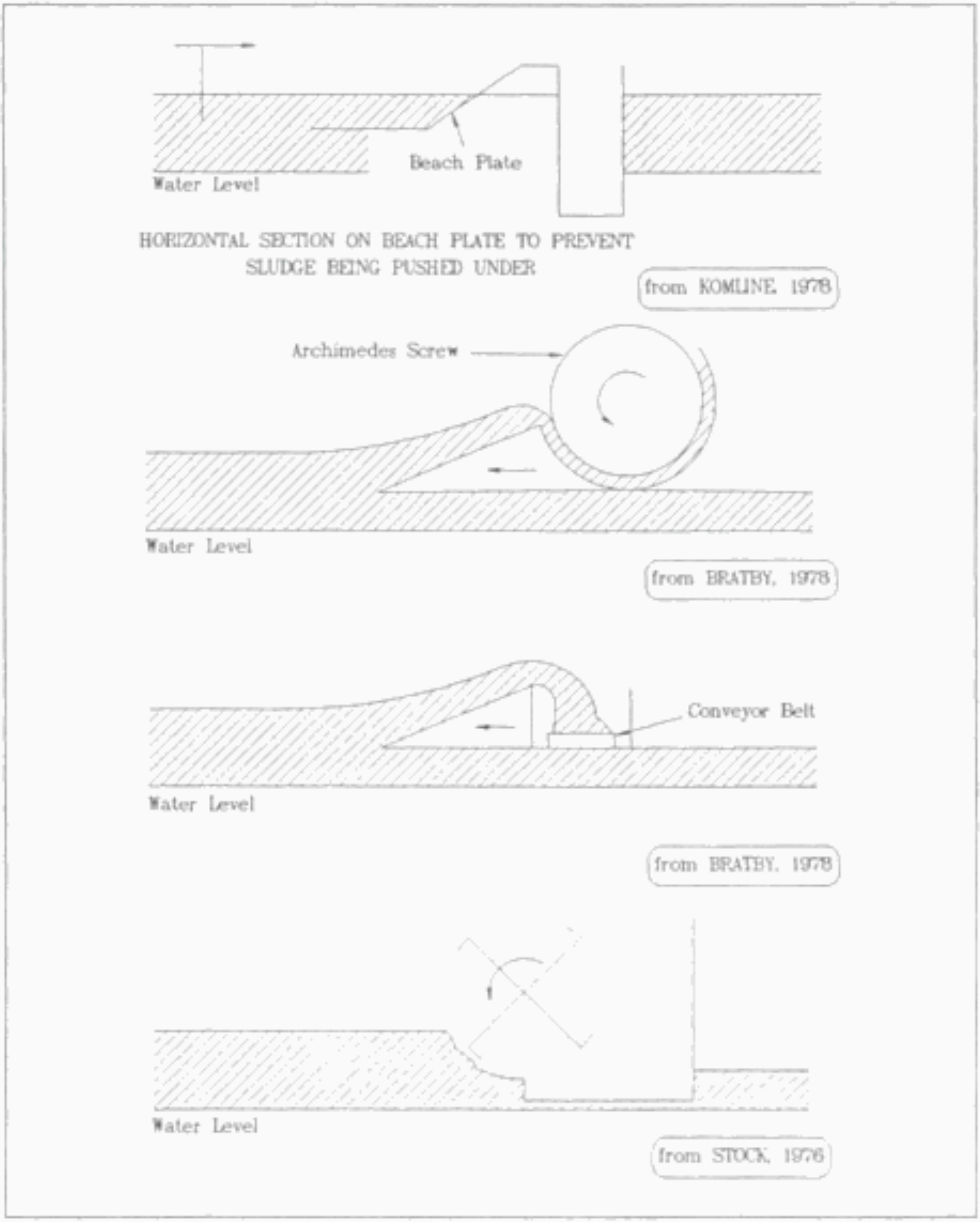


Figure 2.18 : Reported suggestions for improved float layer removal.

Float stabilizing grids

To prevent the disruption of the float layer during scraping and to prevent "rolling" of the float layer, a fixed grid is sometimes placed at the surface of the flotation tank. The float layer then has to "squeeze" through the grid first before the scraper skims the sludge from the upper surface of the grid. This practice is largely confined to clarification applications.

A prominent international manufacturer uses a sludge grid 100 mm deep, with the facility to adjust the water level in the tank relative to the grid to attain the desired float solids concentration. This level is generally found to be 5 to 10 mm below the upper surface of the grid (Conference on, 1977 : 68-76).

Experimentation with a 150 mm by 150 mm grid, 150 mm deep, on a highly eutrophic water source (Van Vuuren & Van der Merwe, 1988) has also established that the water level in the tank must be close to the upper surface of the grid. Moreover, it was found that this level must be kept constant to ensure float layer stability; under these circumstances stability was maintained for five days and C_s as high as 10%.

Care should be taken to design these grids with sufficient structural rigidity to prevent upward deflection when the highly buoyant float layer pushes the grid from below. In the case of a mechanically scraped tank with small tolerance between the grid and the scraper blades, this upward deflection might jam the scraping mechanism.

2.7 Combined flotation/filtration

For water treatment purposes, whether flotation or sedimentation is used, the water is inevitably filtered as a final polishing step. For clarification purposes, the flotation and filter functions are often combined into a single reactor with flotation being carried out on top of the filter bed. The process is commonly known in SA as the DAFF process (dissolved air flotation filtration) and is also known as a FLOFILTER in Europe and as the SANDFLOAT system in the USA. There was no difference between settled and floated water as far as its effect on the subsequent filtration step was concerned (Zabel & Hyde, 1976).

Advantages

The advantages of the combination over separate reactors are twofold. Firstly, there is the obvious cost saving by only having to build one structure instead of two. Secondly, a saving is achieved if the designer opts to leave out the float layer collection mechanism altogether. In this case, the float layer accumulates undisturbed during the filter run and is simply washed out at the end of the filter run with the filter washwater. Some designers of DAFF systems prefer to retain a separate float layer collection mechanism, probably for waters where the float layer does not remain stable for long periods.

DAFF is also used to upgrade conventional treatment plants which did not have flotation as a unit process before. This is a common problem in SA, where deterioration of raw water sources led to a steady decline in filter run lengths. In such cases, DAFF will extend the filter runs by capturing a fraction of the suspended solids in the float layer to decrease the solids loading on the filter bed. Monitoring of a full-scale DAFF plant indicated that about 80 % of the incoming solids are floated in a DAFF reactor, leaving 20% to be filtered and increasing filter in lengths by 63% (Haarhoff & Fouche, 1989). In other examples, settled water caused filter runs of

12 hours, while the addition of DAFF extended the filter runs to 48 hours (Van Vuuren *et al.*, 1983b), and the reduction of head loss after 10 hours from 320 mm to 30 mm by addition of DAFF (Van Vuuren *et al.*, 1983a).

Air binding and head loss development

A principal concern about DAFF is the possibility of drawing those minute air bubbles (with rise rate smaller than the filtration rate) into the sand bed where they will coalesce into larger bubbles which will ultimately block the interstitial pores. This phenomenon is known as air binding and will lead to a rapid increase in the head loss development during the filter run.

Extensive tests on eutrophic surface water showed that the DO concentration in water after flotation was 1 $mg.\ell^{-1}$ higher than the saturation value. During subsequent pressure filtration, no adverse effect on filtration was found, but the authors warned that "...DAF could possibly contribute towards air binding of rapid gravity filters..." (Williams *et al.*, 1985). Studies in the UK on five different pilot plants where flotation preceded filtration, confirmed this finding and concluded that filtration was unaffected by the presence of DAF (Zabel, 1978; Rees *et al.*, 1980b). In one set of experiments on a South African eutrophic source, air binding was experienced when the hydraulic loading exceeded 10 $m.h^{-1}$ (Offringa & Scheepers, 1985). Another study supported this finding, where no evidence of air binding was found below 10 $m.h^{-1}$, but where evidence of air binding at rates higher than 10 $m.h^{-1}$ was found (Vosloo *et al.*, 1985).

There is an indication that the degree of air binding may be dependent on the saturation pressure. During a previous experience by one of the authors (unpublished) with secondary sewage clarification, where fullstream saturation is done, subsequent filtration was ruled out because of the excessive air binding demonstrated during pilot trials. This could be ascribed to the fact that fullstream saturation requires much lower pressure and that the driving force for bubble precipitation was much less than for partstream saturation, causing less complete precipitation.

Interesting results were reported on tests with raw surface water as well as tap water (Van Vuuren *et al.*, 1983b). Water which was supersaturated and then left for a while to release the microbubbles, caused *less* head loss at the very high filtration rate of 47,1 $m.h^{-1}$ than at a normal rate of 5,7 $m.h^{-1}$, in direct conflict with the notion that higher rates will cause larger pressure drop and hence more air precipitation. The conclusion was reached that the retention time in the sand bed had a more important influence on air binding than the filtration rate. When the same experiment was repeated but with no time allowed for the microbubbles to escape first, the head loss increased sharply. This established the point that *direct introduction* of microbubbles is the principal cause for air binding, and not the precipitation of air from supersaturated solution. This points to the danger of producing too small bubbles for DAFF.

Where air binding does occur, the situation can be improved by deliberate air disengagement. In a reported case, this was achieved by only closing the filter outlet valve for 20 s every 1 h. The stop in flow decreased the pressure within the bed just enough to allow accumulated air to escape. With this procedure, the head loss after 30 h was decreased from 290 mm without air disengagement to 90 mm with air disengagement (Van Vuuren *et al.*, 1983).

The head loss development is, of course, also a function of many other parameters. Pilot-scale experimentation with an algal water/oxidation pond blend (Van Vuuren *et al.*, 1983a) indicated the following contributing factors:

- The addition of polymer increased the head loss. Without polymer, the head loss was four times lower than with a polymer.
- The flocculation period is shorter when using polymer. As polymer flocculation was extended, the head loss development decreased. After 8 h, for example, a flocculation period of 25 s caused 280 mm head loss, while the head loss with 200 s and 400 s flocculation was respectively 210 mm and 30 mm.
- With higher recirculation (which means higher air dosage), the head loss was lower. No explanation was advanced for this finding.

The stability of the float layer should also be kept in mind when analysing head loss data from DAFF experiments. In a reported case (Van Vuuren *et al.*, 1983b), the head loss increased abruptly after 45 hours, which was traced to the float layer becoming too thick and unstable after this period.

It is suggested that finer sand should be used for filtering floated water than for settled water, because the suspended material in floated water is generally lower (Rosen & Morse, 1976). In a subsequent discussion, this suggestion was disputed (Conference on, 1977 : 407-416).

CHAPTER THREE

HISTORICAL DEVELOPMENT OF FLOTATION TECHNOLOGY IN SOUTHERN AFRICA

3.1 General

The historical development of flotation technology in Southern Africa can best be reviewed under the main applications rather than a concise chronological sequence of events. These applications can be broadly categorised under :

- Water reclamation
- Sludge thickening
- Clarification of eutrophic waters
- Treatment of industrial effluents.

The initial investigations in the early sixties were mainly focused on the reclamation of sewage effluents for potable and industrial use with secondary attention to industrial effluents. Since the mid seventies the thickening of waste sewage sludges were studied more fundamentally while a few commercial plants were already in operation at the time.

Eutrophic water treatment only became of higher priority during the late seventies, initially for upgrading of existing conventional plants and more recently also for the design of new plants. Under the auspices of the Water Research Commission several industrial effluent types of diverse quality were extensively studied using flotation technology.

The objective of this chapter is to give the reader a broader perspective of local developments in this relatively new field of technology and its great potential for application in the water industry. Most of these developments are published in the literature, of which a bibliography is given in Appendix C.

3.2 Water reclamation

Prior to the sixties, air flotation technology in Southern Africa was mainly utilized for ore beneficiation in the metallurgical industry. This technology remained unexploited for water and effluent treatment. Overseas literature available at the time was rather limited and essentially concerned with research on sludge thickening and industrial waste treatment. For practical scale water clarification, flotation technology lagged behind.

During the early sixties the National Institute for Water Research (NIWR) (now WATERTEK) of the CSIR, in collaboration with the City of Windhoek, investigated the feasibility of reclaiming sewage effluent for potable reuse (Van Vuuren *et al.*, 1965). Conventional biofilter effluent after maturation ponds served as raw water intake for laboratory and pilot investigations. During laboratory tests the phenomenon of poor settleability of flocculated algae consistently proved to be a serious practical problem. Flocculated algae behaved unpredictably in terms of retarded settling and partial flotation. This erratic behaviour was ascribed to low density of algae flocs and the presence of supersaturated oxygen due to photosynthesis.

A simple cascading (pouring) test was devised in the laboratory by which virtually complete flotation of alum/algae floc was achieved using the following procedure:

> The optimum alum dosage was added to each of two empty 1 ℓ measuring cylinders. Maturation pond effluent was introduced in the first (control) cylinder with vigorous stirring followed by gentle stirring for about 5 minutes. In the second cylinder the effluent was poured from a height of about 400 mm thus introducing some air, and violently stirred for about 10 seconds. After a few minutes' standing the results were vastly different in that partial settling occurred in the control cylinder with complete flotation of flocs in the second cylinder, as shown in Figure 3.1.

It was also apparent that non-biodegradable detergents (ABS) which was present at about 1 to 2 mg.ℓ^{-1} appeared to play a supporting role in bubble attachment to the flocculated particles. The cascade method of aeration was subsequently simulated and successfully implemented on pilot scale at Windhoek (see Figure 3.2). Historically, these studies earmarked the beginning of flotation technology for water clarification purposes in Southern Africa (Van Vuuren *et al.*, 1965).

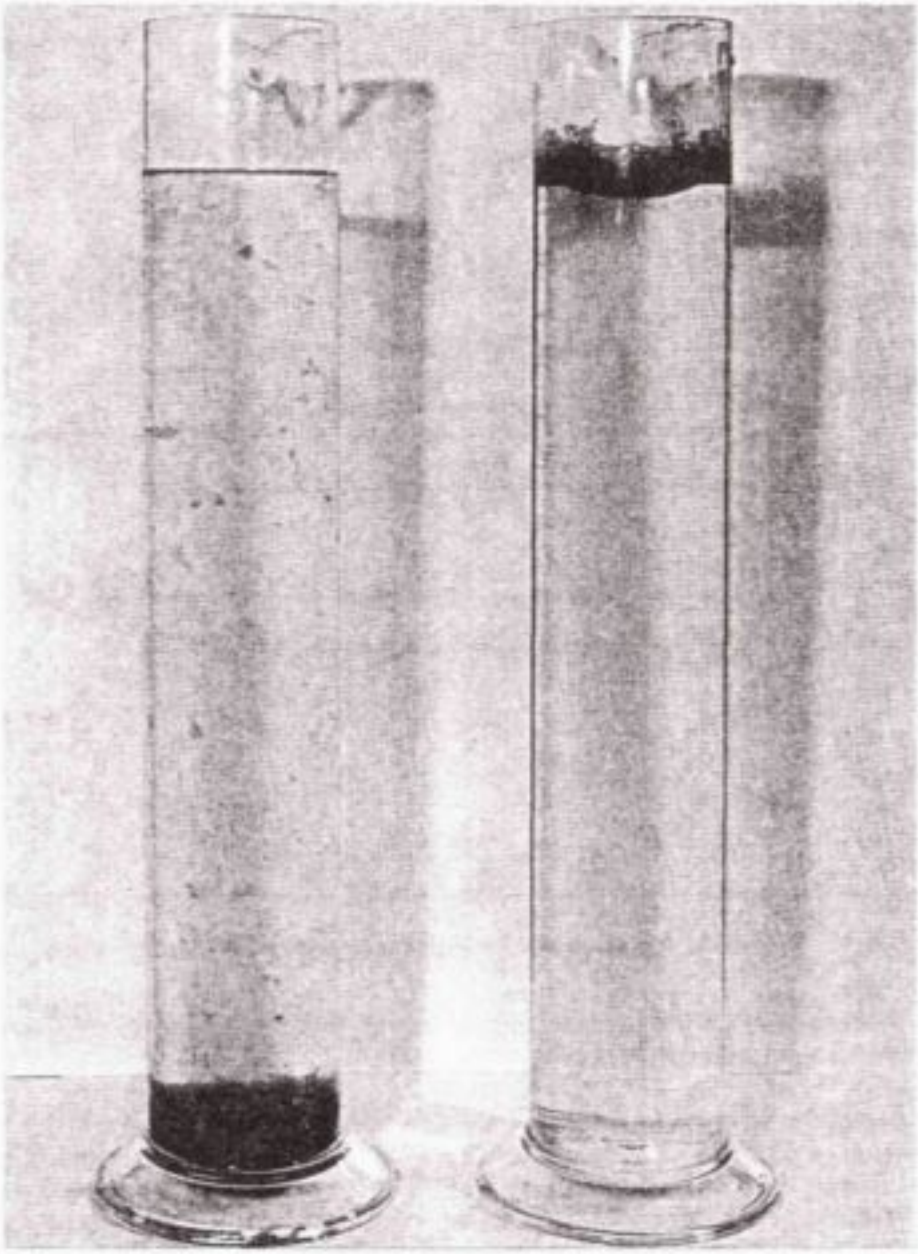


Figure 3.1 : Demonstration of flotation induced by air addition through pouring, compared with a vigorously stirred control without any air addition.

Continued pilot-scale studies

The above success stimulated further development work on flotation tank design, initially using dispersed aeration techniques and subsequently high pressure dissolved aeration methods. These studies were conducted at the CSIR Experimental Site at the Daspoort Sewage Works in Pretoria during the late sixties to early seventies. Different effluent types such as derived from

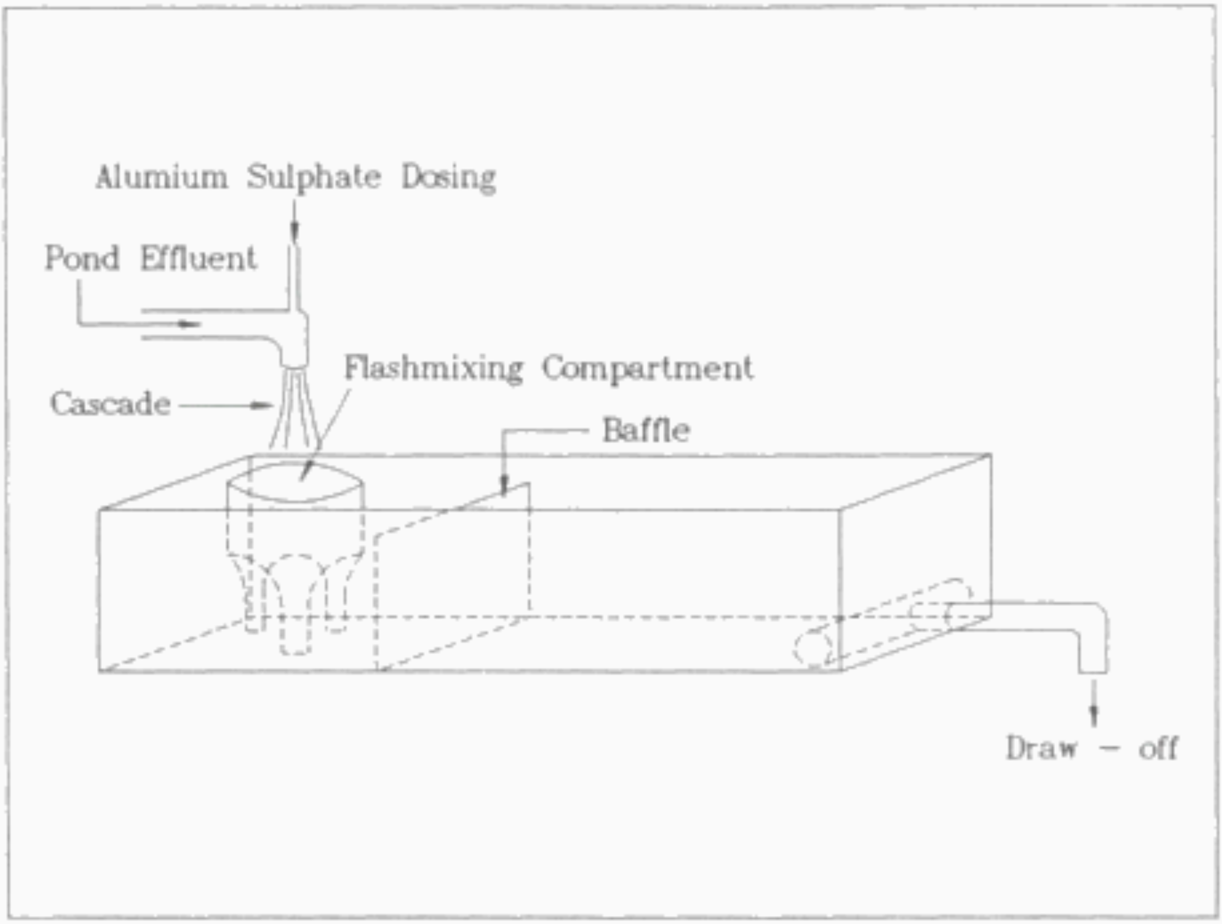


Figure 3.2 : Schematic layout of pilot flotation system with cascade aeration.

maturation/oxidation ponds, humus tanks, raw and settled sewage were tested using various coagulants, including excess lime treatment

During the early stages dispersed aeration was used, by drawing air into pump suction pipes or air eductors in a full stream aeration mode. Particular attention was given to vertical and radial draw-off tanks treating alum-flocculated maturation pond and secondary effluents from humus tanks. (See Figures 3.3 and 3.4 for the radial and vertical flow type pilot plants used). For algal separation, dispersed aeration proved to be successful when supersaturated dissolved oxygen levels were prevalent. For humus tank effluent, reasonable success was achieved with dispersed aeration but superior results required high pressure aeration with part-stream recycle

The indications were also that flotation technology had great potential for the separation of low density suspended solids of organic nature. A portable laboratory flotation unit based on full stream dispersed aeration was constructed during the early seventies for testing various types of industrial effluents derived from fish factories, meat processing plants, and paper and pulp manufacturing (Van Vuuren *et al.*, 1970) (see Figures 3.5 (a) and 3.5 (b))

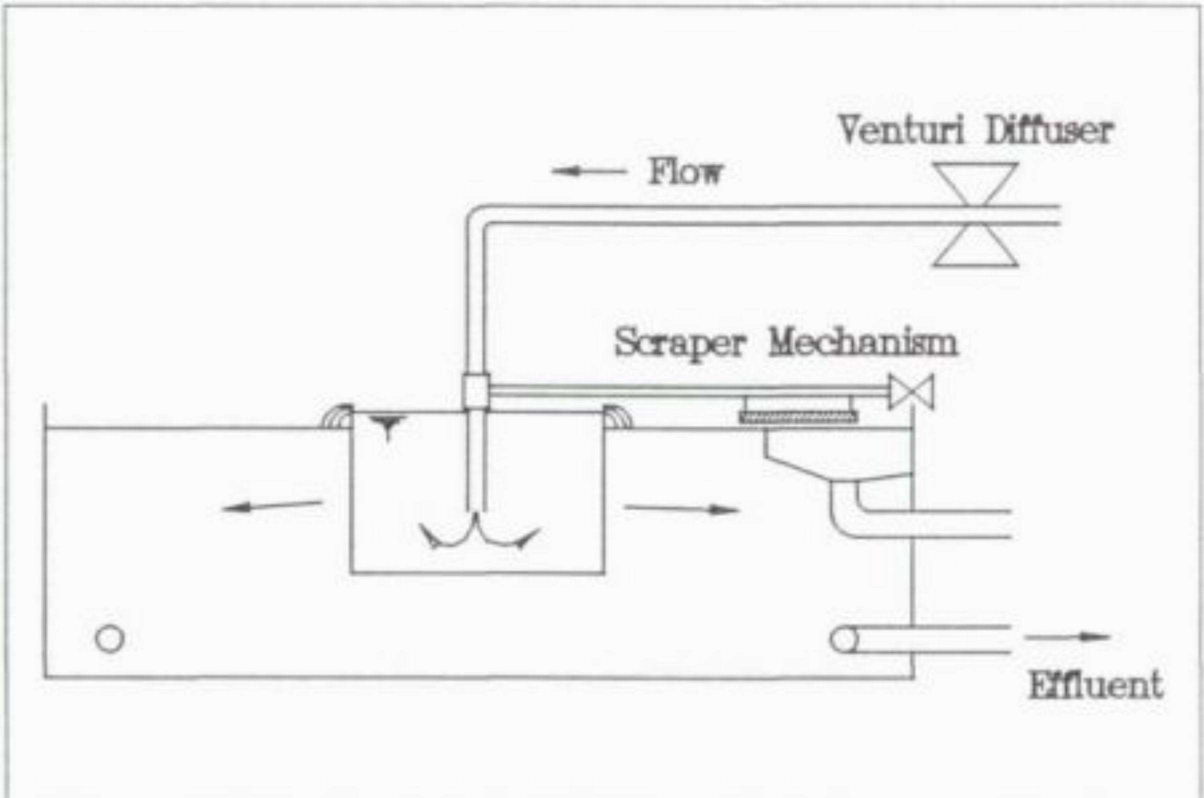


Figure 3.3 : Cross-section of radial-flow flotation tank used in pilot studies.

During parallel tests utilising high lime treatment circular tanks with various geometric arrangements for outflow and desludging were used (Van Vuuren *et al.*, 1970; Van Vuuren *et al.* 1967; Van Vuuren & Van Duuren, 1965). Pilot units for excess or high lime treatment of humus tank effluent and raw (or settled) sewage respectively are shown in Figures 3.6 and 3.7, based on full-stream dispersed aeration.

Ongoing investigations included the direct reclamation of raw or settled sewage. The so-called LFB-process (Integrated Lime Flotation Biological) was extensively studied on pilot scale (Van Vuuren & Ross, 1975) utilising flotation as the primary stage followed by further biological and physical chemical treatment stages.

First full-scale applications

A full-scale reclamation plant of 4.5 $M\ell.d^{-1}$, was commissioned at Windhoek towards the end of 1969 utilising the off-gases from methane combustion as a means of full stream dispersed aeration. The carbon dioxide generated also served the purpose of lowering the pH in order to facilitate alum flocculation. Flotation performance was highly satisfactory and formed an important pretreatment stage of the overall reclamation process (Van Vuuren *et al.*, 1970).

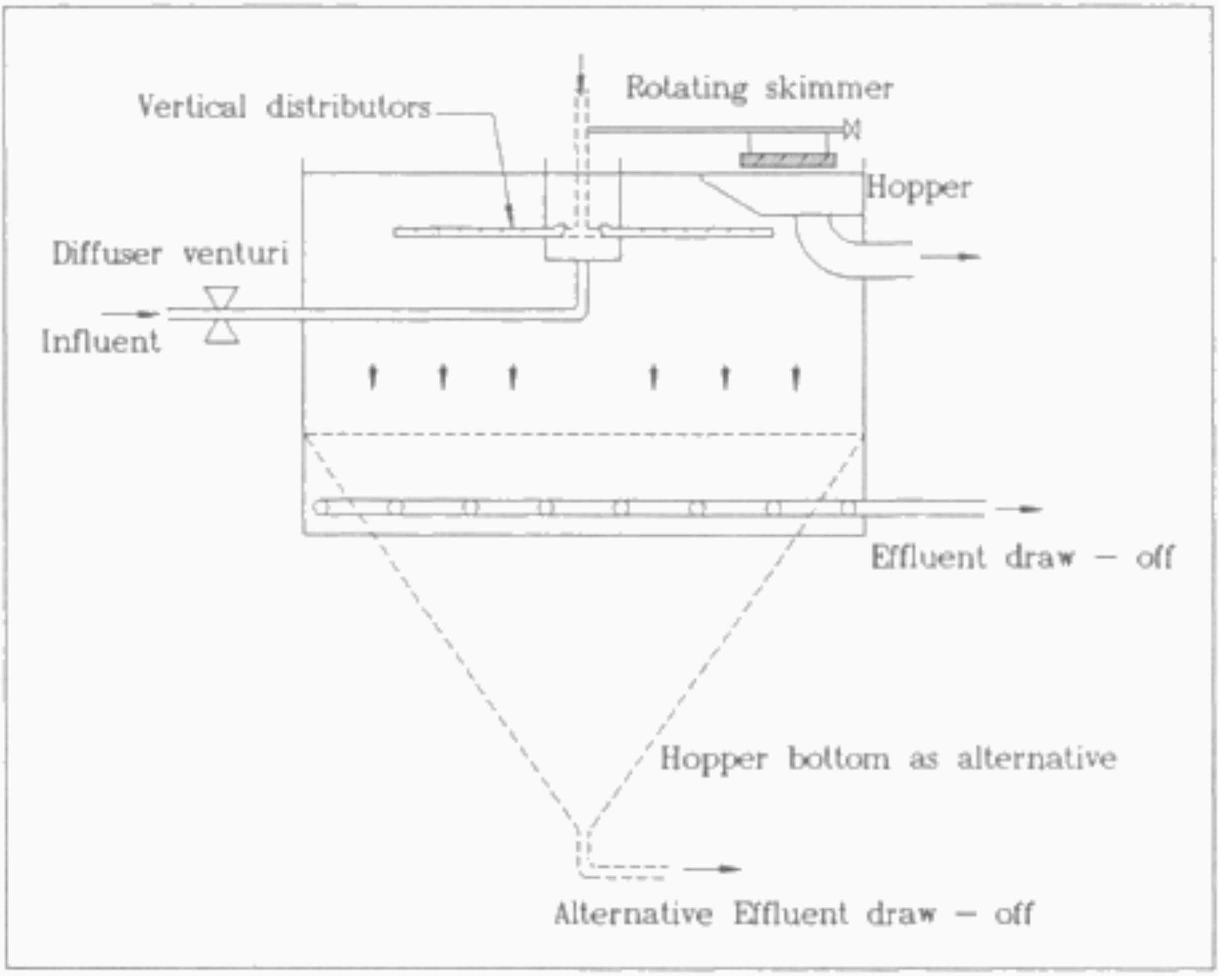


Figure 3.4 : Cross-section of vertical-flow flotation tank used in pilot studies. Tank could be used with flat or hoppered bottom.

A major problem encountered in the reclamation process was the occurrence of high ammonia levels derived from conventional biofiltration during the winter months in particular, which required uneconomically high chlorine dosages for effective disinfection. This problem necessitated the need for introducing high lime treatment and ammonia stripping for which the plant was subsequently modified during the mid-seventies. The excellent settling properties of magnesium hydroxide and calcium carbonate/algae flocs then obviated the need for the flotation stage. A solids contact clarifier was duly installed and the flotation unit bypassed. When the Windhoek sewage works was later extended with activated sludge facilities, the original flotation tank was upgraded and re-commissioned using alum flocculation and high pressure part-stream aeration, which is still used at present (1992). This plant is currently being extended threefold with continued use of DAF being planned. The current flotation system is included with the plants surveyed for this report.

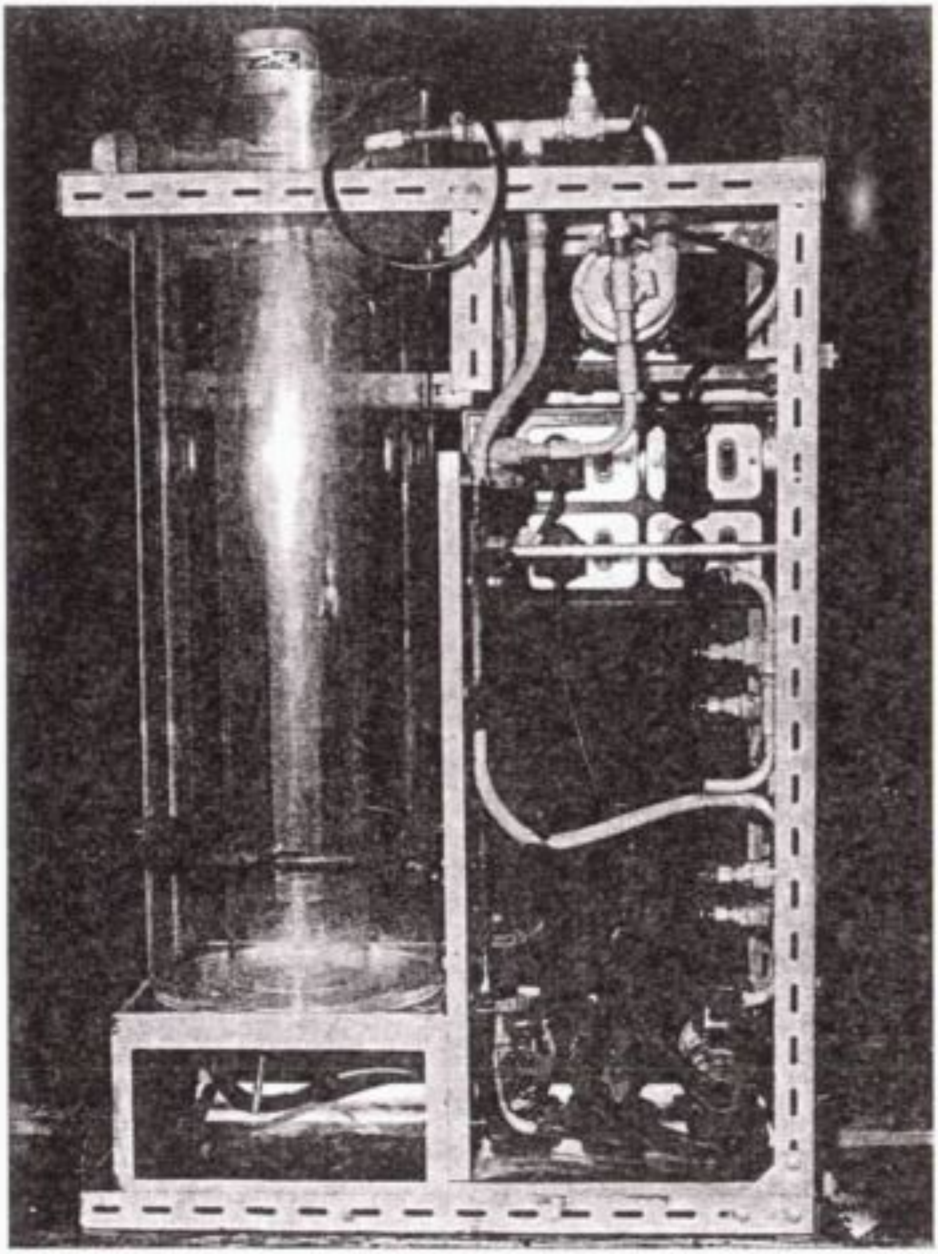
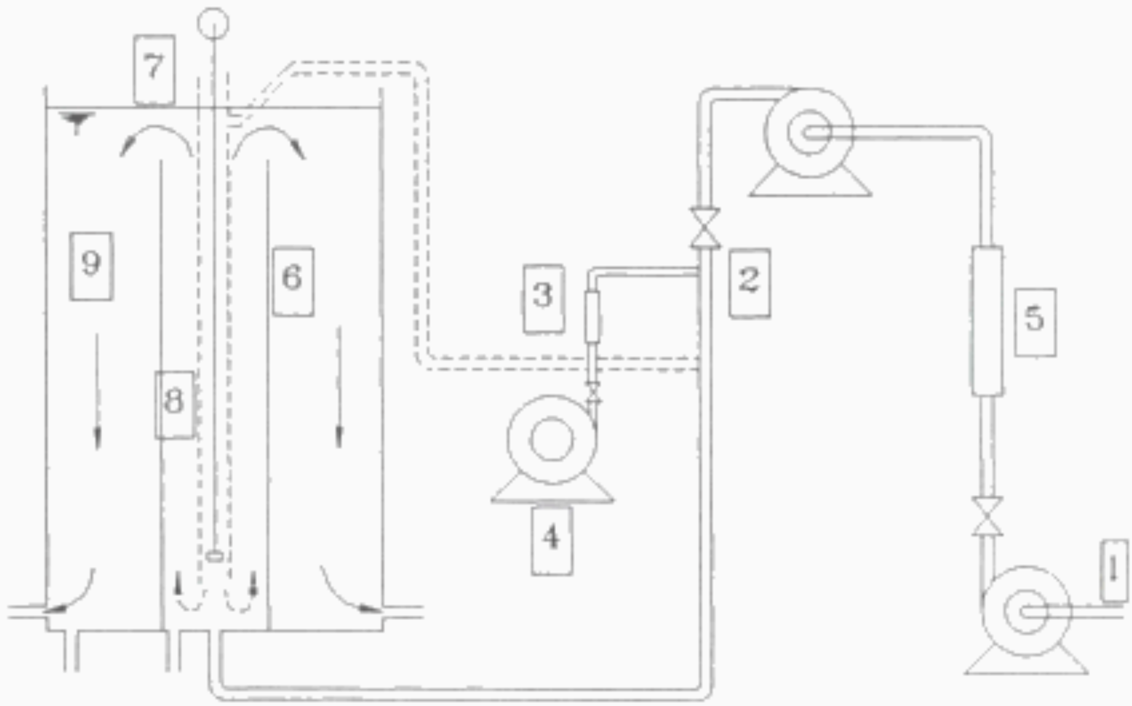


Figure 3.5(a) : Photograph of portable flotation unit with dispersed aeration

Stander water reclamation plant

Concomitant with the Windhoek studies, a research/demonstration plant of 4,5 $M\ell.d^{-1}$ was commissioned during 1970 at Daspoort for reclaiming secondary humus tank effluent. This plant was originally designed for full stream dispersed aeration flotation following high lime treatment but it was converted to two-stage settling with inter-stage recarbonation because of the relatively slow rate of calcium carbonate precipitation which escaped flotation capture. Criteria for high



1 Raw water pipeline
2 Coagulant dosing point
3 Rotameter for coagulant
4 Raw water pump
5 Rotameter for raw water
6 In – line flocculation pipe system
7 Alternative flocculated raw water inlet points
8 Reactor zone of flotation tank
9 Clarification zone of flotation tank

Figure 3.5(b) Schematic section of portable flotation unit shown in Figure 3.5(a)

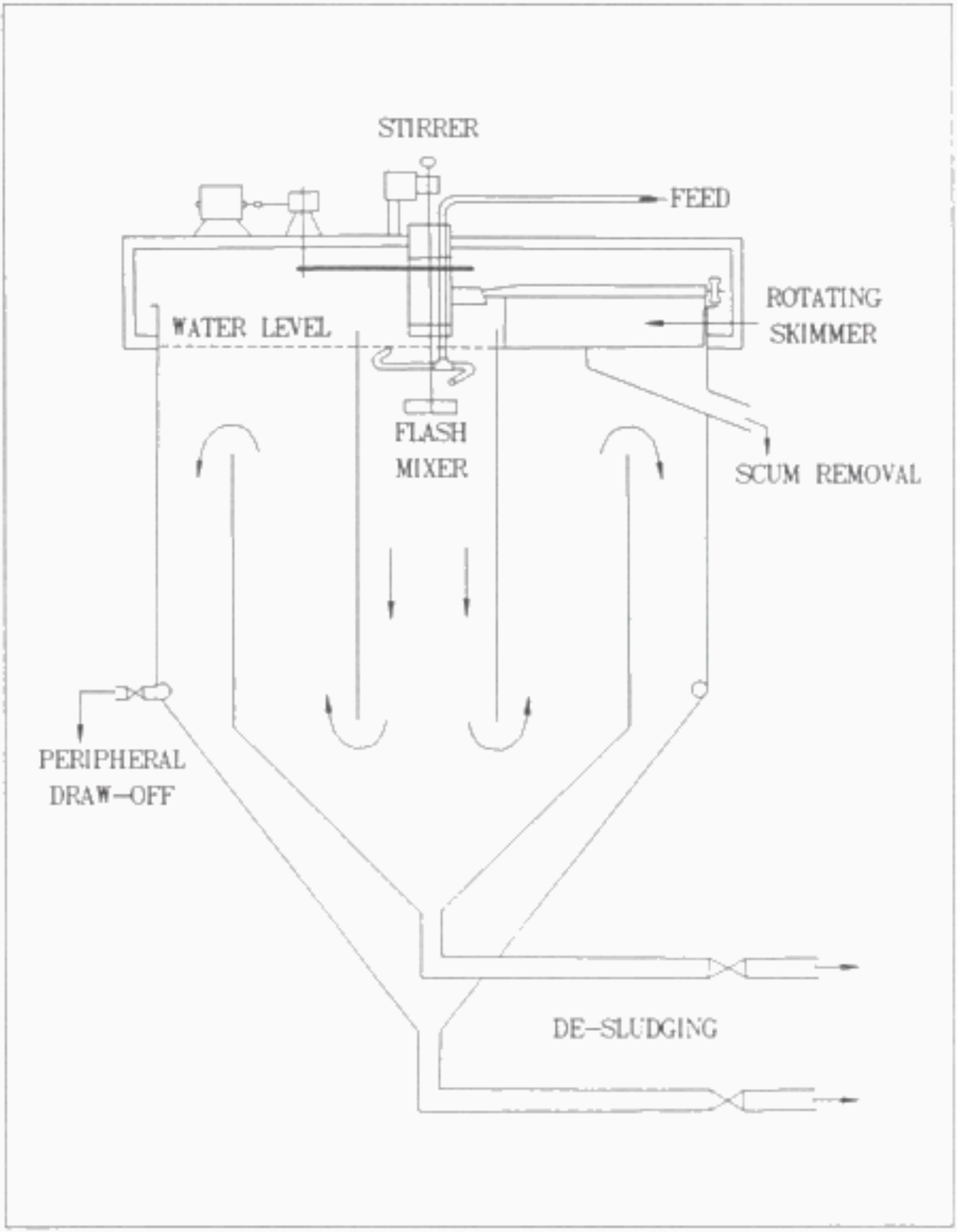


Figure 3.6 Schematic section of pilot flotation unit with full-stream aeration

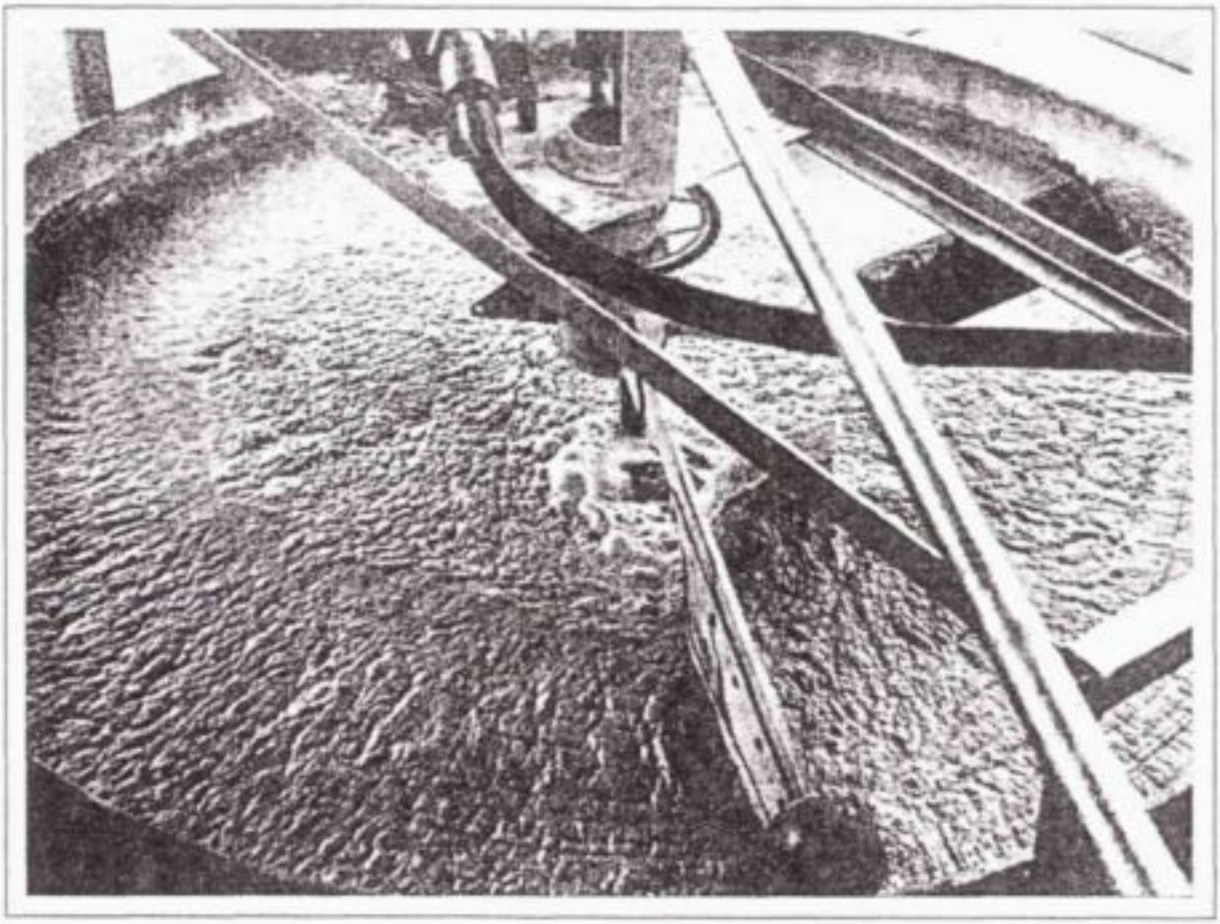


Figure 3.7 : Photograph of working pilot flotation unit with full-stream aeration.

lime treatment with two-stage settling and ammonia stripping which were derived from these investigations were implemented in the modified Windhoek reclamation plant as described above (Van Vuuren *et al.*, 1980).

Fine paper production

Following extensive pilot-scale studies during 1970, a full-scale DAF plant of 25 $M\ell.d^{-1}$ capacity was commissioned during 1971 to reclaim secondary humus tank effluent as process water (Van Vuuren *et al.*, 1972; Jacobs & Smith, 1973). An important feature of this plant was the method of full-stream aeration at moderately low pressure. A great deal of success was reported over a period of 20 years on the production of a high quality paper. The current flotation system is included with the plants surveyed for this report.

Paper mill effluent

During 1973 the laboratory DAF unit shown in Figure 3.5 was used to investigate a grossly polluted effluent from a waste-paper mill. The objectives were to meet effluent quality requirements and to avoid high tariffs as imposed by the local authority for discharge of effluent into the sewerage system. The on-site investigations of only two weeks provided sufficient criteria for the design of a 4 $M\ell.d^{-1}$ DAF plant which currently (1992) is still operative. The existing flotation plant is included with the plants surveyed for this report. A spin-off is the recovery of waste pulp and partial recycle of reclaimed effluent for pulp production (Coertze, 1978). Although this application is essentially an industrial one it is included here because of the re-use connotation.

Water reclamation for mining purposes

During a drought period (early seventies) a major gold mining company in collaboration with NIWR investigated the reuse of sewage effluent for underground applications (De Wet & Van Vuuren, 1980). A DAF plant for treating a flow of 6 $M\ell.d^{-1}$ was successfully commissioned during 1985. This plant is presented with the case studies included in this manual (Leach *et al.*, 1985).

3.3 Sludge thickening

The initial developments described were mainly conducted under the auspices of the NIWR and were essentially directed to *clarification* as a pre-treatment stage for the reclamation of sewage effluent. For sludge *thickening* purposes the initiative for research came from university researchers, consultants and municipalities. Interest was particularly stimulated by the advance of biological nutrient removal technology in South Africa during the seventies (WRC, 1984). The prime purpose was the thickening of phosphate enriched waste activated sludge under aerobic conditions in order to minimize phosphate release experienced under anaerobic conditions. Research by Bratby & Marais and Langenegger are particularly commendable in regard to pilot-scale studies on DAF thickening (Langenegger & Viviers, 1978; Langenegger, 1985). The work by Bratby and Marais was extensively published in the literature (See Appendix C). Their fundamental process variables and proposed design formulations gave great impetus to local development of DAF technology. Some of their most relevant criteria are included in this manual (also see Chapters 2 and 6).

Several full-scale plants were commissioned at various localities since the mid-seventies with a few already at an earlier stage. Design of these plants were based on diverse criteria from local research, and published literature and commercial expertise (Davis, 1977; Offringa, 1986). Most of these plants are included with the case studies reported in this manual. In this regard engineers of the Johannesburg municipality played an important role to implement DAF thickening at several major nutrient removal sewage plants (Davis, 1977; Howell, 1982). Progressively it was also implemented on large scale at other new plants near Cape Town (Anon, 1983) and Pretoria (Everett, 1983).

In comparison with clarification, DAF thickening criteria are evidently of a more complex nature which account for variable measures of success achieved with these plants. Parameters such as air/solids ratio, float thickness, solids loading rates, sludge characteristics, *et cetera* are of different magnitude than for clarification applications. It is apparent that appreciable scope remains to consolidate the diverse criteria for DAF thickening of sludges - which is one of the main objectives for this manual.

3.4 Eutrophic water treatment

DAF applications for clarification of surface waters only gained momentum during the late seventies; the reason being the increased eutrophication of several major impoundments and associated problems encountered with conventional treatment such as short filter runs, taste and odour and THM formation.

Several existing plants were upgraded and new plants constructed during the eighties all with a high degree of success. A number of these flotation systems are included with the plants surveyed for this report. In several cases these designs were preceded by pilot-scale investigations (Figure 3.8), but criteria for design were mainly based on overseas literature sources. In this regard the valuable work extensively published by the Water Research Centre in the UK is particularly noteworthy (Zabel, 1978).

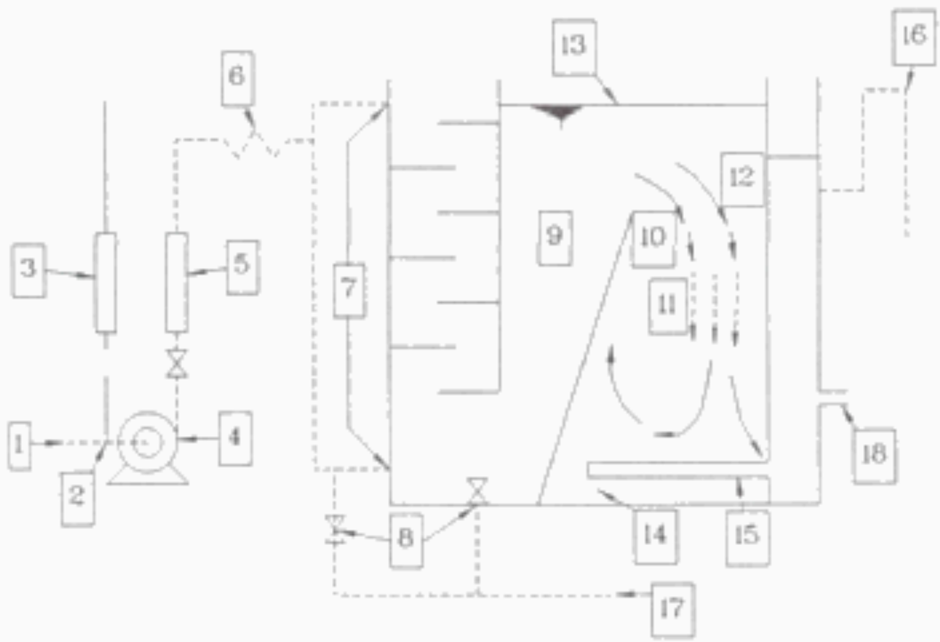
A large-scale DAF plant with nominal capacity of 150 $M\ell.d^{-1}$ was commissioned during the early eighties. This plant was designed to treat eutrophic water from a lake in the Northern Natal coastal region to high quality requirements for use mainly in a fine paper mill (Bernstein *et al.*, 1985). Because of unprecedented flood conditions provision had to be made for high rate pre-settling. These studies showed that DAF had its limitations when inorganic turbidities exceeded a critical value, in this instance about 80 NTU. The experience gained was that seasonal fluctuations in eutrophic water quality with sporadic influx of turbid storm water need to be taken into account during the design of DAF facilities for particular catchments in Southern Africa (Botes & Van Vuuren 1990).

The problem of trihalomethane (THM) formation resulting from chlorination of organically polluted waters received a great deal of research attention from local researchers - particularly under the auspices of the NIWR. The question of pre-chlorination of algal waters as against DAF removal of algae followed by chlorination was addressed. Exploratory tests by Pilkington & Van Vuuren (1981) in this regard endorsed the results of a more detailed investigation by Gehr and Henry (1980) on the value of DAF treatment in this connection, while another study by Gehr *et al.*, (1992) showed that DAF could also be optimized to remove THM precursors from a highly eutrophic water source.

Other NIWR researchers investigated the co-addition of powdered activated carbon in relation to DAF on THM removal (Le Roux & Van der Walt, 1991). In general terms these studies supported the beneficial use of DAF for treatment of eutrophic water sources with the view of minimizing THM formation.

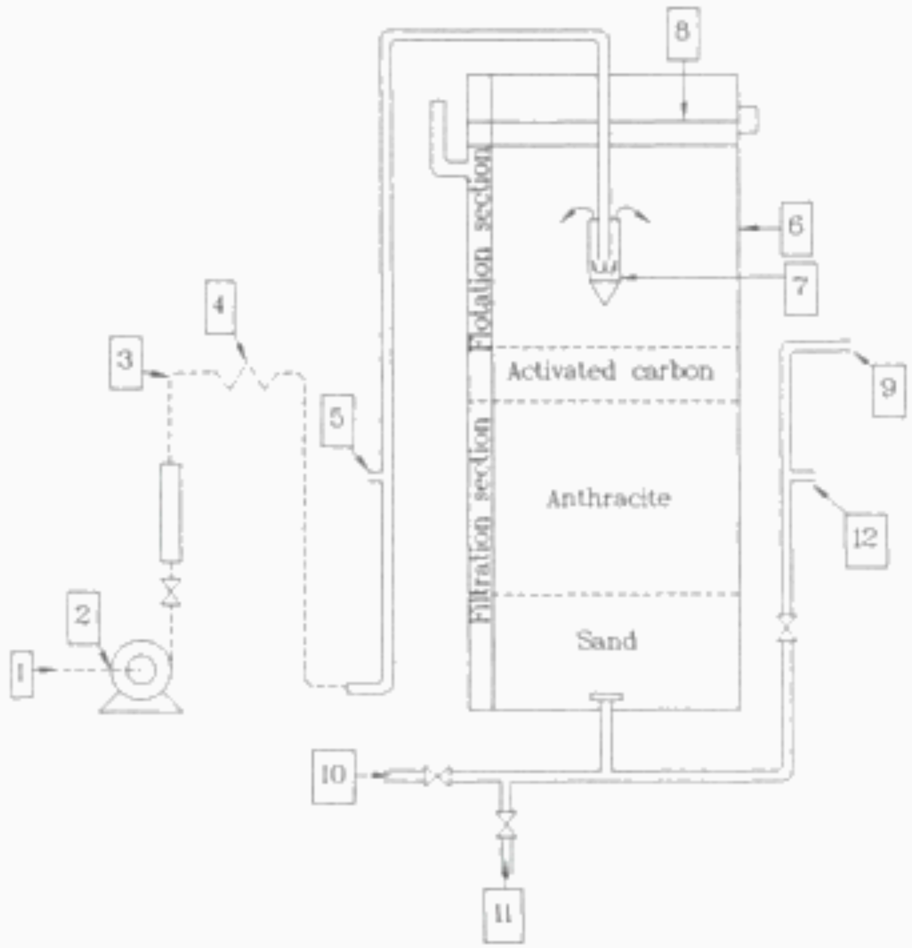
Combined flotation/filtration (DAFF)

During 1978 researchers of the CSIR demonstrated the feasibility of combined flotation/filtration (DAFF) in the laboratory (Van Vuuren *et al.*, 1983a) (see Figure 3.9). A mobile DAFF plant, shown in Figure 3.10, was designed and tested at several localities. The process was subsequently tested on full scale at Richards Bay and more recently practically implemented on full scale at the upgraded Rietvlei water treatment plant near Pretoria (Haarhoff & Fouche, 1989). Both these flotation systems are included with the plants surveyed for this report. Several other DAFF trials were launched during recent years at major supply sources where seasonal eutrophic conditions occur. When high inorganic turbidity prevailed it was evident that DAFF had its limitations in terms of current available expertise.



1 Raw water pipeline
2 Coagulant dosing point
3 Rotameter for coagulant
4 Raw water pump
5 Rotameter for raw water
6 In – line flocculation pipe system
7 Alternative flocculated raw water inlet points
8 Alternative saturated recycle flow inlet needle valves
9 Reactor zone of flotation tank
10 Clarification zone of flotation tank
11 Flow patterns with pipe collector system
12 Flow patterns without pipe collector system
13 Float layer
14 Flotation tank
15 Pipe collector system with orifices
16 Adjustable clarified effluent outlet
17 Saturator feed
18 Saturator draw – off

Figure 3.8 : Schematic section of portable laboratory unit for dissolved air flotation.



1 Raw water pipeline
2 Coagulant dosing point
3 Polyelectrolyte dosing point
4 In – line flocculation pipe system
5 Saturator feed
6 Flotation – filtration tank
7 Circular baffle
8 Float layer
9 Filtered effluent outlet
10 Inlet for backwash water
11 Inlet for scouring air
12 Saturator draw–off

Figure 3.9 : Schematic section of portable laboratory unit for combined dissolved air flotation and filtration.

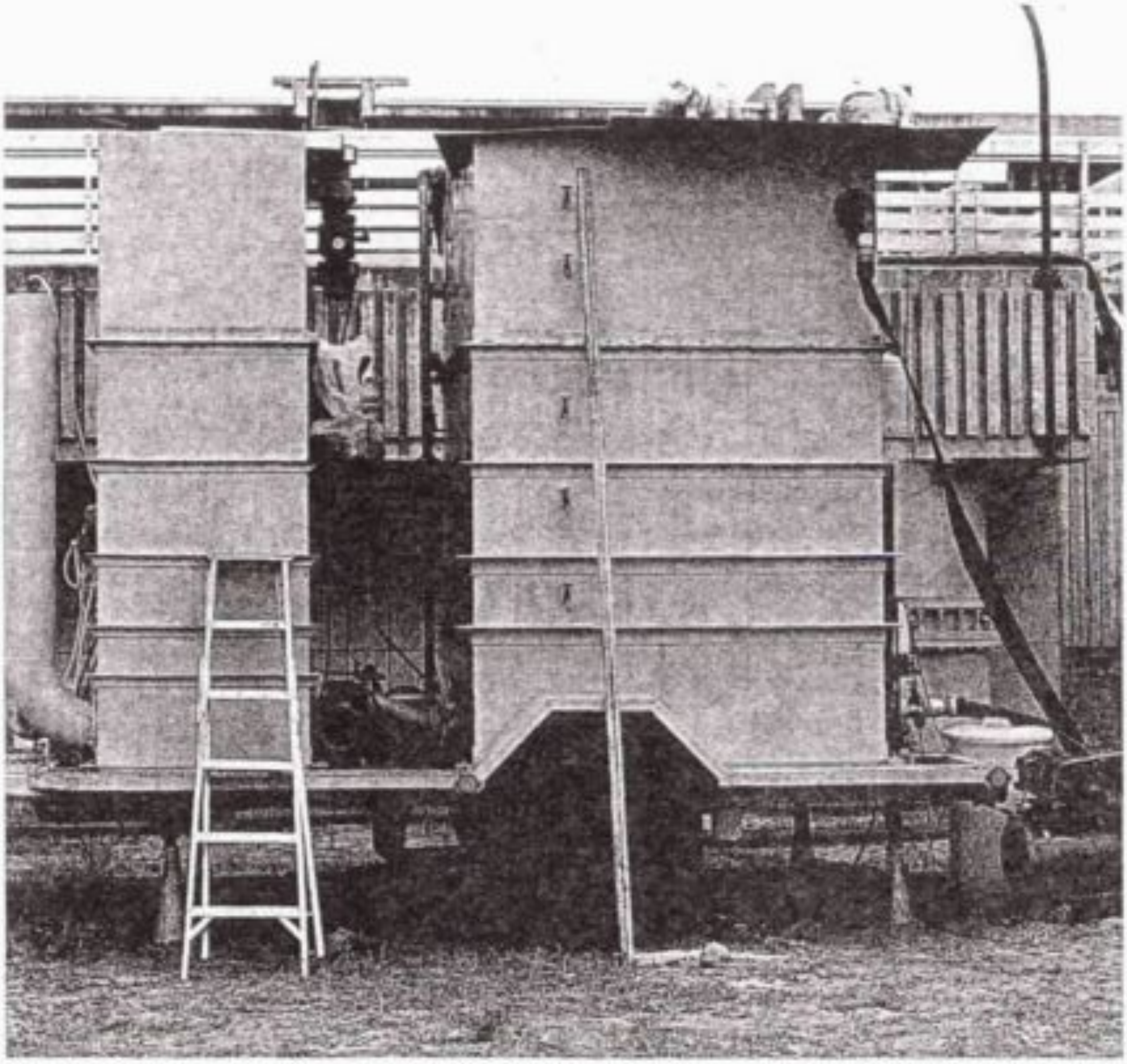


Figure 3.10 : Photograph of portable dissolved air flotation/filtration pilot plant used at many locations in South Africa

Air injection nozzle

The secured production of micro air bubbles from pressurised recycle streams posed to be a problem. Using needle valves for pilot-scale applications gave reasonably satisfactory results. For larger scale applications however the results were rather erratic in terms of sporadic formation of large air bubbles. During the early eighties researchers from the NIWR developed an aeration nozzle based on a 'cup' design, shown in Figure 3.11, which greatly improved micro bubble generation (Van Vuuren *et al.*, 1985). Several modified configurations of this basic 'cup' design have subsequently been developed and successfully implemented by local commercial firms. Research for further refinement is currently undertaken by the Rand Afrikaans University under the auspices of the Water Research Commission.

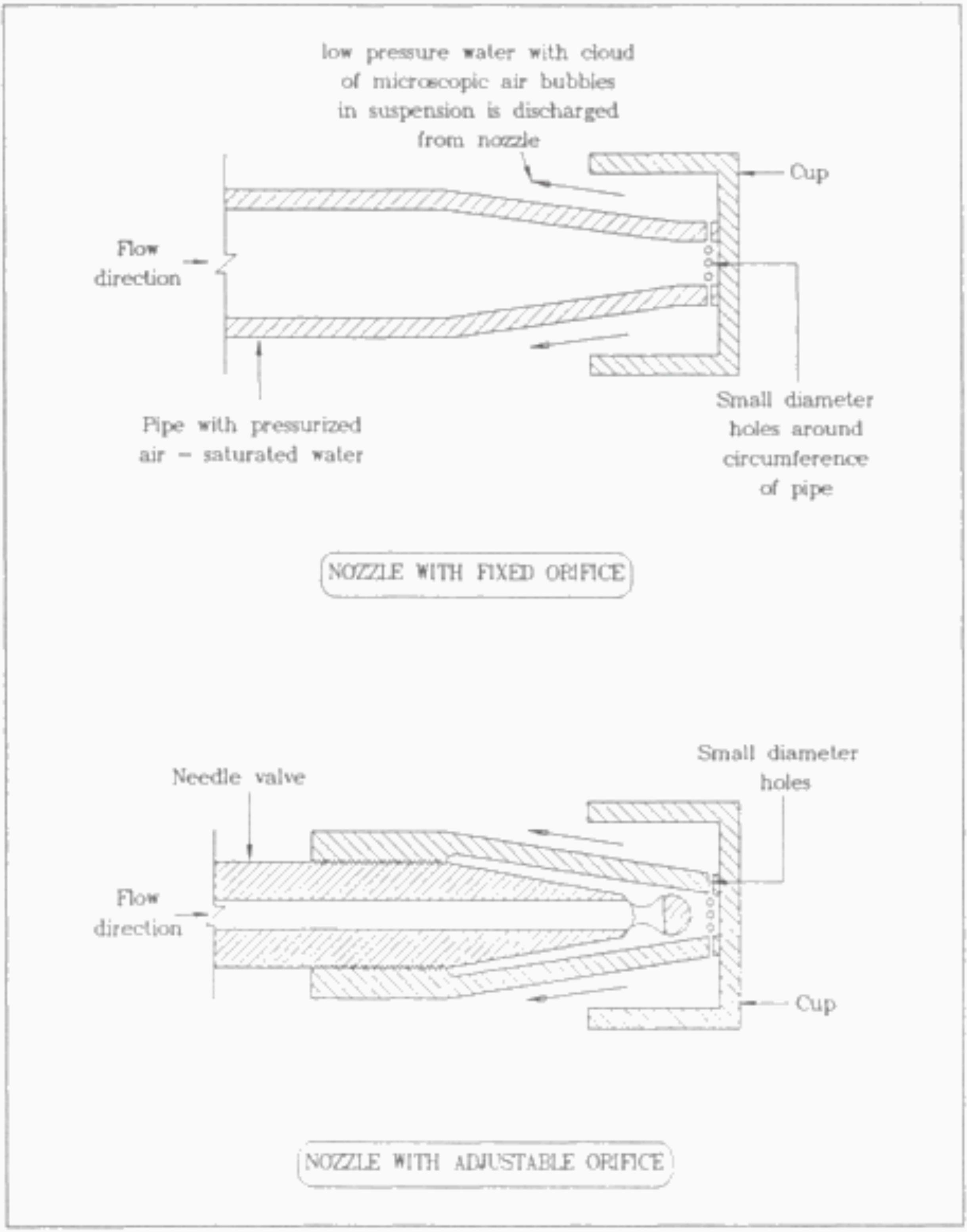


Figure 3.11 : Practical South African injection nozzles developed by the NIWR

Aquaculture

The protein value of algae and the successful separation of algae by flotation methods stimulated special interest locally for harvesting of algae for aquacultural and feedstock purposes. During the mid-seventies, a private South African company, AECI, studied DAF technology to harvest algae as a method for disposal of nutrient enriched effluents via maturation ponds (Sandbank, 1983).

At a later stage the NIWR also pursued this objective using an autoflotation process whereby supersaturated photosynthetic oxygen with polymer addition served as an *in situ* aeration flocculation method (Sandbank & van Vuuren, 1983). Although these pilot studies were generally quite successful the presence of chemical flocculant residues in the harvested product and the competitive market on protein production restricted commercial exploitation.

3.5 Industrial effluents

During the initial investigations as mentioned under 3.1, exploratory tests confirmed the beneficial use of flotation technology for treating various types of organically polluted industrial effluents. At a later stage, under the auspices of the Water Research Commission, several studies were conducted (NATSURV) on flotation treatment of effluents derived from tanneries, meat and vegetable processing etc. Several full-scale applications emanated from these studies.

For tannery wastes the SILFLO-process was developed by local commercial expertise (Roets & Heunis, 1984) and was marketed for both local and overseas use.

Other local applications such as for petroleum refinery wastes and car washing *et cetera* can be briefly mentioned as indicative of the scope for flotation treatment of industrial effluents. The use of aeration techniques such as Induced Air Flotation (IAF) has found beneficial use in some of the industrial applications and design criteria for these flotation systems are not included in this manual. However, empirical design projections from the DAF thickening and clarification criteria presented in this manual may also be applicable to some industrial effluents. A greater need for pilot investigations, however, appears to be warranted in this field.

3.6 Conclusions

Flotation technology has progressively been implemented for diverse applications in the local water and effluent industry during the past three decades. A great deal of success was achieved in water reclamation, eutrophic water treatment, industrial wastes and also with thickening of sewage sludges. The pioneering work by NIWR at Windhoek greatly stimulated research and development of DAF technology. In addition, many design engineers and researchers from various institutions have made significant contributions in promoting flotation technology in Southern Africa during past and recent years. DAF technology is still relatively novel and further refinement and research remain a challenge for the future.

CHAPTER FOUR

ANALYSIS OF SOUTHERN AFRICAN DISSOLVED AIR FLOTATION PLANTS

4.1 General

During 1990 and 1991 a total of 26 dissolved air flotation plants in Southern Africa were visited. Many more plants could have been included in the survey, but the plants that are included represent a fair sampling of the available plants in terms of age, application and size. Written consent was obtained from each authority that it would participate in such a survey and that generalized results may be published in this report.

A comprehensive questionnaire was designed and refined after trial visits to a few plants. These questionnaires were sent beforehand to participating authorities in preparation of the site visits. Most site visits were conducted by the two authors in the company of a site staff member well acquainted with the operation of the flotation plant. After the site inspection, the questionnaire was completed to the satisfaction of all parties. Some queries that emerged during the data analysis phase were referred back to site for verification. After the documentation of each plant was completed it was returned to the participating authorities for final verification and approval. This information is contained in Appendix B.

The primary data upon which this chapter is based, is contained in Appendix B for the benefit of the interested reader and are not repeated again. In this chapter the emphasis is upon the critical comparison of the plant data to determine any trends or patterns that may be evident in Southern African dissolved air flotation plants.

When compiling operational data from full-scale plants, a discrepancy is usually found between the design flow rates and the actual flow rates. Almost all plants, during the first few years of operation, inevitably run below design capacity. Moreover, a plant seldom runs at a smooth, even rate; it more likely follows some diurnal or seasonal pattern. Throughout this survey, two values for each variable are presented. The first is the *design* value, which is included if it could be obtained and the second is the *actual* value, which is taken as the average value during the time of the visit.

Figure 4.1 shows the commissioning dates of the plants included in this survey. It shows that South Africa's experience with DAF dates back more than 20 years - there is, in fact, at least one plant which precedes the earliest one included in this survey. It further shows that the practical implementation of DAF really took off in the early eighties, after and during the period when pioneering DAF research was done at the National Institute of Water Research (now WATERTEK) and the University of Cape Town. The apparent tailing off of construction activity is due to the fact that the plants included in this survey have been selected in 1989. A number of new plants that came into operation since are not included in this survey.

> In this survey, the clarification plants are numbered from # 1 to # 14, and the thickening plants from #21 to #32

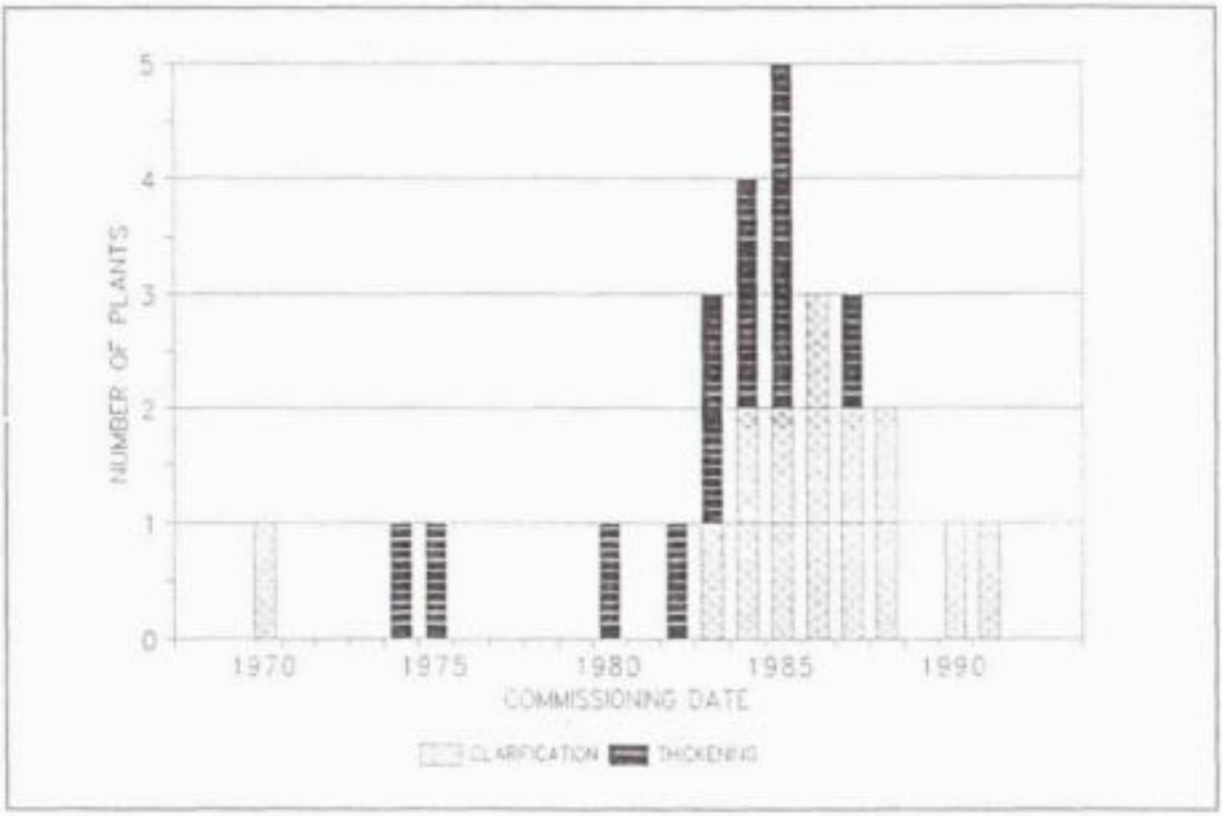


Figure 4.1 : Commissioning dates of the plants included in this survey.

4.2 Reaction zone

The literature survey in Chapter Two indicated the importance of the reaction zone to the DAF process. Practical experience in South Africa provides further evidence of this. During the survey, the authors came across several plants that did not perform satisfactorily upon commissioning. In all the cases, the problem was traced to an improperly designed reaction zone, which could be rectified easily.

The reaction zone layout of each surveyed plant is shown in Appendix B. The reaction zones could be grouped into six broad categories, which are indicated in Figure 4.2. In some cases, there is a clear distinction between the reaction zone and flotation zone. In some cases, this distinction was quite arbitrary and idealized - see Plant #4 as an example. In a few cases, however, the boundary between reaction and flotation zones were indeterminate - see Plants #12 and #30 as examples. Figure 4.2 indicates that there is no standardized South African approach to the design of the reaction zone; there is approximately an even mix of the six basic types indicated.

It was pointed out earlier that a principal difficulty of reaction zone evaluation lies in quantifying the intensity and duration of the turbulence in the reaction zone. For this survey, the mean velocity (hydraulic loading) at the maximum cross-sectional area of the reaction zone was used as an indicator of reaction zone turbulence - this is shown in Figure 4.3 (the reaction zones of Plants #12, #13 and #30 are indeterminate).

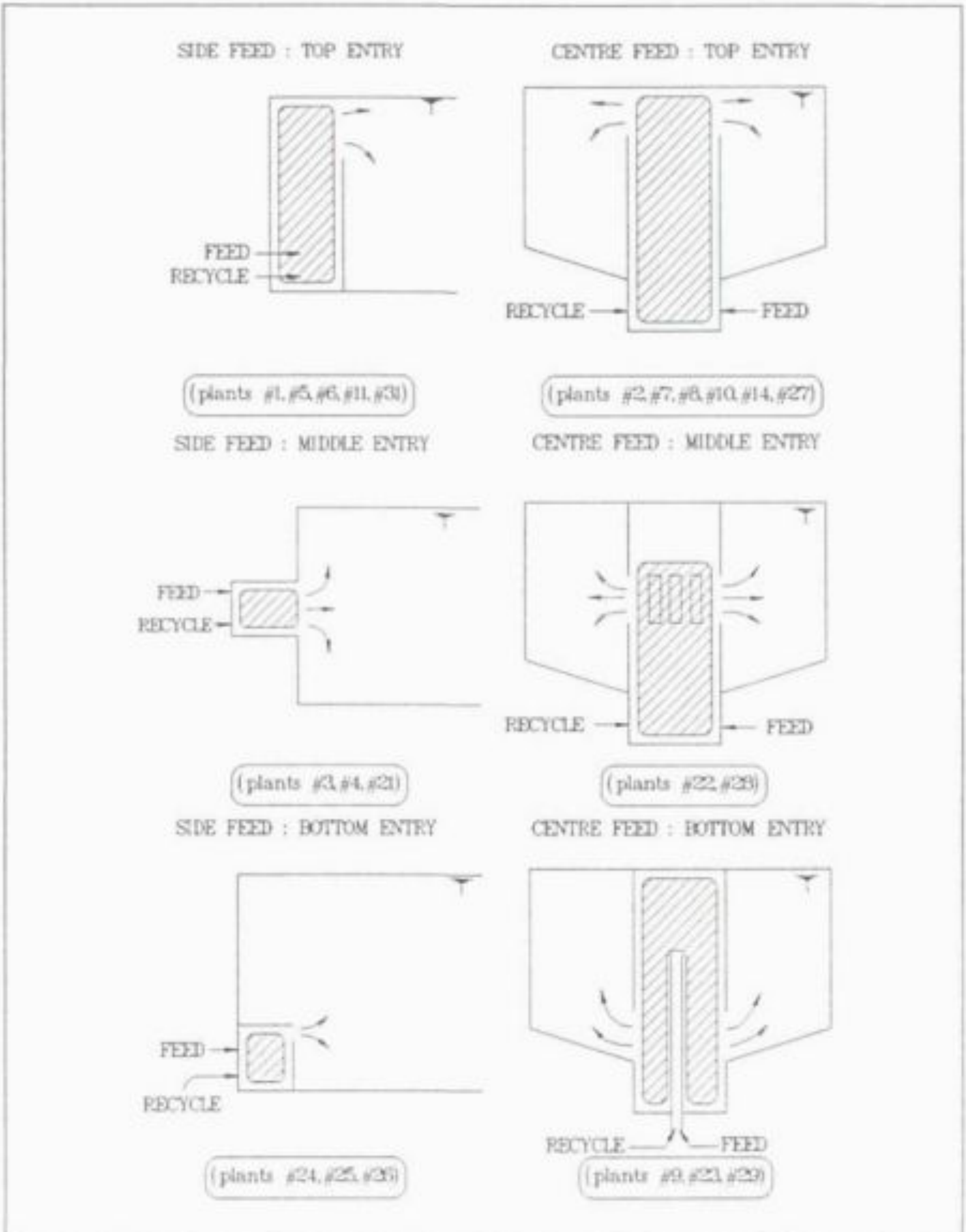


Figure 4.2 : Reaction zone configurations of surveyed plants.

For the clarification plants, the design values lie mostly in an envelope between 50 and 120 $m.h^{-1}$, with the exception of two plants that have higher values. The plant with the highest value, incidentally, is one of the plants which initially experienced difficulties with its reaction zone. For the thickening plants, there is a much wider variation from less than 10 to 1 000 $m.h^{-1}$. It is evident that there is no discernible trend - it appears as if this parameter was not always considered during the design of these thickening plants.

The mean retention time of the water in the reaction zone was used as an indicator of the duration of the mixing in the reaction zone. These values are shown in Figure 4.4 (the missing values are indeterminate as before).

Once again, a fairly clear envelope of 1 to 4 minutes is evident for clarification plants, but the retention time for thickening plants shows an almost random scatter from 0.3 to more than 20 minutes. This apparently is not a parameter that was always considered during the design of these thickening plants.

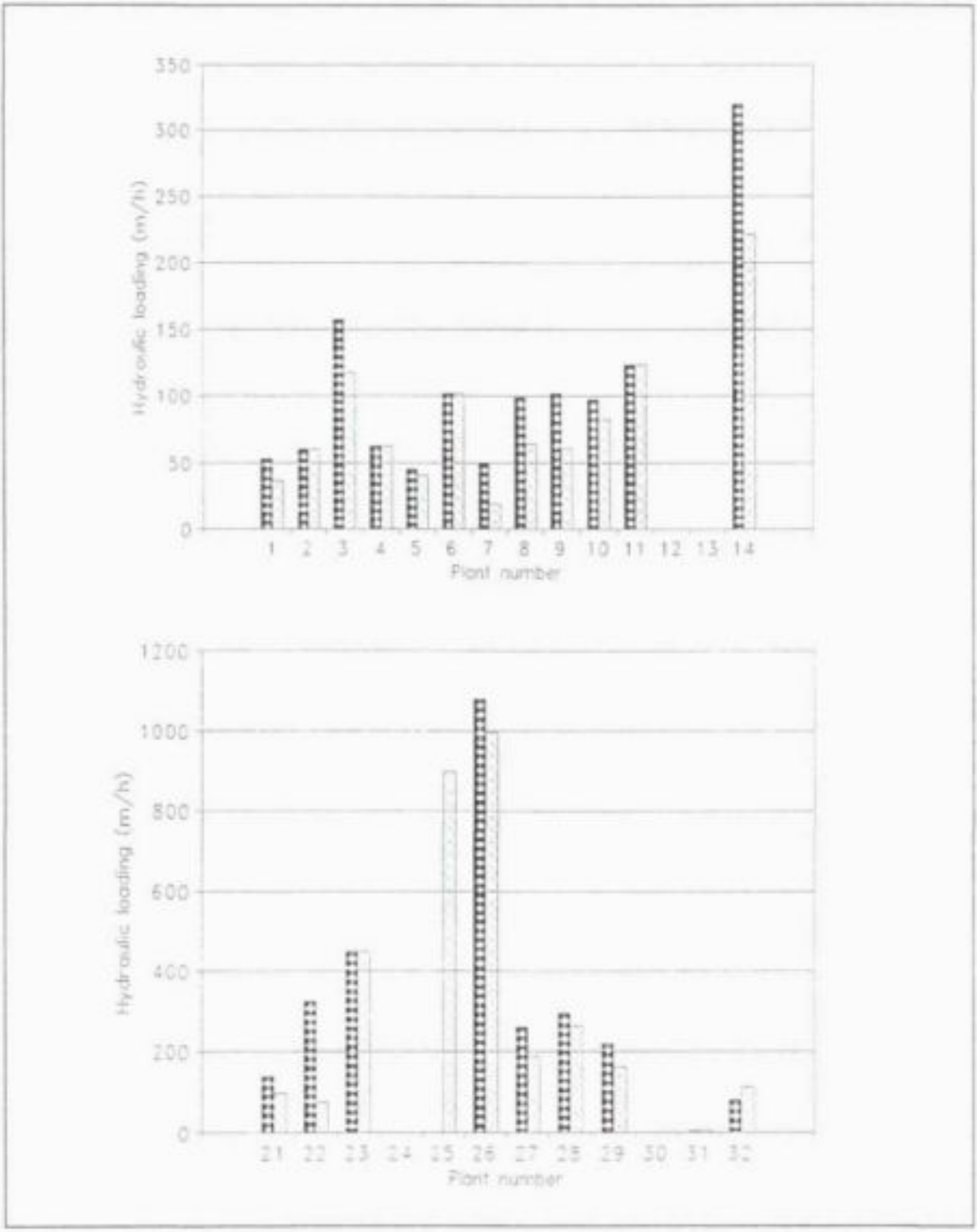


Figure 4.3 : Reaction zone hydraulic loading for (a) clarification and (b) thickening plants. Dark bars denote design values; light bars denote actual average values.

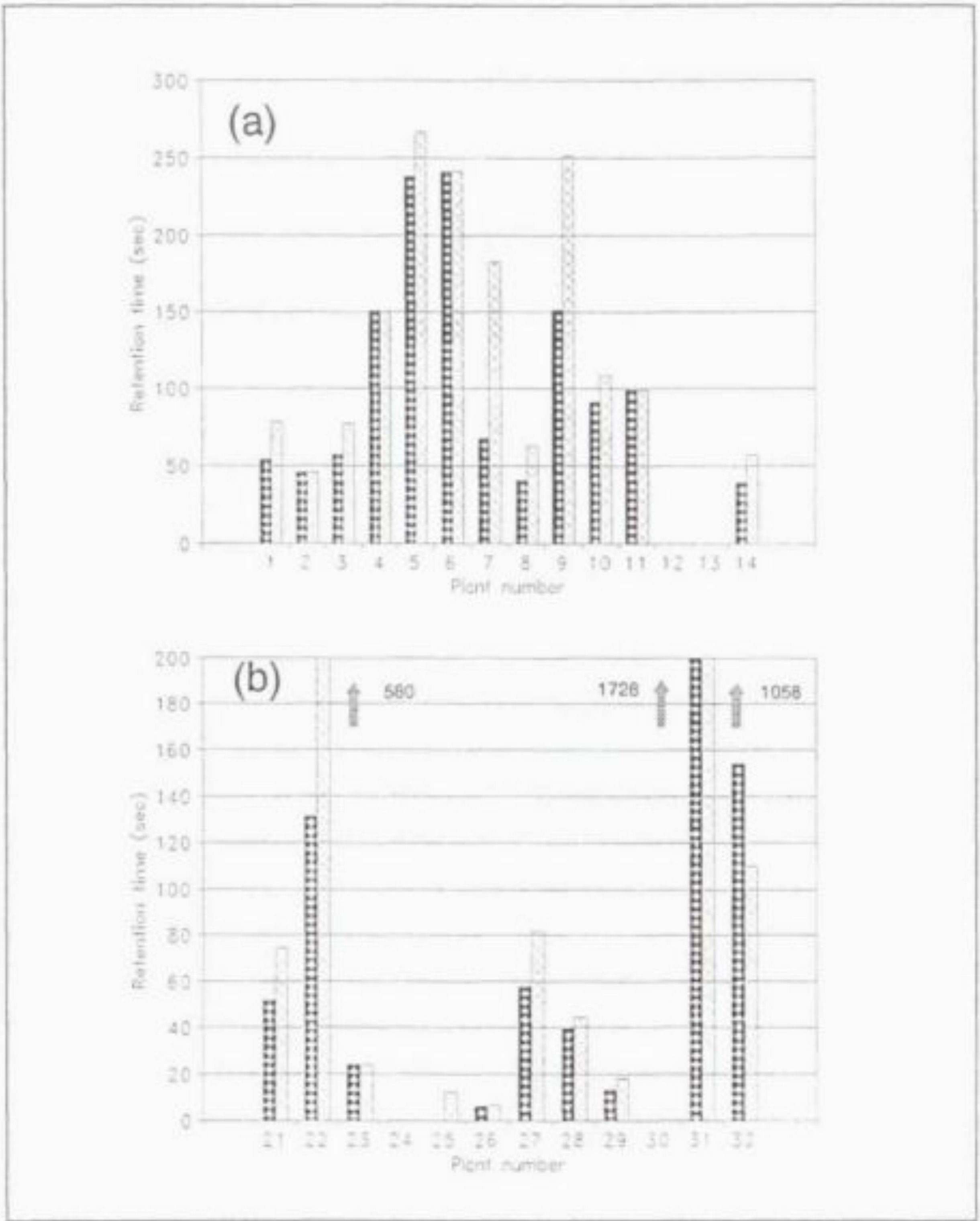


Figure 4.4 : Reaction zone retention time for (a) clarification and (b) thickening plants. Dark bars denote design values; light bars denote actual average values.

4.3 Chemical pretreatment

Table 4.1 summarizes the most important coagulation and flocculation parameters for the DAF plants used for clarification.

TABLE 4.1 : COAGULATION AND FLOCCULATION FOR CLARIFICATION

	Coagulation		Flocculation			
Plant	Coagulant	Dispersion	Type	Time (min)	G (s^{-1})	Gt (-)
#1	$FeCl_3$	weir	pipe	15	45	41 000
#2	$FeCl_3$ polymer	weir weir	unmixed tank	50	*	*
#3	$Al_2(SO_4)_3$ polymer	in-line in-line	pipe	5	110	33 000
#4	$FeCl_3$	in-line	baffled channel	12	80	58 000
#5	$FeCl_3$ polymer	weir weir	stirred tank	120 +	*	*
#6	$Al_2(SO_4)_3$	in-line	stirred tank	10	130	78 000
#7	$Al_2(SO_4)_3$	perforated pipe	stirred tank	8	130	62 000
#8	$Al_2(SO_4)_3$	stirrer	stirred tank	6	140	50 000
#9	$Al_2(SO_4)_3$ polymer	in-line in-line	reactor zone	*	*	*
#10	$FeCl_3$ polymer	in-line weir	stirred tank	21	150	189 000
#11	$Al_2(SO_4)_3$ polymer	stirrer stirrer	stirred tank	15	*	*
#12	$FeCl_3$	in-line	unmixed tank	*	*	*
#13	$FeCl_3$ polymer	in-line in-line	unmixed tank	7	*	*
#14	$FeCl_3$ polymer	in-line weir	stirred tank	6	*	*

* indeterminate

A number of points are evident from Table 4.1:

- The primary coagulant in all cases is either ferric chloride or aluminium sulphate. The choice between the two is predominantly made on economic grounds.
- Polymer (mostly cationic) is used as a flocculation aid at about half the plants surveyed.
- Coagulants are dispersed by mechanical mixers in only 2 cases out of 14 plants.
- Despite the simplicity and good performance of plug-flow flocculators (pipes and baffled channels), they are only used at 3 out of 14 plants. The other plants use back-mix reactors in the form of flow-through turbulent tanks or externally stirred tanks.
- The flocculation time shows a wide scatter between 5 minutes and 2 hours.
- The mixing intensity, on the average, is fairly high with only 1 plant below $G = 80\ s^{-1}$.
- The Gt-product varies, with one exception, within the range 3.10^4 to 8.10^4.

The chemical dosing parameters for DAF thickening applications are not as vital as for clarification. A summary is shown in Table 4.2.

TABLE 4.2 : FLOCCULATION FOR THICKENING

Plant	Flocculant	Sludge type	Note
#21	none	activated sludge	good bioflocculation
#22	polymer	water treatment sludge	dosing into recycle
#23	none	activated sludge	
#24	none	activated sludge	
#25	none	activated sludge	
#26	none	activated sludge	
#27	none	activated sludge	good bioflocculation
#28	none	activated sludge	
#29	$FeCl_3$	activated sludge	dosing for phosphate
#30	none	activated sludge	
#31	polymer	paper mill effluent	in-line dosing
#32	polymer	paper mill effluent	in-line dosing

Chemical dosing is used at all three applications which do not involve activated sludge. In these cases, the use of chemicals is indispensable. For the activated sludge applications, chemical dosing is only used at two out of nine plants. According to the literature, chemical dosing should improve the clarity of the effluent, as well as the float layer concentration. These two parameters are therefore shown in Figure 4.5.

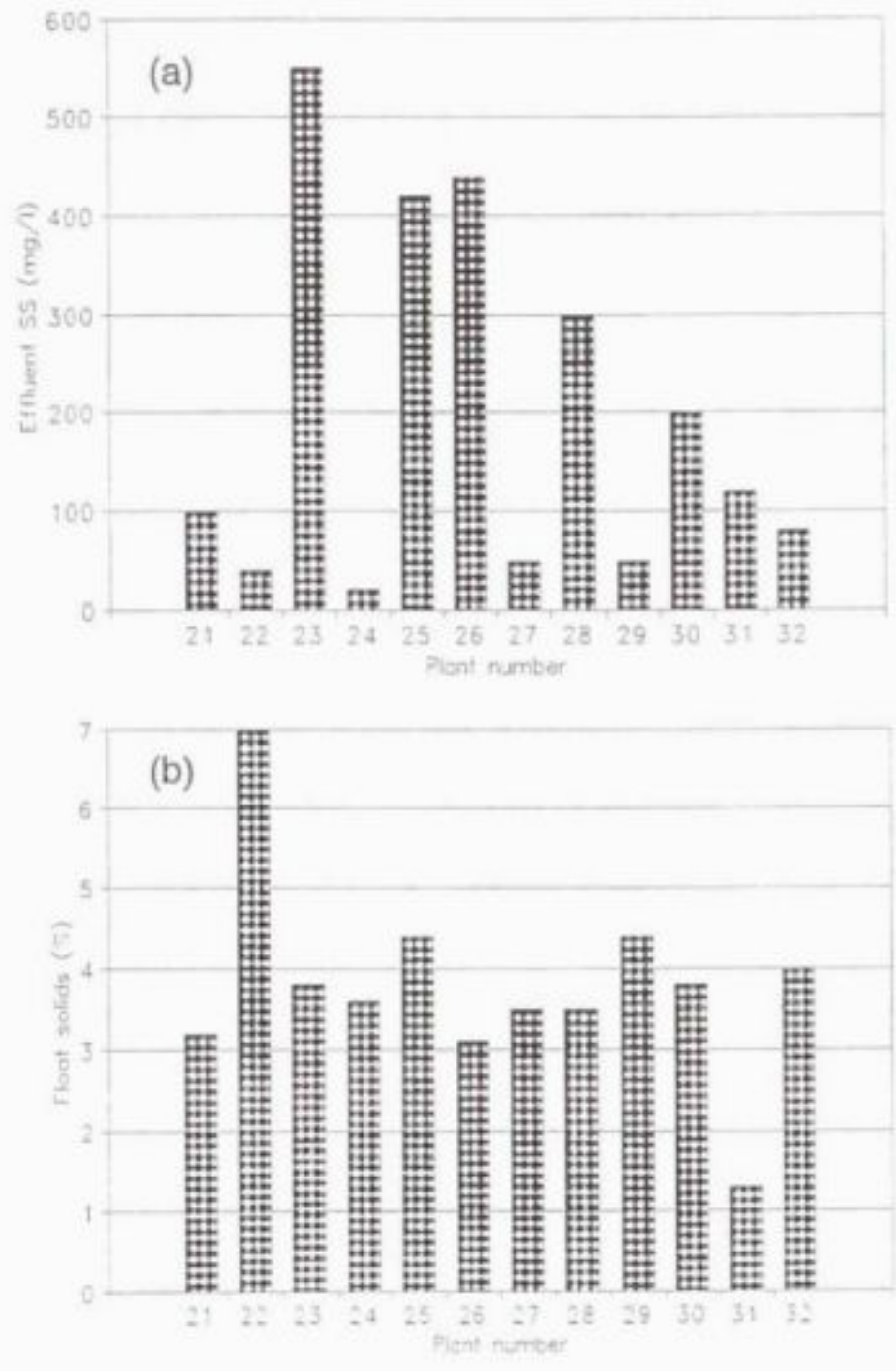


Figure 4.5 : (a) Effluent quality and (b) float solids concentration for thickening plants.

With the exception of plants #22, #31 and #32 (which do not thicken activated sludges), it is clear that the effluent of plants #24, #27 and #29 is superior to the others. Plants #24 and #29 are the ones that use chemical dosing, while "excellent bioflocculation" was reported in the case of plant #27. There are no such dramatic benefits of chemical dosing in terms of float layer concentration. A contributing factor may be the fact that operators deliberately limit the float layer consistency to about 4 % to ease sludge handling further on.

4.4 Bubble production system

Saturator details

The air saturator is the central part of the bubble production system. The most important saturator details are therefore given in Tables 4.3 and 4.4.

TABLE 4.3 : DETAILS OF PACKED SATURATORS

PACKED VERTICAL SATURATORS			
Plant	Packing depth	Assumed efficiency	Water height
#1	0,8 m	75%	25%
#2	0,9 m	75%	*
#3	3,0 m	75%	*
#4	1,5 m	75%	*
#5	2,0 m	75%	*
#7	0,8 m	75%	20%
#8	0,8 m	75%	35%
#10	1,2 m	75%	15%
#12	1,2 m	75%	15%
#13	1,0 m	75%	30%

* not available

TABLE 4.4 : DETAILS OF UNPACKED SATURATORS

VERTICAL SATURATORS WITHOUT INTERNAL RECYCLE				
Plant	Water level (% of height)	Assumed efficiency (%)	Hydraulic loading ($m.h^{-1}$)	Retention time (s)
#6	75	60	34	80
#14	*	60	25	*
#23	50	60	56	32
#25	25	60	57	16
#27	45	60	57	28
#28	*	60	50	*
#29	30	60	69	16
#31	65	60	16	147
#32	50	60	25	107
HORIZONTAL SATURATORS WITHOUT INTERNAL RECYCLE				
Plant	Water level (% of height)	Assumed efficiency (%)	Hydraulic loading ($m.h^{-1}$)	Retention time (s)
#11	50	60	19	146
#22	75	60	7	99
HORIZONTAL SATURATORS WITH INTERNAL RECYCLE				
Plant	Water level (% of height)	Assumed efficiency (%)	Hydraulic loading ($m.h^{-1}$)	Retention time (s)
#21	70	75	30	36
#26	80	75	97	7
#30	*	75	20	*

* not available

The use of packed saturators is confined to clarification plants, where filtered water is mostly recycled to prevent media blockage. Three of the surveyed plants yielded interesting findings:

- One exception was found where DAF effluent was recycled through a packed saturator without any problems of media blockage. In this case, the saturator was routinely opened every two years and no serious blockage could be found.
- On the other hand, a case was found where serious blockage was encountered even with filtered water being recycled- here the saturator had to be cleaned every 6 months.
- In a sludge thickening application, where DAF effluent was directly recycled, a packed saturator was incorrectly specified. Upon commissioning, the saturator blocked within days and the packing had to be removed. The plant is today still running without packing.

Great discrepancies were reported as far as saturator efficiency is concerned. The actual efficiency was very rarely measured. For this reason, a consistent, conservative assumption was made to enable further air dosing calculations:

- All packed saturators are considered to be 75% efficient, regardless of packing depth.
- All unpacked saturators, vertical or horizontal, which have no internal recycle, are considered to be 60% efficient.
- All unpacked saturators with internal recycle, are considered to be 75% efficient.

The water level in the packed saturators varies between 15% and 35% of the total saturator height. This satisfies the two conflicting requirements in saturator design - a low enough water level to keep maximum packing exposed to the pressurized air cushion, but enough water to prevent air entrainment into the saturator outlet pipe. For unpacked saturators the variance in water level of 25 % to 80 % of total height is much greater.

Hydraulic loading

The hydraulic loading had been identified as an important saturator parameter - these values for the packed saturators are shown in Figure 4.6. The saturator hydraulic loading, both as designed as well as the actual loading, varies widely between 20 and 100 $m.h^{-1}$. In 7 out of 10 plants, the variation is between 60 and 100 $m.h^{-1}$.

The difficulty surrounding the efficiency of unpacked saturators was pointed out in Chapter 2. For this reason, a number of potentially important parameters are given in Table 4.4, grouped according to saturator type.

The horizontal saturators without recycle allow the longest retention time (median 123 s) compared to vertical saturators (median 32 s) and horizontal saturators with recycle (median 22 s). In terms of hydraulic loading, the order is different; vertical saturators (median 50 $m.h^{-1}$), horizontal saturators with recycle (median 30 $m.h^{-1}$), and horizontal saturators without recycle (median 13 $m.h^{-1}$).

Saturation pressure

The saturator pressure for both packed and unpacked saturators are shown in Figure 4.7.

For packed saturators, the actual pressure varies between 350 and 550 kPa. The one exception is plant #4, where the actual pressure is 250 kPa. This is the only plant where the air dosage is easily adjustable by the operators and is therefore kept to the minimum. With the exception of plant #9 (the only full-stream application) the pressure variation for unpacked saturators are about in the same range as for packed saturators.

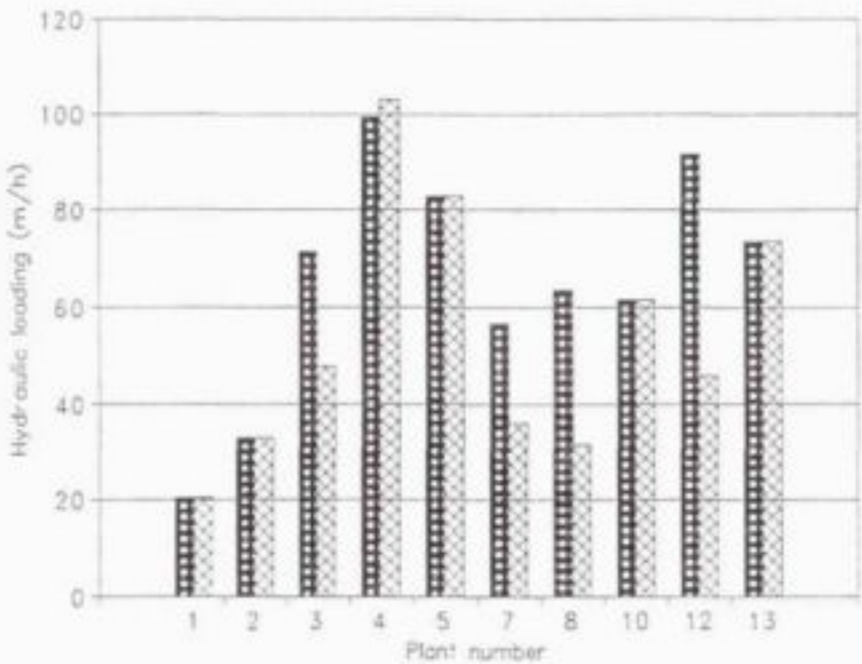


Figure 4.6 : Saturator hydraulic loading for packed saturators.
Dark bars denote design values; light bars denote actual average values.

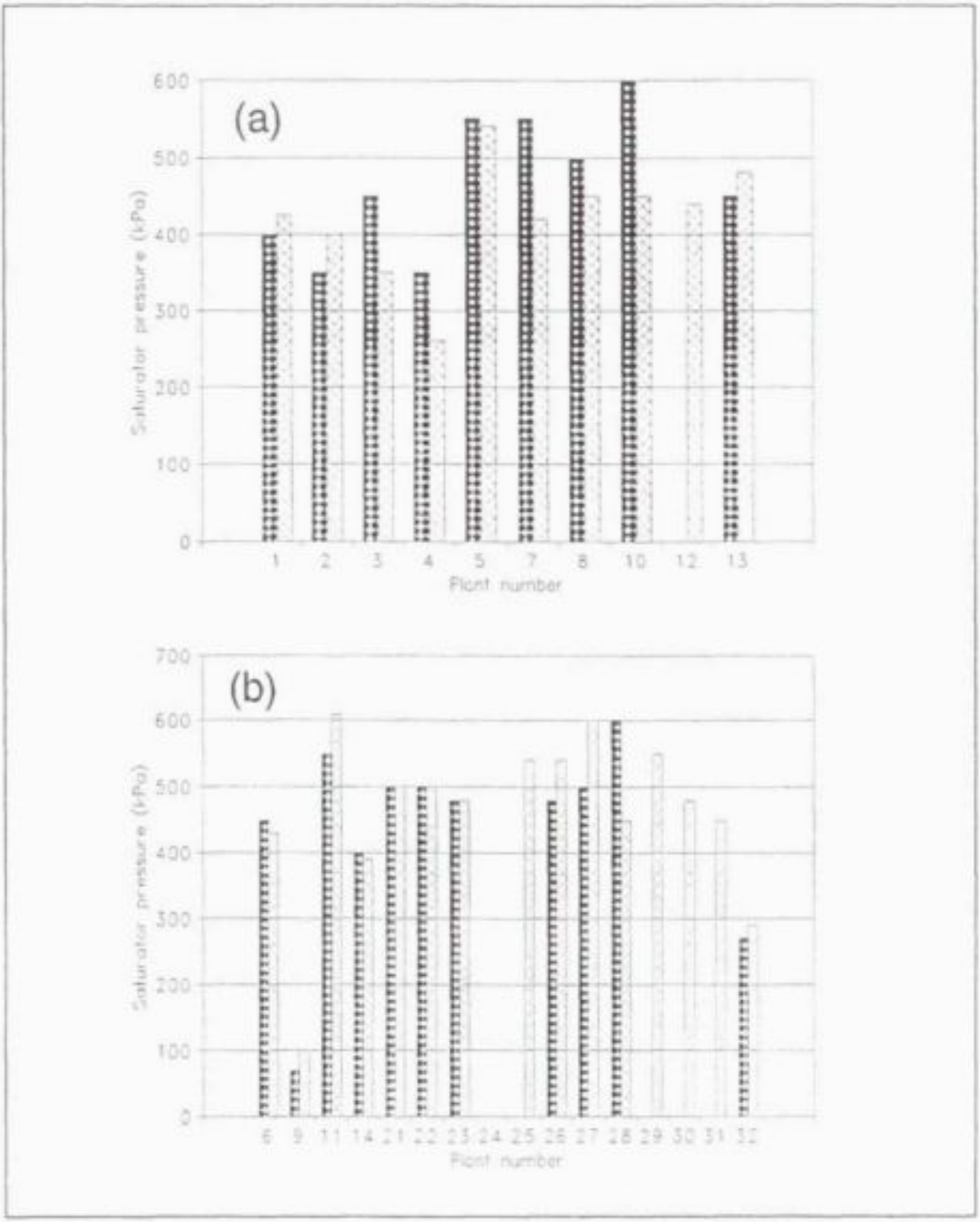


Figure 4.7 : Saturator pressure for (a) packed and (b) unpacked saturators. Dark bars denote design values; light bars denote actual average values.

Recycle rates

The recycle rates, expressed as a fraction of the raw water feed rate, are shown in Figure 4.8 for both clarification and thickening applications. Although the recycle rate *per se* has no significance (the actual air reaching the flotation zone is the important parameter), it is nevertheless shown to determine whether a pattern emerges. The recycle rate is of significance in terms of reactor zone loading and retention time.

For clarification, the average recycle rate is obviously lower because of lower air requirements and ranges between 7% and 18%, with the average around 10%. For thickening, the range is much broader from 30% to 240% to cater for the differing solids concentration in the raw water.

Air dosing

A crucial parameter for flotation performance was shown in Chapter Two to be the air dosing concentration (in the case of clarification) or the air/solids ratio (in the case of thickening). This was not directly measured during the plant visits and was calculated by assuming a conservative saturator transfer efficiency, as indicated before. The calculations for the air dosing concentration, applicable to clarification plants, are illustrated in Table 4.5.

TABLE 4.5 : ACTUAL AIR DOSING FOR CLARIFICATION

Plant	Recycle rate ($m^3.h^{-1}$)	Saturator pressure (kPa)	Saturator efficiency (%)	Air mass ($g.h^{-1}$)	Feed rate ($m^3.h^{-1}$)	Air concentration ($mg.\ell^{-1}$)
1	3,4	425	75%	237	33	7,2
2	12,7	400	75%	834	90	9.3
3	54,2	350	75%	3 116	638	4,9
4	20,7	260	75%	884	167	5,3
5	18,3	540	75%	1 623	184	8,8
6	12,0	430	60%	678	100	6.8
7	48,0	420	75%	3 311	260	12,7
8	18.0	450	75%	1 330	200	6.7
9	0.0	100	60%	9 855	750	13,1
10	25,0	450	75%	1 848	208	8.9
11	60,0	610	60%	4 809	781	6.2
12	36,0	440	75%	2 602	250	10,4
13	1,8	480	75%	142	17	8.3
14	38,0	390	60%	1 947	361	5.4

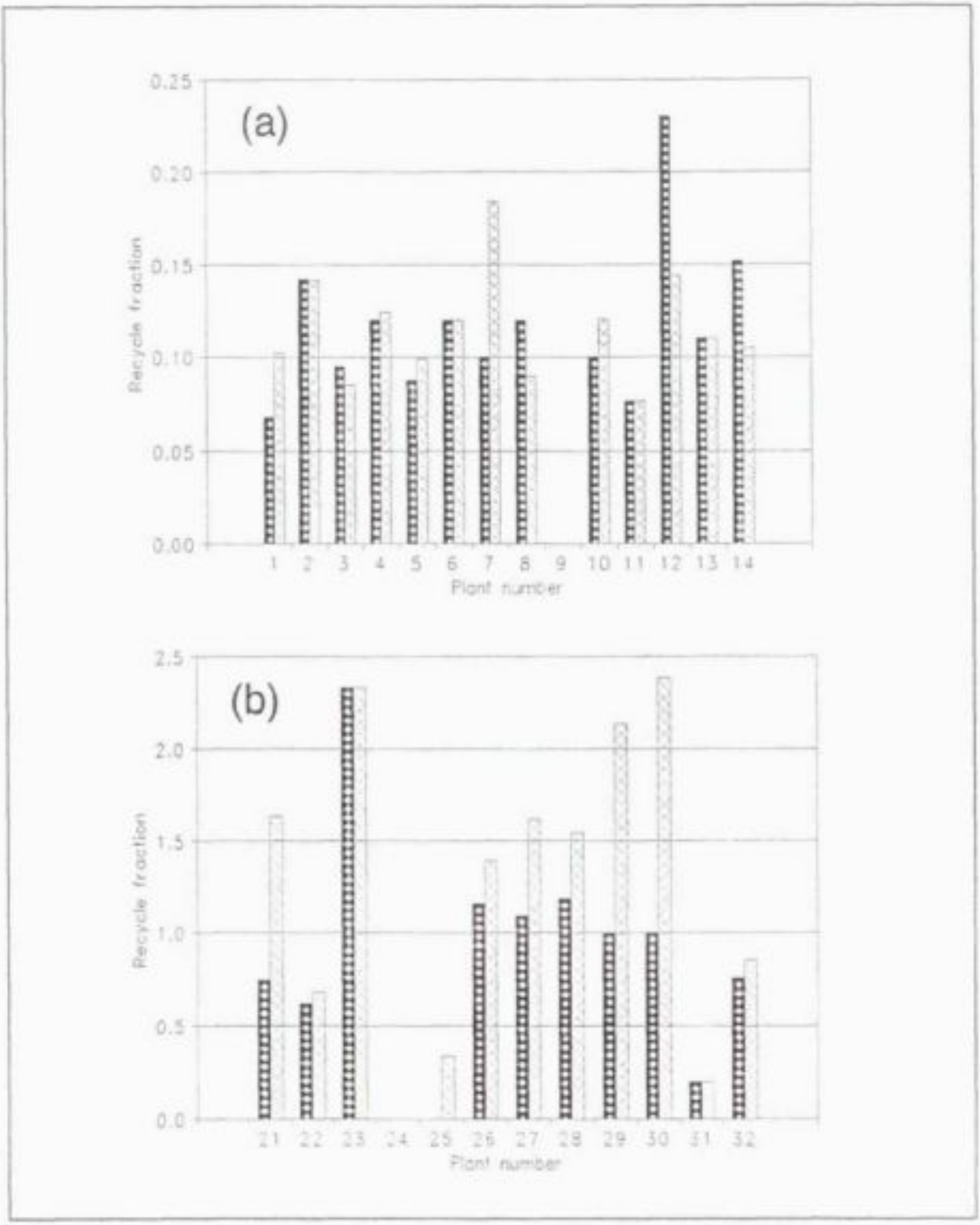


Figure 4.8 : Recycle ratio for (a) clarification and (b) thickening plants.
Dark bars denote design values; light bars denote actual average values.

Similarly, the calculations for the air/solids ratio, applicable to thickening plants, are illustrated in Table 4.6.

TABLE 4.6 : ACTUAL AIR/SOLIDS RATIO FOR THICKENING

Plant	Recycle rate ($m^3.h^{-1}$)	Saturator pressure (kPa)	Saturator efficiency (%)	Air flow ($g.h^{-1}$)	Feed rate ($m^3.h^{-1}$)	SS_{in} ($mg.\ell^{-1}$)	a_s ($mg.mg^{-1}$)
21	180	500	75%	14 783	110	5 500	0,024
22	24	500	60%	1 577	35	6 200	0,007
23	630	450	60%	37 252	270	4 000	0,034
25	180	540	75%	15 965	540	4 200	0,007
26	580	540	75%	51 443	417	4 200	0,029
27	115	600	60%	9 067	71	4 500	0,028
28	128	450	60%	7 569	83	5 000	0,018
29	22	550	60%	1 590	10	3 700	0,043
30	50	480	75%	3 942	21	5 500	0,034
31	8	450	60%	473	41	3 000	0,004
32	206	290	60%	7 850	240	2 800	0,012

The calculated air concentrations and air/solids ratios are illustrated in Figure 4.9.

The minimum air concentration for clarification is 5 $mg.\ell^{-1}$, which is right at the limit suggested by the literature. From here it ranges up to 10 $mg.\ell^{-1}$, with the exception of two plants which are running well below full capacity.

The air/solids ratio for the applications other than activated sludge thickening, is considerably lower, namely around 0,004 to 0,012. The air/solids ratio for activated sludge thickening is generally taken as 0,02 which is almost reached or exceeded by all those plants. The exception is Plant #25, which does not perform to expectations.

Injection nozzles

For clarification, the majority of plants utilized the principle of a shroud surrounding the injection orifice(s), all variations of the well-known NIWR-nozzle described elsewhere. Of the 13 plants for which nozzle details were available, 9 plants use shrouded nozzles - the other 4 use a valve in the recycle line for pressure release. The earlier plants tend to have only one central nozzle with multiple orifices (4 out of 9). The newer plants tend to use a multitude of micronozzles spread over the entire reaction zone by means of a distribution manifold.

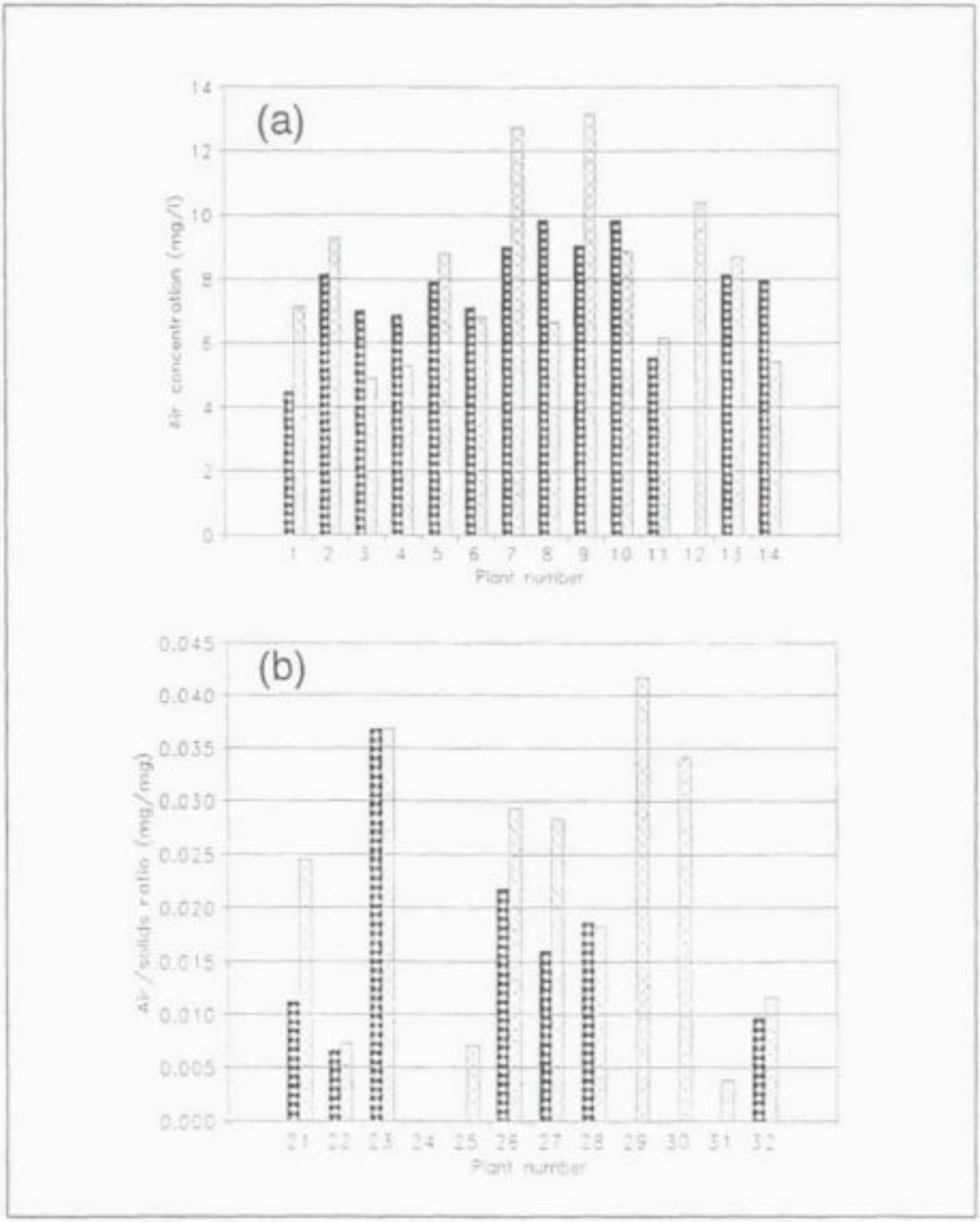


Figure 4.9 : Air dosing for (a) clarification and (b) thickening.
Dark bars denote design values; light bars denote actual average values.

For thickening, the majority (7 out of 12) of plants makes use of a standard industrial valve in the recycle line to relieve the pressure. Of the remainder, 4 plants use an adjustable plug with no shroud. A single plant use multiple nozzles along a manifold.

In this survey, it was found that only a limited number of clarification plants have adjustable nozzles (5 out of 13), while practically all the nozzles for thickening plants are adjustable (11 out of 12).

The nozzle types are shown schematically in Figure 4.10.

4.5 Flotation zone

Crossflow velocity

The first flotation zone parameter that was analyzed, is the crossflow velocity between the reaction zone and flotation zone. It is intuitively felt that too high a velocity would not only disturb the air/floc agglomerates, but would also induce unwanted turbulence in the flotation zone. As in the case of the reaction zone demarcation, the crossflow area had to be defined rather arbitrarily in some cases, but the calculated values are shown in Figure 4.11.

In the case of clarification, the values are mostly below 50 $m.h^{-1}$, and always below 100 $m.h^{-1}$ with one exception. The exception, Plant #14, does not perform to expectations. In the case of thickening, the values are higher, but generally below 200 $m.h^{-1}$, with two exceptions. Once again, the two exceptions are those plants that do not perform to expectations. Figure 4.10 provides limited empirical evidence that the crossflow velocity may be an important flotation zone parameter which is seldomly referred to or experimented with.

Hydraulic/solids loading

The most important flotation zone parameter was indicated to be the hydraulic loading - these values are shown in Figure 4.12.

For clarification, the actual hydraulic loading is at or below 8 $m.h^{-1}$ in all cases, which is a fairly conservative value compared to some evidence in the literature. The actual minimum values are at about 3 $m.h^{-1}$ or below, which would be unnecessarily conservative. These low values are, however, due to newer plants running below capacity. When all the plants run to capacity, the minimum hydraulic loading will be slightly above 4 $m.h^{-1}$ - still a conservative value.

For thickening, the hydraulic loading is of lesser importance as the process is normally controlled by solids loading rate. It is significant to note that some thickening plants actually run at the same hydraulic loadings as the highest clarification plants. Once again, the two thickening plants running at the highest loadings, are those plants that do not meet expectations.

The solids loading rate, the more important parameter for thickening plants, are shown in Figure 4.13. Here a very wide scatter is evident, from less than 2 to 14 $kg.m^{-2}.h^{-1}$. The two highest values are for those plants that do not match expectations.

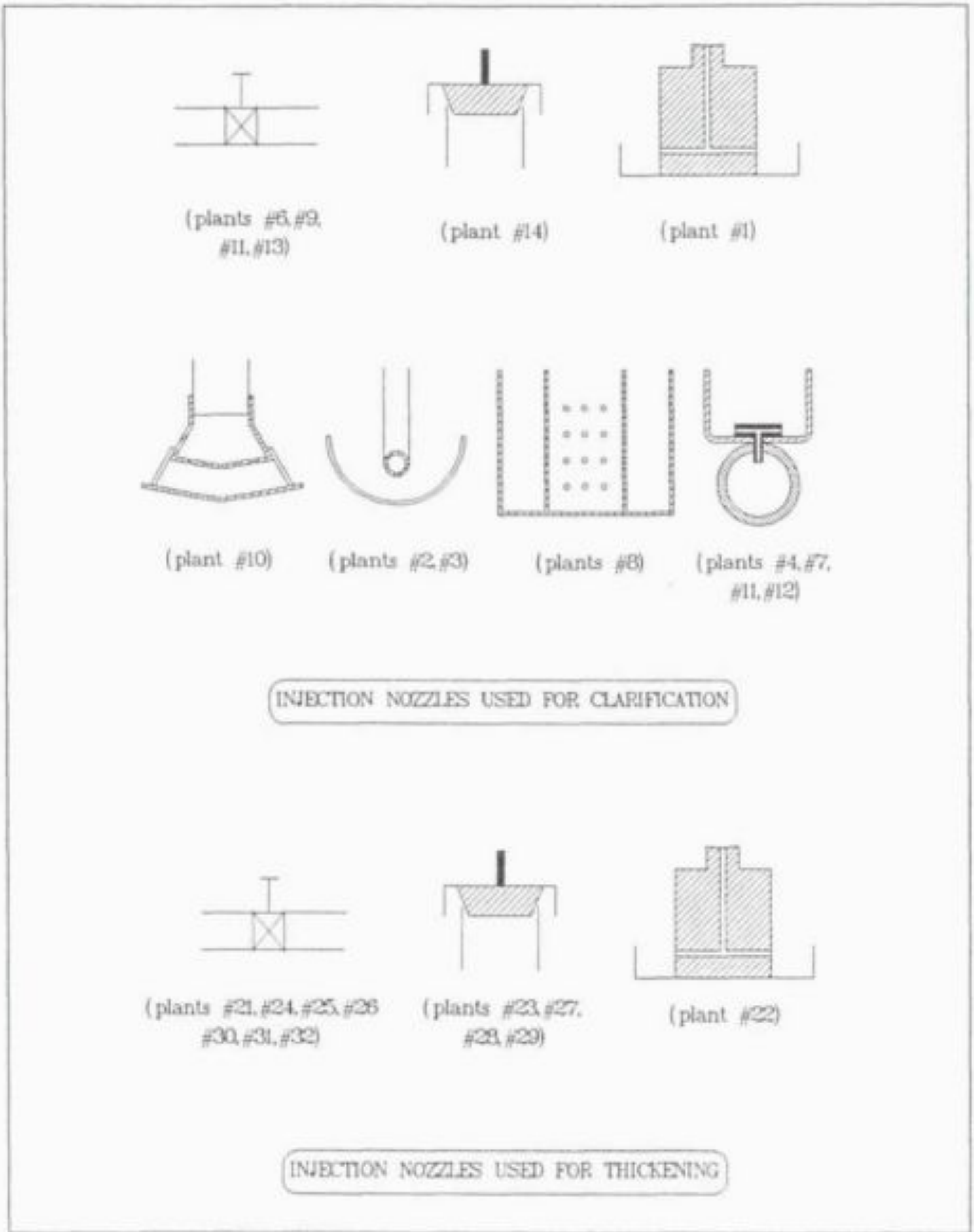


Figure 4.10 : Injection nozzles used in SA plants (schematic).

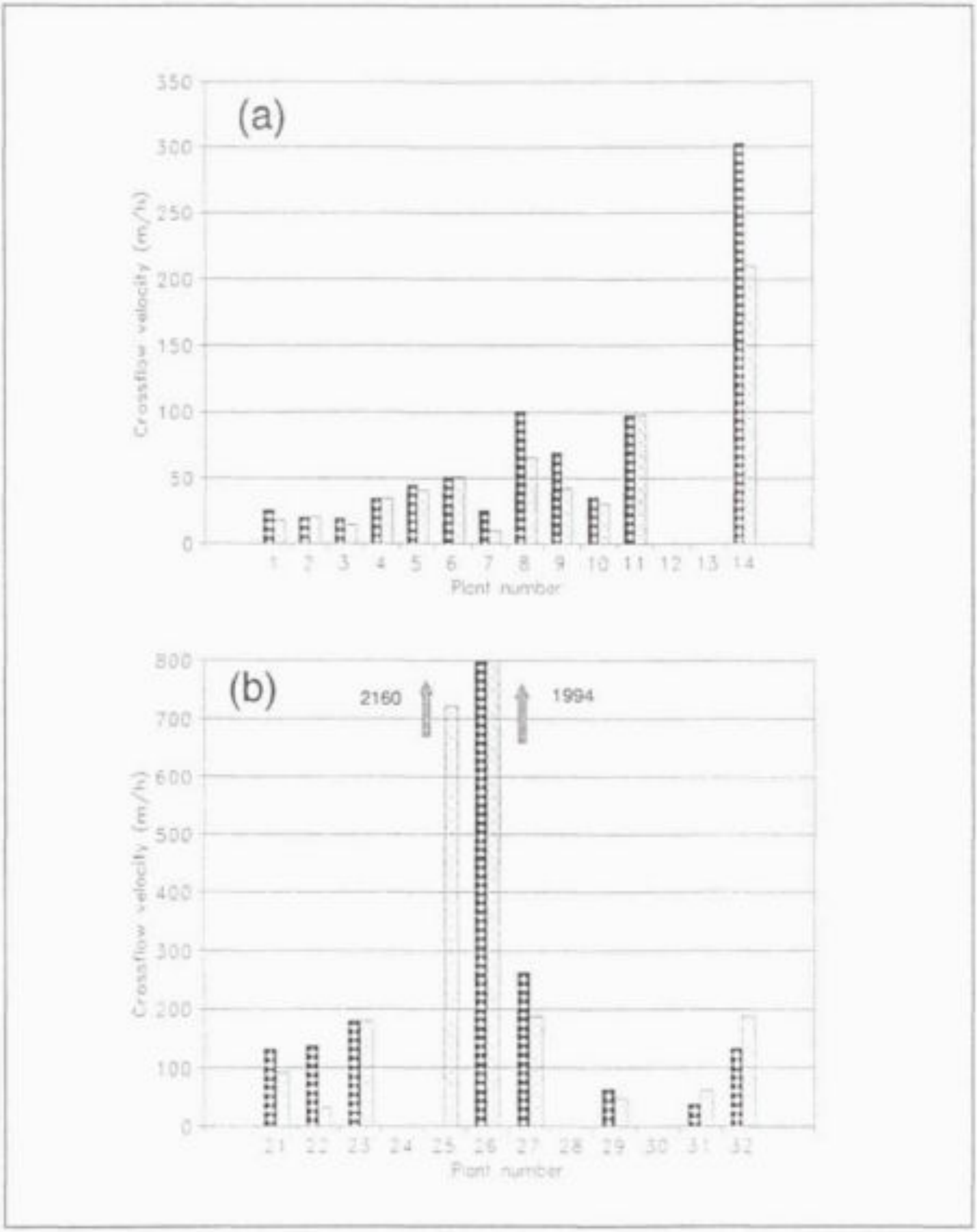


Figure 4.11 : Cross-flow velocity for (a) clarification and (b) thickening plants. Dark bars denote design values; light bars denote actual average values.

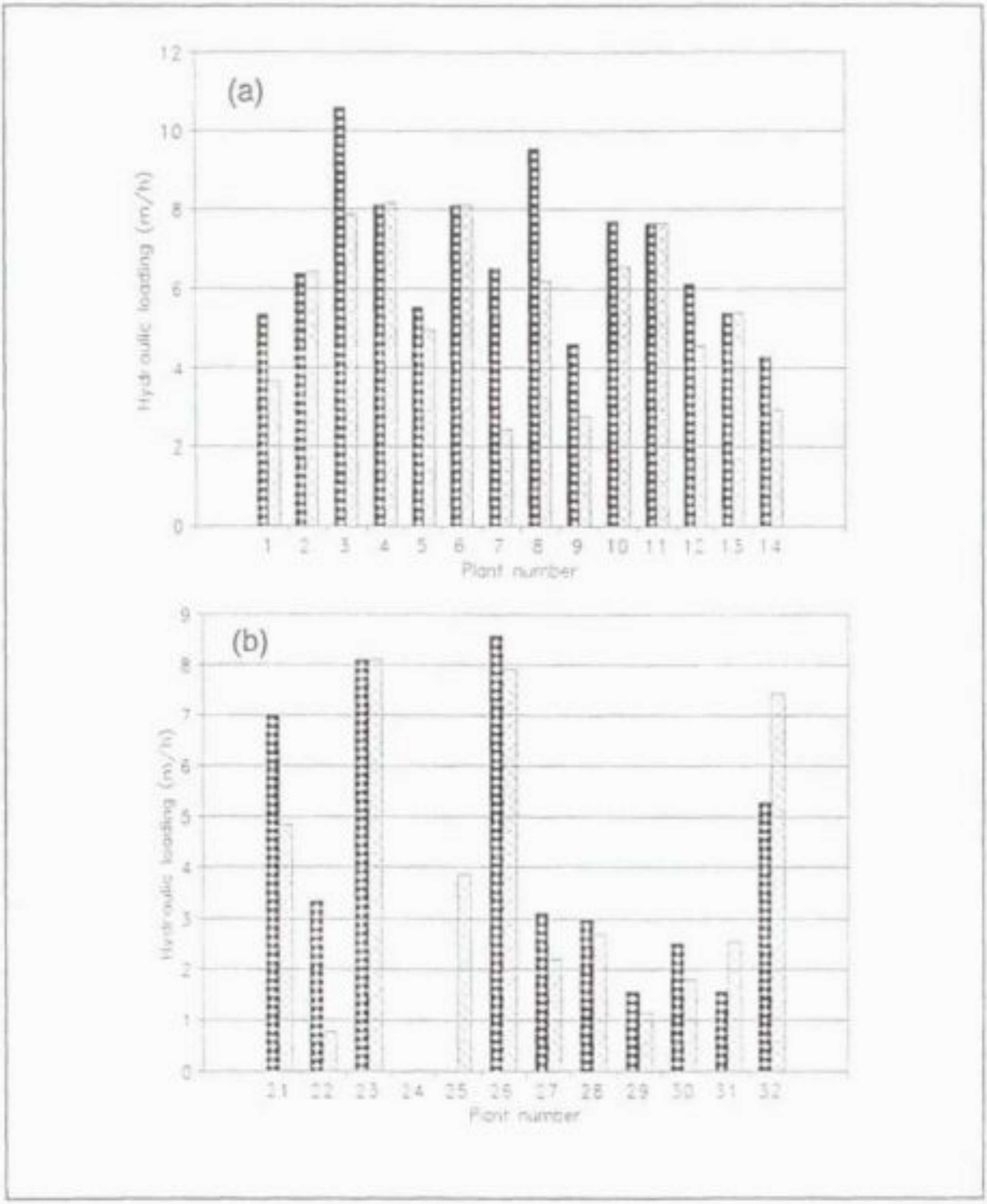


Figure 4.12 : Flotation zone hydraulic loading (based on flotation area only) for (a) clarification and (b) thickening plants.
Dark bars denote design values; light bars denote actual average values.

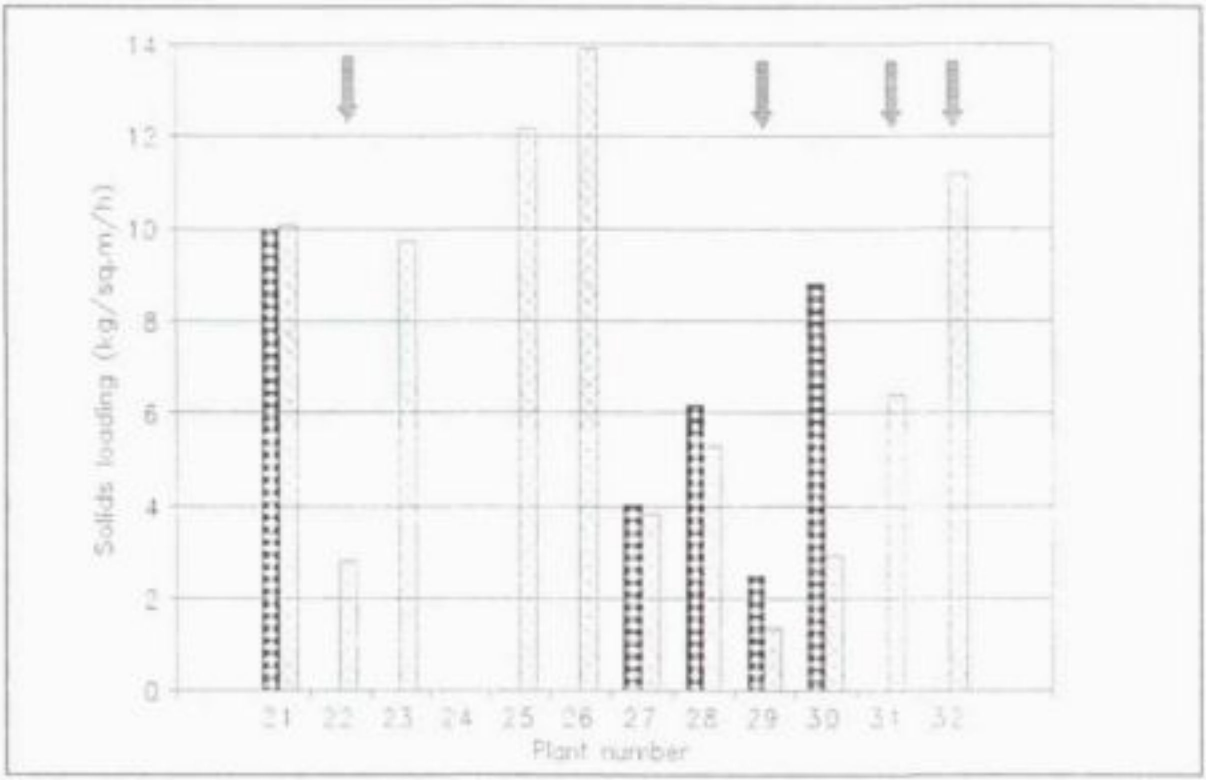


Figure 4.13 : Solids loading rate for thickening plants.
Dark bars denote design values; light bars denote actual average values. Arrows indicate plants where coagulants are used.

Flotation zone geometry

The flotation zone side depths are shown in Figure 4.14. The literature survey indicated that for clarification purposes, there should not be any difference between the side depths required for clarification and thickening, but that additional depth should be allowed for the accumulation of the float layer during thickening. The average side depth of about 2 m for clarification are, in fact slightly lower than the average side depth of about 2.5 m for thickening. In general, it appears that the side depths are considerably more than theoretically required. Those plants with the smallest side depths, operate satisfactorily. The two plants with the greatest side depths, incidentally, are also those plants that do not operate very well.

The length/width ratios of the rectangular plants are shown in Figure 4.15. It is evident that the plants for clarification have a minimum ratio of 1.0 and a maximum of 2.5. The thickening plants are definitely narrower and longer, with the minimum ratio 3.0 and the maximum ratio 4.6. This is probably attributable to an attempt to keep the cost of the float scraping equipment down. For clarification, where the float layer is thin and offers little resistance, the scraper width is not as crucial as for thickening plants, where the scrapers must be much more robust to deal with heavier, thicker float layers.

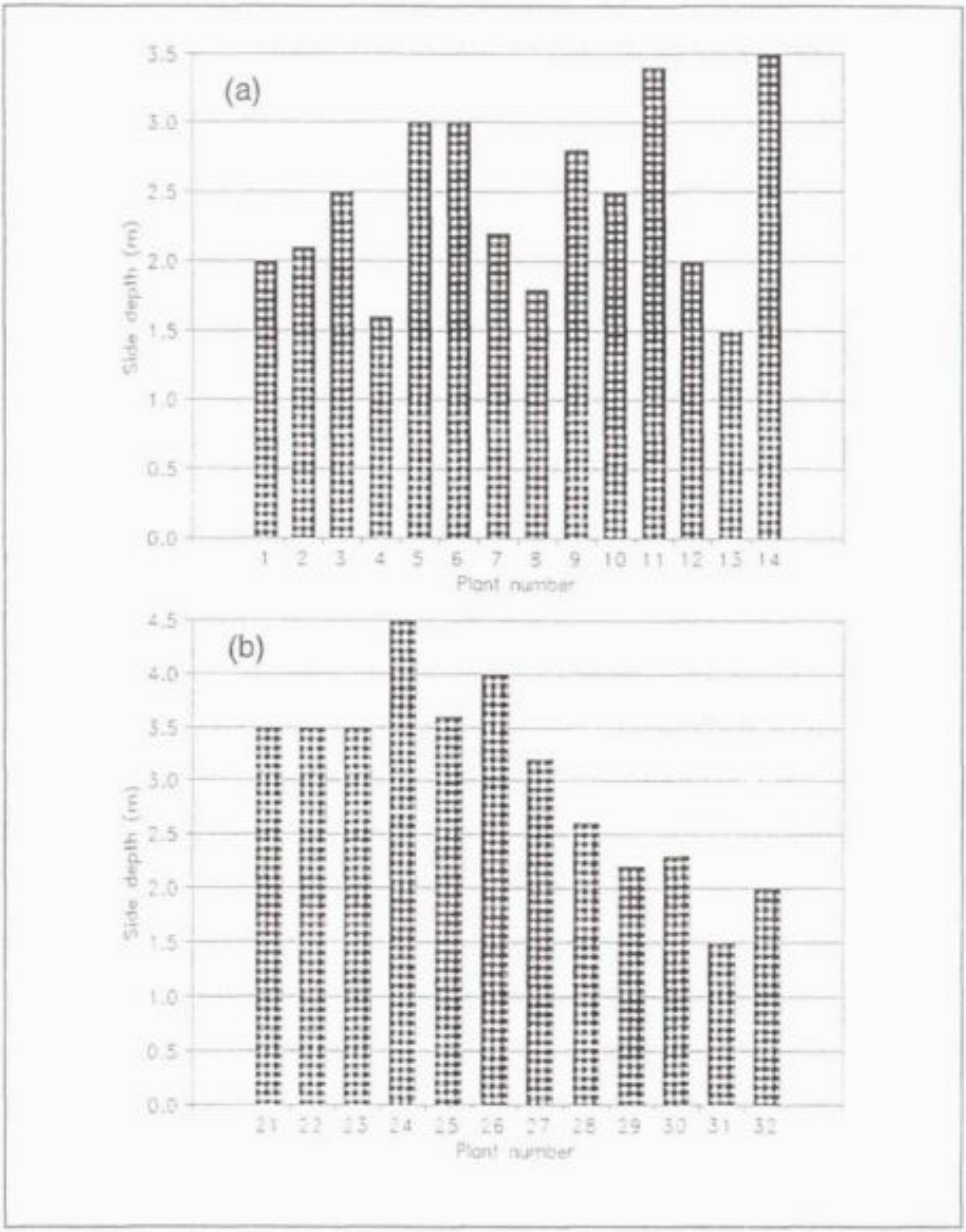


Figure 4.14 : Flotation zone side depth for (a) clarification and (b) thickening plants.

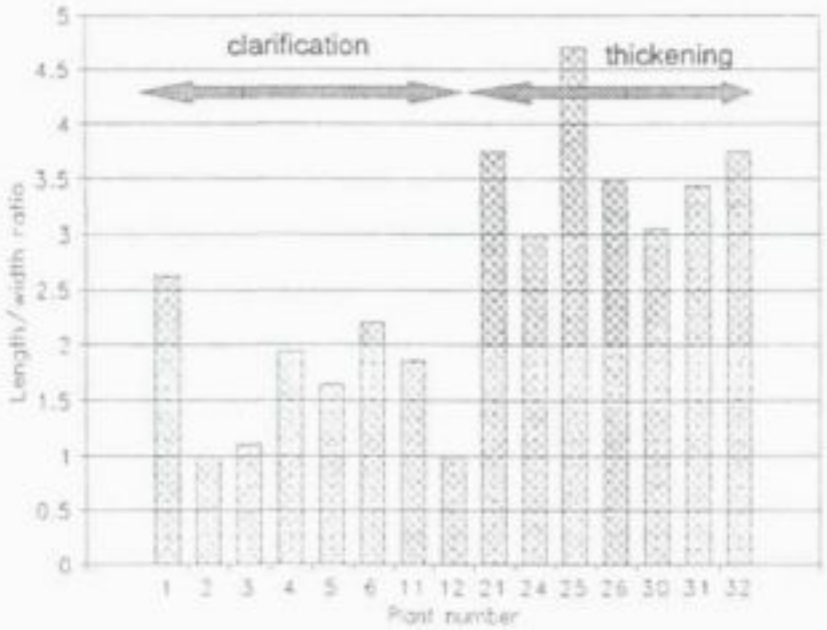


Figure 4.15 : Length/width ratio for rectangular plants.

4.6 Float layer removal

The float layer removal methods applied at the clarification plants are summarized in Tables 4.7 and 4.8.

TABLE 4.7 : FLOAT LAYER REMOVAL FOR RECTANGULAR CLARIFICATION PLANTS

RECTANGULAR FLOTATION TANKS					
Plant	Method	Mode	Spacing (m)	Speed ($m.h^{-1}$)	Bottom scraping
#1	grid	manual	n/a	n/a	no
#2	grid	manual	n/a	n/a	no
#3	flushing	not applicable	n/a	n/a	DAFF
#4	flushing	not applicable	n/a	n/a	DAFF
#5	scraping	continuous	1.3	40	yes
#6	scraping	intermittent	2.5	200	no
#11	scraping	intermittent	7.0	54 min 270 max	no
#12	flushing	not applicable	n/a	n/a	no

TABLE 4.8 : FLOAT LAYER REMOVAL FOR CIRCULAR CLARIFICATION PLANTS

CIRCULAR FLOTATION TANKS					
Plant	Scrapers (no)	Mode	Turn speed ($rev.h^{-1}$)	Tip speed ($m.h^{-1}$)	Bottom scraping
#7	two	continuous	6.0	250	no
#8	two	continuous	5.0	120	no
#9	three	continuous	2.6	160	no
#10	two	continuous	3.2 min 60.0 max	70 min 1320 max	yes
#13	flushing	n/a	n/a	n/a	no
#14	four	continuous	7.0	290	yes

It is evident that scraping (mechanically or manual) is used at most plants, but that bottom scraping is the exception rather than the rule. Only two scrapers operate intermittently. The scraper speed appears to be determined by the maximum blade speed, which shows less scatter than the angular speed.

The float removal methods for thickening plants are summarized in Tables 4.9 and 4.10. Once again, there is a single exception where the scrapers operate intermittently rather than continuously. The majority of the plants (8 out of 12) make use of bottom scraping. The maximum blade speed is about the same as for clarification, namely at about 200 to 300 $m.h^{-1}$.

TABLE 4.9 : FLOAT LAYER REMOVAL FOR RECTANGULAR THICKENING PLANTS

RECTANGULAR FLOTATION TANKS					
Plant	Method	Mode	Spacing (m)	Speed ($m.h^{-1}$)	Bottom scraping
#21	scraping	continuous	2,0	42	yes
#24	scraping	continuous	n/a	n/a	yes
#25	scraping	continuous	n/a	n/a	yes
#26	scraping	intermittent	5,0	129	yes
#30	scraping	continuous	2,0	90	no
#31	scraping	continuous	n/a	n/a	no
#32	scraping	continuous	0,6	72	no

TABLE 4.10 : FLOAT LAYER REMOVAL FOR CIRCULAR THICKENING PLANTS

CIRCULAR FLOTATION TANKS					
Plant	Scrapers (no)	Mode	Turn speed ($rev.h^{-1}$)	Tip speed ($m.h^{-1}$)	Bottom scraping
#22	ten	continuous	2,0	63	yes
#23	two	continuous	6,0	230	yes
#27	four	continuous	8,2	270	yes
#28	four	continuous	5,0	160	no
#29	two	continuous	3,3	63	yes

4.7 Maintenance requirements and problem areas

Operating staff at each of the surveyed plants were asked to identify the most common problems in terms of repair and maintenance. Fifteen plants responded to this question, and the problem areas are ranked as follows:

- By far the most common problem area is that of the float layer scraping equipment (9 out of 15 plants). Specific comments in this category were excessive wear on scraper blades and rollers, regular snapping of scraper chains, high maintenance on the scraper gearbox, and replacement parts that have to be imported. This finding is exactly in line with an earlier observation on British thickening plants, which labelled scraper problems as the only real problem with DAF thickening (Ashman, 1976).

- The second most common complaint concerned the maintenance and wear on the sludge pumps (6 out of 15 plants).
- The third most common complaint was that equipment tend to foul or clog, for example the saturator packing, probes, flow meters, solenoid valves, float switches and narrow pipes (5 out of 15 plants).
- Two plants complained about high maintenance on the compressors used for the saturators.
- Two plants complained about flow control of equalization and flotation tanks.
- One plant each complained about corrosion of equipment and valve maintenance.

4.8 General assessment of flotation performance

The final part of the survey was to get an overall impression from owners and operators on the success of their particular DAF installations. The responses for thickening plants ranged over the full spectrum:

- A few respondents were very positive and indicated that they would undoubtedly duplicate (and have indeed duplicated) their DAF plants if the need arises.
- Some respondents were positive, but qualified their response by adding that they would in future pay particular attention to a few critical areas which had been troublesome in the past.
- One eminent respondent stated that, if given the chance, he would not choose DAF again as its application is flawed by too many technological difficulties, high cost and less than expected efficiency.

How can these opinions, all from experienced operators and managers with a wealth of practical experience, be reconciled? The authors believe that DAF offers a practical, inherently sound method of clarification as well as thickening, but that the process had been improperly engineered in a number of applications. Indirect evidence is contained in the plant survey discussed in this chapter.

- In the case of clarification, there was much less variation of the critical design parameters, and consequently also almost no negative comment from plant operators and managers, a comment echoed by British DAF operators during a recent survey (Longhurst & Graham, 1987). The same conclusion was reached after a Finnish survey (Heinänen, 1988) where the process was satisfactory in terms of quality and quantity, and problems are few.
- In the case of thickening, the variation in some design parameters was tremendous, and it was here where the most negative comments were received. The authors do not imply that plants were negligently designed - there are some real design problems, especially when the process has to be scaled up to very large proportions, and these problems are acknowledged in Chapter Seven, where continuing research needs are identified. The fact, however, is that some parameters in the less successful plants are, seen in retrospect, way outside the range of those plants that are efficient.

CHAPTER FIVE

DESIGN CONSIDERATIONS AND GUIDELINES

5.1 General

The empirical guidelines contained in this chapter are based on

- the literature cited;
- published surveys of overseas flotation plants;
- the data of this Southern African survey;
- personal communications with designers and operators; and
- the authors' own experience.

Before the designer can start with the detailed design of a flotation plant, it is necessary to first consider the overall process design for the entire treatment facility. From this, the exact objectives of the flotation plant can be determined. Typical decisions that could be made at this point, are :

- From the design flow, the number of flotation units can be determined.
- If the sludge is disposed with other waste streams in sludge lagoons, there will be no need for special float layer removal devices and the layer could be simply flushed away.
- If the sludge is to be thickened further by other means, the emphasis should be on float layer thickening within the flotation tank by means of grids, scrapers or polymer addition in the case of waste activated sludge.
- If the removed sludge will be transported over long distances under gravity, it may be pointless to thicken the sludge within the flotation zone beyond the concentration where it will flow spontaneously.
- If a high quality effluent is required in addition to the thickening requirements of waste activated sludge, the use of chemicals is imperative.
- If high raw water turbidity above the critical turbidity where flotation fails (a variable that has to be tested - normally about 60 to 100 NTU) is a possibility, provision should be made for presedimentation. If so, a second facility for dosing, mixing and flocculation between presedimentation and flotation should be provided.

The designer also has to compile and interpret all available data which could impact on the design of the flotation plant. Examples are the variation in flow about the mean, anticipated mode of operation (continuous or stop-start), anticipated raw water quality, and requirements for the flotation effluent and float layer concentration. In the event of an existing plant simply being extended, the assembly of this information may be trivial. Where flotation is to be added as a new process at an existing plant, flotation performance could be estimated by a relatively simple set of bench or pilot tests. Where no existing plant is available, but the feed water will be typical of say eutrophic water or waste activated sludge, the designer could still revert to empirical guidelines such as these. In the worst case, where the water to be treated is of unusual character and no historical data is available, the designer has no option but to proceed with a comprehensive series of bench tests, verified with a pilot programme of sufficient duration to include the full spectrum of raw water variability.

5.2 Reaction zone

The reaction zone of a flotation system has been identified by the authors as being of primary importance, as evidenced from the plant survey and fundamental considerations from the literature. In simple terms the hydraulic conditions should secure rapid, effective contact between bubbles and solids in as short a time as possible.

The configuration of the reaction zone depends in large measure on the shape of the flotation unit. The advantages of each shape were earlier summarized in Table 2.2. As a general rule, one would place the reaction zone in the *centre* of a circular tank, but on the *side* of a rectangular tank. In both cases, the designer further has the option of placing the reaction zone outlet at the *top*, *middle* or *bottom* of the tank. These variations were previously shown in Figure 4.2. During this South African survey, all the different combinations of tank shape and reaction zone positions were encountered, and no obviously advantages amongst the plants could be ascribed to any particular one. It would therefore appear that the reaction zone configuration could be based on practical preference, provided that the rest of the reaction zone design guidelines are adhered to.

A second factor to consider is that of scale. Pilot plants with a single raw water/recycle inlet, regardlesss of how well they perform, cannot be scaled up indefinitely, as was pointed out in the literature review. If more and more raw water and recycle are forced through the same point, the turbulence will increase until bubble coalescence and floc breakup will cause less efficient flotation. In general, it is easier to "spread" the reaction zone over multiple injection points when rectangular flotation tanks are used. In this case, the reaction zone could take the shape of a trough alongside one of the edges, and raw water and recycle could be introduced through a manifold running in the trough. With circular tanks, the reaction zone is necessarily confined to the centre of the tank and it may be more difficult to avoid turbulence. There are no quantitative guidelines available about when the use of multiple injection points becomes necessary; this will in fact be identified as a research need in Chapter Seven.

Analogous to conventional flocculation, it is postulated that two factors determine the reaction efficiency, namely the length of the mixing period and the mixing intensity. The length of the mixing period is simply calculated as the average retention time in the reaction zone. This calculation is based on the *total* flow, i.e. raw water as well as recycle. It is more difficult to quantify the mixing intensity, as there is no measurable energy input into the reaction zone. For these guidelines, the mixing intensity is crudely approximated by the mean flowthrough velocity, based on the largest cross-sectional area in the reaction zone. The reaction zone parameters are summarized in Table 5.1.

TABLE 5.1 : DESIGN GUIDELINES FOR REACTION ZONE

			MINIMUM	MEDIAN	MAXIMUM
CLARIFICATION					
Residence time	(SA survey)	s	46	104	267
Residence time	(recommended)	s	60	-	240
Hydraulic loading*	(SA survey)	$m.h^{-1}$	19	64	222
Hydraulic loading*	(recommended)	$m.h^{-1}$	40	-	100
THICKENING					
Residence time	(SA survey)	s	6	59	1058
Residence time	(recommended)	s	30	-	120
Hydraulic loading*	(SA survey)	$m.h^{-1}$	5	174	997
Hydraulic loading*	(recommended)	$m.h^{-1}$	100	-	200

* based on total flow, i.e. feed plus recycle

5.3 Chemical pretreatment

Clarification

For clarification of surface waters, the use of a coagulant is imperative. There are significant similarities between the chemical pretreatment for flotation and the chemical pretreatment for conventional settling. This is evidenced by the examples cited in the literature where simple jar testing (essentially a simulation of settling) could accurately predict the optimum coagulant dosage for flotation. For the selection of the main coagulant, and determination of the optimum dosage, the designer could thus revert to conventional bench-scale testing with jar testing apparatus in lieu of tests with the more sophisticated bench flotation cells.

It may be advantageous to add a polymeric coagulant aid. For conventional settling, the coagulant aid serves to promote larger, heavier flocs which settle more readily. For flotation, the coagulant aids are rather advantageous from the viewpoint of float layer stability and compaction. This is not possible to test in a laboratory, but will only be evident during full-scale operation. Where bench tests do indicate a light, fragile floc structure after primary coagulation, it may be warranted to design for a coagulant aid feed point should it prove necessary.

Chemical dosing facilities should be designed as for conventional settling with the emphasis on an even stream of coagulant which is rapidly and thoroughly dispersed in the raw water stream.

Indications from the cited theory and practice are that flotation has less stringent flocculation requirements than conventional settling. Provided that the water is properly dosed, only a minimal amount of flocculation is required to get the floc to a certain minimum size. Flocculation beyond this point does not impair the flotation efficiency, but does not improve it either. The flocculation requirements for surface water clarification are summarized in Table 5.2.

TABLE 5.2 : FLOCCULATION REQUIREMENTS FOR CLARIFICATION

			MINIMUM	MEDIAN	MAXIMUM
Velocity gradient	(SA survey)	s^{-1}	45	130	150
Velocity gradient	(recommended)	s^{-1}	50	-	120
Flocculation time	(SA survey)	min	5	13	120
Flocculation time	(Finnish survey*)	min	20	-	127
Flocculation time	(recommended)	min	4	-	15**
Product GT	(SA survey)	-	33 000	58 000	189 000
Product GT	(recommended)	-	40 000	-	80 000

* Heinänen (1988)
** or longer if not for eutrophic water or municipal wastewater reclamation

Thickening

For the thickening of waste activated sludge, the use of a coagulant is optional. From the cited literature, it is evident that the addition of a coagulant (mostly polymers) has definite advantages in terms of float layer concentration as well as effluent quality. With polymer addition, it will also be possible to select a higher solids loading rate. The adoption of a coagulant dosing system obviously implies some capital expenditure, a continuing responsibility to operate and optimize the dosing system, and additional running costs for the plant. The decision to use coagulants or not should preferably be jointly made between the designer and the operating authority.

To determine the dosage requirements for design and cost estimation purposes, the designer could make use of jar testing to find the best coagulant type and to get an indication of the dosages required.

The coagulant can be added at a number of places:

- In the case of activated sludge thickening, the coagulant can be dosed in the aeration zone, well ahead of the DAF unit. In this case, the coagulant would typically be ferric chloride or aluminium sulphate, also added for phosphate precipitation.
- The coagulant can be dispersed into the raw water stream via flash mixers just ahead of the reaction zone.
- The coagulant can be dosed into the recycle stream, most conveniently on the suction side of the recycle pump. In this case, the coagulant would have to be a polymer to prevent fouling of the air saturator by ferric or aluminium hydroxide.

With thickening applications, the particle concentration is very high with ample contact opportunities between particles. For this reason, thickening applications do not require large or elaborate facilities for mixing and flocculation. In most cases, very little provision is made for flocculation other than just relying on the turbulence in the reaction zone to create the desired particle agglomerates.

In summary, the benefits of coagulant addition to a DAF thickening plant for waste activated sludge are threefold:

- Float layer solids concentration is improved from typically 3% to typically 4%.
- Suspended solids in the effluent can be reduced to about 10 mg.ℓ^{-1}.
- Solids loading rates can be increased to 10 kg.m^{-2}.h^{-1} or higher.

5.4 Bubble production system

Size and method of recycle

The designer must know the air requirements of the DAF application before the recycle system can be designed. A given air quantity can be supplied with a large recycle rate at low saturator pressure, or conversely with a small recycle rate at high saturator pressure. The interrelationships were earlier given by Equations 2.3 and 2.4. Table 5.3 shows the range of values encountered in South Africa for the main variables in the above formulas, as well recommended values for typical applications. The selection of the saturator efficiency will be discussed in the next paragraph. There are, additionally, two practical limits for the saturator pressure:

- An air saturator is an industrial pressure vessel and as such must comply with the requirements of the Factory Act.
- Below about 250 kPa, saturator efficiency drops off sharply (Bratby & Marais, 1975b), for reasons that are not altogether clear at present. This represents a practical lower limit which designers should heed.

The water for recycle could be drawn off directly from the bottom of the flotation tank, which is a simple, cheap option at those sites where there is no better water available. This option is generally used at thickening applications. If this option is exercised, the following implications must be borne in mind:

- The recycle will require repressurization to the saturator pressure.
- Unpacked saturators have to be used, since packing will be clogged by the residual fines in the recycle stream.
- Injection nozzles and fittings with fine orifices must be avoided to prevent blockage of the recycle pipe system.
- An ancillary water supply for recycle must be provided (site storage or potable hookup) to enable the DAF plant to start up. Once the plant is running, the ancillary supply can be shut off.

The other option for recycle draw-off, generally found at potable water treatment applications, is to use water *after* at least sand filtration to prevent fouling of the recycle system. In this case, packed saturators and micronozzles can be freely used. The designer has the further option to draw water directly from the site storage reservoir (which implies repressurization through separate recycle pumps) or from the high pressure side of the main high lift pumps (which will not require separate recycle pumps, provided the high lift pressure is the same or higher than the saturator pressure). In the first case, more capital is required but the recycle pumps are sized exactly right for optimum energy use. In the second case, provision must usually be made for a pressure controlled draw-off which would be cheaper, but could dissipate considerable electrical energy already paid for. The choice should be made on economic grounds.

TABLE 5.3 : DESIGN GUIDELINES FOR RECYCLE SYSTEMS

			MINIMUM	MEDIAN	MAXIMUM
CLARIFICATION					
Air requirement C_{air}	(SA survey)	$mg.l^{-1}$	4,5	7,9	9,9
Air requirement C_{air}	(British survey**)	$mg.l^{-1}$	7,0	-	10,0
Air requirement C_{air}	(recommended)	$mg.l^{-1}$	6,0	-	8,0
Recycle ratio r	(SA survey)	-	0,064	0,110	0,230
Recycle ratio r	(Dutch survey*)	-	0,070	0,073	0,075
Recycle ratio r	(British survey**)	-	0,060	-	0,100
Recycle ratio r	(Finnish survey#)	-	0,056	0,100	0,420
Recycle ratio r'	(recommended)	-	0,06	-	0,10
Saturation pressure P	(SA survey)	kPa	260	430	610
Saturation pressure P	(Dutch survey*)	kPa	700	750	750
Saturation pressure P	(British survey**)	kPa	310		830
Saturation pressure P	(Finnish survey#)	kPa	300	580	750
Saturation pressure P	(recommended)	kPa	300	-	600
THICKENING					
Air requirement a_s	(SA survey)	-	0,004	0,024	0,037
Air requirement a_s	(recommended)	-	0,02	-	0,04
Recycle ratio r	(SA survey)	-	0,95	1,542	2,381
Recycle ratio r	(recommended#)	-	0,5	-	2,0
Saturation pressure P	(SA survey)	kPa	290	500	600
Saturation pressure P	(USA survey##)	kPa	414	-	483
Saturation pressure P	(recommended)	kPa	400	-	600

* Kolpa / Wortel
** Longhurst & Graham (1987)
\# Heinänen (1988)
\#\# Komline (1976)
' should be calculated for every case

A good case can be made out for fitting the recycle pumps with variable speed drives. In this way, the operators can change the flow rate as well as the saturator pressure by simply adjusting the pump speed. The air can thus be "dosed" in this manner in the same way as other treatment chemicals. As a large proportion of DAF running costs is ascribed to energy consumption, the operators can therefore trim the air dosage to the minimum. This will be especially advantageous if the recycle pumps were conservatively sized, or if the plant flow deviates considerably from the full design flow.

Saturators

The saturator efficiency η_s plays an important role in determining the recycle rate and is difficult to estimate. Saturator efficiency is a function of many variables and no verified mathematical model is generally used. The efficiency is also not constant for a given saturator; it obviously varies with hydraulic loading, and preliminary investigation also showed that it is strongly affected by temperature (Rykaart, 1991). Empirical design guidelines for saturators make little reference to the efficiency achieved by these guidelines, except to say that an unpacked saturator must be operated at a pressure of about 200 kPa higher than a packed saturator to achieve the same efficiency (Zabel, 1978). If a packed saturator is 75% efficient at 400 kPa, for example, an unpacked saturator then has to be 50% efficient, if this rule applies. Practical measurements have been made by one of the authors on a few full-scale packed saturators, and measured values do indeed differ quite widely. A conservative general estimate of 75% is suggested for packed saturators, and 50% to 60% for unpacked saturators without internal recycle. With internal recycle, the efficiency of the latter obviously goes up depending on the internal recycle rate. Almost no published data is available on the efficiency of unpacked saturators with internal recycle. If this option is chosen, the designer should opt for a performance specification for the efficiency of the saturator.

The principal design variables for *packed* saturators are the hydraulic loading, the packing depth and the packing type. The packing is generally manufactured from plastic with nominal size between 25 mm and 50 mm. The practically encountered values for packing depth and hydraulic loading are shown in Table 5.4 along with recommended values for design.

Important design variables for *unpacked* saturators are considered to be the hydraulic loading, the retention time and the water depth in the saturator. These values with recommendations are also shown in Table 5.4. Another design decision in this case is whether the saturator should stand upright or whether it should lie on its side. An unpacked saturator on its side would require less headroom and would also have a larger air/water interface inside the saturator if the water level is near its centre. In principle, however, the saturator orientation does not affect its operation.

The designer needs to consider two important practical points. The first deals with the *material of construction*. Water with a high air concentration is corrosive towards mild steel, and chrome steel should be considered. The second deals with the *optimum size/number* of saturators. The number of saturators is normally dictated by the site layout. The optimum size of saturators would probably be decided on the basis of cost. The larger the saturator, the greater the care that should be taken to ensure even flow distribution and to prevent vortex formation at the outlet.

It is recommended that a level control system is used which results in an even recycle stream with constant air concentration. For this reason, it is better to regulate the air flow into the saturator than to regulate the water flow rate. It seems to be the simplest to keep the water flow rate constant and to simply stop/start the air flow to maintain the water level within the desired control band.

TABLE 5.4 : DESIGN GUIDELINES FOR SATURATORS

			MINIMUM	MEDIAN	MAXIMUM
PACKED SATURATORS					
Hydraulic loading V_s	(SA survey)	$m.h^{-1}$	18	67	103
Hydraulic loading V_s	(recommended)	$m.h^{-1}$	50	-	80
Packing depth H_p	(SA survey)	m	0,8	1,1	3,0
Packing depth H_p	(recommended)	m	0,8	-	1,2
Water depth	(SA survey)	%	15	23	35
Water depth	(recommended)	%	15	-	25
UNPACKED VERTICAL SATURATORS WITHOUT INTERNAL RECYCLE					
Hydraulic loading (SA survey)		$m.h^{-1}$	16	50	69
Hydraulic loading (recommended)		$m.h^{-1}$	20	-	60
Retention time (SA survey)		s	16	32	147
Retention time (recommended)*		s	20	-	60
Water depth (SA survey)		%	25%	50%	75%
Water depth (recommended)		%	25%	-	50%
UNPACKED HORIZONTAL SATURATORS WITHOUT INTERNAL RECYCLE					
Hydraulic loading (SA survey		$m.h^{-1}$	7	13	19
Hydraulic loading* (recommended)		$m.h^{-1}$	10	-	30
Retention time (SA survey)		s	99	123	146
Retention time* (recommended)		s	100	-	150
Water depth (SA survey)		%	50%	63%	75%
Water depth* (recommended)		%	50%	-	60%
UNPACKED HORIZONTAL SATURATORS WITH INTERNAL RECYCLE					
Hydraulic loading (SA survey)		$m.h^{-1}$	20	30	97
Hydraulic loading* (recommended)		$m.h^{-1}$	20	-	80
Retention time (SA survey)		s	7	22	36
Retention time* (recommended)		s	10	-	40
Water depth (SA Survey)		%	70%	75%	80%
Water depth* (recommended)		%	70%	-	80%

* tentative guideline due to very little available data

Injection nozzles

The injection nozzles have to match the saturator pressure and recycle flow exactly, two parameters which normally have been chosen by the time the nozzle selection or nozzle design has to be done. Two general nozzle types can be used, namely *fixed* nozzles or *adjustable* nozzles. With adjustable nozzles, the final setting of the nozzles can be done after commissioning to ensure the exact combination of recycle rate and pressure drop. With fixed nozzles, the designer must know the exact hydraulic characteristics in order to specify the correct number of nozzles.

For the typical pressure drop across DAF injection nozzles, the water velocity through the nozzle orifice has to be in the region of about 20 $m.s^{-1}$. At this velocity, the erosion by the water jet can be considerable if improper materials are specified for the nozzle, or if an unsuitable valve is adopted as an adjustable nozzle.

The designer has the option to design injection nozzles from first principles (of which a simple example is given in the next chapter), but there are numerous rules of thumb to ensure bubbles of desired size and narrow distribution which makes nozzle design an art as much as a science. The other option is to revert to any of the numerous proprietary nozzle designs used by the major water treatment companies, which will then have to provide the hydraulic data to enable a rational design.

5.5 Flotation zone

The number of flotation units has to be determined first. This is mostly a practical consideration, which has to take into account the likely flow variation, space constraints on the site, modular layout of the treatment plant to facilitate phased construction, *et cetera*. There is an upper limit to the size of individual flotation units - this was set at about 420 $m^3.h^{-1}$ (De Wet; 1980) and 750 $m^3.h^{-1}$ (Zabel & Melborne, 1980) for clarification. For clarification, a maximum size of 750 $m^3.h^{-1}$ is therefore recommended. No guidelines exist for thickening - in this case the practical constraint will be the maximum length of the float layer scrapers.

The shape of the flotation unit is largely determined by the configuration of the reaction zone, which was earlier discussed in Section 5.2.

Clarification

The important parameters for clarification are the crossflow velocity between reaction and flotation zones, the hydraulic loading in the flotation zone and the side depth of the flotation tank. These parameters, with recommendations, are shown in Table 5.5. Note that the cross-flow velocity and hydraulic loading must be calculated on the basis of *total* flow, which includes the recycle.

Thickening

The important parameters for thickening are the crossflow velocity between reaction and flotation zones, the solids loading in the flotation zone, the depth of the float layer and the side depth of the flotation tank. These parameters, with recommendations, are shown in Table 5.5. Note that the crossflow velocity must be calculated on the basis of *total* flow, it is the raw water flow plus the recycle.

Outlet design

A poorly designed outlet could set up a recirculation pattern in the flotation zone, as was indicated in the literature review. In larger flotation tanks, it is recommended that subnatant is not only drawn off at a single point at the far end of the end, but that multiple draw-off points are provided which are evenly distributed over the entire flotation area. This is easily done by providing a few horizontal collection pipes with multiple orifices along the full width of the tank. This particularly applies to rectangular tanks which have a shorter draw-off length in comparison with circular tanks.

It will be frequently required to manipulate the water level in the flotation tank, for example when the float layer must be flushed or scraped. In these cases, the designer should take care to ensure that the water level can be easily and quickly adjusted. At small plants, it can be conveniently done with a pipe overflow with an adjustable sleeve. At larger plants, it may be necessary to use an adjustable rectangular weir.

5.6 Float layer removal

The different options for float layer removal are float layer flushing, float layer scraping with a float layer stabilizing grid, and float layer scraping without a float layer stabilizing grid. The choice amongst these systems is determined by the sludge management system adopted for the entire treatment plant, as was discussed in the literature review.

Float layer stabilizing grids

The survey results included only two float layer stabilizing grids, which have grid openings of 150 mm x 150 mm and 200 mm x 200 mm respectively. These work satisfactorily and provide some guideline to designers. The depth of the grids and the thickness and material of the plate must be sufficient to withstand the float layer uplift force to prevent jamming of the scraper.

The literature review indicated that the grids work best if the water level in the tank is maintained at about 5 to 10 mm below the top of the grids. The designer should allow for water level manipulation around this level.

Float layer scraping

The three variables to be fixed by the designer are:

- the number of scrapers, or scraper spacing;
- the speed of the scrapers; and
- whether to scrape continuously or intermittent.

There are also very important practical scraper details in terms of shape, scraper depth, scraping angle, material of construction, beach plates and discharge troughs which will affect the scraping efficiency. These aspects fall outside the scope of this manual.

TABLE 5.5 : DESIGN GUIDELINES FOR FLOTATION ZONE

			MINIMUM	MEDIAN	MAXIMUM
CLARIFICATION					
Crossflow velocity	(SA survey)	$m.h^{-1}$	14	38	210
Crossflow velocity	(recommended)	$m.h^{-1}$	20	-	100
Hydraulic loading	(SA survey)	$m.h^{-1}$	2,4	5,2	8,2
Hydraulic loading	(Finnish survey)	$m.h^{-1}$	2,5	4,8	8,0
Hydraulic loading	(British survey)	$m.h^{-1}$		4,2*	11,0
Hydraulic loading	(recommended)	$m.h^{-1}$	5,0	-	11,0
Side depth	(SA survey)	m	1,5	2,2	3,5
Side depth	(British survey)	m	1,0	2,4*	3,2
Side depth	(recommended)	m	1,5	-	3,0
THICKENING					
Crossflow velocity	(SA survey)	$m.h^{-1}$	31	91	1994
Crossflow velocity	(recommended)	$m.h^{-1}$	50	-	200
Side depth	(SA survey)	m	1,5	3,4	4,5
Side depth	(recommended)	m	2,0	-	4,0
Without coagulants					
Solids loading	(SA survey)	$kg.m^{-2}.h^{-1}$	2,9	9,7	13,9
Solids loading	(USA survey)	$kg.m^{-2}.h^{-1}$	4,9	6,1	7,3
Solids loading	(recommended)	$kg.m^{-2}.h^{-1}$	2	-	6
With coagulants					
Solids loading	(SA survey)	$kg.m^{-2}.h^{-1}$	1,4	4,6	11,2
Solids loading	(USA survey)	$kg.m^{-2}.h^{-1}$	10,3	15,6	34,1
Solids loading	(recommended)	$kg.m^{-2}.h^{-1}$	6	-	12

* Longhurst & Graham (1987), based on flotation and reaction areas
\+ Mean value
** Heinänen (1988), based on flotation and reaction areas

A mass balance equation for the solids in the flotation zone was given as Equation 2.11 in the literature review, with which the number of scrapers and scraping speed can be estimated. In the case of thickening, where large sludge volumes must be removed, scrapers normally operate continuously. Where a large variation in sludge volume is expected, the scraper could be fitted with a variable-speed control. In the case of clarification, where sludge volumes are relatively small, operation is usually intermittent. Here designers should provide timers for the on-off cycle which can be easily adjustable by the operators to control the depth of the float layer. Table 5.6 provides some guidelines.

Bottom scraping

Very little quantitative data are available on the real need for bottom scraping. Some plants are drained and cleaned at frequent intervals as the normal operation routine; in such cases bottom scraping is probably not justified. For thickening, where much larger volumes of sludge pass through a tank, there is better justification for providing bottom scrapers. For clarification the sludge volumes are very low and bottom scraping not so important.

The results of the survey reflect a similar philosophy amongst designers of SA plants. Of the 12 clarification plants (excluding the DAFF plants where bottom scraping is not possible), only 2 (17%) have provision for bottom scraping. Of the 12 thickening plants surveyed, 8 (67%) have provision for bottom scraping.

For unscraped tanks, hoppered bottoms or partially hopppered bottoms will probably be considered. For scraped tanks, it is recommended that the bottom of the flotation tanks are well sloped and drained for easy rapid drainage and cleaning.

TABLE 5.6 : DESIGN GUIDELINES FOR FLOAT SCRAPING EQUIPMENT

			MINIMUM	MEDIAN	MAXIMUM
RADIAL SCRAPERS (for circular tanks)					
Number of scrapers	(SA survey)	-	2	3	10
Scraper speed	(SA survey)	$rev.h^{-1}$	2.0	5.0	8.2
Tip speed	(SA survey)	$m.h^{-1}$	63	160	290
Tip speed	(recommended)	$m.h^{-1}$	50	-	200
LINEAR SCRAPERS (rectangular tanks)					
Scraper spacing	(SA survey)	m	1,3	2,3	7,0
Scraper speed	(SA survey)	$m.h^{-1}$	40	72	200
Scraper speed	(British survey*)	$m.h^{-1}$	18	-	110
Scraper speed	(recommended)	$m.h^{-1}$	20	-	100

* Ashman (1976)

CHAPTER SIX

DESIGN EXAMPLES

6.1 Introduction

The purpose of this chapter is to illustrate the application of the theoretical principles as well as the empirical data and recommendations covered in the earlier chapters.

The results of this survey have been presented as practically measured parameters, from which a number of empirical design guidelines have been recommended. These parameters, in the opinion of the authors, could be used by designers, provided that the applications are fairly typical of clarification or thickening. The first part of this chapter is consequently devoted to a design for clarification (Section 6.2) and a design for thickening (Section 6.3) based on the empirical guidelines presented earlier.

Two other design approaches, based on more fundamental principles, are also illustrated for those cases where such data may be available. They are the methods of Bratby (Section 6.4) and Gulas (Section 6.5).

Finally, typical calculations are included to illustrate two of the practical hydraulic aspects of DAF. They deal with a typical nozzle sizing problem (Section 6.6) and the interrelationship between the recycle rate, saturator pressure and air dosing (Section 6.7).

The typical design sequence, as will be followed in the following sequence, is summarized in Table 6.1.

6.2 Empirical design of a DAF plant for clarification

Initial parameters

A DAF plant must be designed to clarify chemically flocculated eutrophic surface water. The maximum suspended solids concentration, including treatment chemicals, is 20 mg.ℓ^{-1} and the average suspended solids concentration is 6 mg.ℓ^{-1}. The effluent will be filtered and chlorinated for potable use. The maximum flow rate will be 800 m^3.h^{-1}, and the maximum temperature is expected to be 24°C.

Choose two rectangular DAF units, each with production capacity of 400 m^3.h^{-1}. The sludge has to be landfilled and float layer stabilization grids will be required. The recycle will be drawn from the potable ground storage on site and will have to be repumped through the recycle system.

Table 6.1 : TYPICAL DESIGN SEQUENCE

BACKGROUND INFORMATION, PROJECTIONS AND ASSUMPTIONS
Design flow rate Suspended solid concentration and characteristics Requirements for float layer concentration and subnatant quality Number and shape of units
BUBBLE PRODUCTION SYSTEM
Air requirements Saturator pressure Recycle ratio Saturator sizing Recycle pumps and air compressors Injection nozzles
REACTION / FLOTATION ZONE
Shape / sizing of reaction zone Shape / sizing of flotation zone Side depth Crossflow velocity Draw-off system Level control
FLOAT LAYER REMOVAL
Removal method Estimate volume/concentration Sizing / speed of scraping (if applicable) Sludge discharge system Hoppers / bottom scraping

Recycle flow rate

This is a typical clarification application where the air requirement is expressed as the volumetric concentration based on the raw water flow. Choose a minimum air concentration of 8 mg/ℓ. The solubility constant for saturator air at 24°C is:

$$S_{24} = 0.219 \left(\frac{293}{273+24}\right) 10^{[illegible]} = 0.205\ mg\ \ell^{-1}\ kPa^{-1}$$

Choose a maximum saturator pressure of 500 kPa. As the recycle will be potable water, a packed saturator with assumed efficiency of 75% will be used. The recycle ratio is thus calculated with Equation 2.3 as

$$r = \frac{8}{0.205 \cdot 500 \cdot 0.75} = 0.104$$

The recycle flow rate will therefore be

$$r.Q \;=\; 0{,}104\,.\,800 \;=\; 83\ m^3.h^{-1}$$

and the total flow rate

$$Q\,.\,(1+r) \;=\; 800 + 83 \;=\; 883\ m^3.h^{-1}$$

Saturator

Choose one packed saturator with a maximum hydraulic loading of 80 m.h^{-1}. The saturator cross-sectional area is:

$$A_s \;=\; \frac{83}{80} \;=\; 1{,}04\ m^2$$

and the saturator diameter:

$$D_s \;=\; \sqrt{\frac{4\,.\,1{,}04}{\pi}} \;=\; 1{,}15\ m$$

Choose a packing depth of 1 000 mm with plastic packing between 25 mm and 50 mm, and maintain the water depth just above the bottom of the packing.

Reaction/flotation zone

Choose a rectangular flotation tank with length/width ratio of 2,0, with a layout as shown in Figure 6.1

The designer's attention is again drawn to the fact that the exact boundaries of the reaction zone are unknown and that they have to be arbitrarily assigned. With a tank configuration as shown in Figure 6.1, the reaction zone (for the sake of the plant survey and the design recommendations) was assumed to enclose the water volume from the floor level all the way to the underside of the grid. When applying these recommendations, the same definition has therefore to be used

Choose the hydraulic loading for the reaction zone 60 m.h^{-1}, the crossflow velocity 100 m.h^{-1} and the hydraulic loading for the flotation zone 8 m.h^{-1}. Choose the side depth for flotation to be 2 200 mm from the bottom of the tank to the underside of the float layer stabilizing grid. The area of the flotation zone is therefore (there are two tanks) :

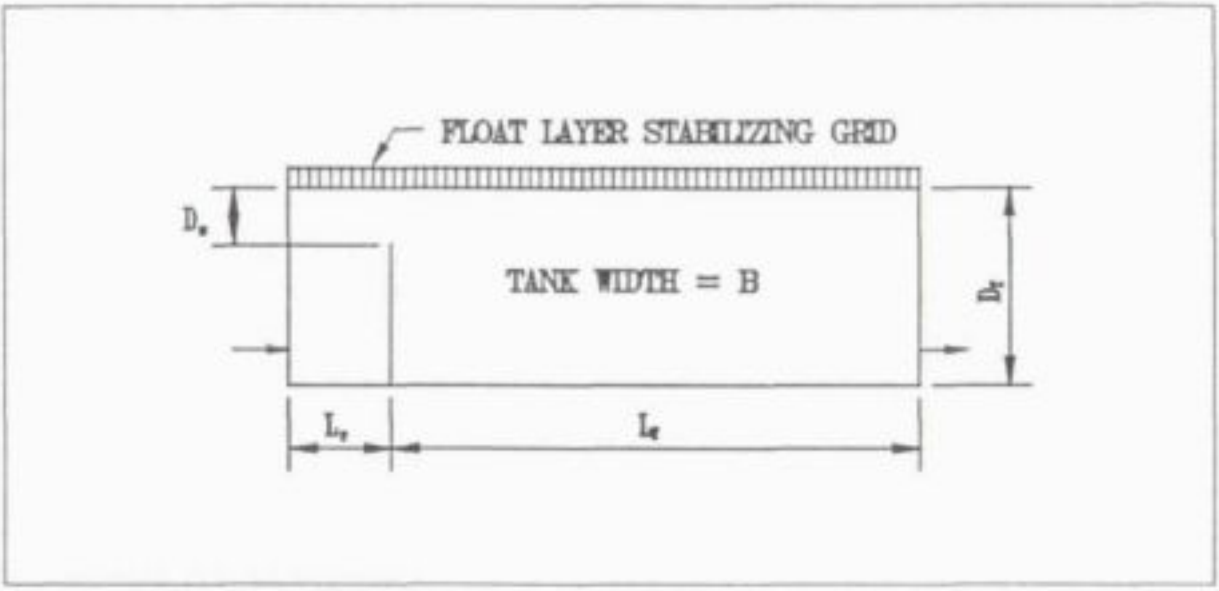


Figure 6.1 : Reaction/flotation zone configuration for empirical design example for clarification.

$$A_f = \frac{883}{2.8.0} = 55 \text{ m}^2.\text{tank}^{-1}$$

The L/W ratio = 2. The dimensions are thus :

$$B = \sqrt{\frac{55}{2}} = 5{,}24 \text{ m}$$

and

$$L_f = 2.5{,}24 = 10{,}5 \text{ m}$$

The clearance between the top of the division wall and the underside of the float layer stabilizing grid is determined by the crossflow velocity:

$$D_c = \frac{883}{2.100.5{,}24} = 0{,}84 \text{ m}$$

The reaction zone width is:

$$L_r = \frac{883}{2.60.5{,}24} = 1{,}40 \text{ m}$$

Check the retention time in the reaction zone:

$$t = \frac{2{,}2 . 5{,}24 . 1{,}40 . 60 . 2}{883} = 132 \text{ s}$$

Float layer removal

It was earlier determined that a float layer stabilizing grid will be used. Choose grid openings of 200 mm by 150 mm, and make the grid 200 mm deep. Estimate the volume of sludge that must be removed under average operating conditions:

$$q = \frac{Q}{C_F} . (SS_{in} - SS_{out}) . 10^{-4}$$

With the assumptions that $C_F = 5\%$ and $SS_{out} \approx 0$:

$$q = \frac{800}{5} (6 - 0) . 10^{-4} = 0{,}096 \text{ m}^3.\text{h}^{-1} = 2{,}3 \text{ m}^3.\text{d}^{-1}$$

Summary sketches

The main design parameters are shown in Figure 6.2.

6.3 Empirical design of a DAF plant for thickening

Initial parameters

Sewage is treated at an activated sludge plant where the mixed liquor has a suspended solids concentration of 2 500 $\text{mg}.\ell^{-1}$. The maximum saturator pressure is 500 kPa and the recycle is drawn directly from the flotation zone. The float layer sludge drains to a secondary sludge handling facility. The raw water flow rate is 10 $\text{m}^3.\text{h}^{-1}$.

Recycle flow rate

This is a typical thickening application where the air requirement is expressed as the air/solids ratio a_s. For activated sludge, choose a minimum a_s of 0.04. The solubility constant for saturator air at 25°C is

$$S_{air} = 0.219 . \left(\frac{293}{273+25}\right) 10^{[illegible]} = 0.202 \text{ mg } \ell^{-1} . \text{kPa}^{-1}$$

As the recycle is drawn unfiltered from the flotation zone, an unpacked saturator has to be used. Omit internal recycling and assume a saturator efficiency of 60%. The recycle ratio is thus:

The recycle flow rate will then be:

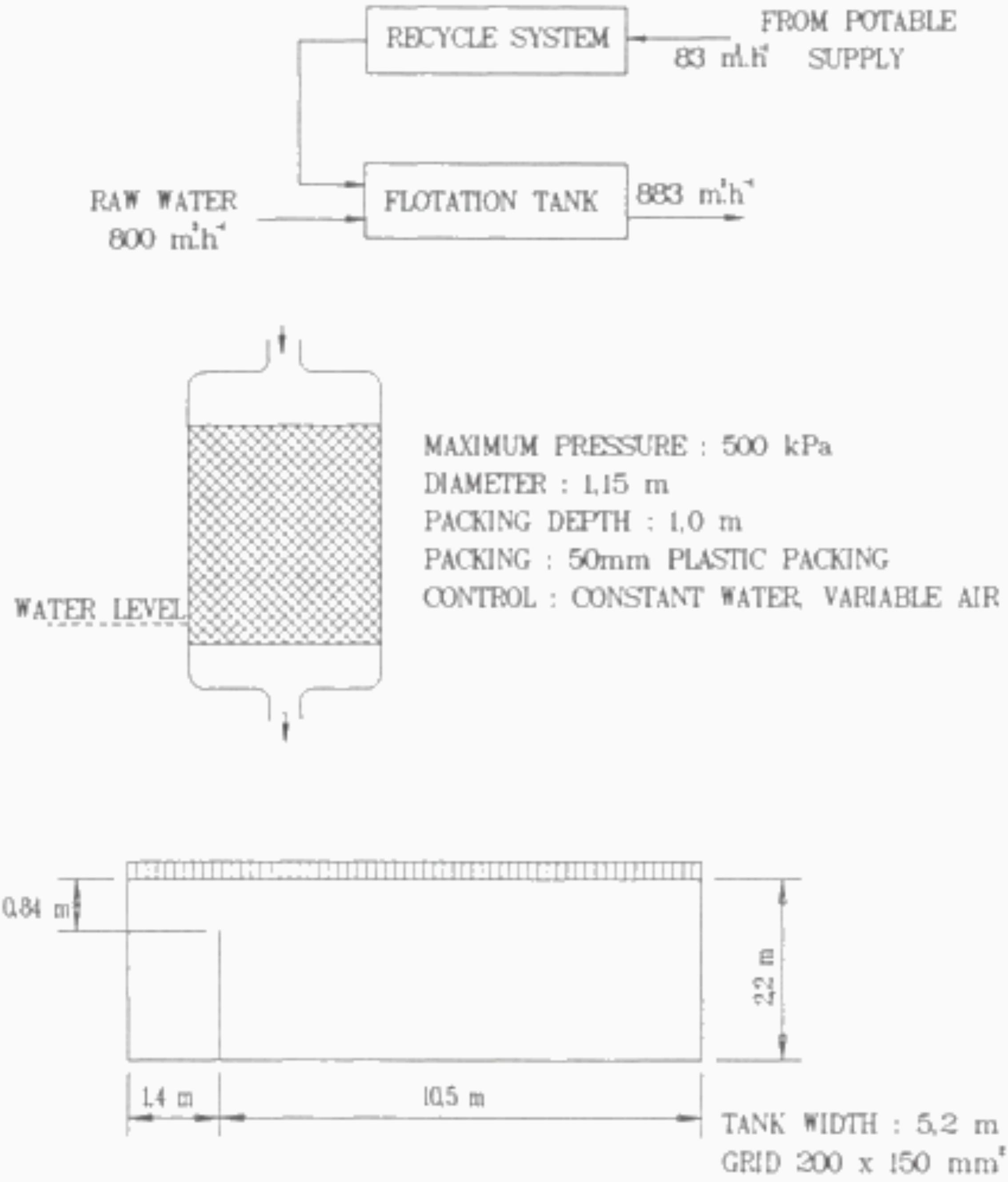


Figure 6.2 Summary sketches for the empirical design example for clarification

$$r = \frac{0{,}04 \, . \, 2500}{0{,}202 \; 500 \, . \, 0{,}60} = 1{,}65$$

$$r . Q = 1.65 \, . \, 10 = 16{,}5 \ m^3 . h^{-1}$$

and the total flow rate:

$$Q . (1+r) = 10 + 16.5 = 26.5 \ m^3 . h^{-1}$$

Saturator

Choose one vertical, unpacked saturator without internal recycle, operated at a hydraulic loading of 50 $m.h^{-1}$. The saturator cross-sectional area is:

$$A_{sv} = \frac{16{,}5}{50} = 0{,}33 \ m^2$$

and the saturator diameter:

$$D_s = \sqrt{\frac{4 \, . \, 0{,}33}{\pi}} = 0{,}648 \ m$$

Allow for 60 seconds mean retention time in the saturator. The water volume is thus:

volume = 16.5 (60 / 3600) = 0.28 m^3

The water depth is then:

depth = 0,28 / 0,33 = 0,83 m

If the water level is to be maintained at 35% of the total saturator height, the total saturator height is:

$$L_s = \frac{0{,}83}{0{,}35} = 2{,}4 \ m$$

Reaction/flotation zone

Choose a circular tank with the following internal layout

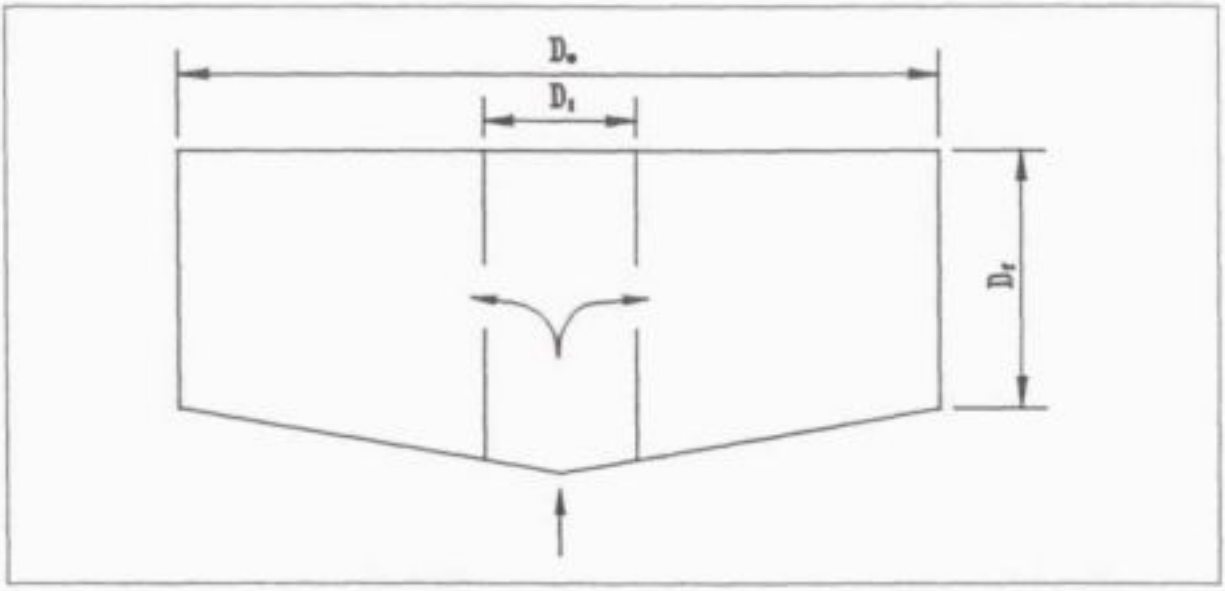


Figure 6.3 : Reaction/flotation zone configuration for empirical design example for thickening.

Choose the hydraulic loading for the reaction zone 100 $m.h^{-1}$, the reaction zone residence time 60 s and the crossflow velocity 100 $m.h^{-1}$. The solids loading for the flotation zone is 5 $kg.m^{-2}.h^{-1}$. Assume a maximum float layer thickness of 600 mm.

The reaction zone diameter is calculated as:

$$D_i = \sqrt{\frac{4.26,5}{\pi.100}} = 0,581 \text{ m}$$

The reaction zone area is :

$$A_r = \frac{\pi.0,581^2}{4} = 0,265 \text{ m}^2$$

and the reaction zone depth is:

$$D_r = \frac{26,5}{0,265} \cdot \frac{60}{3600} = 1,67 \text{ m}$$

The crossflow area is:

$$A_c = \frac{26,5}{100} = 0,265 \text{ m}^2$$

and the crossflow depth :

$$\text{depth} = \frac{0{,}265}{\pi \, . \, 0{,}581} = 0{,}145 \text{ m}$$

The flotation zone area must be:

$$A_f = \frac{26{,}5 \, . \, 2500}{1000 \, . \, 5} = 13{,}25 \text{ m}^2$$

and the outside tank diameter:

$$D_o = \sqrt{\frac{4 \, . \, (0{,}265 + 13{,}25)}{\pi}} = 4{,}15 \text{ m}$$

Allow 0.5 m extra depth for bottom sludge storage. The total tank depth in the centre is then the sum of the following components:

D_t	=	float layer + crossflow area + reaction zone + bottom storage
	=	0,600 + 0,145 + 1,667 + 0,500 = 2,912 m

Float layer removal

The flotation effluent will typically contain suspended solids of about 50 mg ℓ^{-1}, and the float layer will have a concentration of about 4%. The float layer sludge volume can thus be estimated as:

$$q = \frac{Q}{C_f} \, (SS_{in} - SS_{out}) \, . \, 10^{-4}$$

$$= \frac{10}{4} \, (2500 - 50) \, . \, 10^{-4} = 0{,}61 \text{ m}^3 \, . \, h^{-1} = 14{,}7 \text{ m}^3 \, . \, d^{-1}$$

Summary sketches

The main design parameters are summarized in Figure 6.4

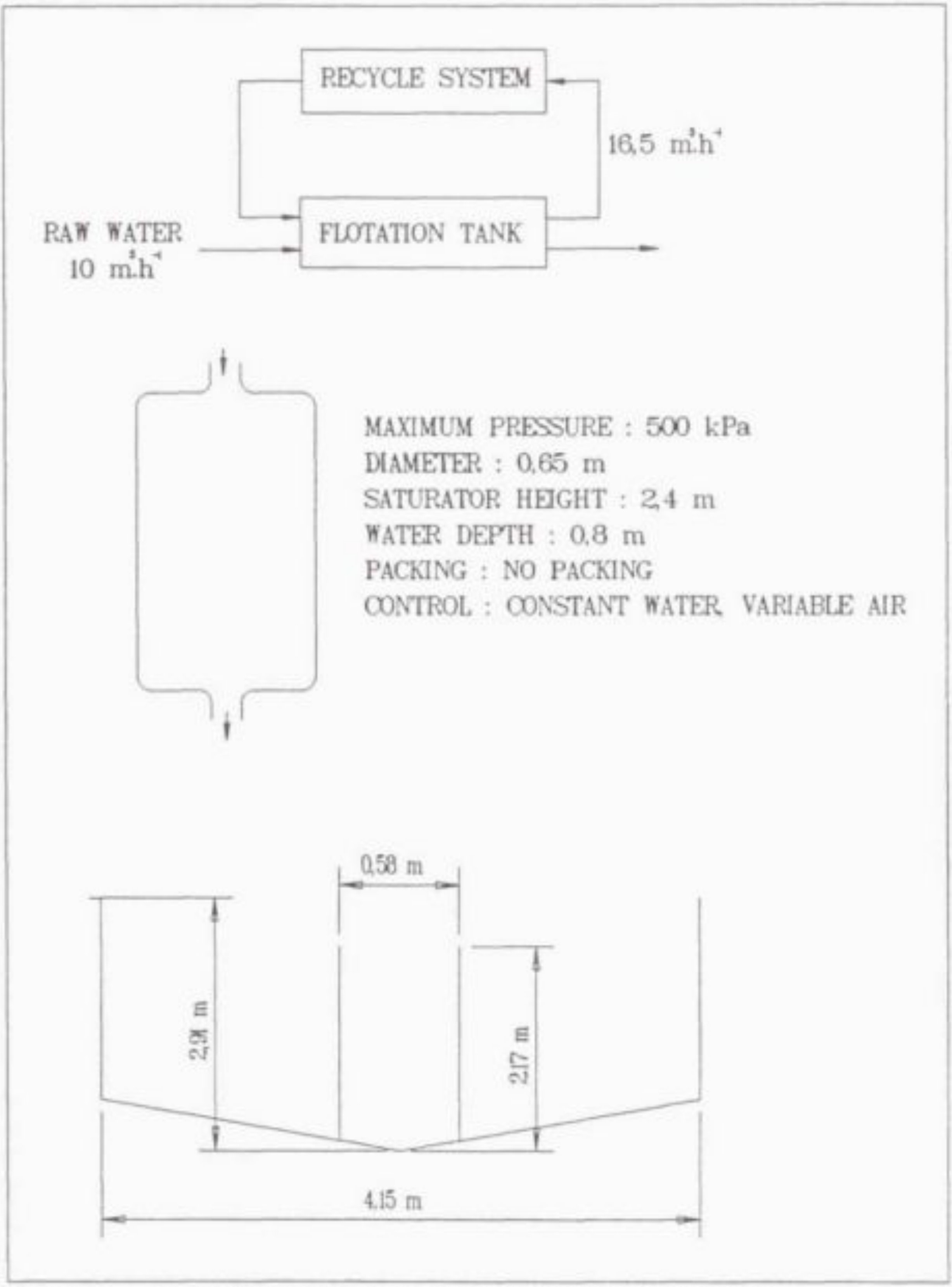


Figure 6.4 : Summary sketches for the empirical design example for thickening.

6.4 The Bratby design approach

Bratby & Marais developed a comprehensive design method for sludge thickening, published in numerous papers, e.g. Bratby & Marais (1975a), Bratby & Marais (1976a) and Bratby (1978). This was based on:

- fundamental considerations and relationships,
- experimentally determined design parameters, and
- a cost analysis to find optimum values for key parameters

Their illustration of the method necessarily included some of the empirical relationships which had already been illustrated in Sections 6.2 and 6.3. In fact, much of the empirical saturator design criteria, for example, had originated from their research. In this section, however, only those parts of the method will be highlighted which are substantially different from the empirical examples given before.

The Bratby design method will be illustrated with the same design problem that was empirically solved in Section 6.3. An additional stipulation is that a float layer concentration of 4% is required.

The optimum air/solids ratio a_r is determined first from the suspended solids concentration in the raw water:

$$\text{Optimum } a_r = \frac{0{,}15}{SS_{in}^{0{,}36}} = \frac{0{,}15}{2500^{0{,}36}} = 0{,}00897$$

The rational assumption is then made that the float layer thickness should be considered in two parts, namely the depth above the water line, and the depth below the water line. The depth above the water line is critical for the float layer concentration, for it affords the interstitial water the opportunity to drain away from the top of the float layer. The depth of the float layer above the water line is then calculated from the required float layer concentration.

$$d_A = (0{,}025 . C_F + 0{,}19) = (0{,}025 . 4 + 0{,}19) = 0{,}290 \text{ m}$$

The solids loading rate Q_s is then calculated from the depth of the float layer above the water line and the required float layer concentration:

$$Q_s = \left(\frac{31{,}74\ d_A^{\ldots}}{C_F}\right)^{\ldots} = \left(\frac{31{,}74\ 0{,}290^{\ldots}}{4}\right)^{\ldots} = 38{,}4 \text{ kg m}^{-2} \text{ d}^{-1}$$

$$= 1{,}60 \text{ kg.m}^{-2}.\text{h}^{-1}$$

From here, it is a simple matter to calculate the required area for thickening:

$$A_t = \frac{SS_r \cdot Q}{Q_t} = \frac{2500 \cdot 10}{1{,}60 \cdot 1000} = 15{,}6\ m^2$$

This area is checked by also calculating the required area for clarification. The recycle flow rate is calculated for the specific saturation system with Equation 2 2

$$S_{25} = 0.202\ mg.\ell^{-1}.kPa^{-1} \text{ from Section 6.3}$$

$$r = \frac{a_r \cdot SS_r}{S_{25} \cdot P \cdot \eta_s} = \frac{0.0089 \cdot 2500}{0.202 \cdot 500 \cdot 0.60} = 0{,}37$$

$$Q\,(1 + r) = 10\,.(1 + 0{,}37) = 13.7\ m^3\,h^{-1}$$

The relationship between the a_r ratio and the limiting downflow rate V_L must be known to calculate the limiting downflow rate V_L. This relationship, which was discussed in Chapter 2, has to be uniquely determined for every sludge that has to be thickened. Using average values for activated sludge, the relationship is:

$$V_L = 5544 . a_r^{0.87} - 36$$

In this case, where the air/solids ratio had already been fixed at 0,0089, the limiting downflow rate is thus:

$$V_L = 5544 \cdot 0{,}0089^{0.87} - 36 = 55{,}8\ m.d^{-1} = 2.32\ m.h^{-1}$$

The area required for clarification including a recommended safety factor of 1.25, is thus:

$$A = \frac{13\ 7 \times 1{,}25}{2{,}32} = 7{,}38\ m^2$$

This is substantially lower than the area required for thickening. The area required for thickening must therefore be used.

As a final step, the total depth of the float layer is calculated from the a_s ratio and the depth of the float layer above the water line:

$$d_s + d_w = \frac{d_w\ (a_s^{0,45} + 0,76)}{a_s^{0,45}} = \frac{0,290\ (0,00897^{0,45} + 0,76)}{0,00897^{0,45}} = 2,13\ \text{m}$$

The total side depth must allow for this float layer thickness as well as a recommended clarification depth of 2,0 m. The final side depth is thus:

side depth = 2,13 (float layer) + 2,0 (clarification) = 4,13 m

Comments

- The calculated a_s of 0,0089 is significantly lower than the minimum value of 0,02 normally proposed in the literature. Bratby & Marais also suggested a minimum value of 0,02 for activated sludge.
- The calculated solids loading rate Q_s 1,60 $kg.m^{-2}.h^{-1}$ is much lower than the empirical values of 1,6 to 14,0 $kg.m^{-2}.h^{-1}$ found in the SA survey, or in British (Ashman, 1976) and US (Komline, 1976) surveys reported earlier.
- The calculated total depth of the float layer of 2,13 m is far in excess of typical US values of 0,2 to 0,6 m (Komline, 1976) or British values of 0,25 to 0,3 m (Ashman, 1976). In the latter case, it is suggested that designers allow for a maximum tolerance of 0,15 to 0,6 m.
- The unique feature of the Bratby/Marais method is that it allows the required float layer concentration to be incorporated in the design process, which empirical methods cannot do. To illustrate the previous two comments, the important design parameters have been calculated for different float layer concentrations, and are shown in Figure 6.5. In practice, float layer concentrations of about 3,5% is achieved without coagulants. At 3,5%, a solids loading of about 2 $kg.m^{-2}.h^{-1}$ and total float layer depth of about 2 m is theoretically required - very much different from practically experienced values.

6.5 The Gulas design approach

The Gulas design approach is described in Gulas and Lindsey (1978) and in Gulas *et al.* (1980). This approach deals specifically with the thickening of waste activated sludge. It requires the experimental determination of three relationships:

- The maximum float layer concentration *versus* the sludge volume index SVI (here called Graph 1).
- A family of curves of the float layer concentration *versus* the feed solids concentration, determined at a number of different a_s ratios (here called Graph 2).
- The limiting downflow rate V_∞ *versus* the a_s ratio (here called Graph 3).

The method starts out by assembling extensive background on the seasonal variation of sludge age, sludge wastage rate, temperature and sludge volume index to determine the critical summer and winter conditions. The design itself is carried out for both the critical summer and winter conditions, and the larger flotation area is finally selected. The calculation steps are:

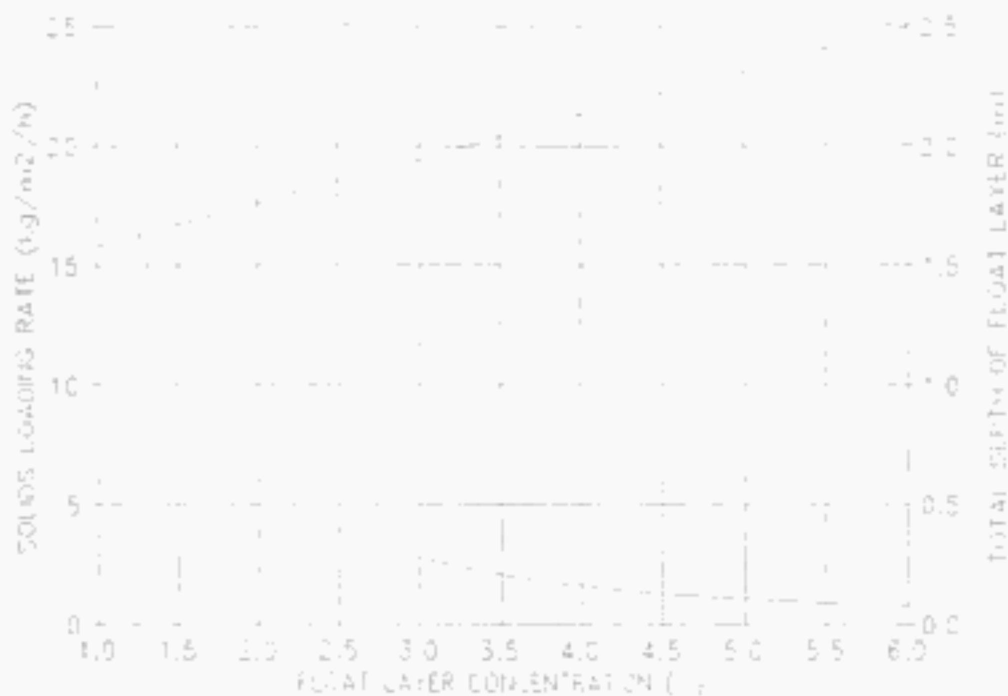


Figure 6.5 : Solids loading rate and total float layer depth as a function of float layer concentration, as calculated by the Bratby/Marais design method for typical activated sludge.

- The maximum attainable float layer concentration is first determined from Graph 1, by using the expected SVI values for the critical periods.
- Enter Graph 2 with the maximum float layer concentration and the feed solids concentration to interpolate for the required a ratio.
- For this a ratio, determine the limiting downflow rate V from Graph 3

From here, with the hydraulic loading and air/solids ratio known, the design proceeds as usual.

Comments

- This procedure requires extensive background information, as well as extensive laboratory tests to determine the required relationships. It may thus have value for existing plants where reliable records are available, but for new plants a more empirical approach will be necessary.
- The thickener area is essentially based on the allowable rise rate (i.e. hydraulic loading), rather than the solids loading rate which is universally acknowledged as the limiting parameter for sludge thickening.
- The method does not provide any information on the sludge layer thickness.
- The method does allow consideration of the sludge volume index, a parameter which is considered to be important for thickening, as shown in the literature review.

6.6 Typical nozzle design

The following illustration was adapted from the actual design calculations for a single nozzle with multiple orifices used at one of the clarification plants included in this survey.

In this case, the design raw water flow rate is 300 $m^3 h^{-1}$. The recycle percentage was fixed at a maximum of 9% at a saturation pressure of 400 kPa. The correct combination of orifice diameter and number of orifices has to be determined.

- Typical discharge coefficients for orifices range from C_d = 0.61 to 0.64. Select a conservative value of C_d = 0.60
- For practical reasons an orifice diameter of 2 mm was selected. The cross-sectional area per orifice is thus

$$\frac{\pi . 0,002^2}{4} = 3.14 . 10^{-6} \quad m^2$$

- The recycle flow rate is calculated from the known recycle percentage and raw water flow rate:

$$\frac{0.09 . 300}{3600} = 0.0075 \; m^3 \; s^{-1}$$

- The pressure drop across the injection nozzle should be about the same as the saturation pressure in a well designed system, namely

$$\frac{400 . 10^3}{1000 . 9.81} = 40.8 \text{ m head of water}$$

- The number of holes can be solved by setting up Equation 2.5

$$\frac{0.0075}{n} = 0.6 . 3.14 . 10^{-6} \sqrt{2 . 9.81 . 40.8}$$

from which

$$n = 140$$

A total number of 140 orifices each with diameter 2.0 mm have to be provided. The designer actually used 12 rows of 12 orifices each, for a total of 144 orifices. The actual nozzle layout is shown in Figure 6.6.

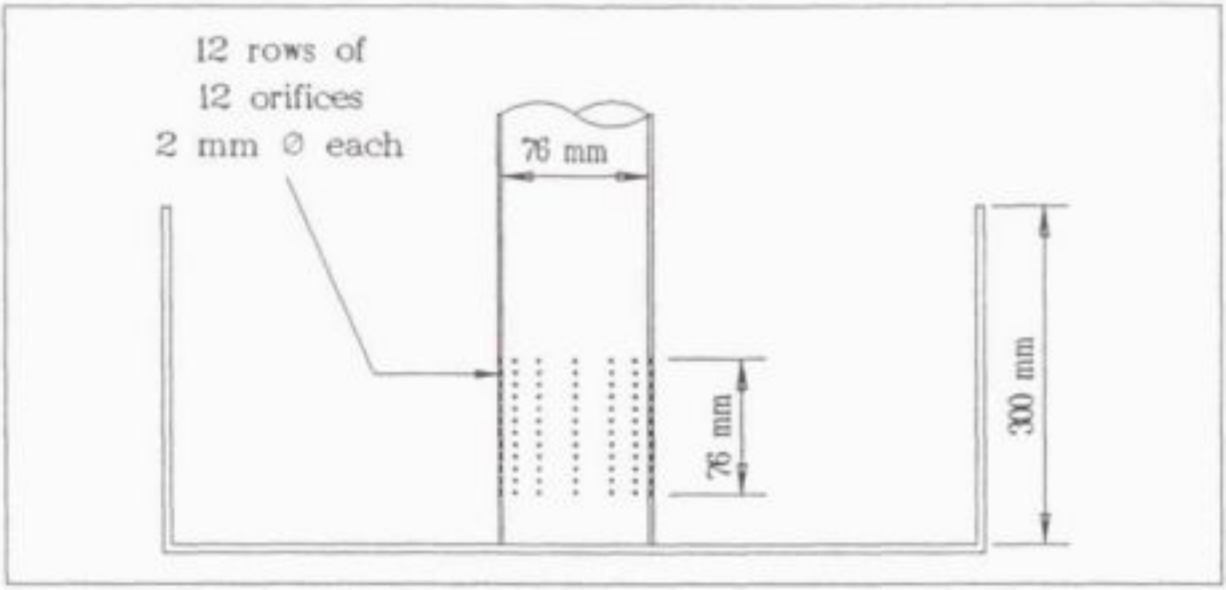


Figure 6.6 : Schematic section of nozzle for which design example is given, based on the NIWR-nozzle principle with enclosing cup (not to scale)

Comment

For a simple nozzle geometry as this, textbook values for the discharge coefficient C_d may be successfully used. For more complicated geometries, for example where the orifice channels are not straight, or the outlets are tightly capped, or for adjustable nozzles, the discharge coefficient must be determined experimentally.

6.7 Effect of recycle flow rate variation

The designer has to provide a recycle system that will provide sufficient air under the *worst* possible combination of conditions, namely the *maximum* air requirement at the *maximum* flow rate at *minimum* air solubility (or highest water temperature). These conditions will be very rarely encountered in actual practice. Under *average* conditions it will very likely be viable to operate the recycle system at a reduced rate and/or reduced pressure to save energy costs. This section presents a typical example of such an analysis.

There are two variables that determine the actual air transfer, namely the saturator pressure (which determines the mass of air that can be forced into the recycle stream) and the recycle flow rate itself. These variables are mathematically linked as shown in Equations 2.3 and 2.4. To manipulate the actual air transfer, either the saturator pressure, or the recycle rate, or both have to be adjusted. In the case of *adjustable* injection nozzles (for example when a valve is used), the adjustment is fairly trivial because the recycle flow rate and the saturator pressure can be independently adjusted (the recycle flow rate by pump speed control or by throttling, and the saturator pressure by adjustment of the injection nozzle). In the case of *fixed* injection nozzles, only the recycle flow rate can be adjusted, which makes the analysis more involved.

When the recycle flow rate is reduced, the saturator pressure is simultaneously reduced. Under normal conditions static head and pipe friction losses are negligible in comparison to the pressure drop across the injection nozzles. The first assumption is thus that the saturator gauge pressure is approximately equal to the pressure drop across the injection nozzles. As the flow through the recycle system is highly turbulent, the saturator pressure is proportional to the square of the recycle flow rate. If Equation 2.3 is set up for *maximum* air transfer conditions, the following relationship follows:

$$r_{max} = \frac{C_{max}}{S_{max} \cdot P_{max} \cdot \eta_s} \quad \alpha \quad \sqrt{P_{max}} \qquad (6.1)$$

If Equation 2.3 is set up once more, but now for *minimum* air transfer conditions, the relationship is:

$$r_{min} = \frac{C_{min}}{S_{min} \cdot P_{min} \cdot \eta_s} \quad \alpha \quad \sqrt{P_{min}} \qquad (6.2)$$

Equation 6.2, divided by Equation 6.1, yields

$$\frac{r_{min}}{r_{max}} = \frac{C_{min}}{C_{max}} \cdot \frac{S_{max}}{S_{min}} \cdot \frac{P_{max}}{P_{min}} = \sqrt{\frac{P_{min}}{P_{max}}}$$

from which the following relationships can be extracted:

$$\frac{P_{min}}{P_{max}} = \left(\frac{C_{min}}{C_{max}} \cdot \frac{S_{max}}{S_{min}}\right)^{\frac{2}{3}} \qquad (6.3)$$

$$\frac{r_{min}}{r_{max}} = \left(\frac{C_{min}}{C_{max}} \cdot \frac{S_{max}}{S_{min}}\right)^{\frac{1}{3}} \qquad (6.4)$$

Example

The air requirement for clarification ranges from a minimum of 6 mg.ℓ^{-1} to a maximum of 10 mg.ℓ^{-1}. The raw water temperature range is from 10°C to 25°C. The plant operates at an minimum of 70% of the maximum design flow of 300 m^3.h^{-1}. The maximum saturator pressure is 550 kPa. If fixed air injection nozzles are used, determine a) the recycle flow rate under conditions of maximum air transfer, b) the saturator pressure under conditions of minimum air transfer, and c) the recycle flow rate under conditions of minimum air transfer. Therefore

P_{max}	=	550 kPa
Q_{min}	=	$0.70\ Q_{max}$
T_{max}	=	25°C
T_{min}	=	10°C
C_{max}	=	10 mg.ℓ^{-1}
C_{min}	=	6 mg.ℓ^{-1}

Calculate solubility constants first:

$$S_{10} = 0.219 \left(\frac{293}{273 + 10}\right) . 10^{[illegible]} = 0.261\ \text{mg}.\ell^{-1}.\text{kPa}^{-1}$$

$$S_{25} = 0.219 \left(\frac{293}{273 + 25}\right) . 10^{[illegible]} = 0.202\ \text{mg}.\ell^{-1}.\text{kPa}^{-1}$$

a) The recycle flow rate under conditions of maximum air transfer is calculated with Equation 6.2 :

$$r_{max} = \frac{10}{0.202 . 550 . 0.75} = 0.120$$

$$r_{max} . Q_{max} = 0.120 . 300 = 36.0\ \text{m}^3.\text{h}^{-1}$$

b) The saturator pressure under conditions of minimum air transfer is calculated with Equation 6.3

$$P_{min} = 550 \left(\frac{6}{10} \cdot \frac{0.202}{0.261}\right) = 330\ \text{kPa}$$

c) The recycle flow rate under conditions of minimum air transfer is calculated with Equation 6.4:

$$r_{min} = 0.120 . \left(\frac{6}{10} . \frac{0.202}{0.261}\right)^{1/3} = 0.093$$

$$r_{min} . Q_{avr} = 0.093 . 0.70 . 300 = 19.5 \ m^3.h^{-1}$$

6.8 Word of caution

The examples presented in this chapter were deliberately simplified only to illustrate the integration of empirically derived recommendations with a theoretical framework that links air solubility, recycle flow rate, air requirements, *et cetera*. The *actual* design problems that will be faced in practice will undoubtedly require much more than contained in this manual, for example:

- At most applications, some practical constraints will force the designer to reverse the illustrated design sequence to some extent.
- Considerable judgment and analytical tests are required to judge whether these empirical guidelines may be followed, or whether supporting pilot-scale testing will be necessary.
- The recommended guidelines do encompass quite a broad range in most cases, and judgment and experience will be required for their proper application.
- Important design decisions will be based on cost considerations, and it will therefore be necessary to design on cost alternative solutions if the designer is not intimately familiar with the typical costs involved for the specialized items such as saturators, injection nozzles, float scraping equipment, *et cetera*.

CHAPTER SEVEN

RESEARCH NEEDS

7.1 General

There is absolutely no doubt that dissolved air flotation had firmly established itself in the water industry as a viable, robust process for both clarification and thickening. This is evidenced by the steady increase of flotation plants that are being constructed all over the world, including South Africa. The process had evolved to the point where general design guidelines for typical applications may even be used without the need for an extensive pilot testing programme, which is the main point of this design manual.

Does this mean that the design problems and uncertainties had been satisfactorily resolved? Before the authors' own opinions are given, it is instructive to quote the recent conclusions of other workers who had reviewed this question. Longhurst and Graham (1987), after a review of British flotation plants, concluded:

> *The water undertakings still regard the WRC (the British Water Research Centre) as the leading authority on DAF, which has restricted the freedom of the engineering companies to introduce their own design criteria. The result is that DAF technology has developed slowly.*

A year later Heinänen (1988), after a review of Finnish flotation plants, came to an independent, but similar conclusion:

> *The design parameters are far from ideal and have resulted in unnecessarily high construction costs. This is because the research institutes have not played their part in the design work. Their role is to conduct investigations and publish the results thus directing development in the right direction.*

Schofield (1991) at a recent international conference on dissolved air flotation, essentially echoed these conclusions:

> *Whilst the lack (of sensitivity to fundamental design criteria) may paint a rather depressing picture of DAF design expertise, it does in fact reflect the lack of progress and whilst DAF has existed as an art form for over 30 years, as a science, it is much less developed.*

The authors, after this review of South African dissolved air flotation practice, have to agree with these findings. The early SA research was conducted when DAF was a relatively novel process, and the driving force behind the research was *whether* DAF could be used as an alternative to the other better known processes. After this had been established, and a number of plants had verified it at full scale, one would have expected that the driving force behind research would shift to ways to *optimize* the process. This, unfortunately, materialized only to a limited extent. Moreover, some excellent fundamental SA research into sludge thickening provided the conceptual basis for the early application of DAF for thickening in SA, but there was a notable lack of integration of full-scale operating experience with the fundamentals.

The remainder of this chapter, therefore, summarizes those areas which the authors found to be particularly weak in terms of practical design parameters, or where our fundamental understanding is lacking even where good practical guidelines do exist.

7.2 Reaction zone

When compared to the voluminous literature on conventional two-phase flocculation, there is an almost total lack of fundamental literature on the analogous growth processes in the three-phase system in the reaction zone. Of specific interest, for example, is:

- Rational expression of the mixing energy within the reaction zone.
- Optimal level of the mixing energy.
- The need for tapering turbulence.
- Optimal retention time within the reaction zone.
- Further development of conceptual models for predicting the effects of key parameters.
- Effect of high concentrations of DOC, which are prevalent in SA surface waters, on hydrophobicity and particle/bubble interaction.

There are also some more empirical needs, such as determining:

- The effect of scale on the reaction zone, and how to compensate for scale effects during design.
- The effect of inlet geometry of feed and recycle respectively.
- Reaction zone geometry

7.3 Chemical pretreatment

There is significant consensus in the literature on the selection of coagulants, and their dosage determination. Where the opinions diverge, however, is on the type, duration and intensity of flocculation. Experience from Europe and Britain dictates longer flocculation times, while empirical SA experience and theoretical trajectory analyses from the US indicate that much shorter times will be adequate. The characterization of the raw waters and the temperature effects, however, were not rigorously documented. Specific needs are:

- The flocculation requirements of those algal genera or species which pose a problem during bloom periods in eutrophic waters.
- The water quality boundaries where the efficiency of DAF starts to decline.
- The water quality boundaries where the efficiency of DAFF starts to decline.
- The performance of DAF with respect to the removal of powdered activated carbon, often used in conjunction with DAF on eutrophic waters for the containment of taste and odour problems.
- Appropriate characterization of sludges to be thickened.

7.4 Bubble production system

The bubble production system requires substantial electrical energy. The following points will lead to the optimization and better performance prediction of the bubble production system:

- The development of a good predictive mathematical model for air saturator efficiency, both for packed and unpacked saturators.
- The measurement of actual air transfer efficiency attained by full-scale air saturators.
- The most cost-effective combination of recycle rate and saturator pressure.

On a more fundamental level, the behaviour of the air injection nozzles needs to be better understood. Narrow bubble size distributions with a specific mean size are required for specific applications. The design of the nozzles, however, is done purely on the basis of previous experience or empirical tests.

7.5 Flotation zone

The greatest need is in the field of thickening, where a discrepancy exists between what rational theory predicts and what is actually measured in full-scale applications. This can only be determined with a close comparison between full-scale and laboratory performance, to determine at which point linear scale-up breaks down, or to identify key parameters which are not incorporated at present into design models.

The hydraulic circulation patterns in sedimentation tanks are known to be of primary importance for sedimentation, and indications were found in the literature that some complex phenomena also do occur in flotation tanks, for example streamlining under the float layer, and recirculation patterns when outlets are not spread out evenly across the flotation area. These observations were mentioned as a matter of interest, and were not made during a systematic investigation into flotation tank hydraulics. There is reason to believe that flotation tanks behave differently due to shielding from wind by the float layer, and to areas of differential buoyancy due to the injection of air bubbles.

The combination of DAF with other processes such as filtration (DAFF) or sedimentation (SEDIDAF) or both (SEDIDAFF) requires adaptation of loading rates and flotation zone geometry which needs to be evaluated and optimized.

7.6 Float layer removal

This aspect of DAF does not lend itself as easily to fundamental analysis, and is influenced to a much greater extent by the empirical design of scrapers, beach plates, drives, gearboxes, *et cetera*. At the same time, however, this area is the one identified in SA and overseas to be one of the main operational headaches of DAF thickening plants. It is thus recommended that a concerted effort is made by suppliers to iron out the difficulties encountered in this regard by further development and testing.

It is felt that the design of surface grids for the compaction of the float layer is too empirical and that relationships are needed to predict the effect of grid depth, size of grid openings, and to calculate the uplift forces on the grid in a rational manner.

7.7 Industrial applications

During this survey of SA flotation plants for thickening and clarification, the authors came across numerous industrial applications of the flotation process, such as in the dairy industry, the fields of meat-processing, industrial washing plants, fish canneries, *et cetera*. There is no doubt that a considerable body of DAF expertise, both in terms of design and operation, had evolved over the years, but which is largely unavailable to the broader engineering community. There is a need to also locate and visit these plants for an industrial DAF survey such as the survey described here.

REFERENCES

ANON (1972) The flotation method of treating industrial wastewater. In : *6th International Conference on Water Pollution Research*, June 1972, Jerusalem.

ANON (1983) Eimco equipment at Athlone sewage works. *Water Sewage and Effluent,* **13** 52.

ANON (1984) Effluent management : UF system demonstrated. *SA Waterbulletin*, August 1984 4-5.

ANON (1986) Optimization of flocculation/flotation in chemical wastewater treatment. In : *Proceedings of the 13th Biennial International Conference on Water Pollution Research and Control*, 1986, Rio de Janeiro : IAWPRC, 1-13.

ASHMAN, PS (1976) Operational experiences of activated sludge thickening by dissolved air flotation at the Aycliffe sewage treatment works. In : *Water Research Centre Conference Felixtone. Flotation for Water and Waste-water Treatment*, 1977, Marlow : WRc

BENNETT, GF (1988) The removal of oil from wastewater by air flotation : a review. *CRC Critical Reviews in Environmental Control,* **18**(3) 189-253.

BERNSTEIN, M, VAN VUUREN, LRJ & BOTES, V (1985) Pilot and full-scale applications of DAF technology to treat eutrophic waters from Lake Nsese. In : *Conference Institute of Water Pollution Control*, 1985, Parhurst : IWPC, 1-9.

BLOMMAERT, MC, MUS, I & JANSSENS, JG (1990) Optimizing dissolved-air flotation through velocity of rise measurements. Poster presented at the IWSA/IAWPRC Joint Specialised Conference on Coagulation, Flocculation, Sedimentation and Flotation, 1990, Jönkoping (Sweden).

BOTES, V & VAN VUUREN, LRJ (1990) Dissolved air flotation for the removal of algae and inorganic turbidity on large scale. *Water Supply,* **8** 133-139.

BRATBY, JR (1978) Aspects of sludge thickening by dissolved-air flotation. *Water Pollution Control*, **77**(3) 421-432.

BRATBY, JR (1982) Treatment of raw wastewater overflows by dissolved-air flotation. *Journal of the Water Pollution Control Federation,* **54**(12) 1558-1565.

BRATBY, JR & MARAIS, GvR (1973) Dissolved air flotation. *Filtration and Separation,* Nov/Des 1974, 614-624.

BRATBY, JR & MARAIS, GvR (1975a) Dissolved air (pressure) flotation - an evaluation of inter-relationships between process variables and their optimisation for design. *Water SA,* July 1975 57-69.

BRATBY, JR & MARAIS, GvR (1975b) Saturator performance in dissolved-air (pressure) flotation. *Water Research,* **9** 929-936.

BRATBY, JR & MARAIS, GvR. (1976a) A guide for the design of dissolved air (pressure) flotation systems for activated sludge processes. *Water SA,* **2**(2) 87-100.

BRATBY, JR & MARAIS, GvR. (1976b) Thickening of brown water sludges by dissolved-air (pressure) flotation. University of Cape Town. Department of Civil Engineering, Research Report No W 16.

CASEY, TJ & NAOUM, IE (1986) Air saturators for use in dissolved air flotation processes. *Water Supply*, **4** 69-82.

CASSELL, EA, KAUFMAN, KM & MATIJEVIC, E (1974) The effect of bubble size on microflotation. *Water Research*, **9** 1017-1024.

CHILDS, AH & REES, AJ (1976) Operational experience with the 2 300 m³/d pilot plant of the Essex water company. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 219-244.

COERTZE, JA (1978) Dissolved air flotation treatment of paper mill effluent. *Progress in Water Technology*, **10**(1/2) 449-457.

CONFERENCE ON Flotation for Water and Wastewater Treatment. (1976 Felixstowe). *Proceedings*. Marlow : Water Research Centre, 1977, 68-76; 157-165; 166-178; 309-316; 401-406; 407-416.

DANIELS, SL (1977) Chemical treatment and dissolved air flotation of oxidation pond effluent. *American Institute of Chemical Engineers Symposium Series*, **73**(166) 358-363.

DAVIS, C (1977) Dissolved air flotation and belt filter pressing of activated sludge in Johannesburg. In : *Symposium on the Treatment and Disposal of Sludges Deriving from Municipal Sewage*, 1977, Pretoria : IWPC, 1-9.

DE GROOT, CPM & VAN BREEMEN, AN (1987) Ontspanningsflotatie en de bereiding van drinkwater. Mededeling van de vakgroep gezondheidstechniek & waterbeheersing. Fakulteit der civiele tegniek. Tegniese Universiteit : Delft.

DE WET, FJ (1980) Flotation of algal waters. Paper presented at a meeting of the Institute of Water Pollution Control, 1980, SA, Pretoria : IWPC, 1-28.

DE WET, FJ, & VAN VUUREN, LRJ (1980) Investigation into the reclamation of sewage effluent for reuse at the Kloof mine. Pretoria : CSIR, NIWR, 1-42 (Contract Report).

DE WET, FJ & VAN VUUREN, LRJ (1982) Investigation into the quality improvement of reclaimed sewage effluent by dissolved air flotation-filtration (DAFF) at SAPPI Adamas. Pretoria : CSIR, NIWR, 1-31 (Contract Report).

EDZWALD, JK, MALLEY, JP & YU, C (1990) Conceptual model for dissolved air flotation in water treatment. *Water Supply*, **8** 141-150.

EDZWALD, JK & WINGLER, BJ (1990) Chemical and physical aspects of dissolved air flotation for the removal of algae. *Aqua*, **39** 24-35.

ETTELT, GA (1964) Activated sludge thickening by dissolved air flotation. In : *Proceedings 19th Industrial Waste Conference*, 1964, Purdue University : IWC, **117** 210-244.

EVERETT, P (1983) Rooiwal sewage purification works - dissolved air flotation system. *Water Sewage and Effluent*, **13** 47-49

FOLKARD, GK, SUTHERLAND, JP & JAHN, SAA (1986) Water clarification with natural coagulants and dissolved air flotation. *Waterlines*, **5**(2) 23-26.

GEHR, R & HENRY, JG (1980) Assessing flotation behaviour of different types of sewage suspensions. *Prog Water Technology*, **12** 1-21.

GEHR, R, SWARTZ, CD & OFFRINGA, G (1991) Dissolved air flotation for the pretreatment of eutrophied surface water for potable use. WRC Final Report No 202/1/91.

GOCHIN, RL & SOLARI, J (1983) The role of hydrophobicity in dissolved air flotation. *Water Research*, **17**(6) 651-657.

GRANT, PE & WILLIAMS, DA (1976) The use of dissolved air flotation in the chemical treatment of food industry waste. In : *Water Research Centre, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 55-67.

GULAS, V & LINDSEY, R (1978) Factors affecting the design of dissolved air flotation systems. *Journal of the Water Pollution Control Federation*, **50**(7) 1835-1840.

GULAS, V, BENEFIELD, L, LINDSEY R & RANDALL C (1980) Design considerations for dissolved air flotation. *Water and Sewage Works*, **127**(7) 30-31, 42.

HAARHOFF, J & FOUCHE, L (1989) Evaluation of combined flotation/filtration of Rietvlei water treatment plant. In : *WISA 1st Biennial Conference and Exhibition*, 1989, Cape Town, 1-13.

HEBDEN, D & BOTHA, GR (1978) A pretreatment method for seawater desalination by reverse osmosis. In : *6th International Symposium on Fresh Water from the Sea*, 1978, Las Palmas, Vol 4, edited by A Delyannis, Athens : EFCE, 209-214.

HEINÄNEN, J (1988) Use of dissolved air flotation in potable water treatment in Finland. *AQUA FENNICA*, **18**(2) 113-123.

HEMMING, ML & COTTRELL, WRC (1976) Experiences in the treatment of domestic sewage by the microflotation process. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 19-42.

HENRY, JG & GEHR, R (1981) Dissolved air flotation for primary and secondary clarification. Ottawa: Canadian Environmental Protection Service, 1-87 (Report SCAT-9).

HOWELL, HO (1982) Johannesburg municipality's northern sewage disposal works operates the world's largest dissolved air flotation plant for activated sludge thickening. *IMIESA*, **7**(6) 53,55.

HYDE, RA (1975) Water clarification by flotation : design and experimental studies on a dissolved-air flotation pilot plant treating 8,2 m^3/h of river Thames water. WRc, Technical Report TR 13, 1-34.

JACOBS, HJ & SMITH, L (1973) Advanced treatment of purified sewage for the production of high-brightness pulp and paper. *Tappi*, **56**(12) 170-174.

JONES, AD & HALL, AC (1981) Removal of metal ions from aqueous solution by dissolved-air flotation. *Filtration and Separation*, **18**(5) 386-388.

KEGEL, T (1987) Sewerage effluent treatment to supplement mine service water supply. *Mine Service Water Supply* - Kloof G.M. Co Ltd 1-22.

KING, RP (editor) (1982) *Principles of flotation*. Johannesburg : South African Institute of Mining and Metallurgy. (Monograph Series no. 3).

KITCHENER, JA & GOCHIN, RJ (1981) The mechanism of dissolved air flotation for potable water: basic analysis and a proposal. *Water Research*, **15**(5) 585-590.

KOMLINE, TR (1976) Sludge thickening by dissolved air flotation in the USA. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow: WRc, 130-156.

KOMLINE, TR (1978) Dissolved air flotation tackles sludge thickening. *Water and Wastes Engineering*, **15**(2) 63-67, 69.

LANGENEGGER, O & VIVIERS, JMP (1978) Thickening of waste activated sludge with a dissolved air flotation unit. *Water Pollution Control*, 79-86.

LEACH, GW, SOUTAR, BN & PULLEN, AW & VAN VUUREN, LRJ (1985) Sewage effluent reclamation for potable industrial mine service water. (NIWR Reprint RW 0996), 1-43.

LEININGER, KV & WALL, DJ (1974) Available air measurements applied to flotation thickener evaluations. *Deeds & Data*, May 1974 3-7.

LE ROUX, JD & VAN DER WALT, CJ (1991) Development of a two-stage counter-current powdered activated carbon-dissolved air flotation system for the removal of organic compounds from water. WRC, Final Report No 224/1/91, 1-72.

LONGHURST, SJ & GRAHAM, NJD (1987) Dissolved air flotation for potable water treatment : A survey of operational units in Great Britain. *Public Health Engineer*, **14**(6) 71-76.

LOVETT, DA & TRAVERS, SM (1986) Dissolved air flotation for abattoir wastewater. *Water Research*, **20**(4) 421-426.

LUNDGREN, H (1970) Recent advances in air flotation technology. *Tappi*, **53**(2) 287-289.

LUTHY, RG, SELLECK, RE & GALLOWAY, TR (1978) Removal of emulsified oil organic coagulants and dissolved air flotation. *Journal of the Water Pollution Control Federation*, **50**(2) 331-346.

MADDOCK, JEL (1976) Research experience in the thickening of activated sludge by dissolved air flotation. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 78-101.

MADDOCK, JEL & TOMLINSON, EJ (1980) The clarification of effluent from an activated-sludge plant using dissolved-air flotation. *Water Pollution Control*, **79**(1) 117-125.

MARTIN, MH & BHATTARAI, RP (1991) More mileage from gravity sludge thickeners. *Water Environment & Technology*, July 1991.

OFFRINGA, G (1986) Dissolved air flotation treatment of final effluent from three South African board mills. *Paper Southern Africa*, Jan/Feb 1986, 16-23. (NIWR Reprint RW 1086)

OFFRINGA, G & SCHEEPERS, JA (1985) Removal of algae by dissolved air flotation of Swartwater Dam, Qwaqwa. Pretoria : CSIR, NIWR, 1-18 (Contract Report).

OFFRINGA, G, VENNEKENS, MJA & BOTHA A, CARSTENS, PA & SCHOONBEE, A (1986) Upgrading of the SAPPI, ENSTRA dissolved air flotation plant from 20 Mℓ/d to 30 Mℓ/d. Pretoria : CSIR, NIWR, 1-30 (Contract Report).

OFFRINGA, G, VENNEKENS, MJA & DE VILLIERS HA, & BOTHA A (1987) Dissolved air flotation studies in the pulp and paper industry. *National Institute for Water Research, CSIR*, 1-11.

PACKHAM, RF & RICHARDS, WN (1975a) The determination of dissolved air in water. WRc, Technical Memorandum TM 106.

PACKHAM, RF & RICHARDS, WN (1975b) Water clarification by flotation - 3. Marlow : WRc, Technical Report TR 2.

PILKINGTON, NH & VAN VUUREN, LRJ (1981) Formation of trihalomethanes during chlorination of algae laden surface waters. Paper presented at the Australian Water and Wastewater Association Conference, 1981, Perth.

RAMIREZ, ER (1979) Comparative physicochemical study of industrial wastewater treatment by electrolytic dispersed air and dissolved air flotation technologies. In : *Proceedings 34th Industrial Waste Conference*, 1980, Ann Arbor : AASP, 6.

REAY, D & RATCLIFF, GA (1973) Removal of fine particles from water by disperse air flotation : effects of bubble size and particle size on collection efficiency. *Can J Chem Eng*, **51** 178-185.

REES, AJ, RODMAN, DJ & ZABEL, TF (1979) Water clarification by flotation - 5. WRc, Technical Report TR 114.

REES, AJ, RODMAN, DJ & ZABEL, TF (1980a) Evaluation of dissolved-air-flotation saturator performance. WRc, Technical Report TR 143, 1-16.

REES, AJ, RODMAN, DJ & ZABEL, TF (1980b) Operating experiences with dissolved air flotation on various raw waters. *Aqua*, **8** 170-177.

RICHARD, Y & DAUTHUILLE, P (1984) Thickening of sludge and new development in sedimentation. In : *Fransa Seminar*, 1984, Johannesburg : FRANSA, 1-19.

ROBERTS, KL, WEETER, DW & BALL, RO (1978) Dissolved air flotation performance. In : *Proceedings 33 rd Industrial Waste Conference*, 1978, Ann Arbor : Ann Arbor SCI PUB, 194-199.

ROETS, SD & HEUNIS, EA (1984) The application of an induced air flotation process in the treatment of proteinaceous waste. *Water Sewage and Effluent*, **19**(7) 9-11.

ROSEN, B & MORSE, JJ (1976) Practical experience with dissolved air flotation on various waters in Sweden and Finland. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 318-338.

ROVEL, JM (1976) Experiences with dissolved air flotation for industrial effluent treatment. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 43-54.

RULEV, NN (1985) Effect of coalescence of gas bubbles on efficiency of microflotation. *Khimiya I Teknologiya Vody*, **7**(3) 9-12.

RYKAART, EM (1991) Die modellering van lugversadigers vir opgelostelugflottasie. Undergraduate thesis presented to the Rand Afrikaans University.

SANDBANK, E (1983) Algae harvesting. In : *Proceedings Conference Institute of Water Pollution Control*, 1983, Thohoyandou : IWPC, 1-16.

SANDBANK, E & VAN VUUREN, LRJ (1983) Algae harvesting. *Paper presented at the Environmental Engineering and Water Resources Centre*, 1983, Technion Haifa, 1-10.

SCHADE, H (1982) Optimierung von Flotationsanlagen durch Einsatz eines TPF-Abscheiders (Tiltable Plate Flotator) in der Abwassertechnik. *Korrespondenz Abwasser*, **29**(2) 629-634, 636.

SCHOFIELD, T (1991) The Severn Trent Water Experience with DAF. Workshop on Dissolved Air flotation (DAF). *Proceedings*, Antwerp IWSA/IAWPRC.

SCOTT, PD & SQUIRES, RC (1983) Protein recovery from abattoir effluent. *Chemsa*, **9**(6) 100-103.

SHANNON, WT & BUISSON, DH (1980) Dissolved air flotation in hot water. *Water Research*, **14**(7) 759-765.

SPEECE, RE, SIDDIQI, RH & AUBERT, RP (1975) Application of a lower energy pressurized gas transfer system to dissolved air flotation and oxygen transfer. In : *Proceedings 30th Industrial Waste Conference*, 1977, Ann Arbor : Ann Arbor SCI PUB, 465-478.

STOCK, PF (1976) The Flockation process : Investigations and results obtained by treating river and well water for potable supply. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 349-416.

TAVERY, MA (1979) DAF prove-out : 12-years running. *Water and Sewage Works*, **126**(1) 55-57.

VAN PUFFELEN, J (1990) Flotatie : Een topper in de waterzuivering. Presentation at a Conference, 1990, Technical University Delft.

VAN VUUREN, LRJ, CLAYTON, AJ & VAN DER POST, DC (1980) Current status of water reclamation at Windhoek. *Journal Water Pollution Control Federation*, **52** 661-671.

VAN VUUREN, LRJ, DE WET, FJ & CILLIE, GG (1983a) Treatment of water from eutrophied impoundments. *Water Supply*, **1**(1) 145-156.

VAN VUUREN, LRJ, FUNKE, JW & SMITH, L (1972) The full-scale refinement of purified sewage for unrestricted industrial use in the manufacture of fully-bleached craft pulp and fine paper. In : *6th International Conference on Water Pollution Research*, June 1972, Jerusalem, 18-24.

VAN VUUREN, LRJ, HENZEN, MR, STANDER, GJ & CLAYTON, AJ (1970) The full-scale reclamation of purified sewage effluent for the augmentation of the domestic supplies of the city of Windhoek. In : *5th International Conference on Water Pollution Research*, 1970. San Francisco. 1-32/1

VAN VUUREN, LRJ, MEIRING, PGJ & HENZEN, MR (1965) The flotation of algae in water reclamation. *Int J Air Water Pollution*, **9** 823-832.

VAN VUUREN, LRJ, OFFRINGA & G, WILLIAMS, PG (1985) Development and application of DAF technology in South Africa. In : *The Institute of Water Pollution Control Yearbook and Buyers Guide*, 1985-1986, 75-89.

VAN VUUREN, LRJ & OFFRINGA, G (1985) Production of potable supplies from algae-laden waters. In : *Proceedings of Symposium on the Impact of Phosphate on South African Waters*, 1985, Pretoria : CSIR, 1-17.

VAN VUUREN, LRJ, PRINSLOO J & DE WET, FJ (1983b) Pilot-scale studies for the treatment of eutrophied water by dissolved air flotation filtration. In : *10th Federal Convention of the Australian Water and Wastewater Association*, 1983, Sydney : AWWA, 14/1-14/10.

VAN VUUREN, LRJ & ROSS, WR (1975) Integrated physical, chemical and biological treatment systems. Paper presented at Symposium on Water Research : Selected in-depth Studies (Part II), 1975, Pretoria.

VAN VUUREN, LRJ, STANDER GJ, HENZEN, MR, MEIRING, PGJ & VAN BLERK, SHV (1967) Advanced purification of sewage works effluent using a combined system of lime softening and flotation. *Water Research*, **1**(7) 463-474.

VAN VUUREN, LRJ & VAN DER MERWE, PJ (1988) Thickening of sludges by dissolved air flotation. *Water Science Technology*, **21** 1771-1774.

VAN VUUREN, LRJ & VAN DUUREN, FA (1965) Removal of algae from wastewater maturation pond effluent. *Journal Water Pollution Control Federation*, **37** (9) 1256-1262.

VOSLOO, PBB & LANGENEGGER, O (1979) Possible methods for dealing with phytoplankton at water purification works. *Paper presented at Monthly Meeting of the Institute of Water Pollution Control*. 1979, SA : IWPC, 1-14.

VOSLOO, PBB, WILLIAMS, PG & RADEMAN, RG (1985) Pilot and full-scale investigations on the use of combined dissolved air flotation and filtration (DAFF) for water treatment. In : *Biennial Conference and Exhibition of the Institute of Water Pollution Control*. 1985, SA Branch, Durban : IWPC, **1** 1-12.

VRABLIK, ER (1959) Fundamental principles of dissolved-air flotation of industrial wastes. In : *Proceedings 14th Industrial Waste Conference*, 1959. Purdue University : IWC. 743-779.

WATER RESEARCH COMMISSION (WRC) (1984) Theory, design and operation of nutrient removal activated sludge process. Water Research Commission of South Africa, P O Box 824, Pretoria 0001.

WILLIAMS, PG & VAN VUUREN, LRJ (1984) *Windhoek Water Reclamation Plant Process Design for Dissolved Air Flotation*. Pretoria : CSIR, NIWR. 1-20.

WILLIAMS, PG, VAN VUUREN, LRJ & VAN DER MERWE, PJ (1985) Dissolved air flotation upgrades a conventional plant treating eutrophic water. In : *Proceedings 11th International Convention of the Australian Water and Wastewater Association*, 1985, Melbourne : AWWA, 189-196.

ZABEL, T (1978) Flotation. In : *Proceedings of the 12th Congress of the International Water Supply Association*, 1978, London : IWSA, PF1-PF10.

ZABEL, T (1985) The advantages of dissolved-air flotation for water treatment. *Journal of the American Water Works Association*, **77**(5) 42-46.

ZABEL, TF & HYDE, RA (1976) Factors influencing dissolved air flotation as applied to water clarification. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow : WRc, 180-218.

ZABEL, TF & MELBOURNE, JD (1980) *Development in water treatment*. Chapter 6. 140-191. Edited by WM Lewis.

ZABEL, TF & REES, AJ (1976) The WRC flotation working group's 2 300 m^3/d pilot plants. In : *Water Research Centre Conference, Felixstowe. Flotation for Water and Wastewater Treatment*, 1977, Marlow: WRc, 245-283.

ZIEMINSKI, SA, CARON, MM & BLACKMORE, RB (1967) Behaviour of air bubbles in dilute aqueous solutions. *I & EC Fundamentals*, **6**(2) 233-242.

ZHANG, Y (1985) Chinese show how DAF can reduce MLSS. *Water Engineering and Management*, **132**(8) 38-42.

APPENDIX A

LIST OF PARTICIPATING INSTITUTIONS

Atomic Energy Corporation
P.O Box 582
0001 PRETORIA

Bophuthatswana Water Supply Authority
P O Box 4500
8681 MMABATHO

Borough of Amanzimtoti
P.O. Box 26
4125 AMANZIMTOTI

Bronkhorstspruit Town Council
P.O. Box 40
1020 BRONKHORSTSPRUIT

Durban Corporation
P O Box 2443
4000 DURBAN

Goldfields of South Africa
P.O. Box 1167
2000 JOHANNESBURG

Hartbeespoort Town Council
P.O. Box 976
0216 HARTBEESPOORT

Johannesburg City Council
P.O. Box 4323
2000 JOHANNESBURG

Kosmos Town Council
P.O. Box 1
0250 KOSMOS

Mhlatuze Water Board
P O Box 1264
3900 RICHARDS BAY

Morgenzon Town Council
P.O. Box 9
2315 MORGENZON

Nampak Paper
P. O. Box 40
7535 BELLVILLE

Pretoria City Council
P.O. Box 1409
0001 PRETORIA

Richards Bay Town Council
Private Bag X1004
3900 RICHARDS BAY

Sappi Fine Papers
P O Box Enstra
1560 SPRINGS

Sappi Adamas
P.O. Box 2164
6056 NORTH END

Umgeni Water
P.O. Box 9
3200 PIETERMARITZBURG

Vereeniging City Council
P.O. Box 35
1930 VEREENIGING

Western Cape Regional Services Council
P O Box 173
8000 CAPE TOWN

Windhoek City Council
P.O. Box 59
WINDHOEK
Namibia

APPENDIX B

DETAILS OF PLANTS SURVEYED

The details of the plants included in the survey are summarized in this appendix. The details for each plant are summarized as follows :

- A description of the plant, with comments on application, chemical pretreatment, details on the tank configuration and bubble production system, and some remarks on operational performance and problems.
- A table showing the quantitative parameters for each plant, including physical dimensions and operational variables.
- A sketch to show the layout of the entire flotation tank.
- A sketch to show the layout of the reaction zone.

Plants #1 to #14 are clarification plants; plants #21 to #32 are thickening plants.

NOTE

Clarification plants are numbered from #1 to #14

Thickening plants are numbered from #21 to #32

DESCRIPTION OF FLOTATION PLANT #01

Objective

The plant treats water from a highly eutrophic impoundment for drinking-water purposes. Flotation follows chemical treatment and flocculation, and precedes rapid sand filtration. Its primary aim is to increase filter run length by reduction of the solids load, and the secondary aim to compact the float to the point where it can be manually handled and hauled.

Feed water

The raw water from the impoundment has very low turbidity, but often contains very high numbers of bluegreen algae, with concomitant taste and odour problems. Turbidity varies between 1 and 5 NTU, pH between 7.5 and 9.6, TDS between 300 and 400 $mg.\ell^{-1}$ and alkalinity above 100 mg $CaCO_3.\ell^{-1}$.

Chemical conditioning

The water is immediately dosed with powdered activated carbon, and approximately two minutes later with ferric chloride. Flocculation follows in a pipe flocculator which provides contact time of 15 minutes at G-values tapering from $70.s^{-1}$ to $20.s^{-1}$. After flocculation lime is added for pH correction. Chlorination follows flotation and gravity filtration.

Flotation tank(s)

Flotation is done in a single rectangular tank with a flat bottom. The feed water enters the tank at the bottom through 5 identical pipes which are throttled with a pinched rubber section to provide the desired head loss. The recycle water is injected through 9 small injection nozzles in the tank itself, in close proximity of the feed inlets. The subnatant is withdrawn through a submerged pipe with 6 evenly spaced orifices. The float layer is compacted by the rectangular steel grid, manually scraped off and hauled away. The level in the flotation tank is maintained by an overflow weir.

Recirculation system

The recycle is pressurized through a single vertical saturator packed with plastic rings. The water is supplied at a constant, set pressure and the water flow rate is controlled with an internal float. The air is supplied at a slightly lower pressure than the water and therefore requires no separate control.

Performance

The flotation tank performs very well and frequently delivers subnatant with turbidity less than 1 NTU. During rapid raw water changes due to changes in wind speed or direction, performance suffers because chemical dosing changes are not immediately made. The air compressor requires frequent maintenance.

PLANT NUMBER # 01

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	5.5	
Width	W	m	2.1	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	2.0	
Number flotation cells			1	
Flotation area	A_f	m^2	10.0	
Reactor area	A_r	m^2	1.0	
Reactor volume	V_r	m^3	0.8	
Crossflow area	A_c	m^2	2.1	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	50.0	33.3
Recycle rate/cell	q	$k\ell.h^{-1}$	3.4	3.4
Saturator pressure	P	kPa	400	425
Concentration in	C	NTU		2.8
Concentration out	C_e			

*** Saturation ***

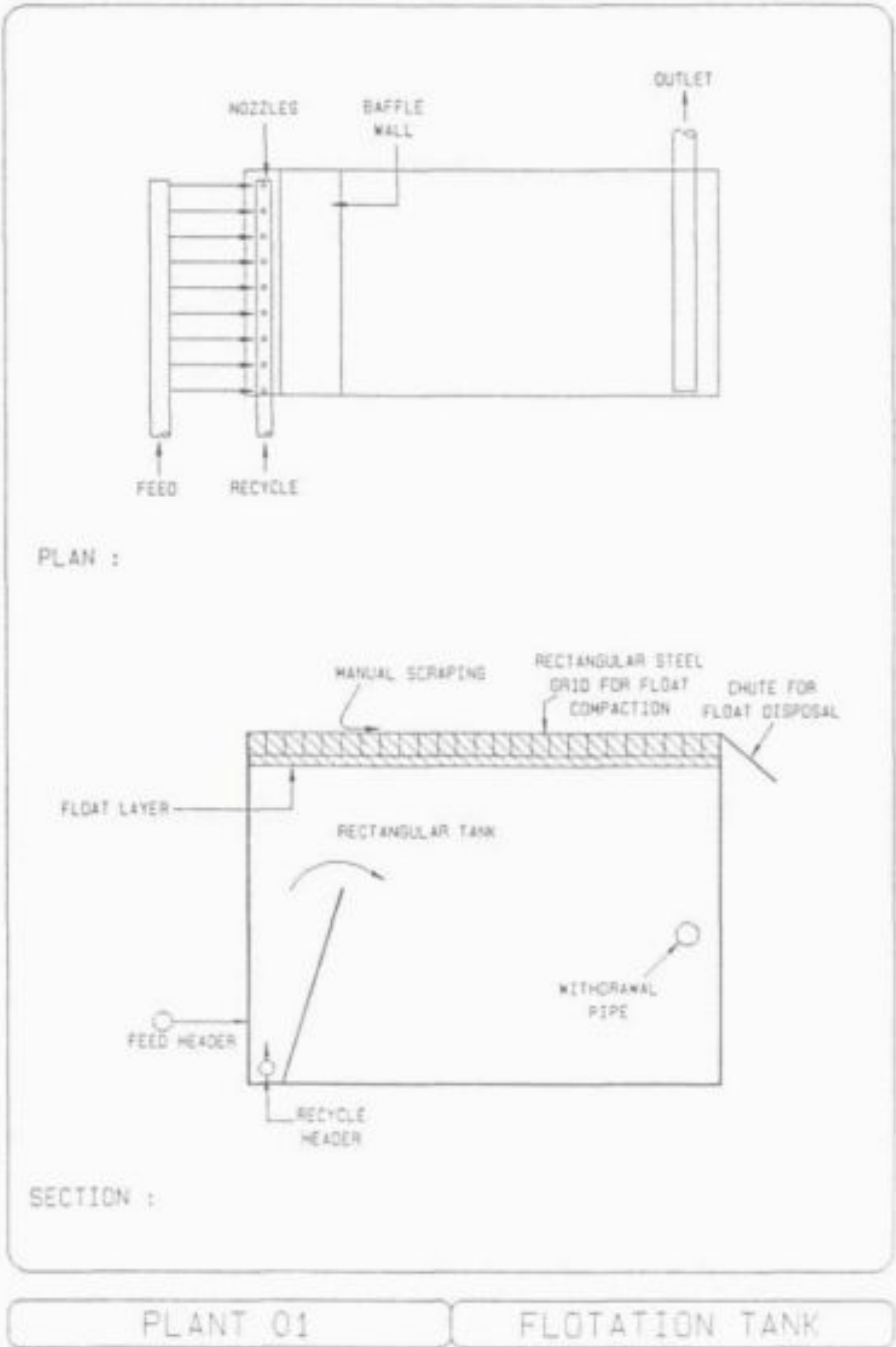
Position (H/V)			V
Diameter	D_s	m	0.46
Length	L_s	m	1.6
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D_p		0.8
Number of saturators			1

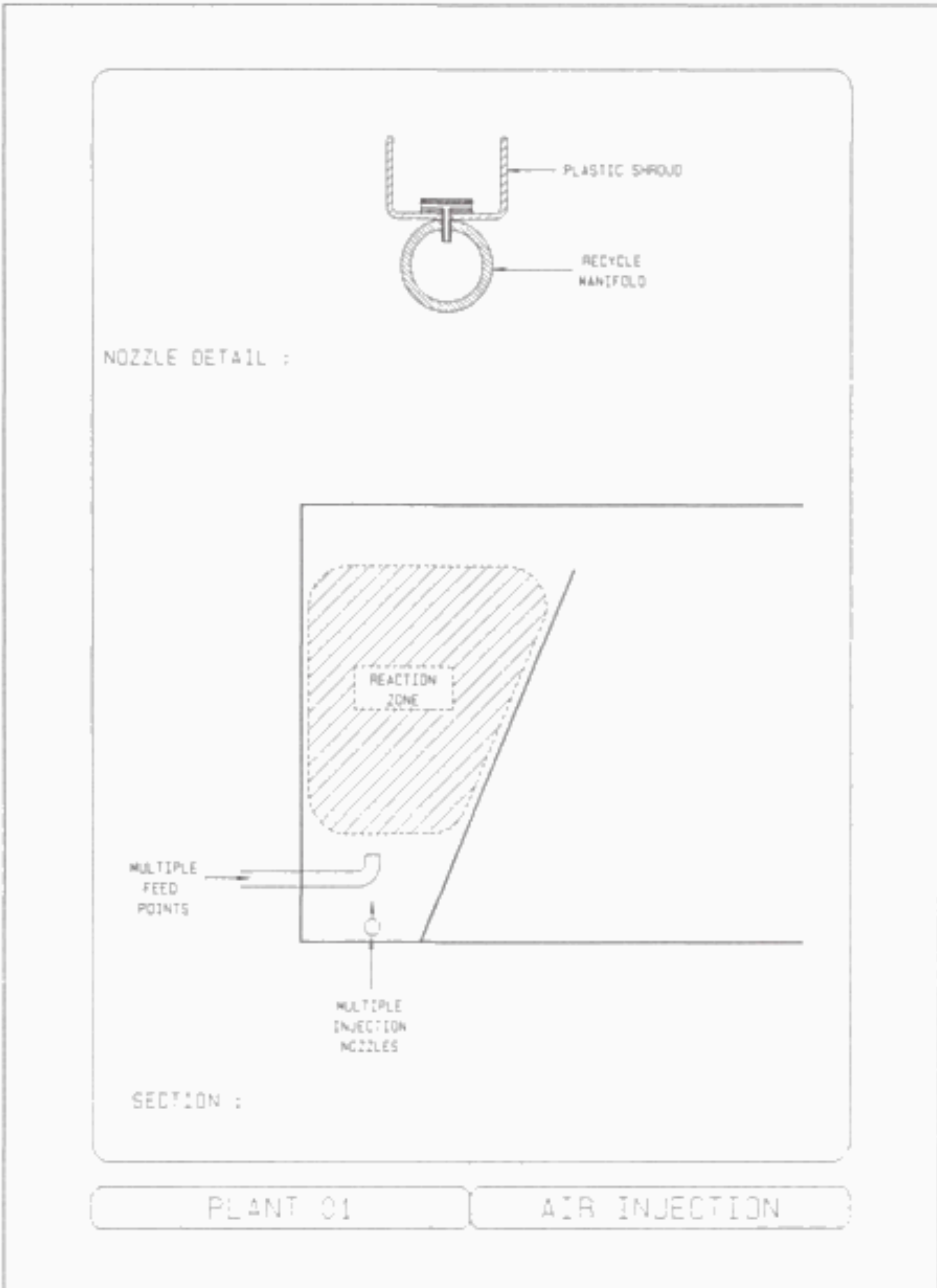
*** Float removal ***

Grid (Y/N)	Y
Scraper (Y/N)	N
Spacing scrapers	n/a
Speed scrapers	n/a
Continuous scraping (Y/N)	n/a
Bottom desludging (Y/N)	N

*** Notes ***

Tank drained and cleaned once per week.
Grid openings 150 mm by 150 mm. Manual scraping


OUTLET
NOZZLES
BAFFLE WALL
FEED
RECYCLE
PLAN :
MANUAL SCRAPING
RECTANGULAR STEEL GRID FOR FLOAT COMPACTION
CHUTE FOR FLOAT DISPOSAL
FLOAT LAYER
RECTANGULAR TANK
WITHDRAWAL PIPE
FEED HEADER
RECYCLE HEADER
SECTION :
PLANT 01
FLOTATION TANK


PLASTIC SHROUD
RECYCLE MANIFOLD
NOZZLE DETAIL :
REACTION ZONE
MULTIPLE FEED POINTS
MULTIPLE INJECTION NOZZLES
SECTION :
PLANT 01
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #02

Objective

The plant treats water from a highly eutrophic impoundment for drinking-water purposes. Flotation follows chemical treatment and flocculation, and precedes pressure filtration through sand. Its primary aim is to increase filter run length by reduction of the solids load, and the secondary aim to compact the float to the point where it can be manually handled and hauled.

Feed water

The raw water from the impoundment has very low turbidity, but often contains very high numbers of bluegreen algae, with concomitant taste and odour problems. Turbidity varies between 1 and 5 NTU, pH between 7,5 and 9,5, TDS between 300 and 400 mg.ℓ^{-1} and alkalinity above 100 $CaCO_3.\ell^{-1}$.

Chemical conditioning

The water is simultaneously dosed with one or more of powdered activated carbon, ferric chloride, polymer or sodium hydroxide. Upflow flocculation follows in a single square tank with a hoppered bottom which provides contact time of 50 minutes. Chlorination follows flotation and pressure filtration.

Flotation tank(s)

Flotation is carried out in 2 identical square, flat-bottomed tanks which are operated in parallel. The feed to the flotation tanks enters in the bottom centre and is contacted with the recycle in an inverted conical reaction zone. The recycle is injected with a single nozzle with multiple orifices. The injection nozzle is shrouded to enhance even bubble precipitation. The subnatant is collected by an evenly spaced grid of vertical pipes which withdraw close to the bottom of the tanks. The float layer is compacted by the rectangular steel grid, manually scraped off and hauled away. The level in the flotation tank is maintained by a float valve.

Recirculation system

The recycle is drawn off from the high-pressure distribution system and passed through 2 vertical saturators (one for each flotation tank) packed with plastic rings. The water is supplied at a constant, set pressure and the water flow rate is controlled with an internal float. The air is supplied at a slightly lower pressure than the water and therefore requires no separate control.

Performance

The flotation tanks perform satisfactorily and produce water of good quality. From time to time the plastic packing in the saturators needs to be manually cleaned from a slimy deposit. The injection nozzles also require periodic cleaning.

PLANT NUMBER			# 02	
***** Tank geometry *****				
Shape (Round/Rect)			Rect	
Length	L	m	4.0	
Width	W	m	4.0	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	2.1	
Number flotation cells			2	
Flotation area	A_f	m^2	16.0	
Reactor area	A_r	m^2	1.7	
Reactor volume	V_r	m^3	1.3	
Crossflow area	A_c	m^2	5.2	

*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	179.0	90.0
Recycle rate/cell	q	$k\ell.h^{-1}$	12.7	12.7
Saturator pressure	P	kPa	350	400
Concentration in	C_i	SS		2.8
Concentration out	C_o			

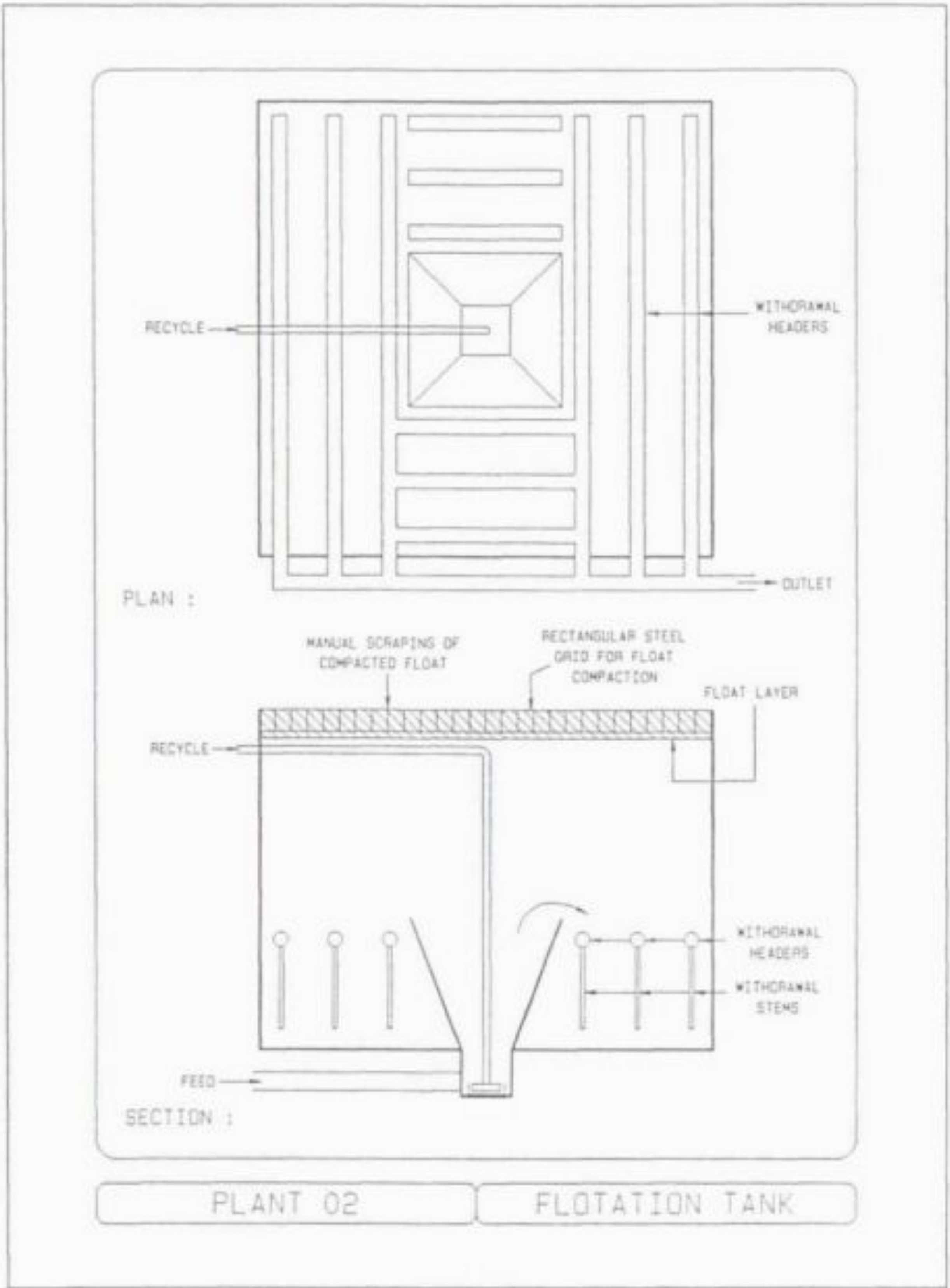
*** Saturation ***			
Position (H/V)			V
Diameter	D_s	m	0.70
Length	L_s	m	1.9
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D_p	m	0.9
Number of saturators			2

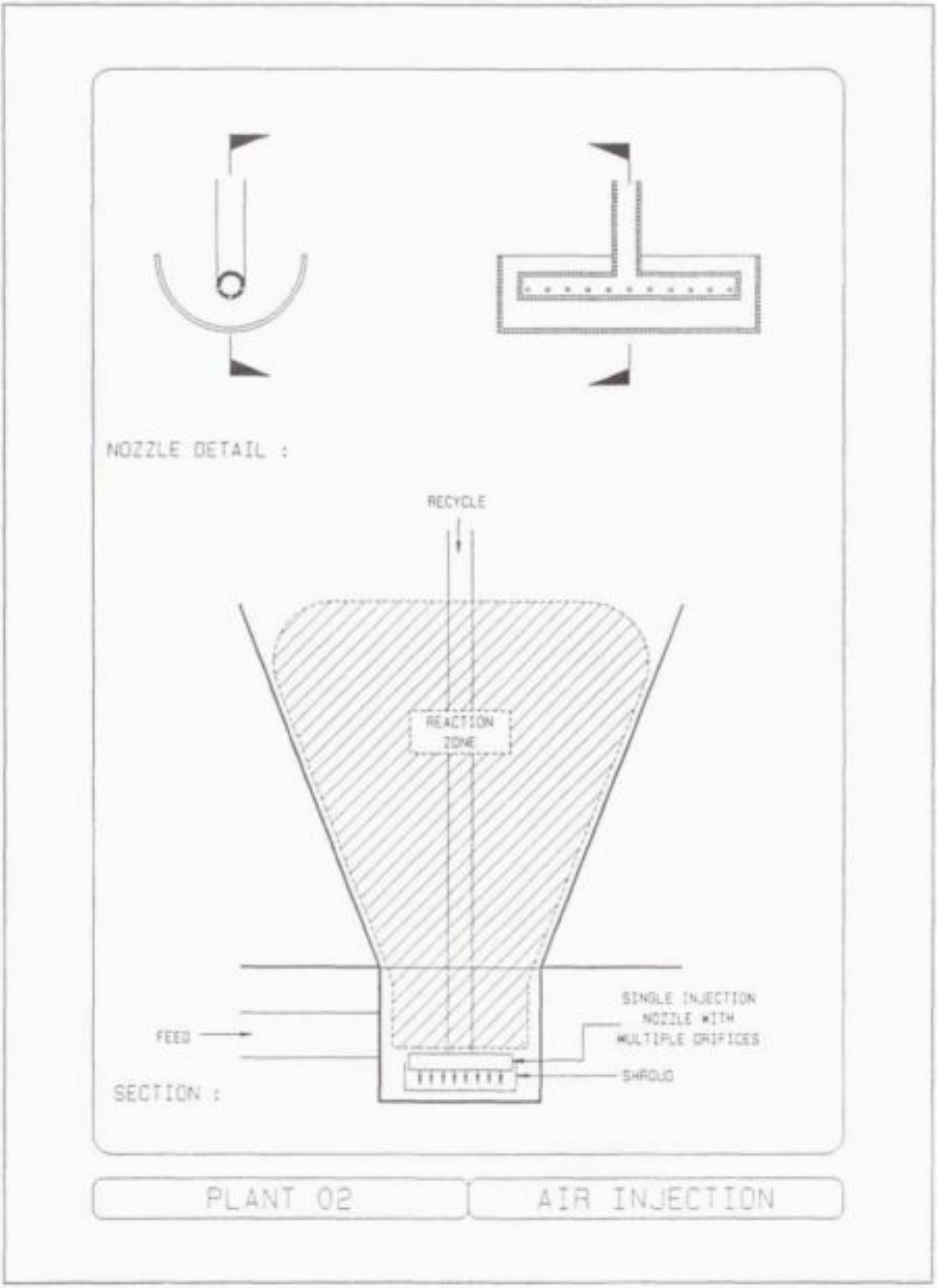
*** Float removal ***	
Grid (Y/N)	Y
Scraper (Y/N)	N
Spacing scrapers	n/a
Speed scrapers	n/a
Continuous scraping (Y/N)	n/a
Bottom desludging (Y/N)	N

*** Notes ***

Tank drained and cleaned once per week

Grid openings 200 mm by 200 mm x 2 mm high. Manual scraping


RECYCLE
WITHDRAWAL HEADERS
OUTLET
PLAN :
MANUAL SCRAPING OF COMPACTED FLOAT
RECTANGULAR STEEL GRID FOR FLOAT COMPACTION
FLOAT LAYER
RECYCLE
WITHDRAWAL HEADERS
WITHDRAWAL STEMS
FEED
SECTION :
PLANT 02
FLOTATION TANK


NOZZLE DETAIL :
RECYCLE
REACTION
ZONE
SINGLE INJECTION
NOZZLE WITH
MULTIPLE ORIFICES
FEED
SHROUD
SECTION :
PLANT 02
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #03

Objective

The plant treats water from a clear inland lake for drinking-water purposes. The water is treated most of the time by direct filtration, but occasional algal blooms are encountered which greatly reduce filter run lengths and bring the plant to a virtual standstill. For this reason, flotation is only infrequently employed during algal blooms only to reduce the solids load on the filters. Under these conditions, flotation follows chemical treatment and flocculation and is carried out in the filter bay above the filter sand.

Feed water

The raw water from the impoundment has very low turbidity, but has infrequent, severe algal blooms. Turbidity varies between 1 and 5 NTU, pH between 7.0 and 8.0 and alkalinity above 45 mg $CaCO_3.\ell^{-1}$.

Chemical conditioning

The treatment chemical (aluminium or polymer) is injected into the raw water pipelines leading into the plant. The chemical is dispersed with an in-line mixer and flocculation occurs in the remainder of the pipe leading to the combined flotation/filter units. The flocculation time is about 5 minutes with a velocity gradient of 110 s^{-1}. Chlorination follows filtration.

Flotation tank

Flotation is carried out above the sand in a rectangular rapid sand filtration bay. The feed is distributed along the length of the filter on the floor of the filter backwash channel. The recycle is similarly injected, close to the concrete surface to enhance even bubble precipitation. The filter backwash channel serves as the reaction zone. The float layer is allowed to accumulate on the water surface until it is washed out at the end of each filtration cycle with the dirty filter backwash water. The water level is kept constant by a hydraulic device downstream of the filter.

Recirculation system

Filtered water is abstracted directly downstream of the filter and pumped through a single saturator. The saturator is mounted vertically and packed with plastic packing. The water flow rate is initially set to the desired rate by throttling of the delivery valve. Once set, the water level in the saturator is maintained by controlling the air flow to the saturator.

Performance

When in use, flotation works satisfactorily and does allow continuous operation of the treatment plant during algal blooms. Problems were initially encountered in that the feed and recycle inlets were placed too closely together, but that had been successfully overcome. High maintenance is required against corrosion and erosion of the recycle injection orifices, drilled into ordinary galvanized steel piping.

PLANT NUMBER # 03

*** Tank geometry ***

Shape (Round/Rect)			Rect
Length	L	m	9.9
Width	W	m	8.9
Outer diameter	D		n/a
Inner diameter	D		n/a
Depth	D	m	2.5
Number flotation cells			1
Flotation area	A	m^2	87.7
Reactor area	A	m^2	5.9
Reactor volume	V	m^3	14.8
Crossflow area	A	m^2	49.3

*** Variable ranges ***

			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	850	638
Recycle rate/cell	q	$k\ell.h^{-1}$	81	54
Saturator pressure	P	kPa	450	350
Concentration in	C			
Concentration out	C			

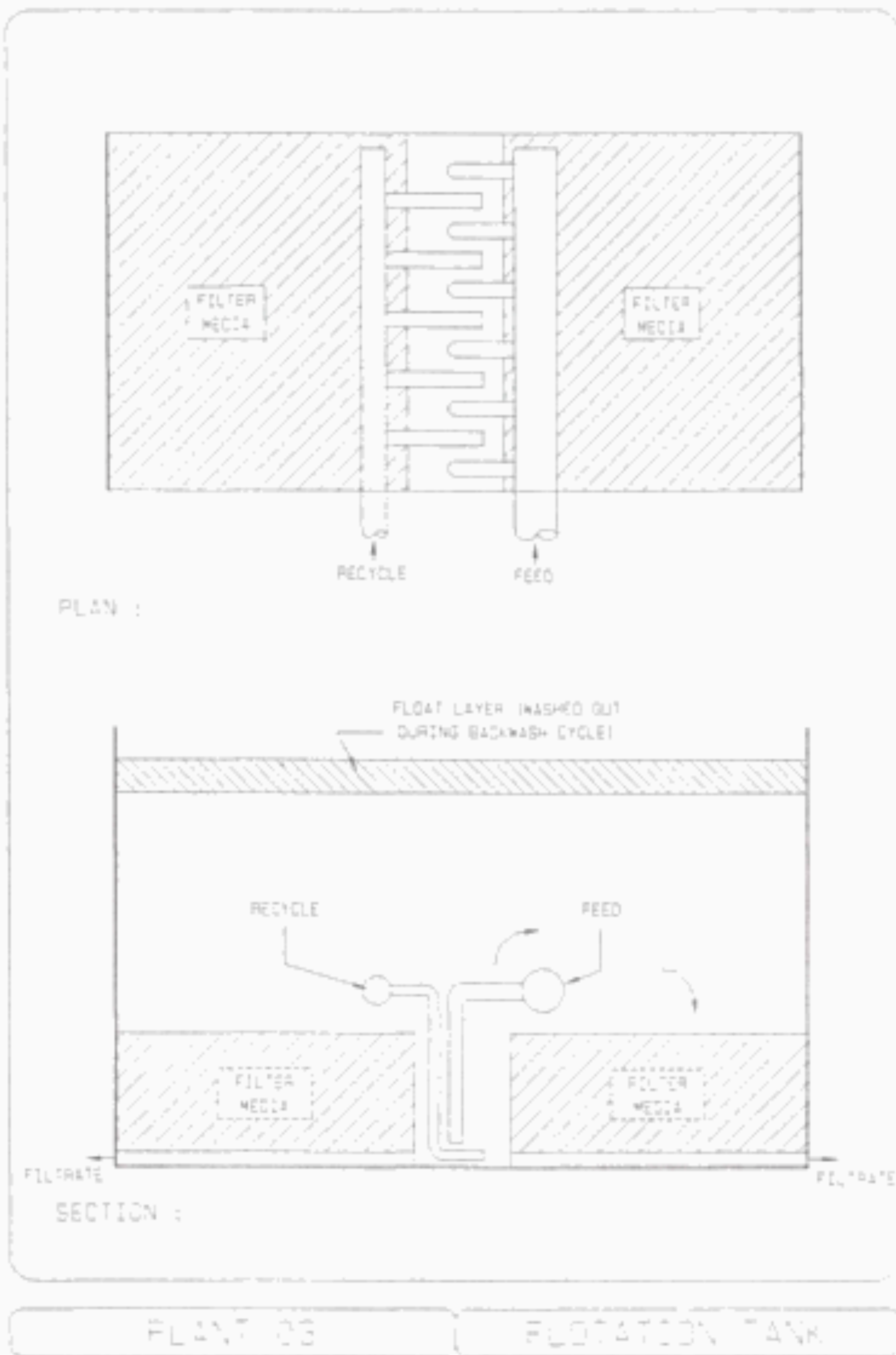
*** Saturation ***

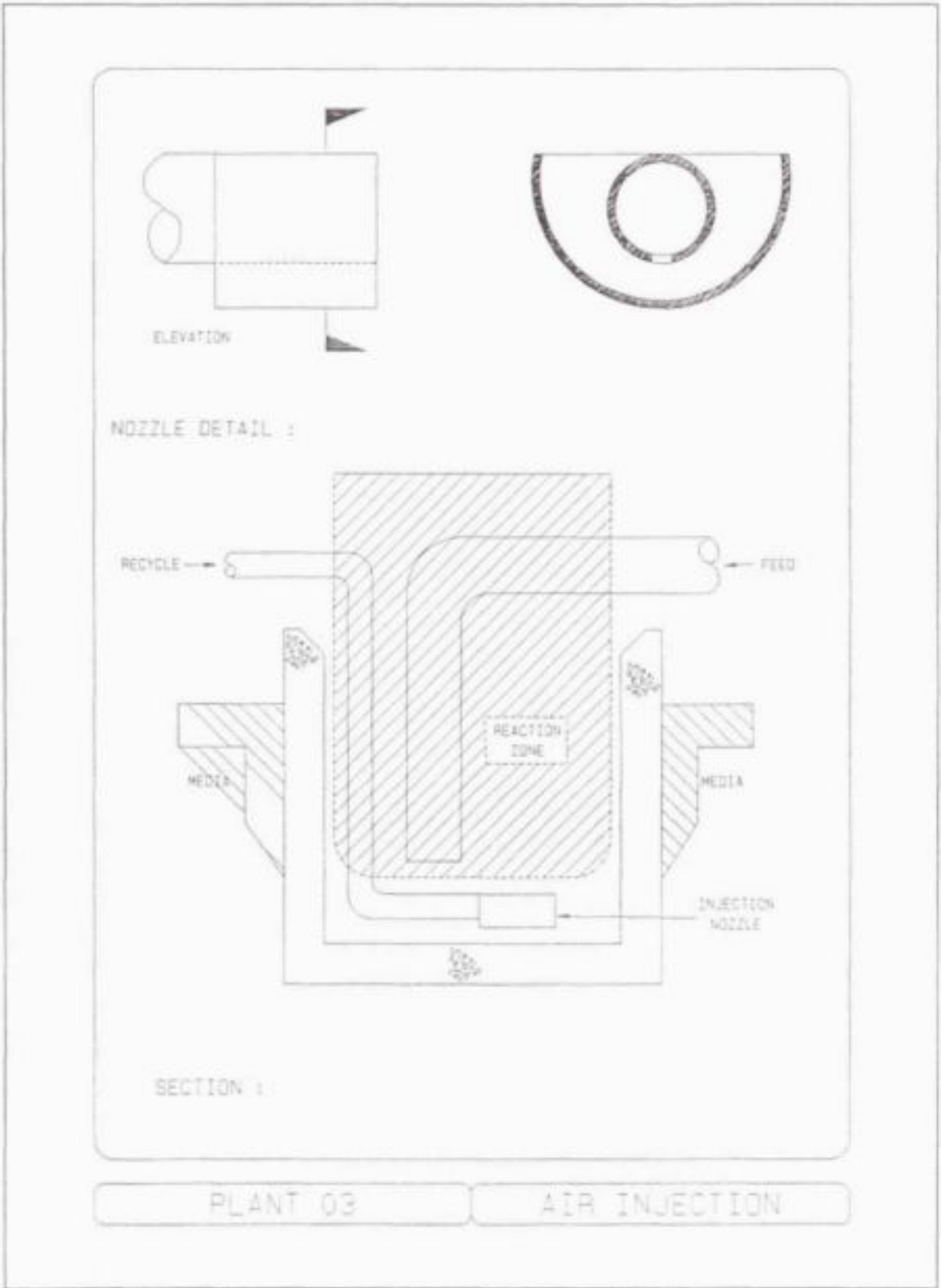
Position (H/V)			V
Diameter	D	m	1.20
Length	L	m	3.0
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D		3.0
Number of saturators			1

*** Float removal ***

Grid (Y/N)	N
Scraper (Y/N)	N
Spacing scrapers	n/a
Speed scrapers	n/a
Continuous scraping (Y/N)	n/a
Bottom desludging (Y/N)	backwash

*** Notes ***


FILTER MEDIA
FILTER MEDIA
RECYCLE
FEED
PLAN :
FLOAT LAYER (WASHED OUT DURING BACKWASH CYCLE)
RECYCLE
FEED
FILTER MEDIA
FILTER MEDIA
FILTRATE
FILTRATE
SECTION :
PLANT 03
FLOTATION TANK


ELEVATION
NOZZLE DETAIL :
RECYCLE
FEED
REACTION ZONE
MEDIA
MEDIA
INJECTION NOZZLE
SECTION :
PLANT 03
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #04

Objective

The plant treats water from an eutrophic impoundment for drinking-water purposes. Flotation follows chemical treatment and flocculation and is carried out together with rapid sand filtration in a combined flotation/filtration unit. Its primary aim is to increase filter run length by reduction of the solids load.

Feed water

The raw water from the impoundment has very low turbidity, but often contains high numbers of green algae and diatoms, with occasional bluegreen algae. Turbidity varies between 1,5 and 6 NTU, chlorophyll a between 5 and 50 $\mu g.\ell^{-1}$, pH between 8,0 and 9,5 and alkalinity above 100 $CaCO_3.\ell^{-1}$.

Chemical conditioning

Raw water is dosed with ferric chloride in an in-line static mixer. Flocculation follows in 2 parallel baffled channels which provide 12 minutes of flocculation at a G-value of 80.s^{-1}. Chlorination follows flotation/filtration.

Flotation tank(s)

Flotation takes place in the head space above the sand of a regular rapid sand filter. There are 10 units operated in parallel. The feed is evenly distributed through 8 inlets along the length of, and on the bottom of the filter backwash channel. The recycle is, likewise, distributed immediately above the feed header through 54 injection nozzles. The subnatant leaves through the sand bed. The float layer is left to accumulate on the water surface and is flushed out at the end of the filter run with the dirty filter backwash water.

Recirculation system

Treated water is withdrawn after chlorination and repressurized by centrifugal pump to 2 vertical saturators which are operated in parallel. The saturators are packed with plastic packing. The recycle rate is adjustable by means of speed control on the recycle pump. Once set by the operator, the water flow rate is fixed. The air supply to the saturator is switched on and off by a level sensor within the saturator, which maintains an air cushion. The recycle is distributed to individual flotation/filtration units through a ring feed.

Performance

The process works very well and has consistently produced the desired quality at full capacity. The subnatant is consistently in the range of 1 to 3 NTU.

PLANT NUMBER # 04

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	7.2	
Width	W	m	3.7	
Outer diameter	D		n/a	
Inner diameter	D		n/a	
Depth	D	m	1.6	
Number flotation cells			10	
Flotation area	A	m^2	23.0	
Reactor area	A	m^2	3.0	
Reactor volume	V	m^3	7.8	
Crossflow area	A	m^2	5.5	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell\,h^{-1}$	167	167
Recycle rate/cell	q	$k\ell\,h^{-1}$	20	21
Saturator pressure	P	kPa	350	260
Concentration in	C	SS		17
Concentration out	C	SS		4

*** Saturation ***

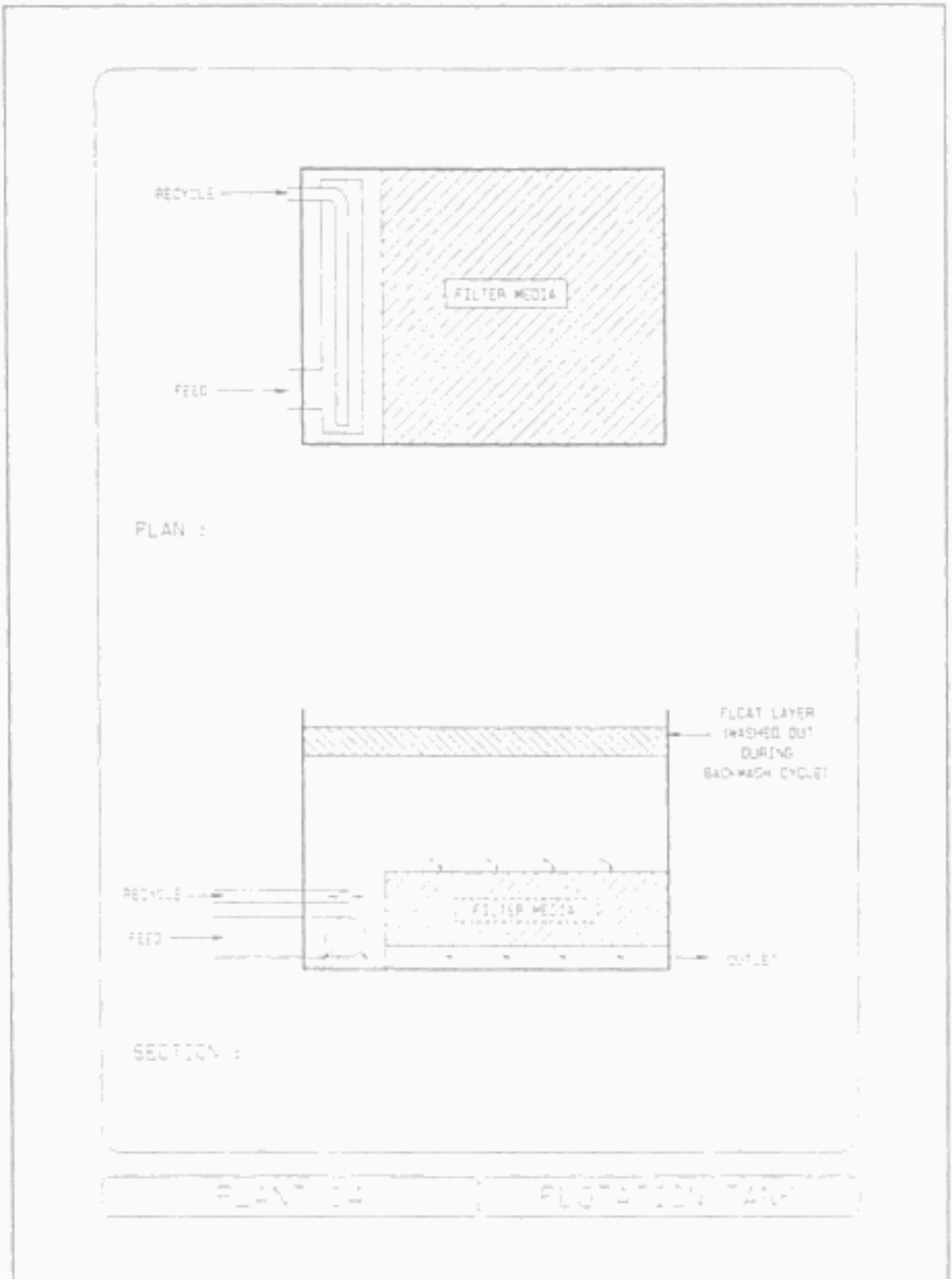
Position (H/V)			V
Diameter	D	m	1.60
Length	L	m	2.0
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D	m	1.5
Number of saturators			2

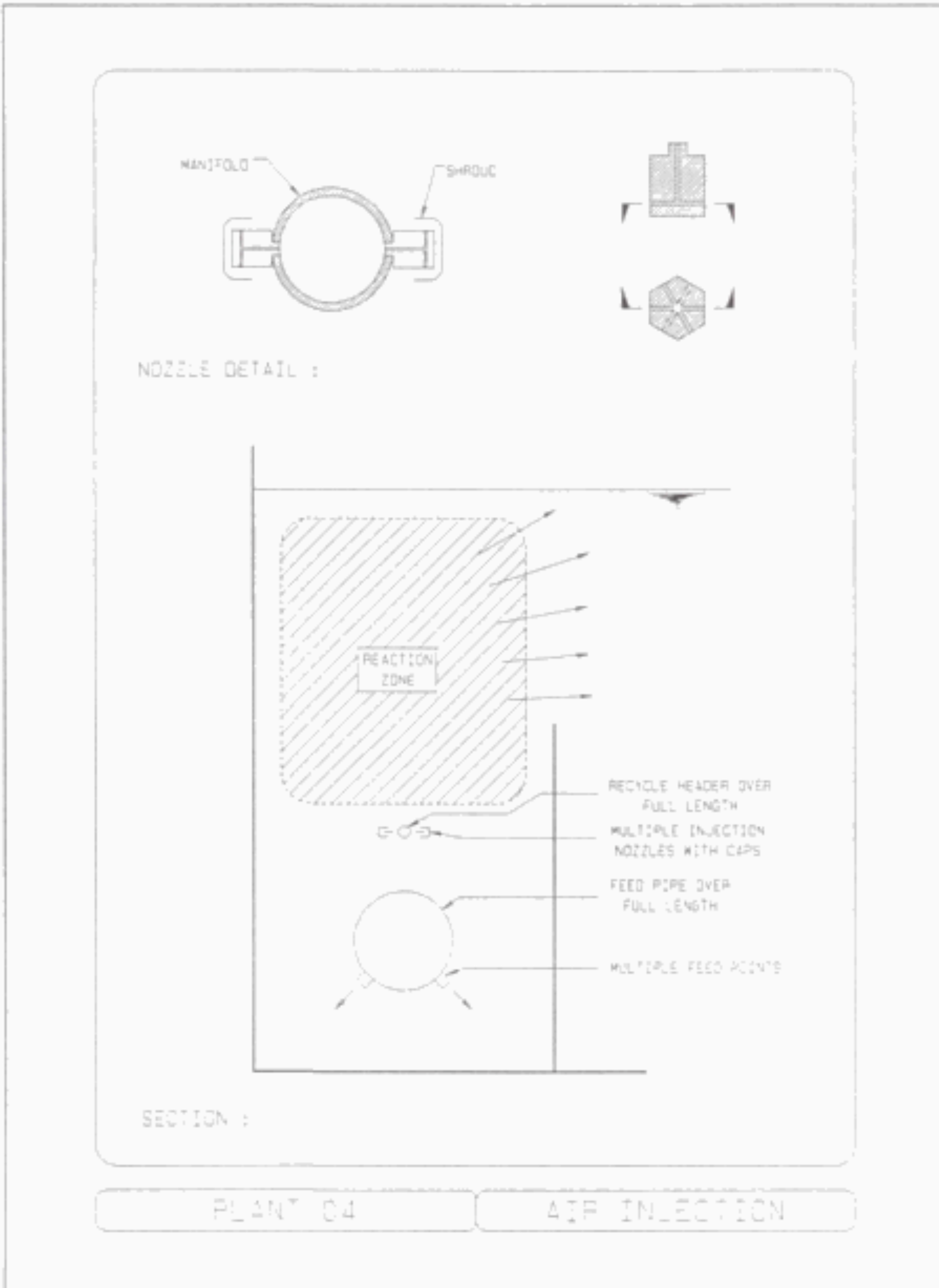
** Float removal ***

Grid (Y/N)	N
Scraper (Y/N)	N
Spacing scrapers	n/a
Speed scrapers	n/a
Continuous scraping (Y/N)	N
Bottom desludging (Y/N)	N

*** Notes ***

This plant has combined flotation / filtration units


RECYCLE
FILTER MEDIA
FEED
PLAN :
FLOAT LAYER
(WASHED OUT
DURING
BACKWASH CYCLE)
RECYCLE
FILTER MEDIA
FEED
OUTLET
SECTION :


MANIFOLD
SHROUD
NOZZLE DETAIL :
REACTION ZONE
RECYCLE HEADER OVER FULL LENGTH
MULTIPLE INJECTION NOZZLES WITH CAPS
FEED PIPE OVER FULL LENGTH
MULTIPLE FEED POINTS
SECTION :
PLANT 04
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #05

Objective

The plant treats water from a small impoundment for drinking-water purposes The water is chemically treated and flocculated by mechanical stirrers before flotation Rapid sand filtration follows flotation. The primary purpose of the flotation is to reduce the solids load on the filters

Feed water

The water has, on the average, low turbidity with eutrophic character during dry spells

Chemical conditioning

The water is dosed with ferric chloride aluminium sulphate or polymer at the inlet weir Extended flocculation follows in 2 mechanically stirred tanks in series which allows for a flocculation period in excess of 2 hours per tank Chlorination and lime addition follows filtration

Flotation tank(s)

Flotation is carried out in 6 rectangular tanks with flat bottoms The water level in the tank is controlled by a fixed overflow weir Each tank is equipped with both a bottom and float scraper The float scraper is done with a series of blades which reciprocates continuously and driven from the high pressure system.

Recirculation system

The recycle is drawn from the filtered water supply and pumped through a single, vertical, unpacked saturator. The air supply is controlled with a magnetic switch to maintain a constant water level in the saturator The recycle is introduced from a manifold to individual tanks and pressure released through 32 cupped nozzles at the bottom of each tank where it is blended with the feed

Performance

In general, the performance is satisfactory and low supervision and maintenance was reported. The hydraulic loading is at present only about 50 % of design capacity

PLANT NUMBER # 05

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	8.2	
Width	W	m	5.0	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	3.0	
Number flotation cells			6	
Flotation area	A_f	m^2	41.0	
Reactor area	A_r	m^2	5.0	
Reactor volume	V_r	m^3	15.0	
Crossflow area	A_c	m^2	5.0	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	208.3	184.2
Recycle rate/cell	q	$k\ell.h^{-1}$	18.3	18.3
Saturator pressure	P	kPa	550	540
Concentration in	C	NTU		10
Concentration out	C_e	NTU		3

*** Saturation ***

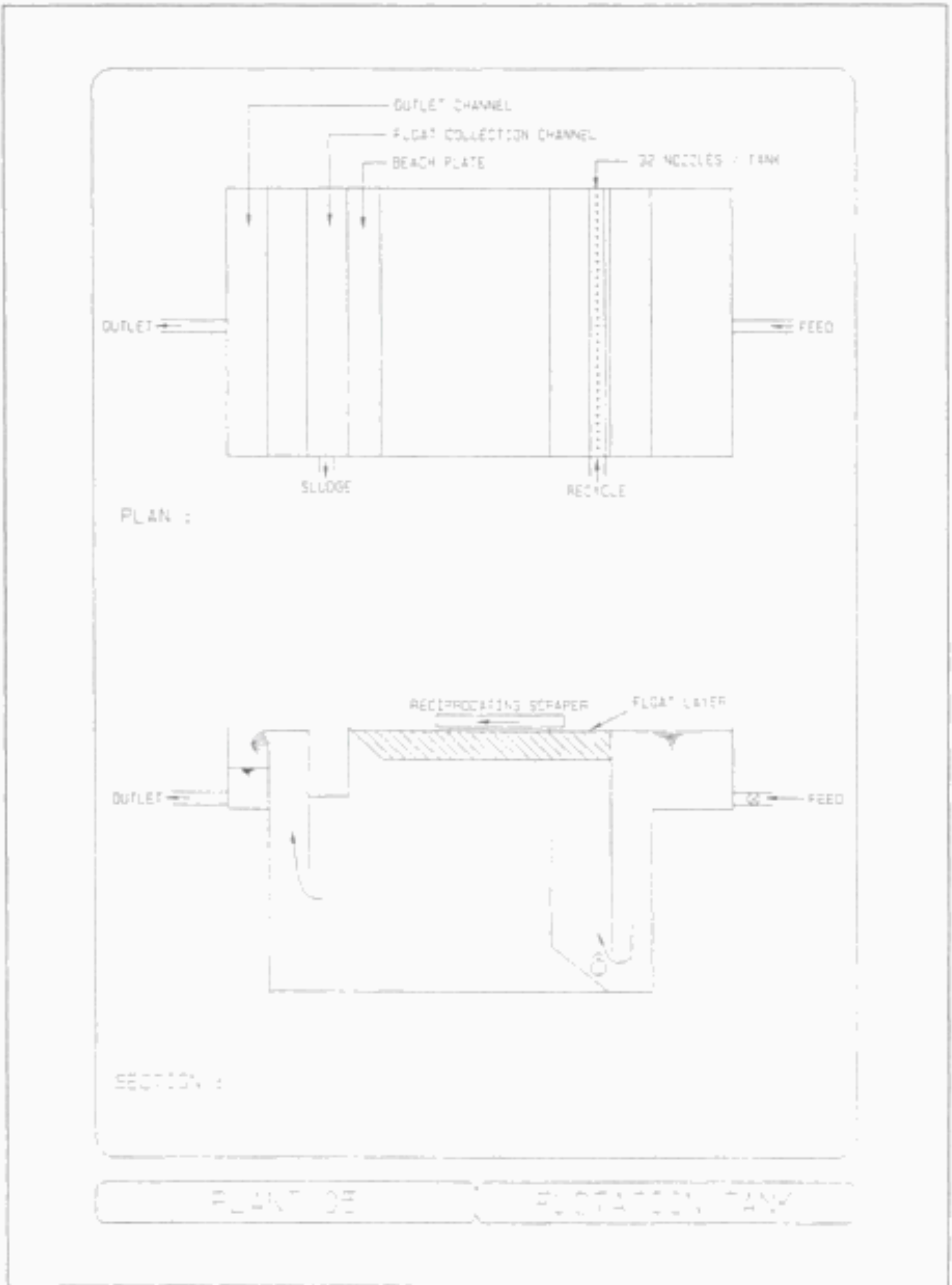
Position (H/V)			V
Diameter	D_s	m	1.30
Length	L_s	m	3.0
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D_p		2.0
Number of saturators			1

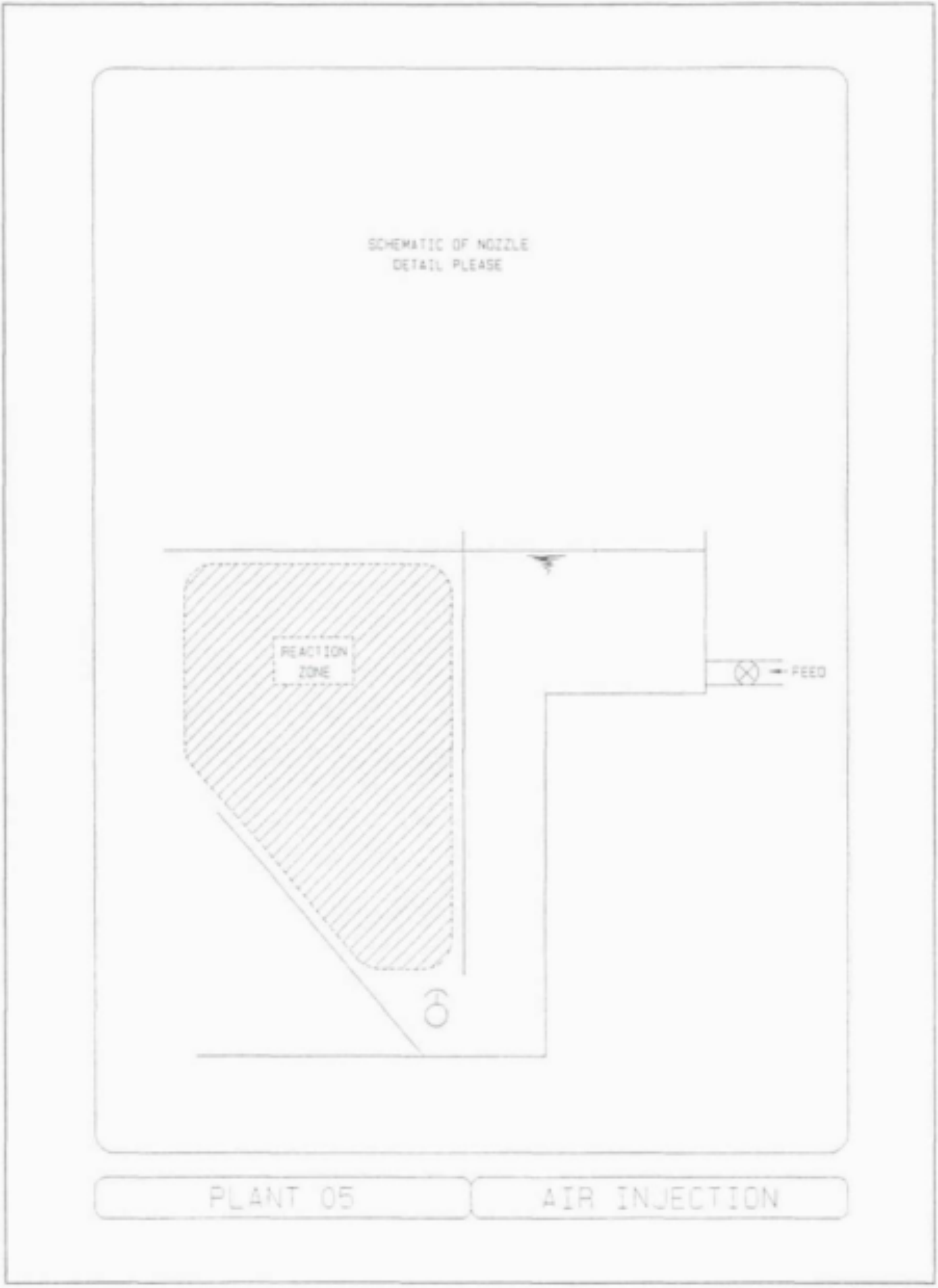
*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	m	1.3
Speed scrapers	m/h	40.0
Continuous scraping (Y/N)		Y
Bottom desludging (Y/N)		backwash

*** Notes ***

Bottom desludging once a week


OUTLET CHANNEL
FLOAT COLLECTION CHANNEL
BEACH PLATE
32 NOZZLES / TANK
OUTLET
FEED
SLUDGE
RECYCLE
PLAN :
RECIPROCATING SCRAPER
FLOAT LAYER
OUTLET
FEED


SCHEMATIC OF NOZZLE
DETAIL PLEASE
REACTION
ZONE
FEED
PLANT 05
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #06

Objective

The objective of the treatment plant is to treat an industrial effluent to meet the legal discharge standards before disposal into a natural stream. Flotation follows chemical treatment and flocculation. The water is discharged immediately after flotation.

Feed water

The raw water treated is an approximately equal mix of cooling tower blowdown water and secondary sewage effluent and collected in a balancing tank, where limited settling takes place. The composition of the water therefore is fairly constant and it is characterized by low turbidity and mostly organic solids. The pH varies between 6,0 and 9,0, the turbidity between 2,5 and 8,0 NTU and the COD between 10 and 80 $mg.\ell^{-1}$.

Chemical conditioning

The water is chemically treated by aluminium sulphate, which is dispersed in an in-line mixer. This is followed by flocculation for about 10 minutes with a mechanical stirrer, capable of inducing a maximum G-value of $130.s^{-1}$. During the occasional mixer down-time, it was found that the flotation still performed reasonably well.

Flotation tanks

Flotation is carried out in 2 rectangular tanks. The feed water enters each tank at a single point through an enlarged vertical pipe section. The recycle is added at the point where the feed enters the enlarged pipe section. The recycle pressure release is done through an adjustable diaphragm valve a short distance away from the entry point. Each tank is equipped with a reciprocating float scraper operating intermittently. No bottom scraper is installed, and regular manual desludging is not necessary. The subnatant is abstracted through 2 horizontal pipe manifolds, and the water level is controlled by an adjustable overflow weir.

Recirculation system

The recyle water is withdrawn immediately after flotation and pumped through a single, unpacked vertical saturator. The water feed rate is kept constant, and the water level in the saturator is kept constant by a level sensor in the tank and a solenoid valve on the air supply. The water level is maintained at a high level: about 25% of the total saturator depth from the top. A second level sensor near the bottom acts as a safeguard against the saturator running dry; this sensor activates a valve on the saturator outlet which stops the water flow abruptly when this level is reached.

Performance

This plant operates exceptionally well with the minimum supervision and maintenance. On the average, the turbidity is reduced from 4 NTU to 1,5 NTU. The turbidity of the subnatant routinely reaches 0,5 NTU. The phosphate (as P) is, on the average reduced from 2,0 $mg.\ell^{-1}$ to 0,3 $mg.\ell^{-1}$.

PLANT NUMBER # 06

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	5.5	
Width	W	m	2.5	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	3.0	
Number flotation cells			2	
Flotation area	A_f	m^2	13.8	
Reactor area	A_r	m^2	1.1	
Reactor volume	V_r	m^3	7.5	
Crossflow area	A_c	m^2	2.2	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	100100	
Recycle rate/cell	q	$k\ell.h^{-1}$	12	12
Saturator pressure	P	kPa	450	430
Concentration in	C_i	NTU		4.0
Concentration out	C_o	NTU		1.5

*** Saturation ***

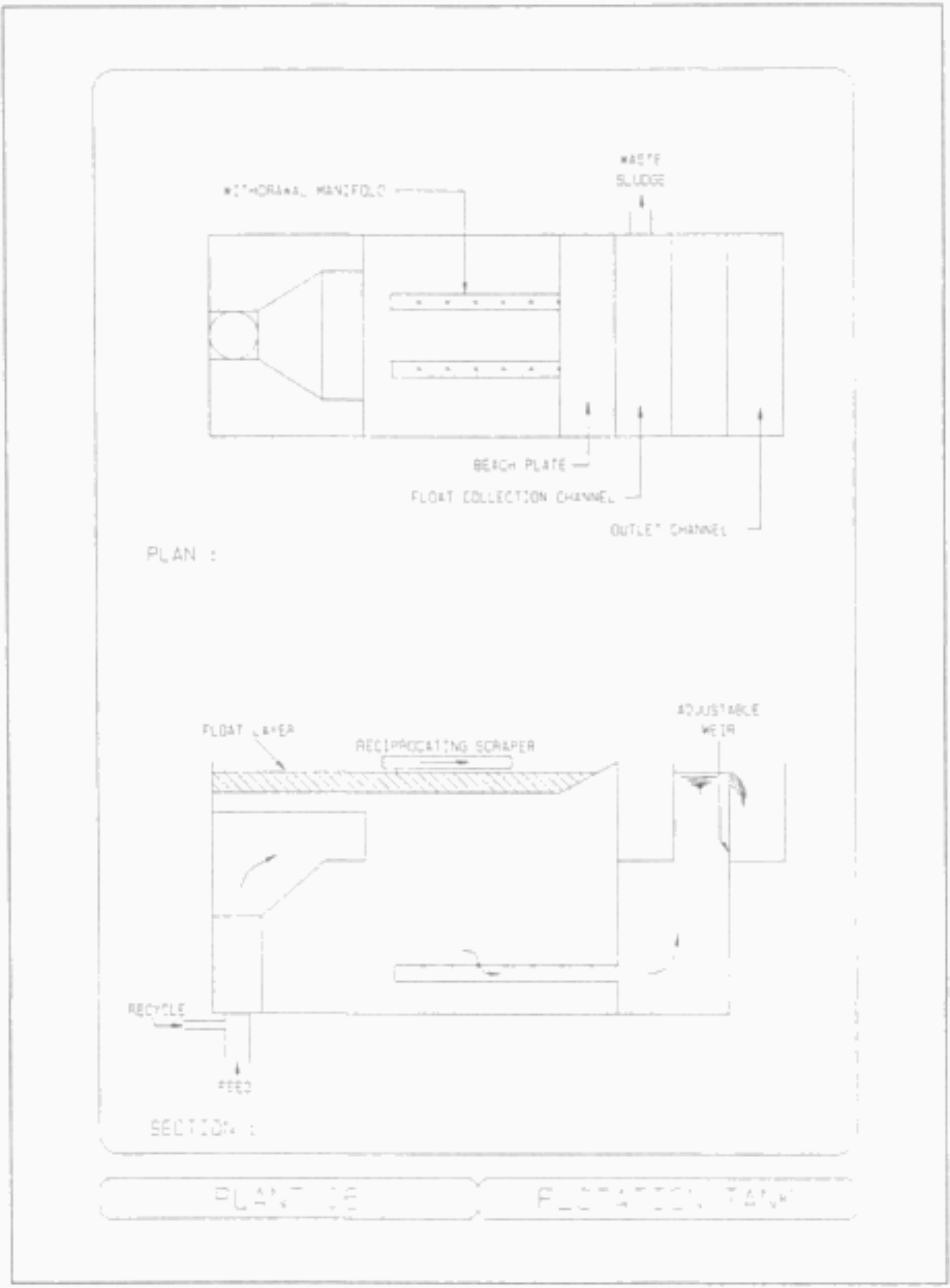
Position (H/V)			V
Diameter	D_s	m	0.95
Length	L_s	m	1.5
Packing (Y/N)			N
Recirculation (Y/N)			N
Depth packing	D_p		n/a
Number of saturators			1

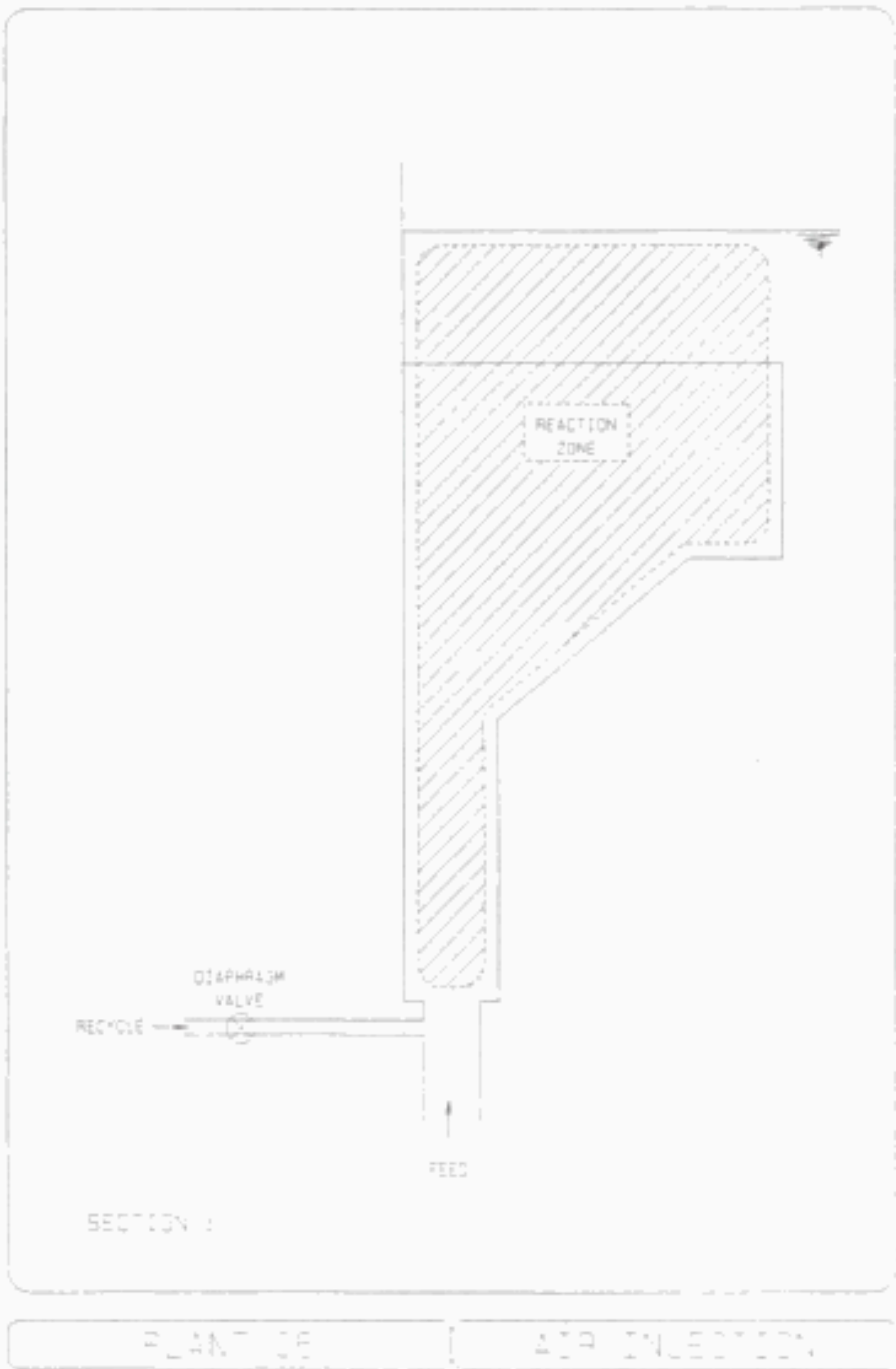
*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	m	2.5
Speed scrapers	m/h	200.0
Continuous scraping (Y/N)		N
Bottom desludging (Y/N)		N

*** Notes ***

Five minutes between reciprocal scraping.


WASTE SLUDGE
WITHDRAWAL MANIFOLD
BEACH PLATE
FLOAT COLLECTION CHANNEL
OUTLET CHANNEL
PLAN :
ADJUSTABLE WEIR
FLOAT LAYER
RECIPROCATING SCRAPER
RECYCLE
FEED
SECTION :
FLOTATION TANK


REACTION
ZONE
DIAPHRAGM
VALVE
RECYCLE
FEED

DESCRIPTION OF FLOTATION PLANT #07

Objective

The objective of the treatment plant is to reduce the phosphate concentration of an industrial effluent before discharge into a natural stream. Flotation is the final process before discharge, and is preceded by chemical treatment and flocculation. The float layer is dried on drying beds before disposal. Effluent is used in the cooling system.

Feed water

The feed water consists of cooling tower blowdown water, which in turn is derived from secondary sewage effluent. Because of a balancing tank, the flow rate into the plant can be kept very constant and the plant operates permanently. The suspended solids, before chemical treatment, varies between 7 and 73 mg.ℓ^{-1} with an average of about 20 mg.ℓ^{-1}.

Chemical conditioning

Aluminium sulphate is used as the only coagulant and is injected through a multitude of orifices into a small dosing tank. Provision is made for flocculation for between 7 and 10 minutes with mechanical stirrers, capable of providing a maximum G-value of 130.s^{-1}, but the mixers are seldom turned on. The plant works well even if the mixers are turned off. Provision is also made for final pH correction with caustic soda, but this has never been necessary.

Flotation tank

There is one circular flotation tank with a reaction zone in the centre. The feed water enters from below into the reaction zone. Immediately above the feed inlet, the feed is evenly distributed into the reaction zone with a horizontal plate with a number of orifices. The recycle is distributed above this plate by 250 small injection nozzles. From the reaction zone, the water flows radially outward to the flotation zone. The inlet rate into the plant is fixed and the tank level is controlled by a pressure sensor and a control valve on the outlet pipe. There are 2 radial float scrapers, continuously operated. There is no bottom scraper and bottom desludging is not routinely done.

Recirculation system

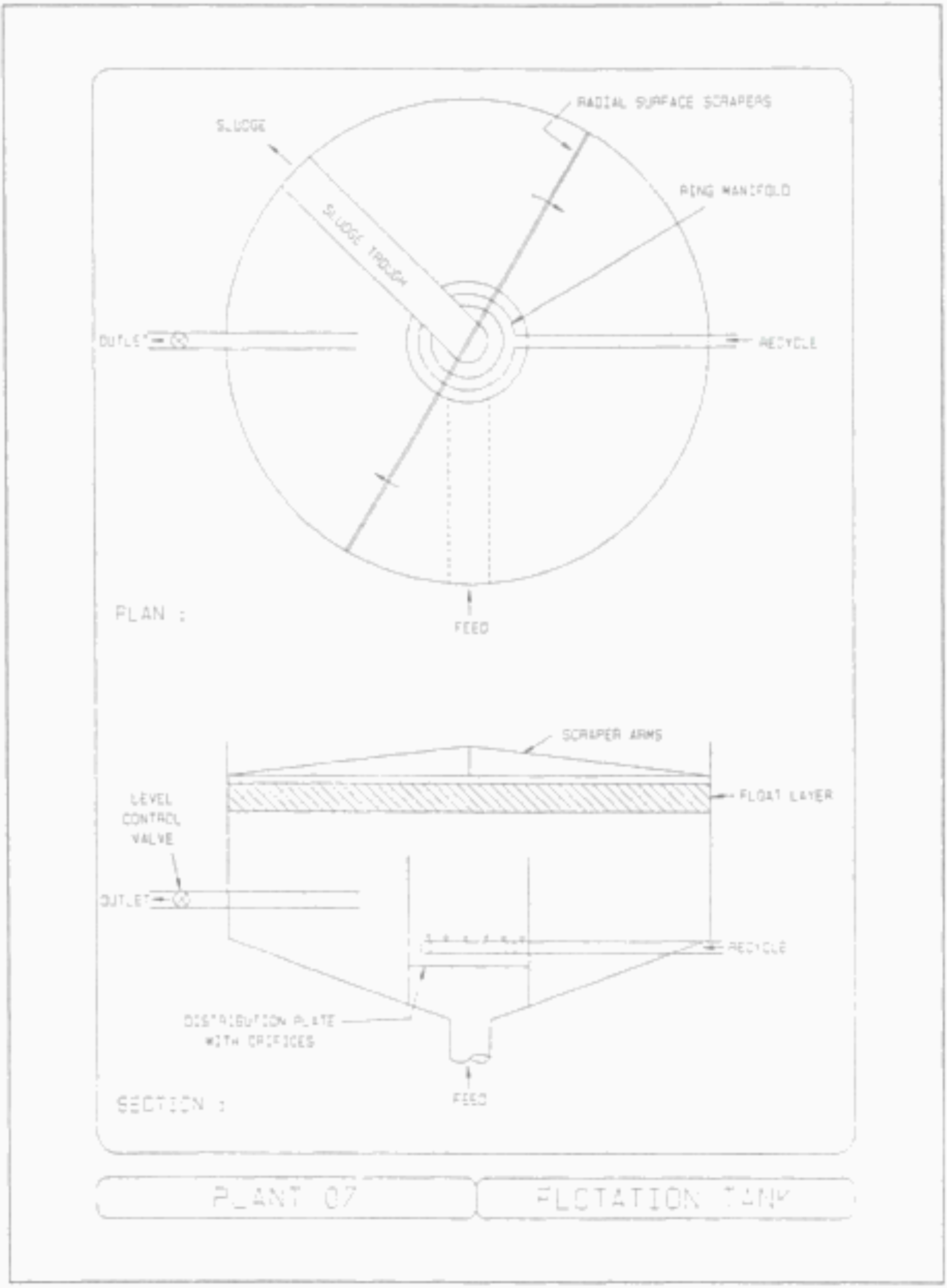
The recycle is drawn from subnatant and pumped through any one of 2 vertical saturators, packed with plastic packing. The water level in the saturator is kept constant with a solenoid valve, controlled by 2 capacitance probes in a level sensing tube outside the saturator.

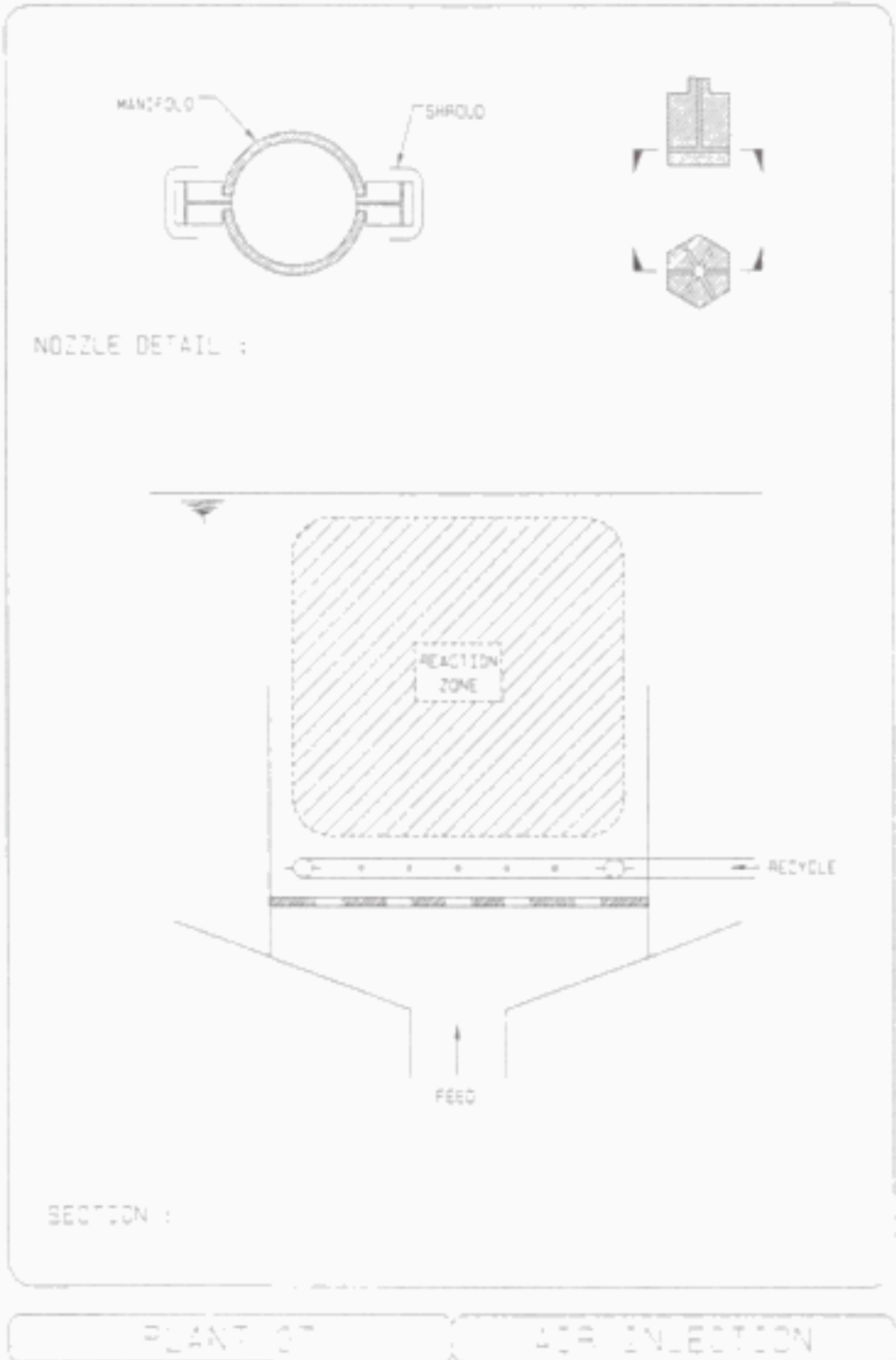
Performance

The flotation plant performs well. On the average, the suspended solids is reduced from 20 mg.ℓ^{-1} to about 5 mg.ℓ^{-1}, and phosphate (as P) is easily kept below 1 mg.ℓ^{-1}. The above description is of the plant as it is at present. Originally, the reaction zone was configured differently with only 2 recycle inlets, but this system never worked satisfactorily and had to be replaced.

PLANT NUMBER			# 07	
***** Tank geometry *****				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D	m	13.5	
Inner diameter	D	m	4.6	
Depth	D	m	2.2	
Number flotation cells			1	
Flotation area	A	m^2	126.5	
Reactor area	A	m^2	16.6	
Reactor volume	V	m^3	15.6	
Crossflow area	A	m^2	31.8	
***** Variable ranges *****			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	750	260
Recycle rate/cell	q	$k\ell.h^{-1}$	75	48
Saturator pressure	P	kPa	550	420
Concentration in	C	SS		40
Concentration out	C	SS		9
***** Saturation *****				
Position (H/V)			V	
Diameter	D	m	1.30	
Length	L	m	2.5	
Packing (Y/N)			Y	
Recirculation (Y/N)			N	
Depth packing	D	m	0.8	
Number of saturators			1	
***** Float removal *****				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		no	2	
Speed scrapers		$r.h^{-1}$	6.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			N	

*** Notes ***


RADIAL SURFACE SCRAPERS
SLUDGE
SLUDGE TROUGH
RING MANIFOLD
OUTLET
RECYCLE
PLAN :
FEED
SCRAPER ARMS
FLOAT LAYER
LEVEL CONTROL VALVE
OUTLET
RECYCLE
DISTRIBUTION PLATE WITH ORIFICES
FEED
SECTION :
PLANT 07
FLOTATION TANK


MANIFOLD
SHROUD
NOZZLE DETAIL :
REACTION ZONE
RECYCLE
FEED
SECTION :
PLANT OF
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #08

Objective

The objective of the plant is to produce municipal drinking water. The water is chemically dosed, flocculated and then passes on to the flotation tank. After flotation, the water runs through a primary settling tank and is then filtered through rapid sand filters, sent through granular activated carbon columns and chlorinated.

Feed water

The feed water consists of treated sewage effluent. The pH of the water is between pH 6,7 and pH 8.9 and with turbidity between 2 NTU and 15 NTU. The COD of the water is between 27 and 42 mg ℓ^{-1} and the DOC concentration varies between 11 and 15 mg ℓ^{-1}

Chemical conditioning

The water entering the plant is dosed with aluminium sulphate (typically 120 mg ℓ^{-1}, expressed in terms of the kibbled product) into the first flocculation compartment. Agitation in this tank is achieved by means of a flash mixer at a velocity gradient of about 140.s^{-1}. The retention time is 5 to 8 minutes. Lime (between 15 and 30 mg.ℓ^{-1}) is added as a slurry at the inlet to the reactor zone in the flotation zone. The water is fairly well buffered and the pH drop after coagulant addition is small.

Flotation tank

This is one circular flotation tank with a central reaction zone. This tank was converted from an obsolete settling tank. There is no overflow weir; the water level is manually controlled with a valve on the discharge line. The feed enters in the bottom centre of the tank; the recycle is released immediately above the feed inlet. The central reaction zone is separated from the flotation by a circular skirt extending from the bottom of the tank almost to the top of the tank. The subnatant is collected with 16 radial collector pipes which join onto a common ring manifold. The tank is equipped with 2 surface scrapers, but without bottom scrapers.

Recirculation system

The recycle is drawn from the subnatant and pumped through a single, vertical, packed saturator. The water level in the saturator is maintained by an automatic control valve on the water supply. The air supply is constant and can be set at a predetermined rate.

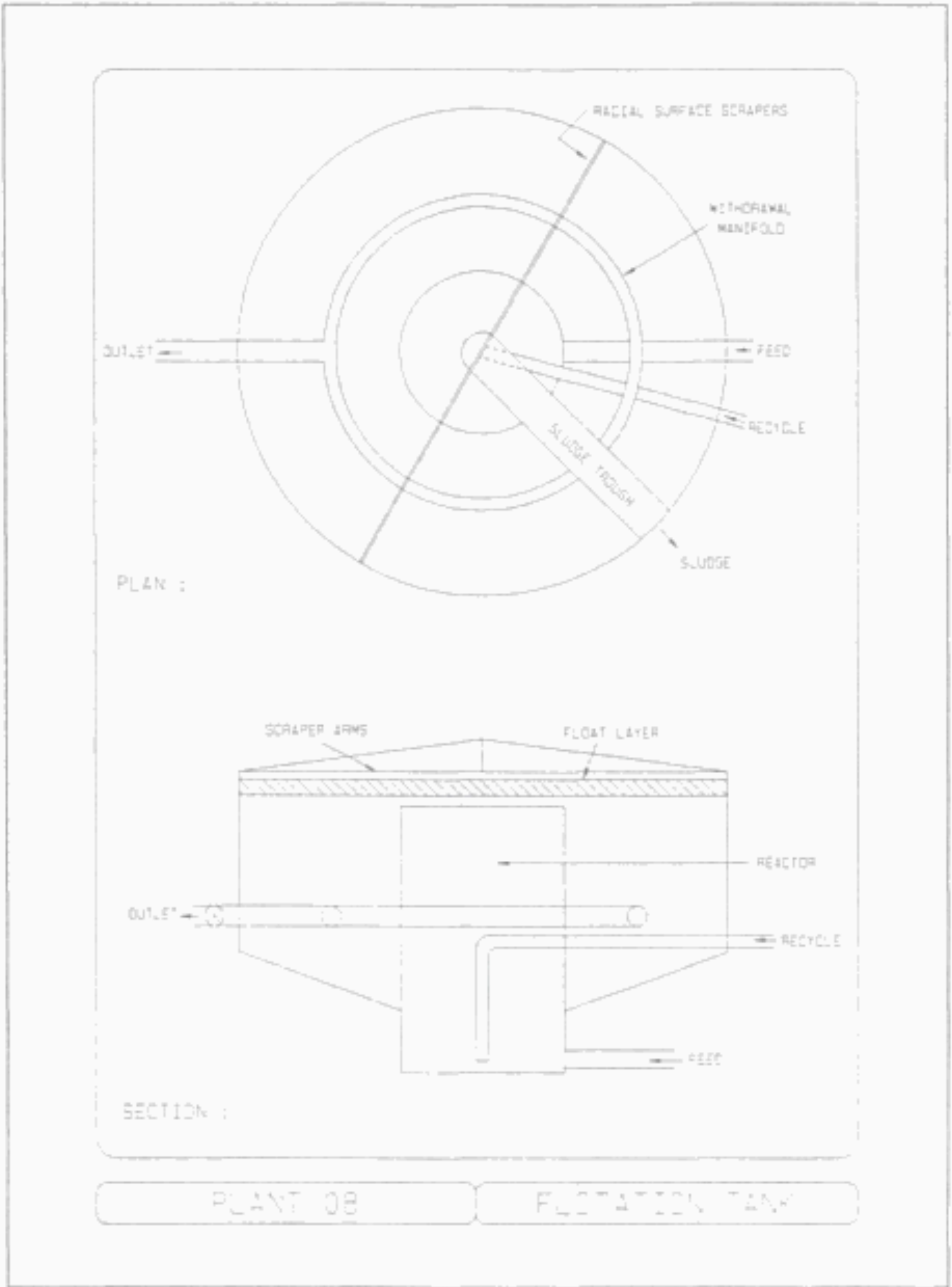
Performance

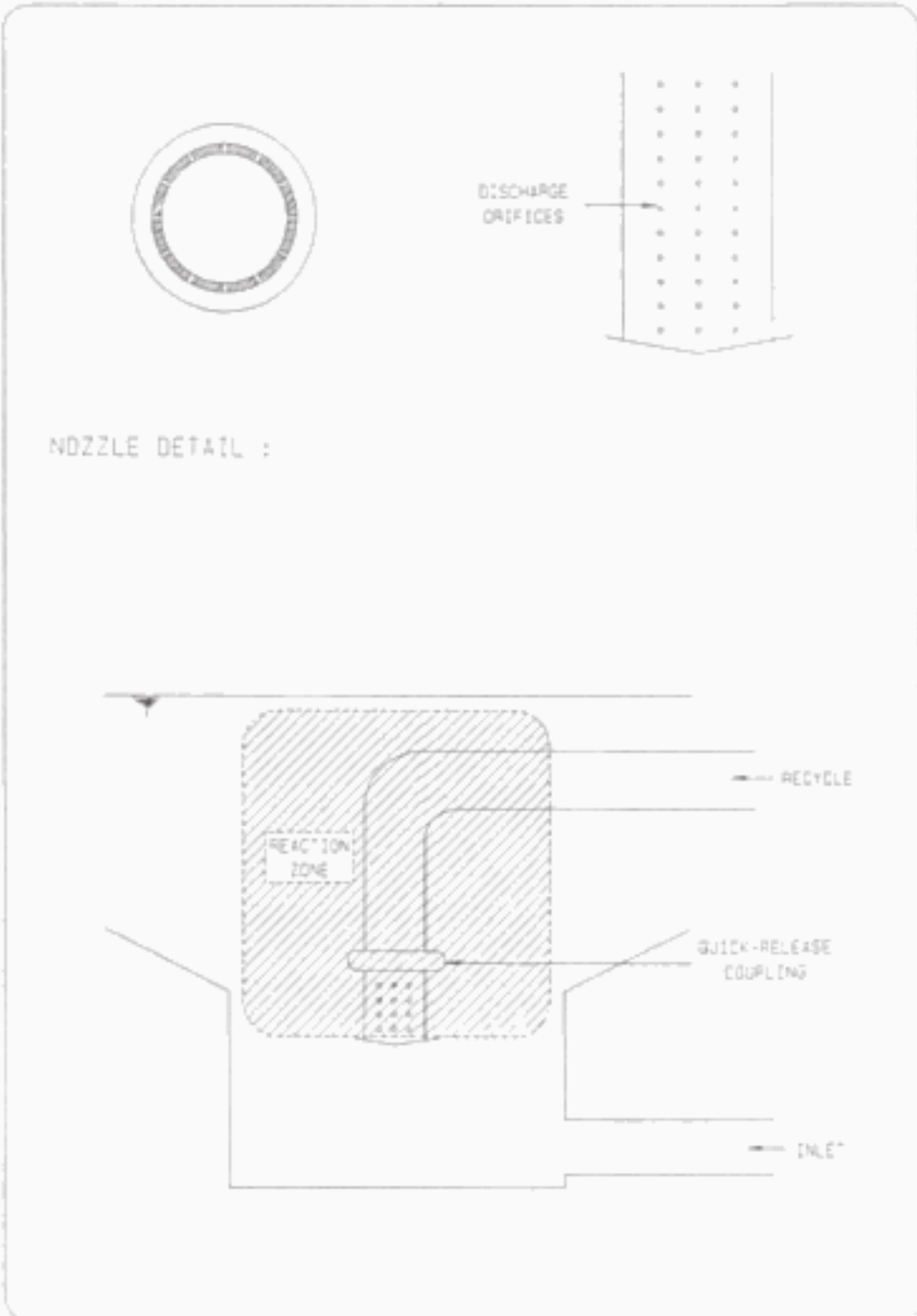
The turbidity reduction during flotation is reasonably good. The data for a typical recent month, for example, shows that the influent turbidity ranged between 2,0 NTU and 15,0 NTU with the effluent turbidity between 1.5 NTU and 2.8 NTU.

PLANT NUMBER			# 08	
*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	7.6	
Inner diameter	D	m	2.1	
Depth	D_t	m	1.8	
Number flotation cells			1	
Flotation area	A_f	m^2	42.0	
Reactor area	A_r	m^2	3.4	
Reactor volume	V_r	m^3	3.8	
Crossflow area	A_c	m^2	3.4	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	7200	4800
Recycle rate/cell	q	$k\ell.h^{-1}$	864	432
Saturator pressure	P	kPa	500	450
Concentration in	C_i	NTU		3.8
Concentration out	C_o	NTU		2.2
*** Saturation ***				
Position (H/V)			V	
Diameter	D_s	m	0.85	
Length	L_s	m	1.7	
Packing (Y/N)			Y	
Recirculation (Y/N)			N	
Depth packing	D_p		0.8	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		no	180.0	
Speed scrapers		$r.h^{-1}$	5.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			N	

*** Notes ***

The tank is manually cleaned every 2 - 3 weeks


RADIAL SURFACE SCRAPERS
WITHDRAWAL MANIFOLD
OUTLET
FEED
RECYCLE
SLUDGE TROUGH
SLUDGE
PLAN :
SCRAPER ARMS
FLOAT LAYER
REACTOR
OUTLET
RECYCLE
FEED
SECTION :
PLANT 08
FLOTATION TANK



PLANT 08 | AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #09

Objective

The flotation plant is used for colour removal of sewage effluent prior to the water being used in the production of high-brightness pulp and paper.

Feed water

The feed water is a blend of the sewage effluents from 2 municipal waste-water treatment plants. After conventional treatment with biological filters, the effluent is filtered through rapid sand filtration units and then taken to a raw water storage impoundment. The pH ranges between pH 7,0 and pH 7,5 and the suspended solids between 6 and 9 $mg.\ell^{-1}$. The chemical oxygen demand is between 40 $mg.\ell^{-1}$ and 50 $mg.\ell^{-1}$.

Chemical conditioning

Aluminium sulphate is injected into the pipe connecting the saturator with the flotation tank; the aluminium sulphate dosage being automatically controlled with a streaming current detector. Polymer is injected directly into the reaction zone. There is no separate flocculation zone and flocculation takes place in the tank inlet pipe and in the central reaction zone inside the tank. After flotation, the water is stabilized with caustic soda and disinfected by chlorination and bromination.

Flotation tank

There is a single circular tank with a central reaction zone. Because of the full-stream saturation, there is only one inlet at the bottom of the tank. The water is then directed through a series of annular vertical baffles before it enters the flotation zone. The water is withdrawn from a fixed peripheral overflow weir. There are 3 radial surface scrapers, but no bottom scrapers.

Recirculation system

This plant makes use of full-stream saturation in a vertical, unpacked saturator. The water enters from the top, but the air is introduced at the bottom of the saturator. There is a high-speed turbine at the base of the saturator immediately below the air introduction point,which disperses the air into the water column. An automatically controlled valve on the outlet keeps the level in the saturator constant at about 75 % of the saturator height.

Performance

The plant has been working well for more than 20 years. The tank is drained and cleaned once per month, but not much bottom sludge is removed. During its early years, rapid sand filtration was attempted immediately after flotation, but had to be abandoned because of excessive air binding and very short filter runs.

PLANT NUMBER # 09

*** Tank geometry ***

Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	19.4	
Inner diameter	D	m	5.6	
Depth	D.	m	2.8	
Number flotation cells			1	
Flotation area	A_f	m^2	271.0	
Reactor area	A_r	m^2	12.3	
Reactor volume	V_r	m^3	52.3	
Crossflow area	A_c	m^2	18.0	

*** Variable ranges ***

			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	1250	750
Recycle rate/cell	q	$k\ell.h^{-1}$	0	0
Saturator pressure	P	kPa	69	100
Concentration in	C_i			
Concentration out	C_o			

*** Saturation ***

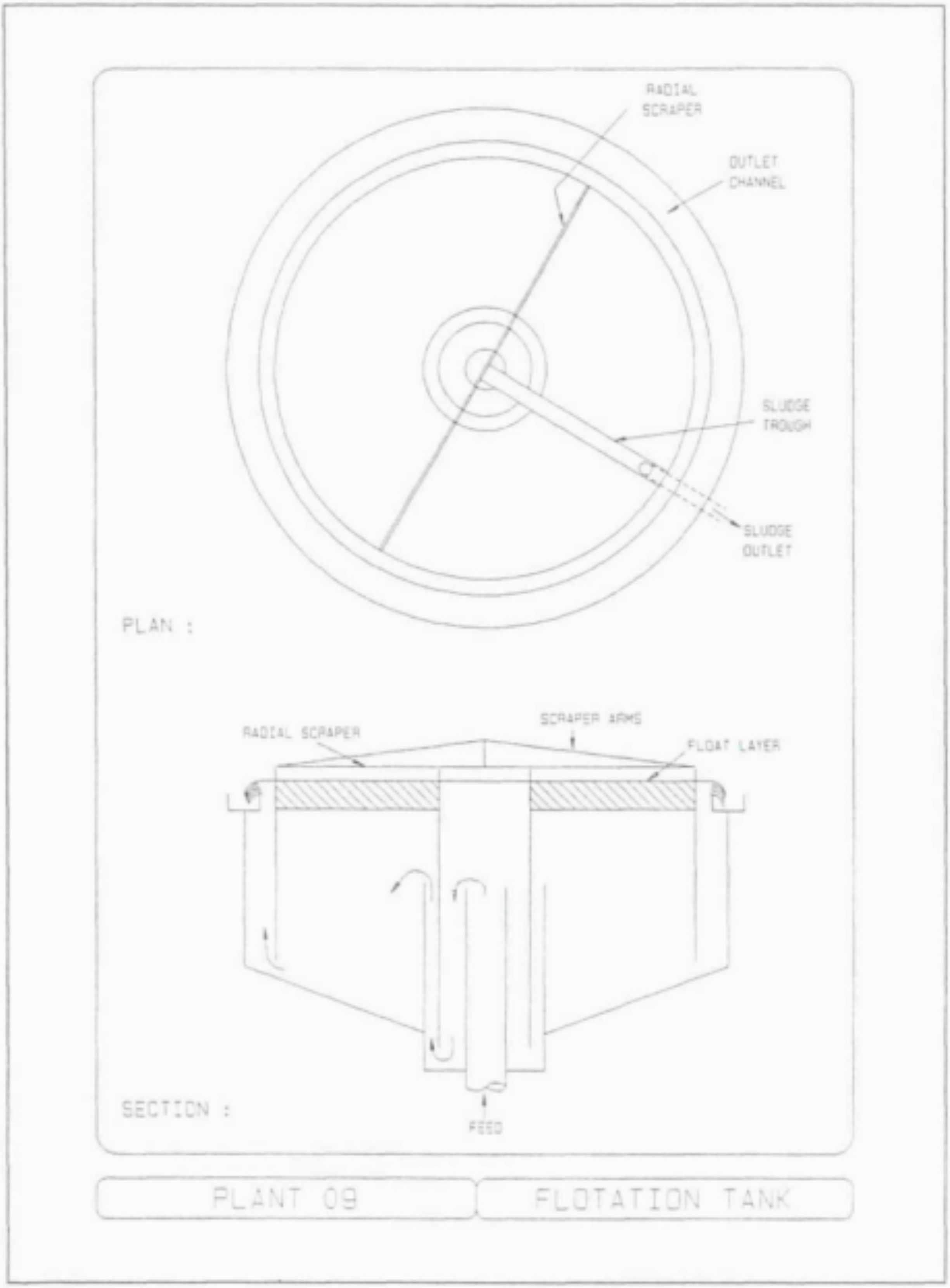
Position (H/V)			V
Diameter	D_s	m	2.00
Length	L_s	m	3.9
Packing (Y/N)			N
Recirculation (Y/N)			N
Depth packing	D_c		n/a
Number of saturators			1

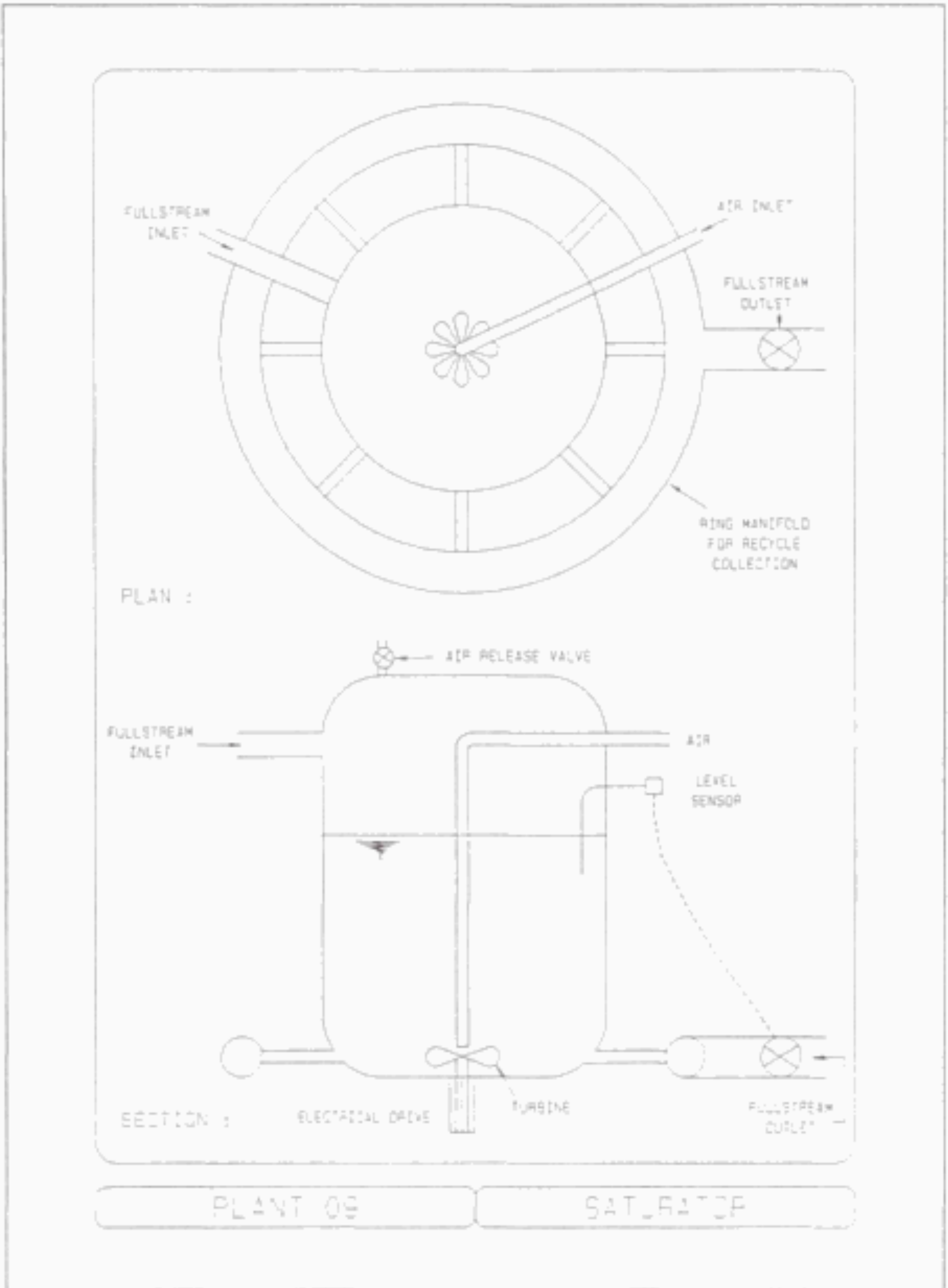
*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	no	120
Speed scrapers	$r.h^{-1}$	2.6
Continuous scraping (Y/N)		Y
Bottom desludging (Y/N)		N

*** Notes ***

Reactor and crossflow areas estimated.


RADIAL
SCRAPER
OUTLET
CHANNEL
SLUDGE
TROUGH
SLUDGE
OUTLET
PLAN :
RADIAL SCRAPER
SCRAPER ARMS
FLOAT LAYER
SECTION :
FEED
PLANT 09
FLOTATION TANK


FULLSTREAM INLET
AIR INLET
FULLSTREAM OUTLET
RING MANIFOLD FOR RECYCLE COLLECTION
PLAN :
AIR RELEASE VALVE
FULLSTREAM INLET
AIR
LEVEL SENSOR
SECTION :
ELECTRICAL DRIVE
TURBINE
FULLSTREAM OUTLET
PLANT 09
SATURATOR

DESCRIPTION OF FLOTATION PLANT #10

Objective

The objective of the plant is to treat maturation pond effluent for underground use as mine service water. The water is filtered by upflow filtration after flotation.

Feed water

Domestic sewage is being treated by a waste-water treatment plant consisting of conventional biological filtration. After treatment, the effluent is discharged in a series of 8 maturation ponds. From here, a fraction is pumped to a flow balancing tank. The water is finally pumped again from the balancing tank to maintain a constant flow rate to the flotation plant. The COD varies between 30 $mg.\ell^{-1}$ to 50 $mg.\ell^{-1}$ and the pH between pH 7,0 to pH 9,0.

Chemical conditioning

The water is flocculated in a three-compartment flocculation zone. Ferric chloride is added in the pipe just before the first compartment, and polymer at the weir between the second and third compartments. The mechanically stirred compartments provide about 7 minutes of flocculation each, at a velocity gradient of between $130.s^{-1}$ to $180.s^{-1}$. After flotation, the water is stabilized by adding soda ash solution. Originally, a lime slurry was used, but this was abandoned in favour of soda ash.

Flotation tank

There is a single circular flotation tank with central reaction zone. The feed enters from below and the recycle is introduced at the same point. Reaction takes place in a funnel-shaped reaction compartment before the water flows outward to the flotation zone. The subnatant is collected at a fixed peripheral overflow weir. There is a travelling bridge which supports 2 surface scrapers as well as 2 bottom scrapers. The rotation speed can vary up to 1 r/min.

Recirculation system

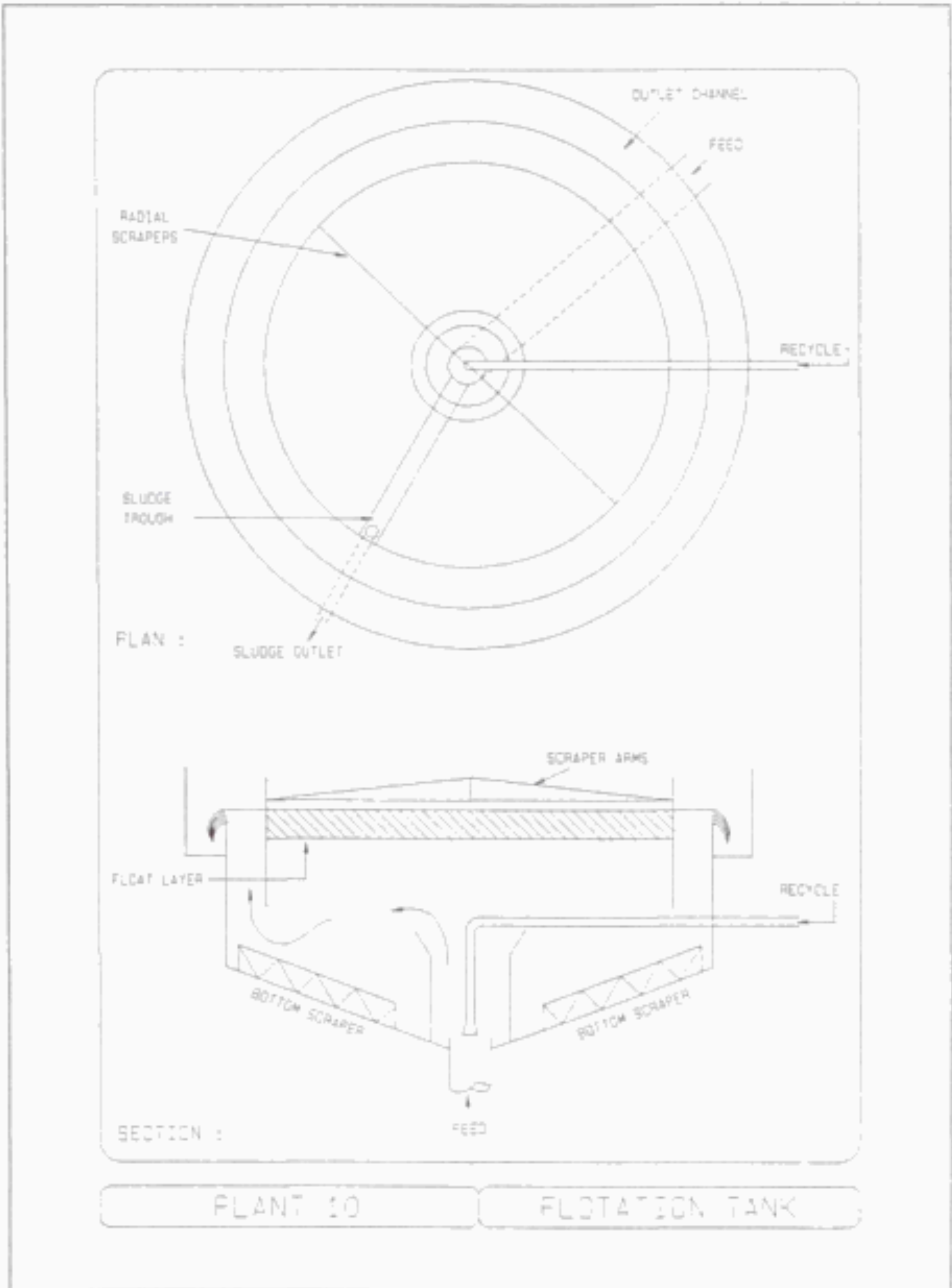
Pressurized water is withdrawn from the high pressure water reticulation system. There are 2 saturators - one working and the other on standby. The saturators are vertical and packed with plastic packing. The water level is maintained by controlling the withdrawal rate from the mine reticulation system.

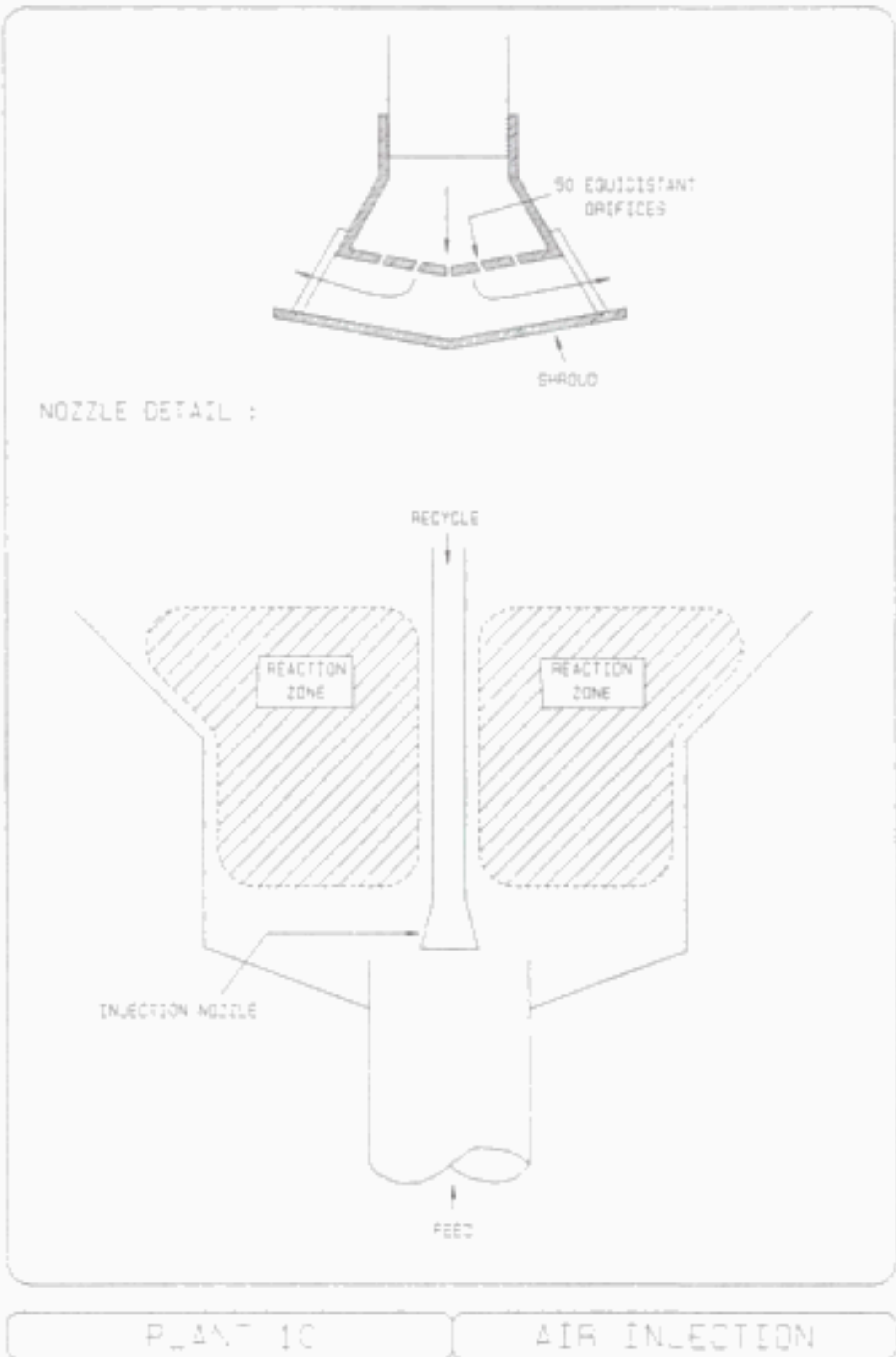
Performance

The plant operates satisfactorily. The air release nozzle has to be cleaned once per week. The tank is completely drained and cleaned once every six months. The COD is reduced to about 15 $mg.\ell^{-1}$ and the suspended solids to between 10 $mg.\ell^{-1}$ and 20 $mg.\ell^{-1}$.

PLANT NUMBER			# 10	
*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D	m	7.0	
Inner diameter	D	m	1.9	
Depth	D	m	2.5	
Number flotation cells			1	
Flotation area	A	m^2	35.6	
Reactor area	A	m^2	2.8	
Reactor volume	V	m^3	7.0	
Crossflow area	A	m^2	7.8	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	250	208
Recycle rate/cell	q	$k\ell.h^{-1}$	25	25
Saturator pressure	P	kPa	600	450
Concentration in	C			
Concentration out	C			
*** Saturation ***				
Position (H/V)			V	
Diameter	D	m	0.72	
Length	L	m	2.5	
Packing (Y/N)			Y	
Recirculation (Y/N)			N	
Depth packing	D	m	1.2	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		no	180	
Speed scrapers		$r.h^{-1}$	3.2	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***


OUTLET CHANNEL
FEED
RADIAL SCRAPERS
RECYCLE
SLUDGE TROUGH
PLAN :
SLUDGE OUTLET
SCRAPER ARMS
FLOAT LAYER
RECYCLE
BOTTOM SCRAPER
BOTTOM SCRAPER
FEED
SECTION :
PLANT 10
FLOTATION TANK


50 EQUIDISTANT
ORIFICES
SHROUD
NOZZLE DETAIL :
RECYCLE
REACTION
ZONE
REACTION
ZONE
INJECTION NOZZLE
FEED
PLANT 10
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #11

Objective

The plant treats water from a shallow eutrophic impoundment. The quality of purified water is better than the SABS 241, 1984 Specification for Domestic Supplies regarding colour and total suspended solids.

Feed water

At certain times of the year, the water is low in turbidity but has a high algal concentration. At other times, the turbidity is excessively high due to floods or wind action on the impoundment. Moreover, the water quality changes are very rapid and chemical dosing rates are hourly verified and adjusted if necessary. The raw water turbidity ranges from 20 to 1800 NTU, the conductivity from 16 to 109 $mS.m^{-1}$, the pH from pH 7.4 to pH 8.6 and the chlorophyll *a* from 5 to 33 $\mu g.\ell^{-1}$.

Chemical conditioning

The water is first dosed with polymer (if the turbidity is high) and settled in a presedimentation tank. This strategy is highly succesful for reducing the otherwise very sharp turbidity peaks, and the maximum turbidity after presedimentation can be kept to below 75 NTU. Further chemical dosing with aluminium sulphate and polymer follows with flocculation in 8 parallel, mechanically stirred flocculation basins. The flocculation time is about 15 minutes. The flocculated water is transferred with short lengths of pipe to the flotation tanks. Chlorination follows after rapid sand filtration.

Flotation tanks

There are 8 rectangular, flat-bottomed flotation tanks. The feed enters at one end through 6 vertical pipes per tank, flared towards the top. The recycle is injected into every short pipe length connecting the flocculation and flotation tanks. The flow into each injection point is individually adjustable. The subnatant is collected at the far end of the tank over a fixed weir. The float layer is scraped with a rubber-bladed scraper towards the float collection channel.

Recirculation system

Four horizontal, unpacked saturators are used for saturation of the recycle. Subnatant from the flotation tanks is pumped back through the saturators. The water and air flow into the saturators are controlled by flow control valves which are controlled by the water level in the saturator. The air supply is drawn from a pressure vessel fed by an air compressor.

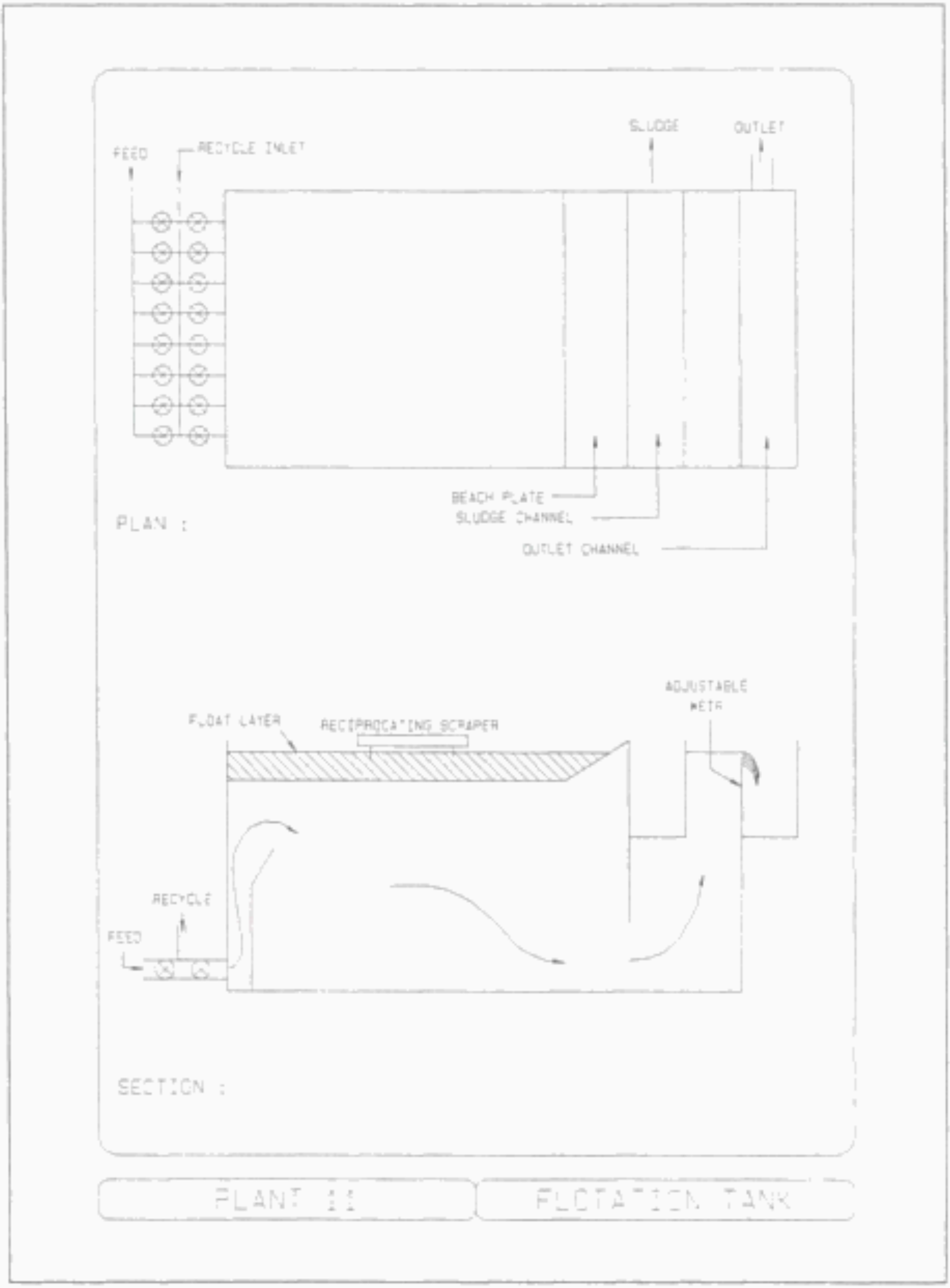
Performance

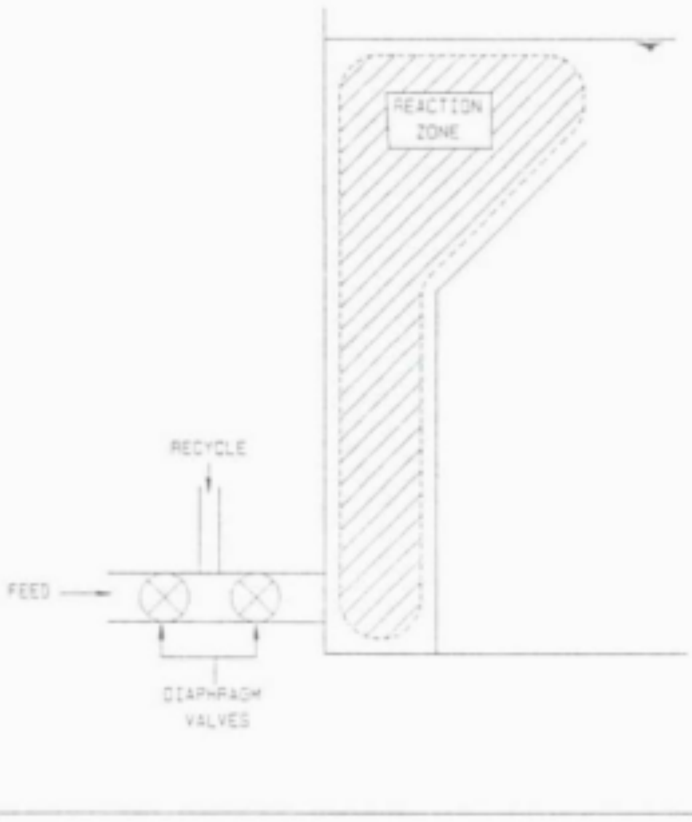
The plant was originally designed on the basis of a maximum raw water turbidity of 75 NTU. When turbidities of more than 75 NTU were experienced, preclarifiers were added to reduce turbidity to the design level. Since the commissioning of these clarifiers, no further problems were experienced in the operation of the flotation system.

PLANT NUMBER			# 11	
*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	14.3	
Width	W	m	7.7	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	3.4	
Number flotation cells			8	
Flotation area	A_f	m^2	110.1	
Reactor area	A_r	m^2	6.8	
Reactor volume	V_r	m^3	23.1	
Crossflow area	A_c	m^2	6.6	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	781	781
Recycle rate/cell	q	$k\ell.h^{-1}$	60	60
Saturator pressure	P	kPa	550	610
Concentration in	C_i	NTU		25
Concentration out	C_o	NTU		5
*** Saturation ***				
Position (H/V)			H	
Diameter	D_s	m	2.00	
Length	L_s	m	3.1	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D_p		n/a	
Number of saturators			4	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		m	7.0	
Speed scrapers		$m.h^{-1}$	54-270	
Continuous scraping (Y/N)			N	
Bottom desludging (Y/N)			N	

*** Notes ***

Scrapers one hour off, five minutes on.


SLUDGE
OUTLET
FEED
RECYCLE INLET
BEACH PLATE
SLUDGE CHANNEL
OUTLET CHANNEL
PLAN :
ADJUSTABLE WEIR
FLOAT LAYER
RECIPROCATING SCRAPER
RECYCLE
FEED
SECTION :
PLANT 11
FLOTATION TANK



PLANT 11 | AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #12

Objective

The plant treats water from an eutrophied surface source for drinking-water purposes. Flotation follows chemical treatment and flocculation, and is followed by rapid sand filtration, chlorination and stabilization by lime.

Feed water

Water is abstracted from a natural stream which predominantly carries purified municipal and industrial effluent. The water, during certain times, has a high algal growth potential. The chlorophyll *a* level averages 30 $\mu g.\ell^{-1}$ with a maximum of about 100 $\mu g.\ell^{-1}$. The turbidity varies between 10 NTU and 40 NTU.

Chemical conditioning

Provision is made for dosing of lime and ferric chloride as the water enters the plant. The lime is added at the inlet box and the ferric chloride shortly thereafter in an in-line mixer. Flocculation is performed in a back-mix reactor immediately below the flotation tank. No external mixing is provided in the flocculation compartment, mixing is induced by the turbulence of the incoming water only.

Flotation tank

There is one square flotation tank, located on top of the flocculation compartment. The flocculated water passes into the flotation tank through 4 funnel-shaped inlets. The recycle water is released immediately outside these inlets at the base of the flotation tank. There is no separate reaction zone, and reaction between feed and recycle takes place within the larger flotation zone. Subnatant is withdrawn through 12 pipes near the bottom of the tank, discharging through a telescopic weir which can be adjusted to control the water level in the flotation tank. The float layer is periodically flushed into 4 launders with the telescopic weir. There is no bottom scraper; the tank is manually cleaned at infrequent intervals.

Recirculation system

Water is withdrawn form the filtered water supply and pumped through a single vertical saturator packed with 50 mm plastic Pall rings. Level control is through the air valve.

Performance

As the plant was originally designed, there was excessive upward turbulence at the point where the water entered the flotation tank from below. This caused float disruption with a concomitant build-up of sludge on the floor of the flotation tank. This was then slightly modified without success. It was finally modified once more with plates at the inlets which deflect the water sideways upon entry, and towards the area where the recycle is released. This change caused a considerable improvement in the performance of the plant. The details of the plant given in this report is based on the latter arrangement. Turbidity after flotation ranges between 4 NTU and 13 NTU.

PLANT NUMBER # 12

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	7.9	
Width	W	m	7.9	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	2.0	
Number flotation cells			1	
Flotation area	A_f	m^2	62.8	
Reactor area	A_r		ID	
Crossflow area	A_c		ID	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	313	
Recycle rate/cell	q	$k\ell.h^{-1}$	72	36
Saturator pressure	P	kPa		440
Concentration in	C_i			
Concentration out	C_o			

*** Saturation ***

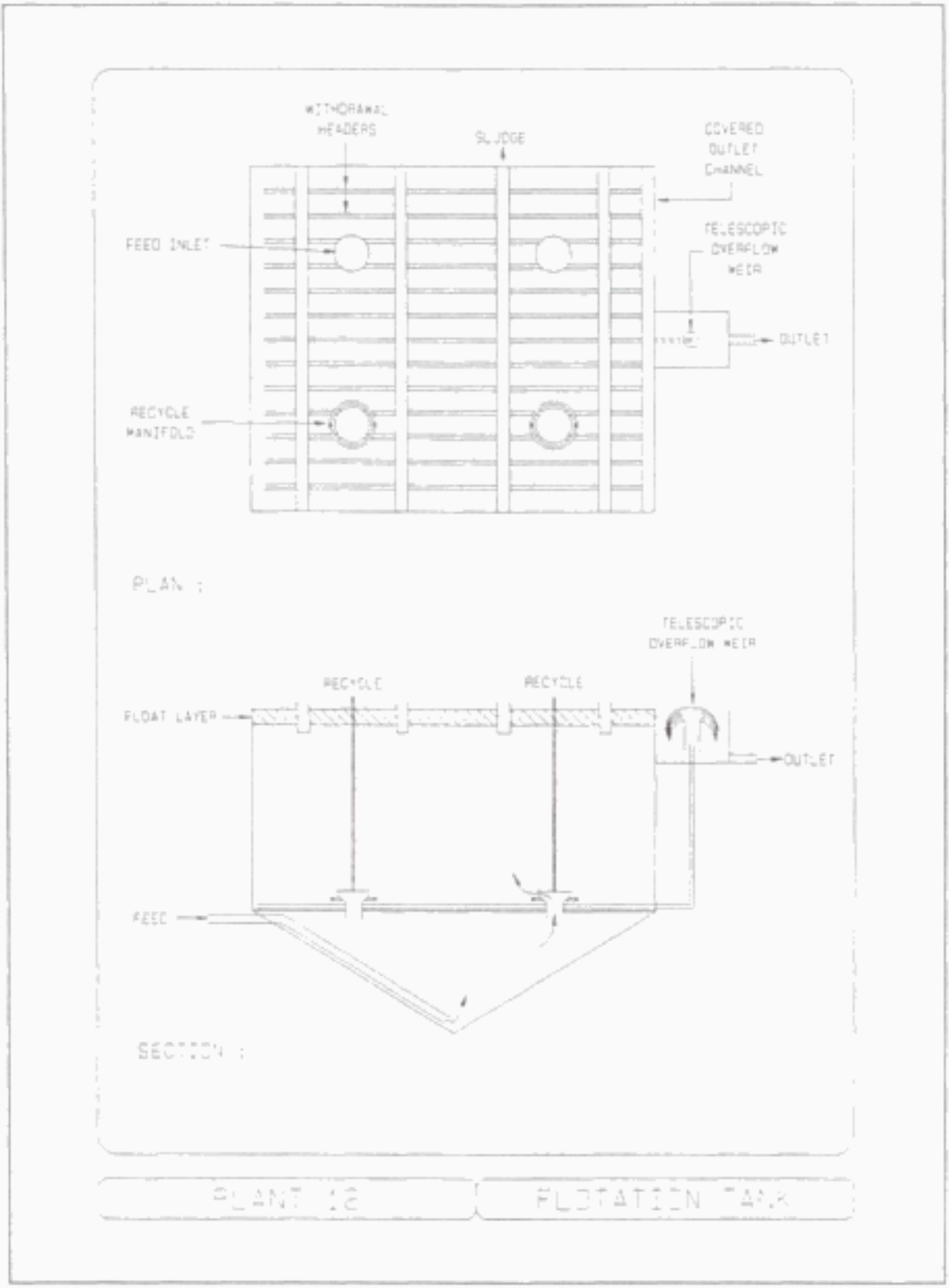
Position (H/V)			V
Diameter	D_s	m	1.00
Length	L_s	m	2.0
Packing (Y/N)			Y
Recirculation (Y/N)			N
Depth packing	D_p	m	1.2
Number of saturators			1

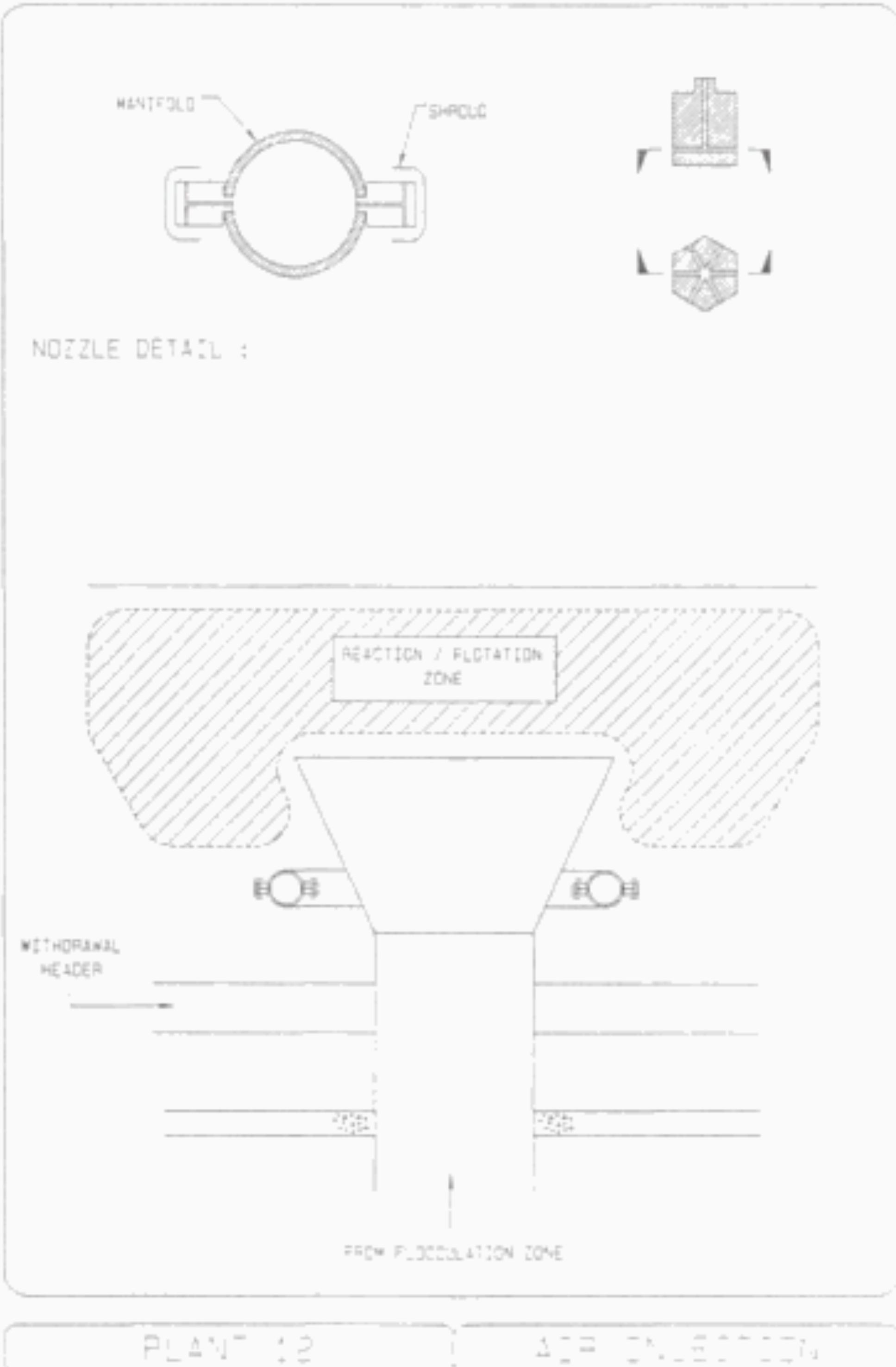
*** Float removal ***

Grid (Y/N)	N
Scraper (Y/N)	N
Spacing scrapers	n/a
Speed scrapers	n/a
Continuous scraping (Y/N)	N
Bottom desludging (Y/N)	N

*** Notes ***

Tanks emptied and cleaned every three months


WITHDRAWAL HEADERS
SLUDGE
COVERED OUTLET CHANNEL
FEED INLET
TELESCOPIC OVERFLOW WEIR
OUTLET
RECYCLE MANIFOLD
PLAN :
TELESCOPIC OVERFLOW WEIR
RECYCLE
RECYCLE
FLOAT LAYER
OUTLET
FEED
SECTION :
PLANT 12
FLOTATION TANK


MANIFOLD
SHROUD
NOZZLE DETAIL :
REACTION / FLOTATION ZONE
WITHDRAWAL HEADER
FROM FLOCCULATION ZONE
PLANT 12
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #13

Objective

The objective of the treatment plant is to produce drinking-water for a small community. Flotation follows a period of raw water storage, chemical dosage and flocculation. After flotation, the water is filtered through pressure filters and chlorinated.

Feed water

The feed water is drawn from a small impoundment in a stream. The water is pumped to a raw water storage dam, which allows some initial presedimentation. After presedimentation, the water passes a 4 mm screen as it enters the treatment plant. The turbidity after presedimentation ranges from 30 NTU to 60 NTU and the pH from pH 7,5 to pH 8,5. The impoundment is plagued by sporadic algal blooms.

Chemical conditioning

Flocculant and polymer (depending on the season) are added in the preconditioning stage and is dispersed with a series of static mixers. If ferric chloride is added before the first mixer, lime is added before the second mixer. Polymer, when used, is added before the third mixer. The water then passes through a down-flow column flocculator which provides about 7 minutes of flocculation time.

Flotation tank

There are 5 circular flotation tanks, each 2,1 m in diameter. The feed enters at the top of the tank, and the recycle near the bottom of the tank, which induces a counter-current flow between feed and recycle. There are, therefore, no separate reaction and flotation zones. The top of the tank is in the shape of an inverted cone which causes the float layer to concentrate in an area considerably smaller than the bottom diameter of the tank. The float layer is periodically washed out by closing the tank outlet.

Recirculation system

There is a single vertical saturator, packed with 25 mm plastic Pall rings. The water level is sensed by 2 probes set at the minimum and maximum water levels. The probes are used to control the air supply to the saturator. The water is distributed at the top of the saturator by means of a splash plate.

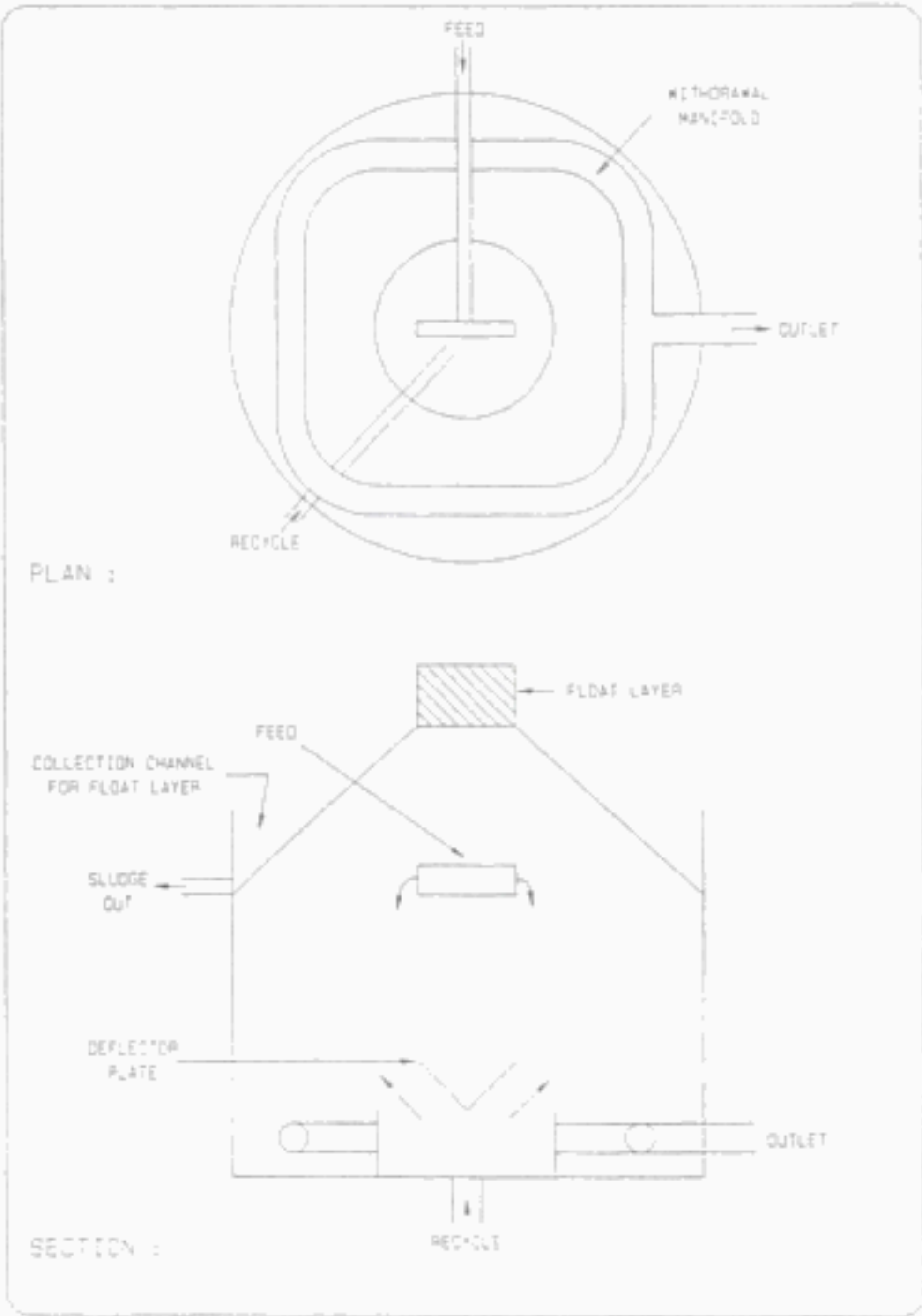
Performance

The plant operates satisfactorily. Maintenance problems are occasionally experienced with the lime and chemical dosing pumps. The flotation tanks are drained and desludged at weekly intervals.

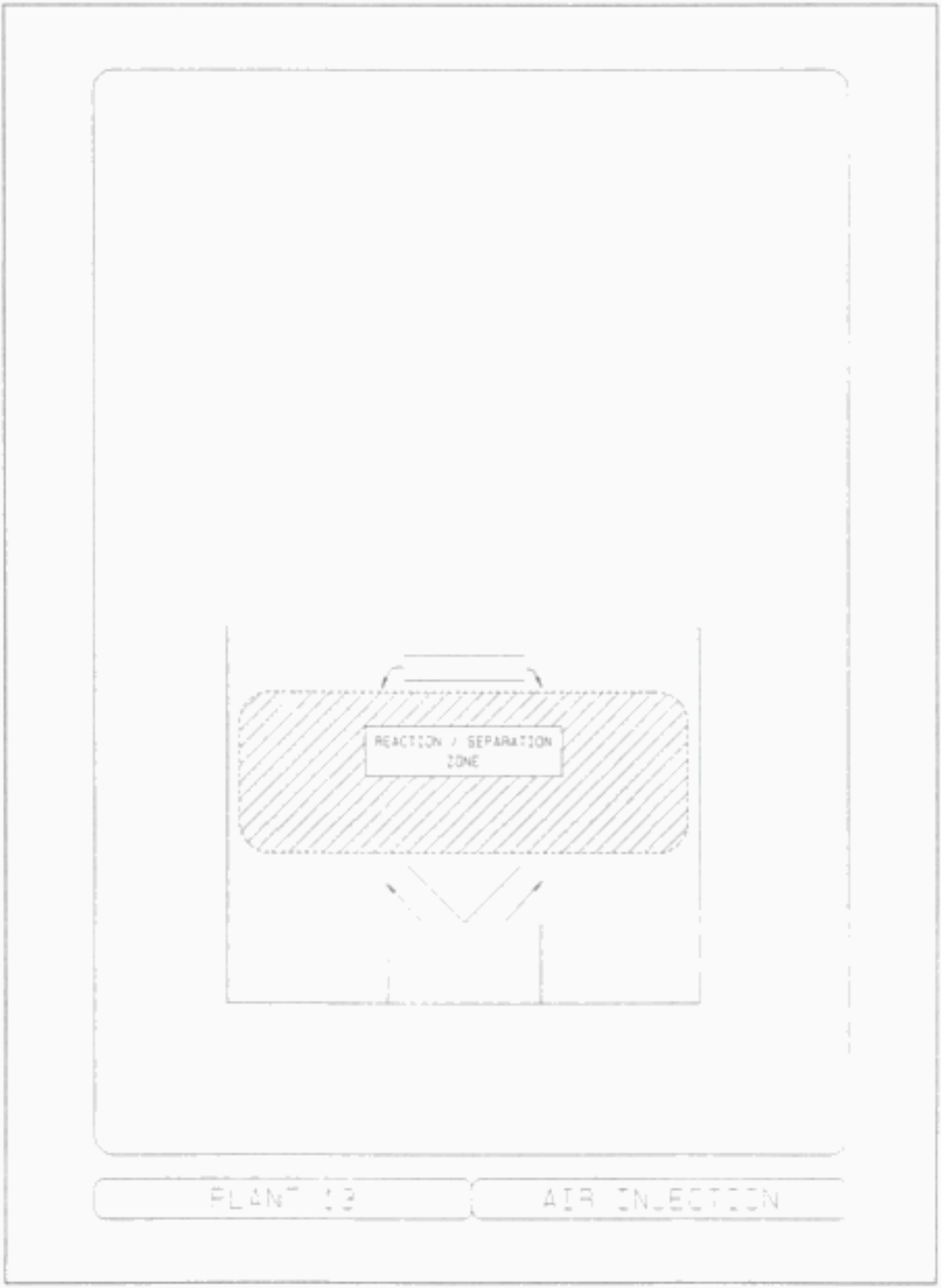
PLANT NUMBER			# 13	
*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D	m	2.1	
Inner diameter	D		n/a	
Depth	D	m	1.5	
Number flotation cells			5	
Flotation area	A	m^2	3.5	
Reactor area	A		ID	
Reactor volume	V		ID	
Crossflow area	A		ID	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell\,h^{-1}$	17	17
Recycle rate/cell	q	$k\ell\,h^{-1}$	2	2
Saturator pressure	P	kPa	450	480
Concentration in	C			
Concentration out	C			
*** Saturation ***				
Position (H/V)			V	
Diameter	D	m	0.40	
Length	L	m	2.0	
Packing (Y/N)			Y	
Recirculation (Y/N)			N	
Depth packing	D	m	1.0	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			N	
Spacing scrapers			n/a	
Speed scrapers			n/a	
Continuous scraping (Y/N)			n/a	
Bottom desludging (Y/N)			N	

*** Notes ***

Float removal by periodic flushing



PLANT 13 | FLOTATION TANK


REACTION / SEPARATION
ZONE
PLANT 13
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #14

Objective

Flotation is used to treat the effluent from a waste-water treatment plant prior to disposal into a natural stream. The effluent is chemically dosed for phosphate precipitation and flocculation when the phosphate concentration is above 1 mg P ℓ^{-1}, and the solids are then removed by flotation. The water is finally chlorinated before discharge.

Feed water

The effluent comes from a large waste-water treatment plant which treats municipal sewage. The treatment plant is designed for biological nutrient removal.

Chemical conditioning

The effluent is chemically treated with ferric chloride and polymer when required. The coagulants are added at consecutive overflow weirs which provide the energy for mixing. Flocculation is done for about 6 minutes in a backmix reactor which is mechanically stirred by an electrically driven mixer.

Flotation tank

There are 3 circular flotation tanks with central reaction zones. The feed and recycle are separately introduced into the reaction zone. The recycle is introduced through a valve (adjustable from the top of the tank) which also acts as an injection nozzle. The water level in the tank is maintained by a fixed, peripheral overflow weir. There are 4 radial surface scrapers per tank, which are operated at about 7 revolutions per hour. There are bottom scrapers which scrape the bottom sludge towards a collection hopper. The hopper is drained once per day.

Recirculation system

There are 3 saturators, 1 for each tank. The saturators are vertical and unpacked, and draws water from the flotation subnatant. The water level in the saturator is maintained by a magnetic switch which controls the air supply to the saturator.

Performance

The flotation did not work well at all when the plant was commissioned. At that time, the feed and recycle were mixed some distance away from the reaction zone, and the recycle pressure release was crudely done across a valve. The system was then modified to that described above, and from then the plant operates very well, with the suspended solids in the subnatant less than 4 mg.ℓ^{-1}. This indicates the extreme importance of a well designed air introduction system.

PLANT NUMBER # 14

*** Tank geometry ***

Shape (Round/Rect)			Round
Length	L		n/a
Width	W		n/a
Outer diameter	D_o	m	13.2
Inner diameter	D_i	m	1.5
Depth	D_f	m	3.5
Number flotation cells			3
Flotation area	A_f	m^2	135.0
Reactor area	A_r	m^2	1.8
Reactor volume	V_r	m^3	6.3
Crossflow area	A_c	m^2	1.9

*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	500	361
Recycle rate/cell	q	$k\ell.h^{-1}$	36	36
Saturator pressure	P	kPa	400	390
Concentration in	C_i	SS	100	15
Concentration out	C_o	SS	4	4

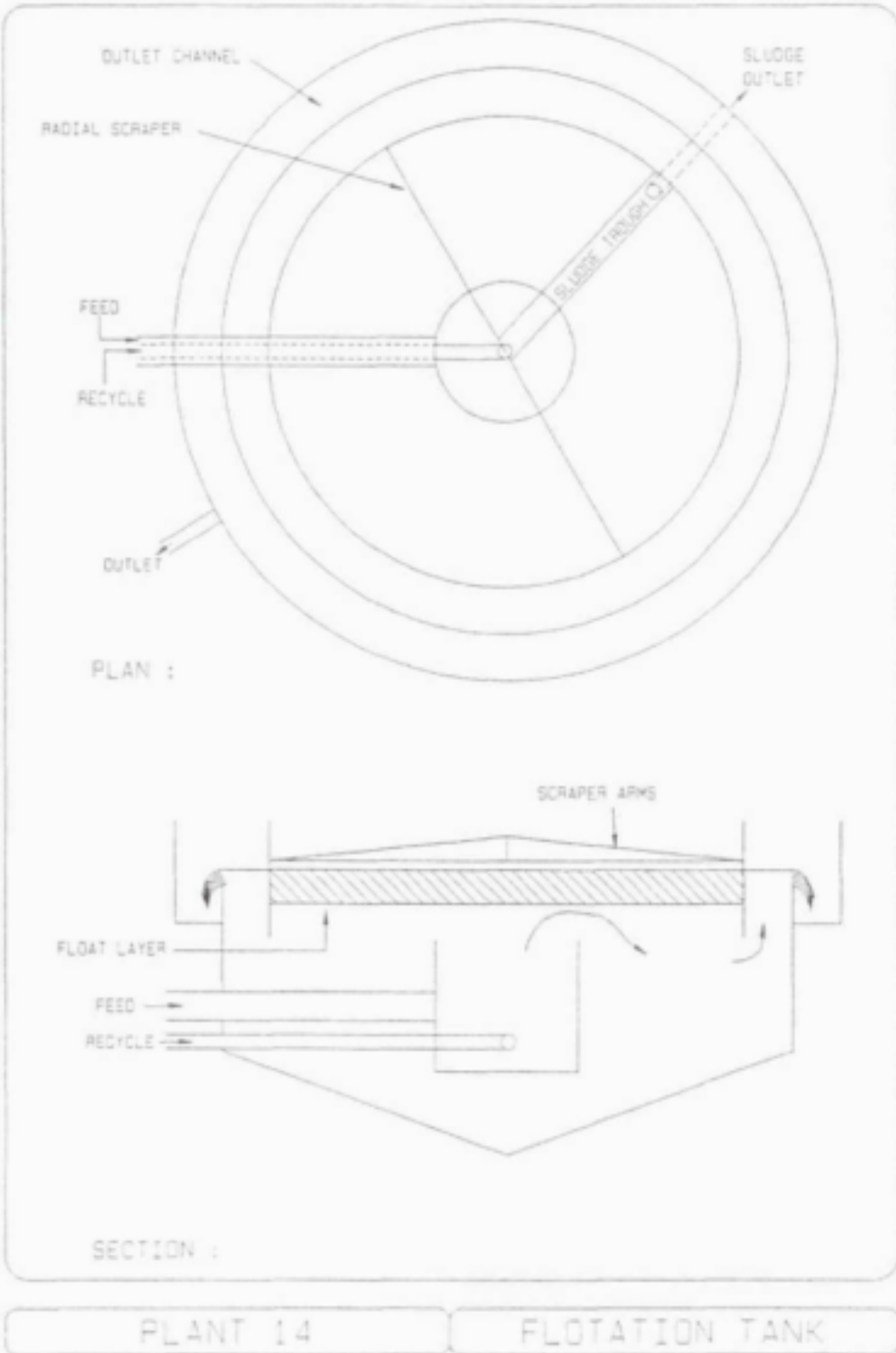
*** Saturation ***

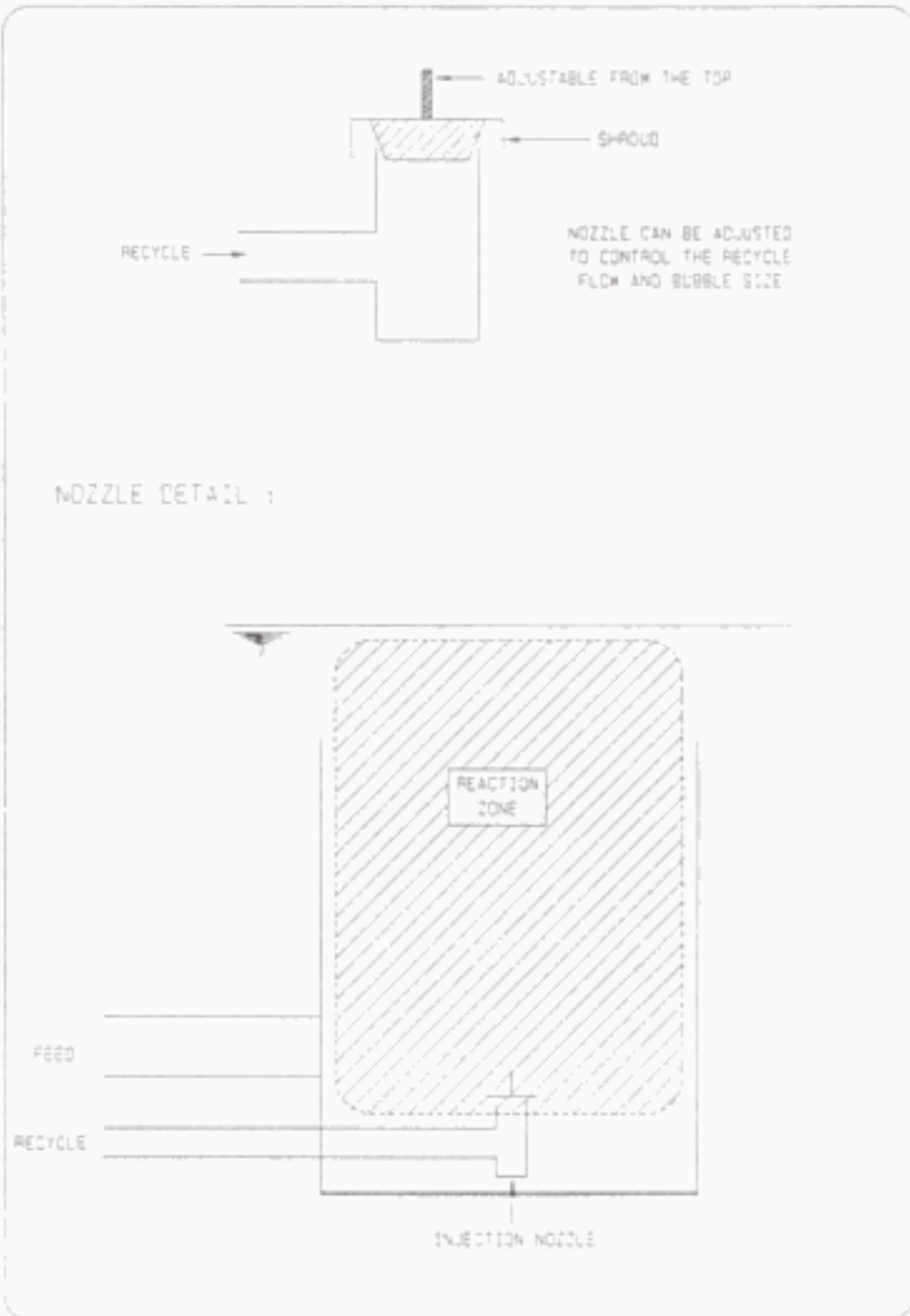
Position (H/V)			V
Diameter	D_s	m	1.35
Length	L_s	m	2.0
Packing (Y/N)			N
Recirculation (Y/N)			N
Depth packing	D_p		n/a
Number of saturators			3

*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	no	4
Speed scrapers	$r.h^{-1}$	7.0
Continuous scraping (Y/N)		Y
Bottom desludging (Y/N)		Y

*** Notes ***


OUTLET CHANNEL
SLUDGE OUTLET
RADIAL SCRAPER
SLUDGE TROUGH
FEED
RECYCLE
OUTLET
PLAN :
SCRAPER ARMS
FLOAT LAYER
FEED
RECYCLE
SECTION :
PLANT 14
FLOTATION TANK



PLANT 14 AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #21

Objective

The plant is designed to thicken waste activated sludge prior to centrifuging

Feed

The sludge concentration is approximately 5 500 $mg.\ell^{-1}$ and is derived from a conventional activated sludge plant treating sewage with approximately 40% industrial effluent. Of this, about 50% is brewery waste. Because of excellent bioflocculation, no chemicals are added.

Flotation tanks

Flotation is done in 2 parallel flat-bottomed units of approximately similar design. A hopper section is provided near the influent end for the capture of coarse solids and bottom sludge. The water level is controlled with an adjustable weir. The float layer is continuously scraped at a rate of about 40 $m.h^{-1}$ using 15 blades spaced 2 m apart. Daily volume of sludge removed varies between 110 to 180 m^3.

Recirculation system

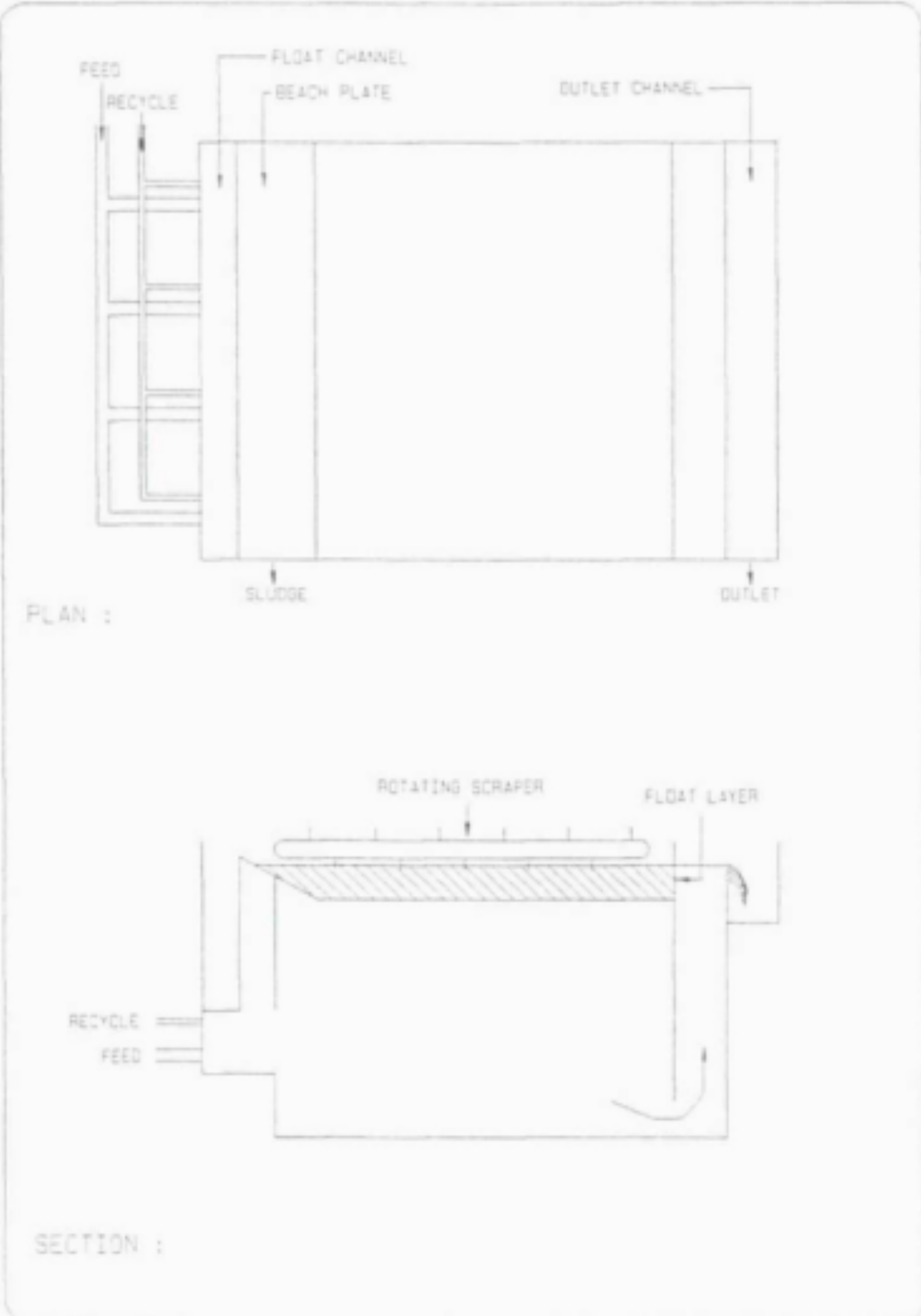
Tank effluent is pressurized and recycled to 2 horizontal saturators with provision for internal recycling. No packing is provided on account of high risk for blockages. The air is released in the flotation tank by means of a number of diaphragm valves immediately outside the tank. Level control probes are provided to maintain the water level at 60% to 80% of vertical depth.

Performance

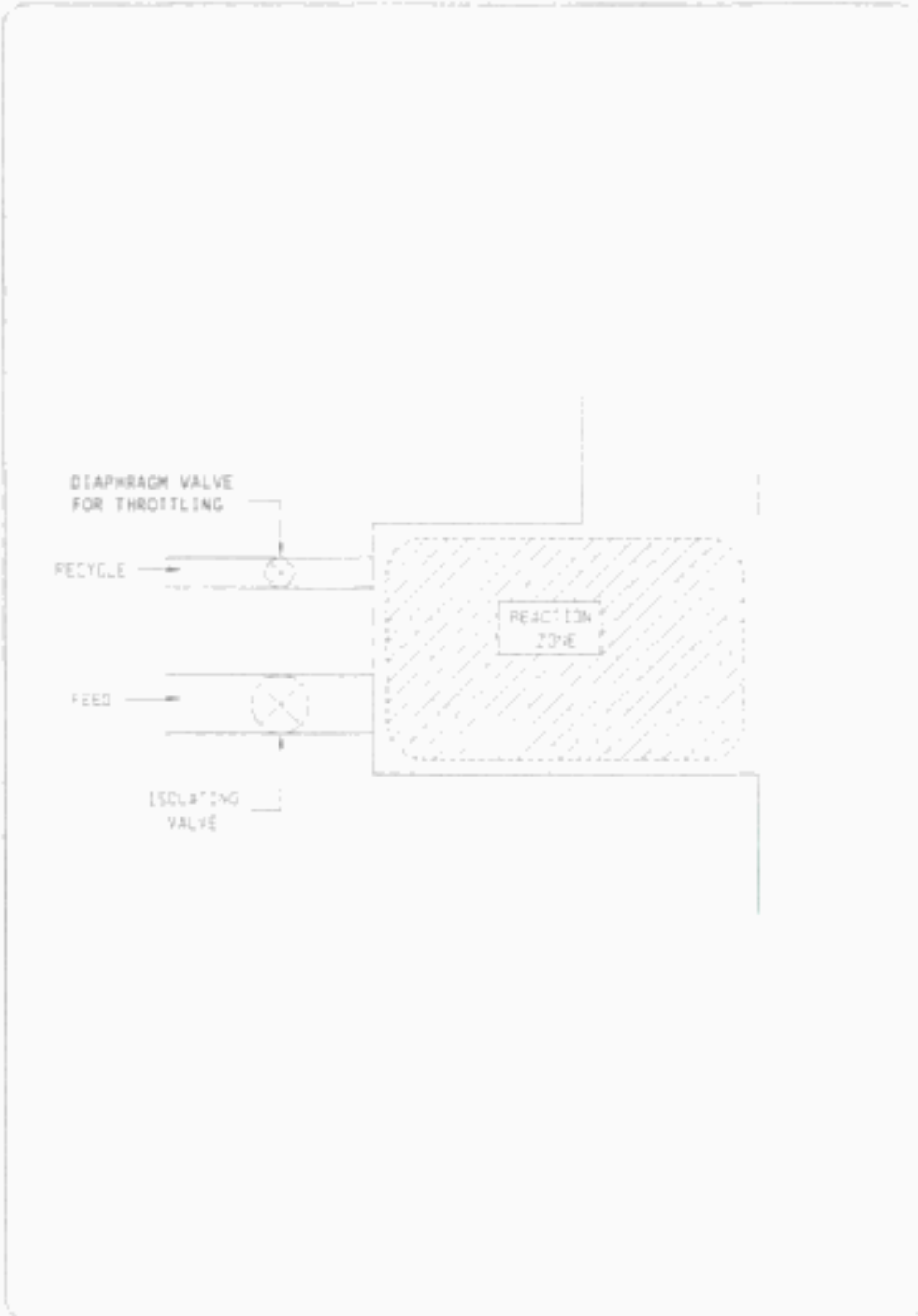
The flotation process performs satisfactory achieving about 3,5 % concentration of float solids. The sludge is further thickened by centrifuge to about 12 % to 22 %.

PLANT NUMBER			# 21	
*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	15.0	
Width	W	m	4.0	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_t	m	3.5	
Number flotation cells			2	
Flotation area	A_f	m^2	60.0	
Reactor area	A_r	m^2	3.0	
Reactor volume	V_r	m^3	6.0	
Crossflow area	A_c	m^2	3.2	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	240.0	110.0
Recycle rate/cell	q	$k\ell.h^{-1}$	180	180
Saturator pressure	P	kPa	500	500
Concentration in	C	SS		5500
Concentration out	C_e	SS	200	100
*** Saturation ***				
Position (H/V)			H	
Diameter	D_s	m	1.50	
Length	L_s	m	4.0	
Packing (Y/N)			N	
Recirculation (Y/N)			Y	
Depth packing	D_p		n/a	
Number of saturators			2	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		m	2.0	
Speed scrapers		$m.h^{-1}$	42.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***



PLANT 21 | FLOTATION TANK



PLANT 21 | AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #22

Objective

The plant is designed to thicken waste clarifier sludge derived from a large water treatment plant. The float is further concentrated by centrifuge for final disposal to a dumping site by a waste disposal contractor

Feed

The feed comes from the underflow of the clarifiers and consists mainly of polyaluminium chloride, lime grit and turbidity components in the original raw water. The solids concentration varies between 5 $g.\ell^{-1}$ and 7,5 $g.\ell^{-1}$.

Chemical conditioning

A polymer is introduced as a flotation aid on the suction side of the recycle pump.

Flotation tank

Flotation is done in a circular tank with a central reactor zone. The subnatant is withdrawn from a fixed peripheral weir, which also controls the water level in the tank. Recycle and feed are introduced at the bottom of the reactor zone. Precipitation of dissolved air is achieved using cupped aeration nozzles. A surface scraper for float removal is normally operated continuously at about 2 revolutions per hour. The same assembly also concentrates settled sludge with a bottom scraper. The thickened bottom sludge is discharged intermittently through a timer controlled discharge valve

Recirculation system

The recycle is drawn from a recycle holding tank and repumped through the saturation system. It is pumped through a horizontal unpacked saturator. The level in the saturator is measured with capacitance probes and the level is maintained by on/off control on the delivery valve of the recycle pump.

Performance

The solids concentration of the float is between 5 % and 8 %, while that of the centrifuge cake is about 35 %. Saturation efficiency of 50 % to 60 % is reported

PLANT NUMBER # 22

*** Tank geometry ***

Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	10.0	
Inner diameter	D_i	m	1.0	
Depth	D_r	m	3.5	
Number flotation cells			1	
Flotation area	A_f	m^2	77.8	
Reactor area	A_r	m^2	0.8	
Reactor volume	V_r	m^3	9.5	
Crossflow area	A_c	m^2	1.9	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	160	35
Recycle rate/cell	q	$k\ell.h^{-1}$	100	24
Saturator pressure	P	kPa	500	500
Concentration in	C	SS		6200
Concentration out	C_e	SS		40

*** Saturation ***

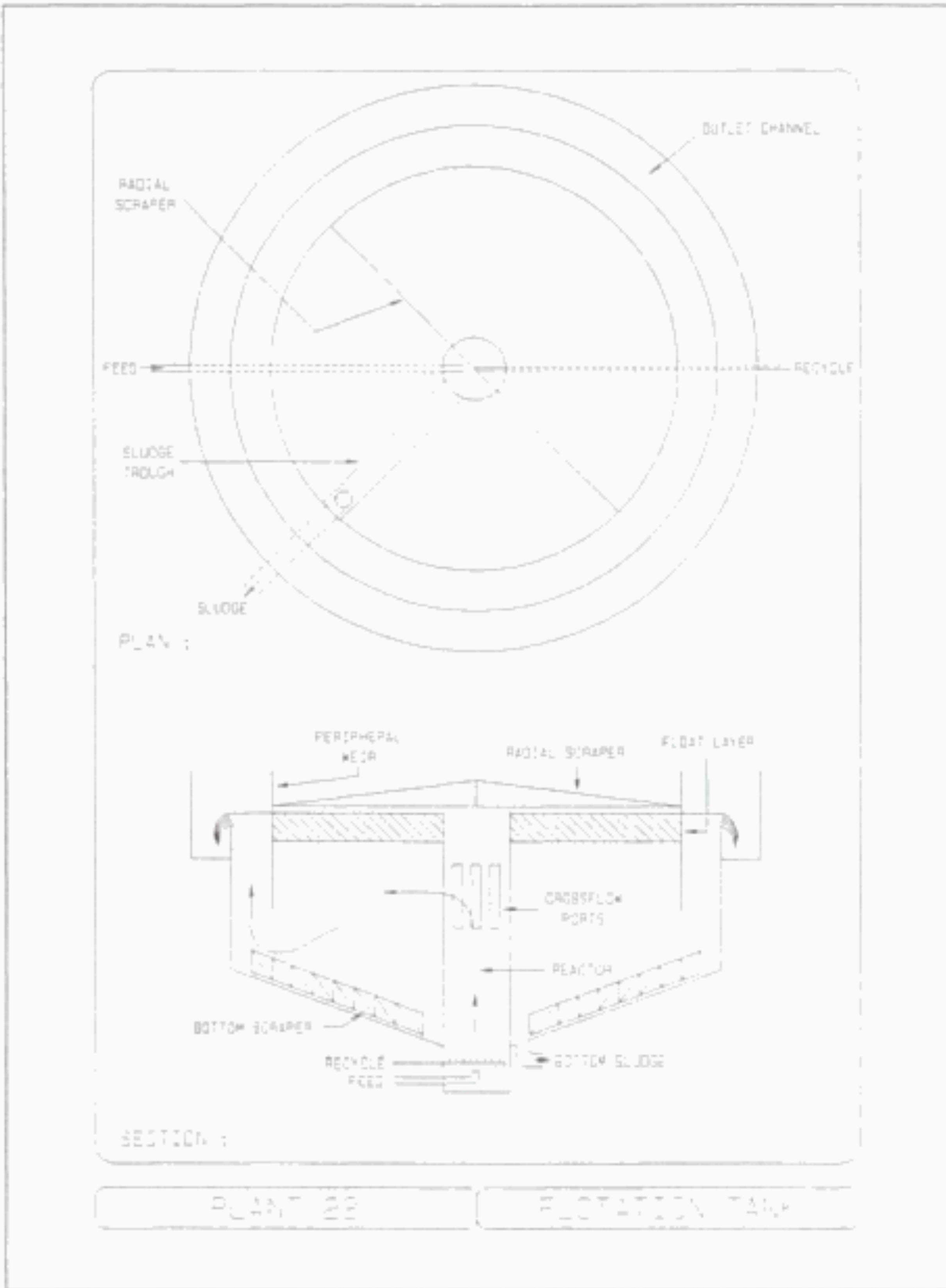
Position (H/V)			H
Diameter	D_s	m	1.20
Length	L_s	m	3.0
Packing (Y/N)			N
Recirculation (Y/N)			N
Depth packing	D_p		n/a
Number of saturators			1

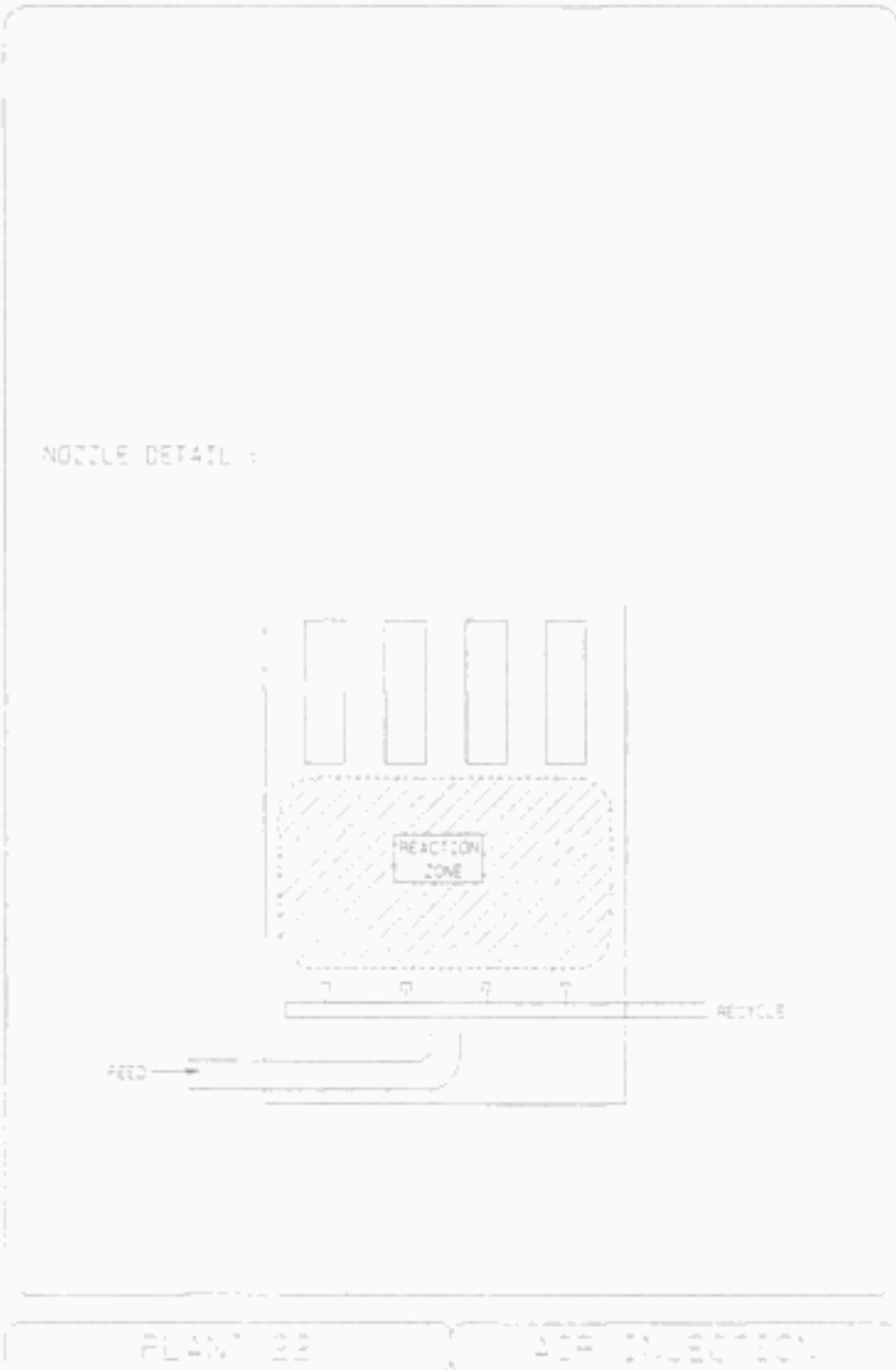
*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	no	36
Speed scrapers	$r.h^{-1}$	2.0
Continuous scraping (Y/N)		note
Bottom desludging (Y/N)		Y

*** Notes ***

Scrapers either continuous: or 3 minutes on, 15 minutes off.


OUTLET CHANNEL
RADIAL SCRAPER
FEED
RECYCLE
SLUDGE TROUGH
SLUDGE
PLAN :
PERIPHERAL WEIR
RADIAL SCRAPER
FLOAT LAYER
CROSSFLOW PORTS
REACTOR
BOTTOM SCRAPER
RECYCLE
FEED
BOTTOM SLUDGE
SECTION :
PLANT 28
FLOTATION TANK


NOZZLE DETAIL :
REACTION ZONE
RECYCLE
FEED
PLANT 22

DESCRIPTION OF FLOTATION PLANT #23

Objective

The plant is designed for the thickening of waste activated sludge prior to anaerobic digestion.

Feed

The sludge is derived from a large sewage treatment plant based on biological nutrient removal. The suspended solids vary from 3 000 $mg.\ell^{-1}$ to 5 000 $mg.\ell^{-1}$. No chemicals are added.

Flotation tanks

Each circular tank has a central reactor zone. The feed enters from the bottom and the recycle is introduced from the top of the reactor zone. The air is released through an adjustable plug valve. The water level in the tank is maintained by the fixed overflow weir. The tank is fitted with radial surface and bottom scrapers. There are 2 radial arms per tank for surface scraping, turning at about 9 revolutions per hour. The bottom sludge is bled off for about 10 minutes per shift.

Recirculation system

Flotation subnatant is repumped through a vertical, unpacked saturator. The saturator is controlled by a magnetic level switch which controls the air supply.

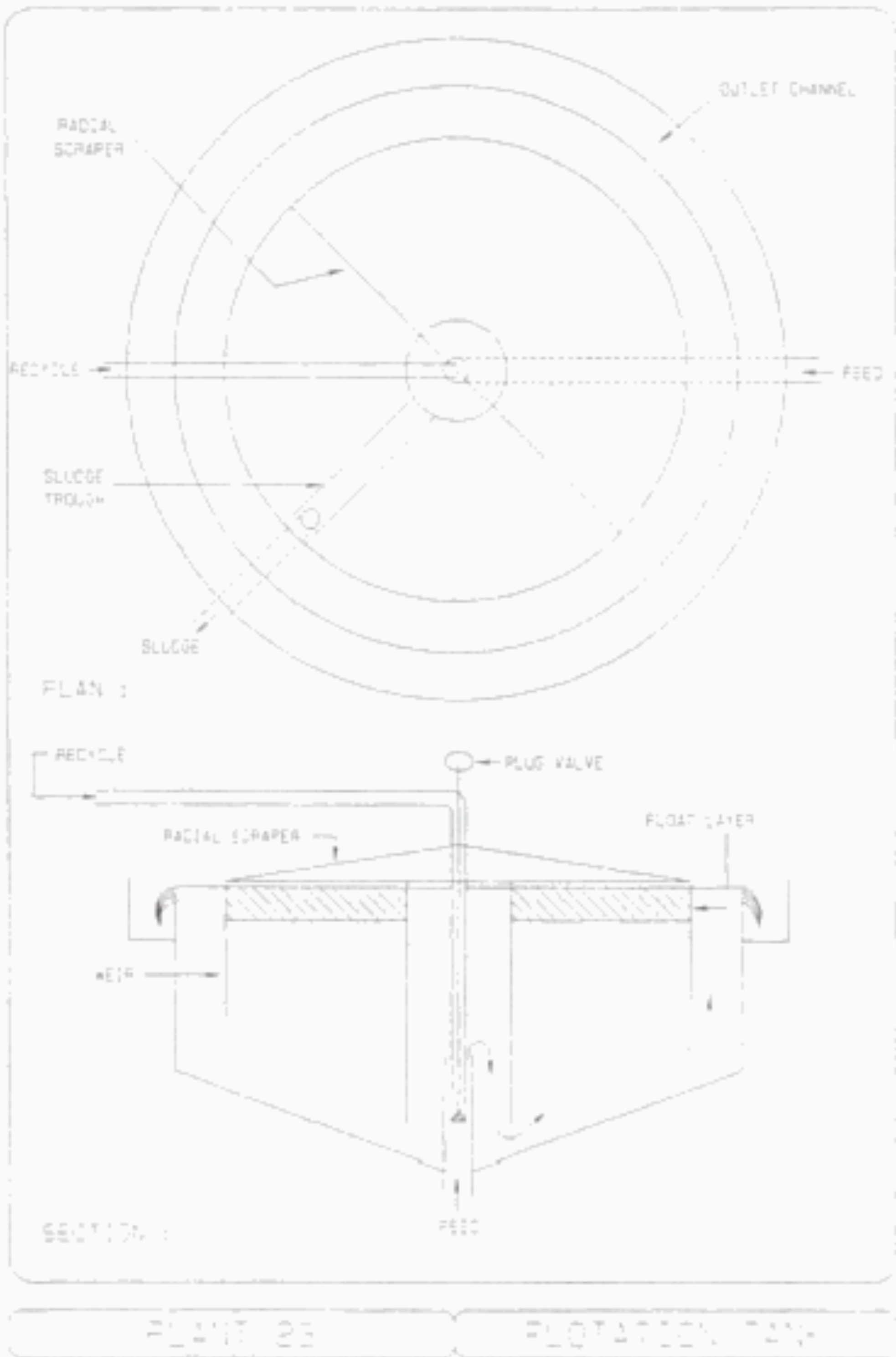
Performance

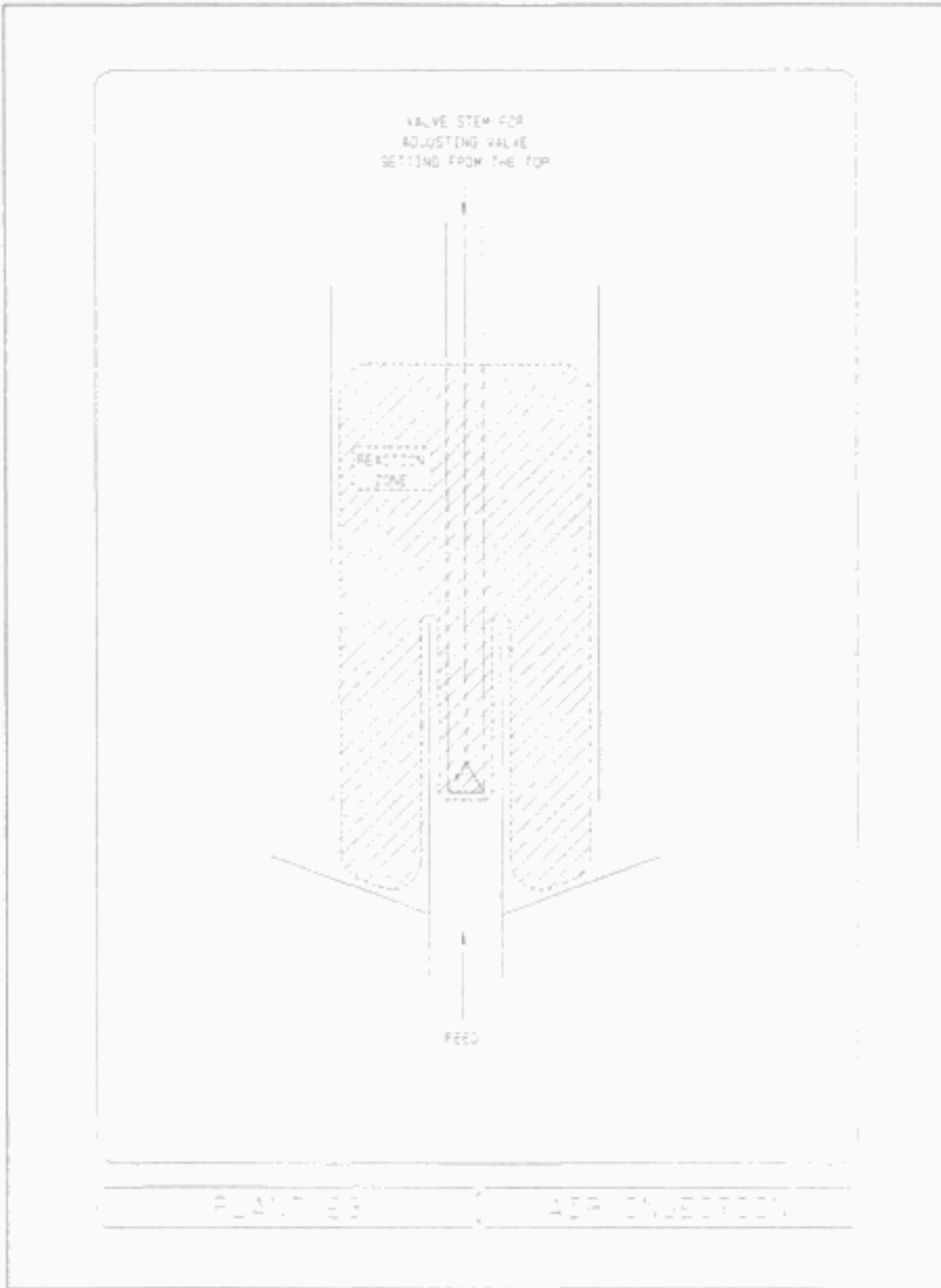
An average float concentration of 3.8 % is achieved but underflow solids concentration is poor, on the average 550 $mg.\ell^{-1}$. The scrapers and the sludge pumps require frequent maintenance

PLANT NUMBER			# 23	
*** Tank geometry ***				
Shape (Round, Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D	m	12.0	
Inner diameter	D	m	1.6	
Depth	D	m	3.5	
Number flotation cells			2	
Flotation area	A	m^2	111.1	
Reactor area	A	m^2	2.0	
Reactor volume	V	m^3	6.0	
Crossflow area	A	m^2	5.0	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	270	270
Recycle rate/cell	q	$k\ell.h^{-1}$	630	630
Saturator pressure	P	kPa	480	480
Concentration in	C	SS		4000
Concentration out	C	SS		550
*** Saturation ***				
Position (H/V)			V	
Diameter	D	m	3.80	
Length	L	m	2.0	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D		n/a	
Number of saturators			2	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		no	2.0	
Speed scrapers		$r.h^{-1}$	6.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***

Bottom scraper operated 9 times per day
Top scraper 3 minutes rest between cycles


OUTLET CHANNEL
RADIAL SCRAPER
RECYCLE
FEED
SLUDGE TROUGH
SLUDGE
PLAN
RECYCLE
PLUG VALVE
FLOAT LAYER
RADIAL SCRAPER
WEIR
FEED
SECTION
PLANT 25
FLOTATION TANK


VALVE STEM FOR
ADJUSTING VALVE
SETTING FROM THE TOP
ZONE
FEED

DESCRIPTION OF FLOTATION PLANT #24

Objective

This plant was designed for the thickening of waste activated sludge enriched with phosphate. It operated for 2 years and then mothballed because of poor performance, high maintenance and power cost. It was recommissioned during a period when ferric chloride was added to the activated sludge to promote phosphate removal, during which period the performance was good. Subsequently it was mothballed once more on account of the high cost of ferric chloride dosing. It is included in this survey for assessment of design parameters under conditions of no chemical addition.

Feed water

Suspended solids in the feed averaged 4 000 mg.ℓ^{-1}.

Flotation tank

The tank has a rectangular shape, and the water level is controlled by 2 fixed overflow weirs on the opposite ends of tank. The tank is symmetrical and the feed is introduce at the centre of the tank. There are 2 sets of surface scrapers, each scraping from the centre towards one of the sides. The tank is also equipped with bottom scrapers.

Recirculation system

Water is withdrawn from the subnatant and pumped through a horizontal, unpacked saturator. Provision is made for internal recycling. The water level in the saturator is measured, which in turn controls the air flow into the saturator. The recycle is added to the feed immediately outside the tank, and the pressure release is done across a valve.

Performance

Underflow suspended solids were poor (up to 1 000 mg.ℓ^{-1} at times) when ferric chloride dosing was absent. With ferric chloride dosing, average values of about 20 mg.ℓ^{-1} could be attained. This points towards the importance of floc structure towards floatability. In this regard the sludge volume index has been reported to be an important characteristic. A mean solids concentration of 3,6 % in the float was achieved.

PLANT NUMBER # 24

*** Tank geometry ***

Shape (Round/Rect)			Rect
Length	L	m	15.0
Width	W	m	5.0
Outer diameter	D_o		n/a
Inner diameter	D_i		n/a
Depth	D_t	m	4.5
Number flotation cells			2
Flotation area	A_f	m^2	75.0
Reactor area	A_r	m^2	2.6
Reactor volume	V_r	m^3	4.1
Crossflow area	A_c	m^2	2.0

*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$		
Recycle rate/cell	q	$k\ell.h^{-1}$		
Saturator pressure	P	kPa		
Concentration in	C	SS		4000
Concentration out	C	SS		20

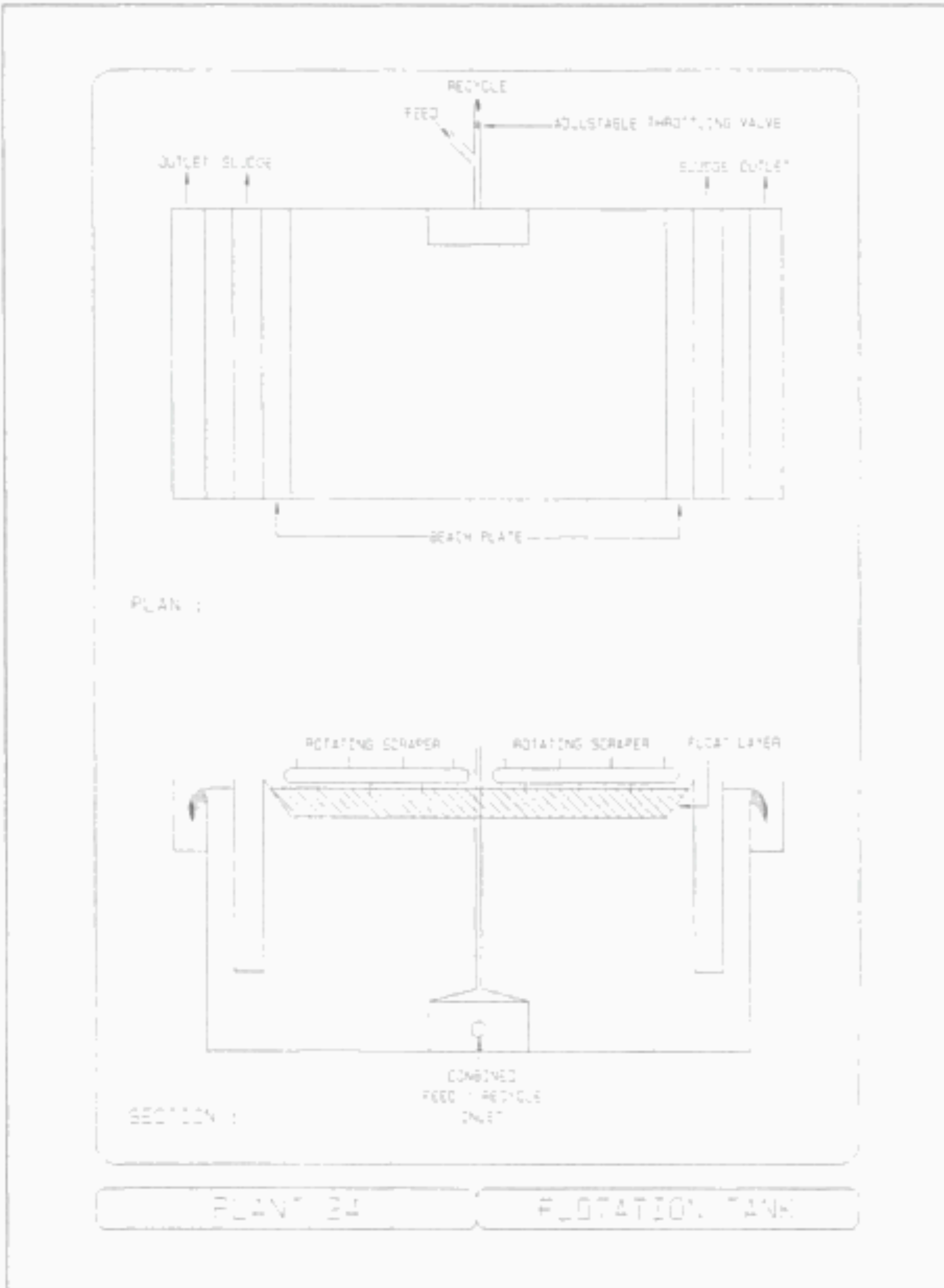
*** Saturation ***

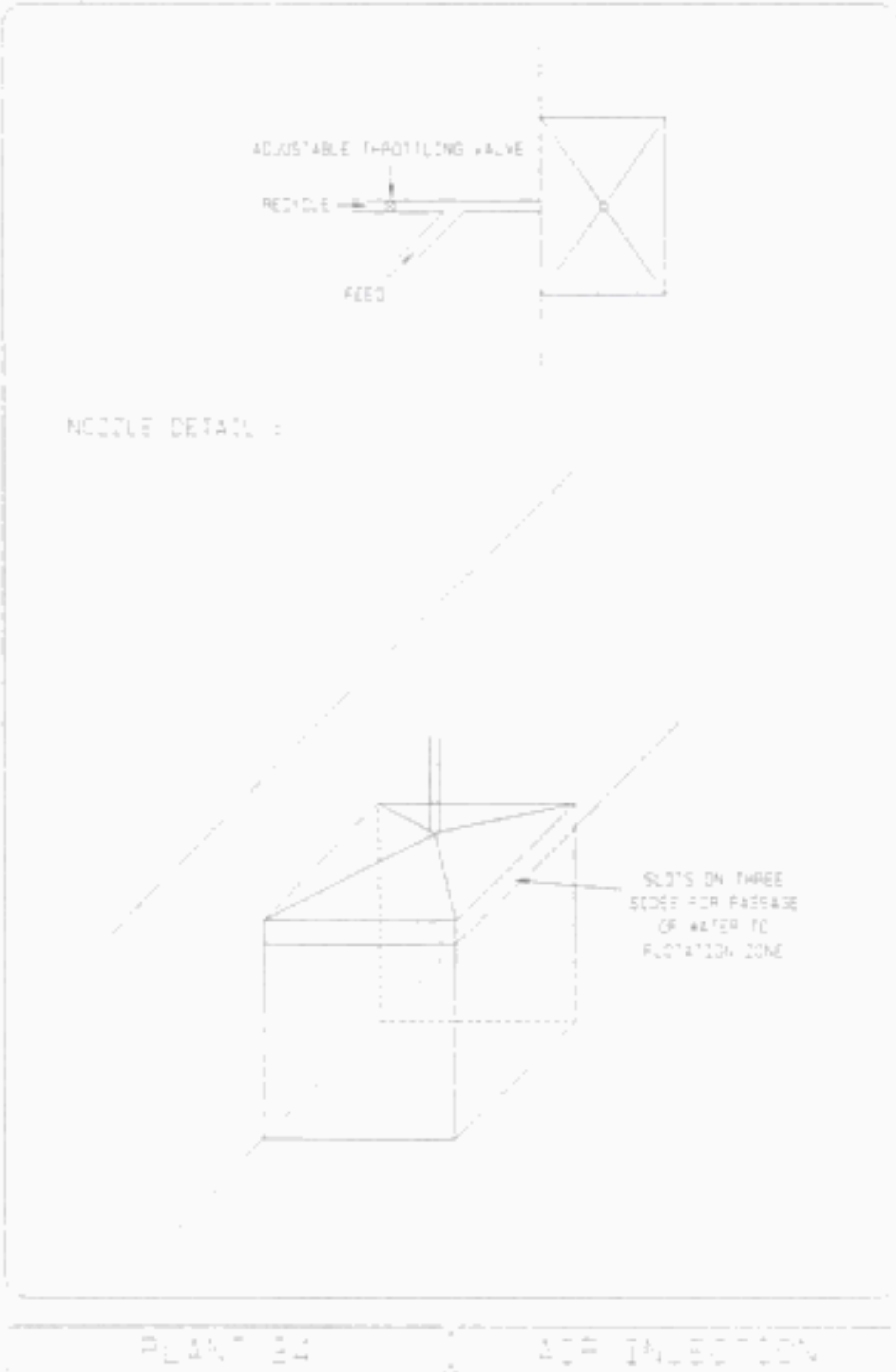
Position (H/V)			H
Diameter	D_s	m	1.30
Length	L_s	m	4.0
Packing (Y/N)			N
Recirculation (Y/N)			Y
Depth packing	D_p		n/a
Number of saturators			2

*** Float removal ***

Grid (Y/N)			N
Scraper (Y/N)			Y
Spacing scrapers		m	?
Speed scrapers		$m.h^{-1}$	?
Continuous scraping (Y/N)			Y
Bottom desludging (Y/N)			Y

*** Notes ***


RECYCLE
FEED
ADJUSTABLE THROTTLING VALVE
OUTLET SLUDGE
SLUDGE OUTLET
BEACH PLATE
PLAN :
ROTATING SCRAPER
ROTATING SCRAPER
FLOAT LAYER
COMBINED
FEED + RECYCLE
INLET
SECTION :
FLOTATION TANK


ADJUSTABLE THROTTLING VALVE
RECYCLE
FEED
NOZZLE DETAIL :
SLOTS ON THREE
SIDES FOR PASSAGE
OF WATER TO
FLOTATION ZONE
PLANT 24
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #25

Objective

The purpose of the plant is to thicken waste activated sludge from a nutrient removal plant prior to anaerobic digestion.

Feed water

The solids concentration varies between 3 000 $mg.\ell^{-1}$ to 7 500 $mg.\ell^{-1}$ with an average of 4 300 $mg.\ell^{-1}$. No chemicals are added.

Flotation tanks

Two similar, rectangular tanks are used. The water level is controlled with a fixed overflow weir. Recycle is introduced externally to the feed and a pipe length of about 3 m serves as the reactor zone. The blended flow enters the flotation tank at 4 bottom discharges with impingement on deflector discs. The recycle pressure is released across a diaphragm valve.

Recirculation system

There are 2 vertical, unpacked saturators, which are fed by recycle pumps withdrawing water from the flotation subnatant. The air supply is controlled to maintain a constant water level.

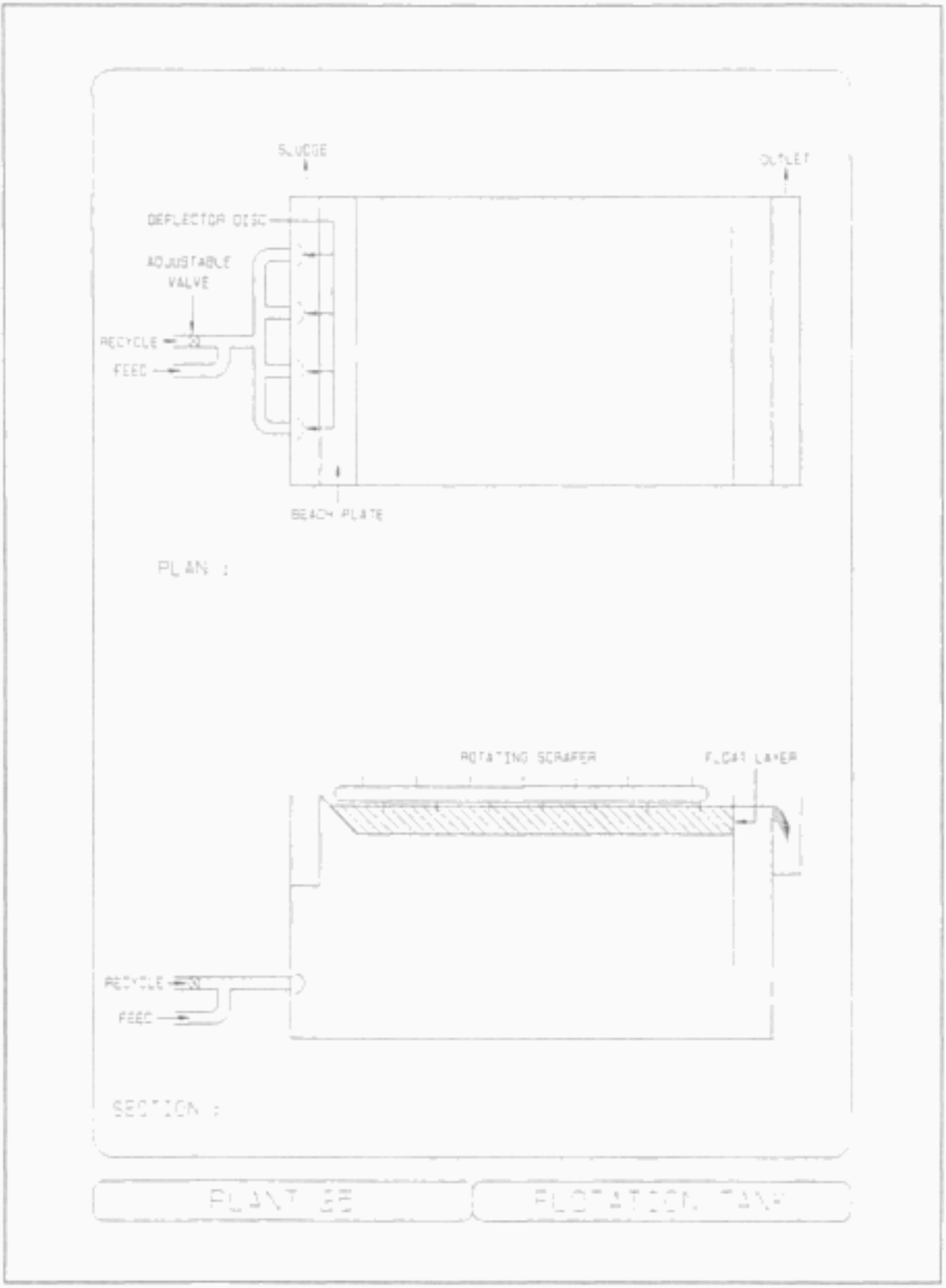
Performance

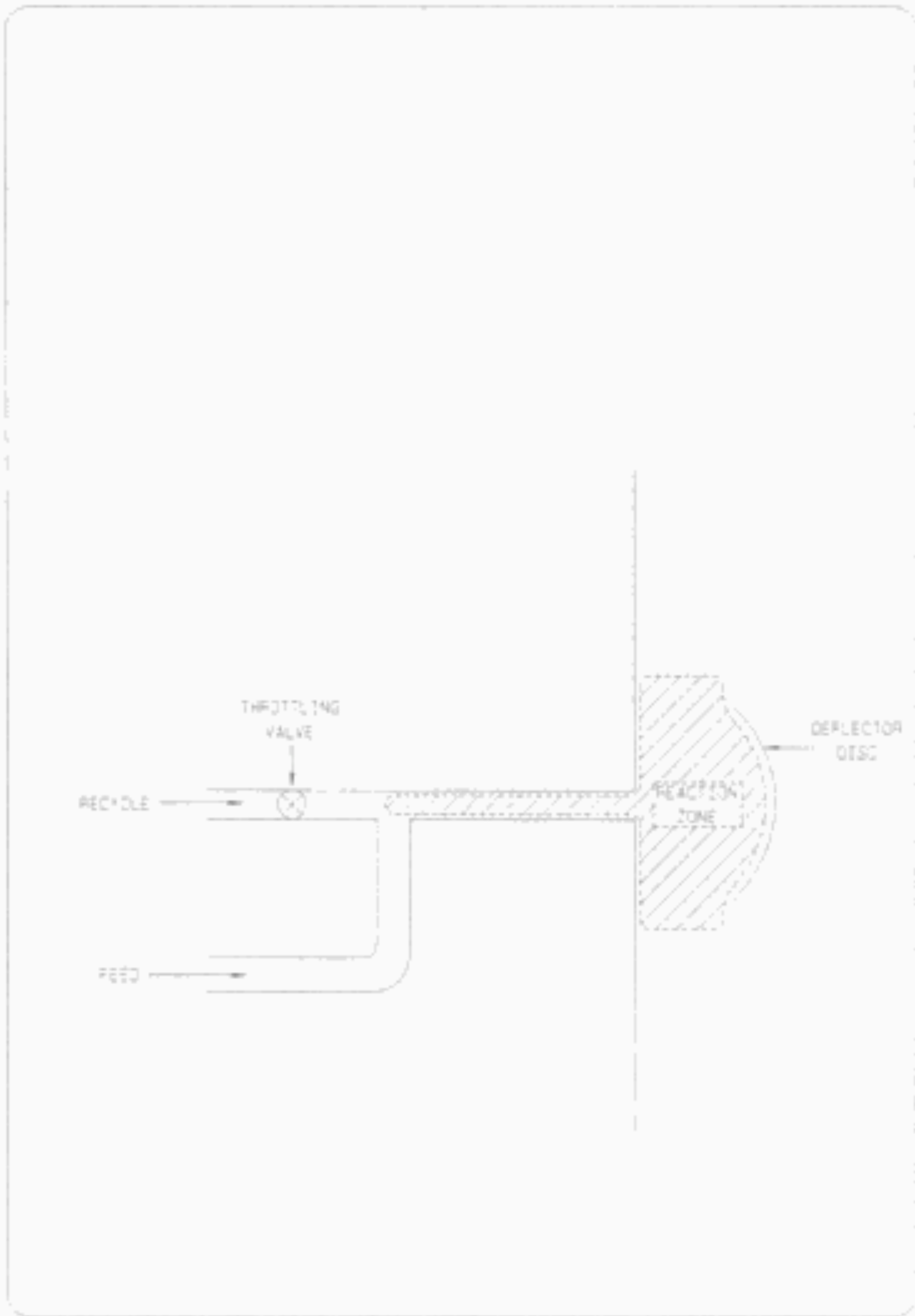
The flotation system is susceptible to overloading when the solids concentration exceeds 4 000 $mg.\ell^{-1}$. Float concentrations of 3.3 % to 5.9 % are achieved with underflow varying between 80 $mg.\ell^{-1}$ and 2 200 $mg.\ell^{-1}$, with an average of about 400 $mg.\ell^{-1}$. High maintenance on bottom as well as top scrapers was reported, with a delay with the importation of parts.

PLANT NUMBER			# 25	
*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	29.6	
Width	W	m	6.3	
Outer diameter	D		n/a	
Inner diameter	D		n/a	
Depth	D	m	3.6	
Number flotation cells			2	
Flotation area	A	m^2	186.5	
Reactor area	A	m^2	0.8	
Reactor volume	V	m^3	2.4	
Crossflow area	A	m^2	1.0	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$		540
Recycle rate/cell	q	$k\ell.h^{-1}$		180
Saturator pressure	P	kPa		540
Concentration in	C	SS		4200
Concentration out	C	SS		420
*** Saturation ***				
Position (H/V)			V	
Diameter	D	m	2.00	
Length	L	m	3.0	
Packing (Y/N)			N	
Recirculation (Y/N)			Y	
Depth packing	D_d		n/a	
Number of saturators			2	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		m	?	
Speed scrapers		$m.h^{-1}$	?	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***

Bottom scraper operated 9 times per day.
Top scraper 3 minutes rest between cycles.


SLUDGE
OUTLET
DEFLECTOR DISC
ADJUSTABLE VALVE
RECYCLE
FEED
BEACH PLATE
PLAN :
ROTATING SCRAPER
FLOAT LAYER
RECYCLE
FEED
SECTION :
PLANT 33
FLOTATION TANK



PLANT 35 | AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #26

Objective

The plant thickens waste activated sludge by flotation as pretreatment before centrifugation

Feed water

The suspended solids concentration varies from 3 400 $mg.\ell^{-1}$ to 4 700 $mg.\ell^{-1}$, with an average of about 4 200 $mg.\ell^{-1}$. Ferric sulphate is dosed in the anoxic zone of 1 module of the activated sludge plant, at a concentration of about 30 $mg.\ell^{-1}$

Flotation tank

Two rectangular tanks are used, of which the water level is controlled by adjustable overflow weirs. Adjustable surface scrapers are operated intermittently with a 3-minute rest period between cycles. 5 blades are fitted for each tank. The recycle pressure is released across a globe valve where the recycle is blended with the feed immediately outside the tank. The blend is introduced into a small rectangular box which acts as a reactor compartment. The inlet arrangement was subsequently modified to a pipe with baffled holes.

Recirculation system

There are 2 horizontal, unpacked saturators, with provision for internal recirculation. The recycle is withdrawn from the flotation subnatant and pumped through the saturators. The water level in the saturator is maintained by means of a solenoid valve on the air supply, activated from a level sensor.

Performance

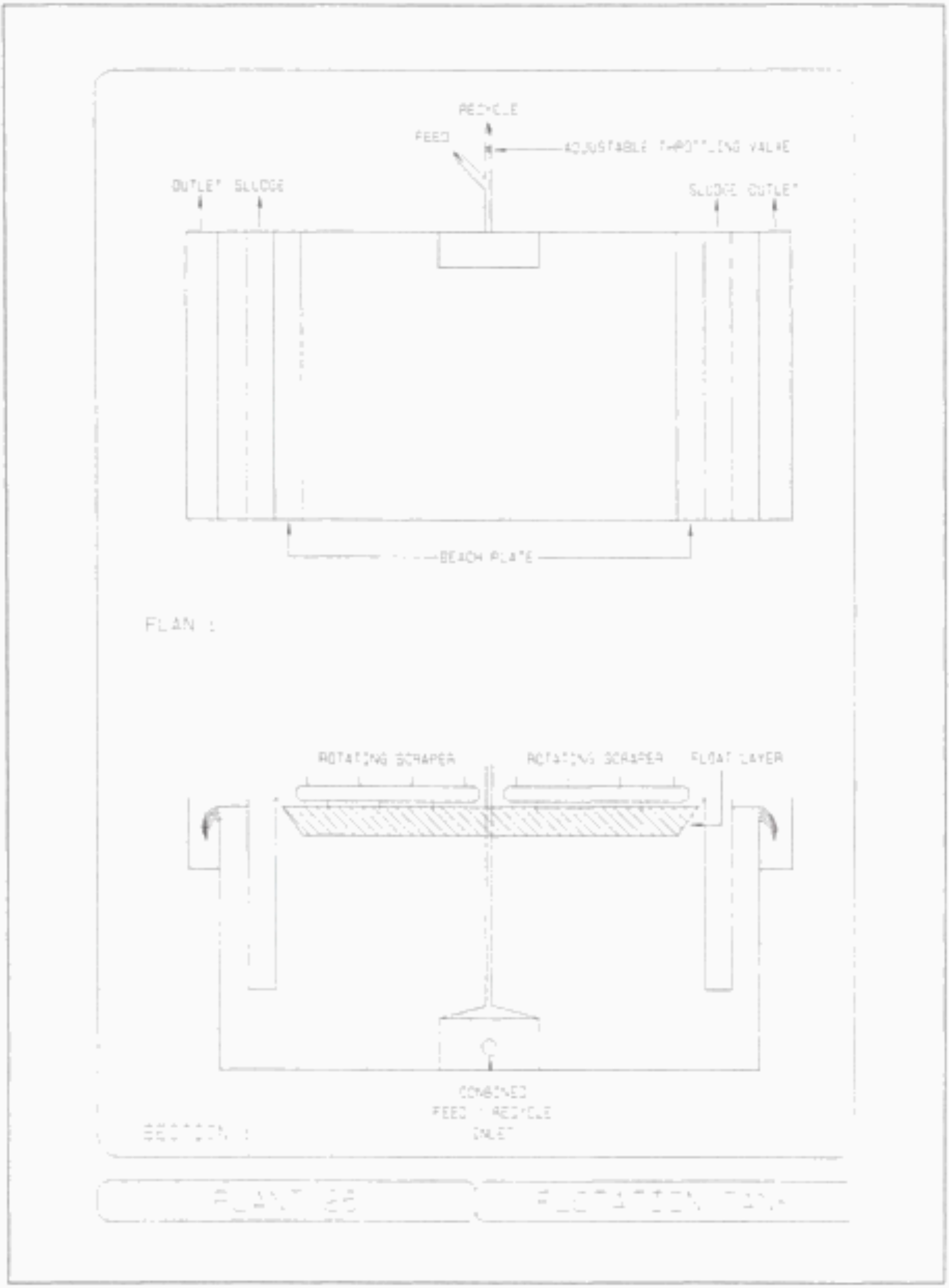
Float solids concentration varies between 2,7 % and 3,6 %, with a mean of 3,1 %. The underflow concentration varies from 80 $mg.\ell^{-1}$ to 2 300 $mg.\ell^{-1}$ with a mean of 400 $mg.\ell^{-1}$. Problems with maintenance on scrapers, valves and pump packings were reported. The tanks are emptied twice per year for cleaning.

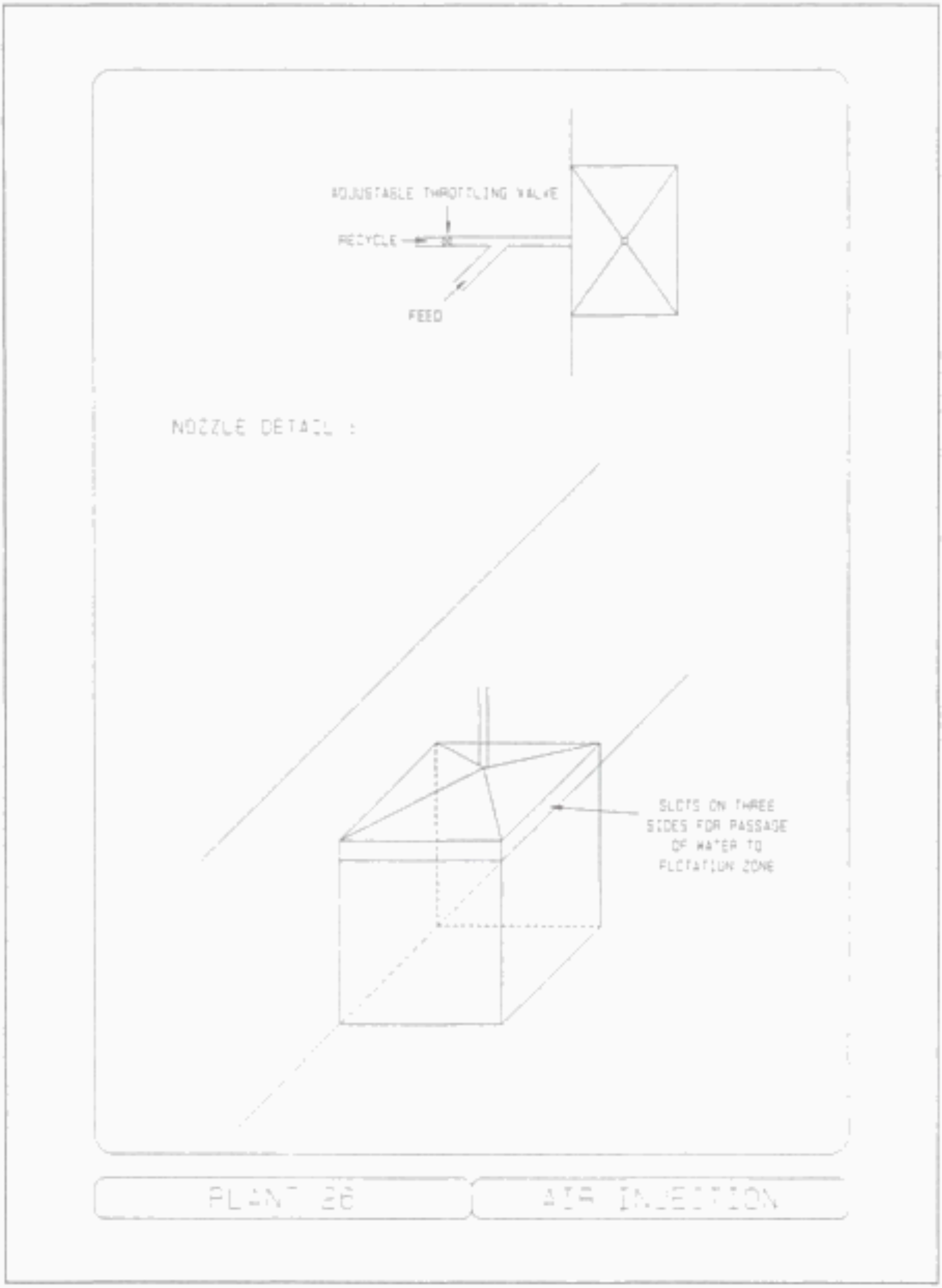
PLANT NUMBER # 26

*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	21.0	
Width	W	m	6.0	
Outer diameter	D		n/a	
Inner diameter	D		n/a	
Depth	D	m	4.0	
Number flotation cells			2	
Flotation area	A	m^2	126.0	
Reactor area	A	m^2	1.0	
Reactor volume	V	m^3	1.8	
Crossflow area	A	m^2	0.5	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell\,h^{-1}$	500	417
Recycle rate/cell	q	$k\ell\,h^{-1}$	580	580
Saturator pressure	P	kPa		540
Concentration in	C	SS		4200
Concentration out	C	SS		430
*** Saturation ***				
Position (H/V)			H	
Diameter	D	m	1.70	
Length	L	m	3.5	
Packing (Y/N)			N	
Recirculation (Y/N)			Y	
Depth packing	D		n/a	
Number of saturators			2	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers			m	5.0
Speed scrapers			m/h	129.0
Continuous scraping (Y/N)			N	
Bottom desludging (Y/N)			Y	

*** Notes ***

Bottom scraper operated 9 times per day
Top scraper 3 minutes rest between cycles


RECYCLE
FEED
ADJUSTABLE THROTTLING VALVE
OUTLET
SLUDGE
SLUDGE
OUTLET
BEACH PLATE
PLAN :
ROTATING SCRAPER
ROTATING SCRAPER
FLOAT LAYER
COMBINED
FEED + RECYCLE
INLET
SECTION :
PLANT 25
FLOTATION TANK


ADJUSTABLE THROTTLING VALVE
RECYCLE
FEED
NOZZLE DETAIL :
SLOTS ON THREE SIDES FOR PASSAGE OF WATER TO FLOTATION ZONE
PLANT 26
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #27

Objective

The flotation plant thickens waste activated sludge from a biological waste-water treatment plant designed for nutrient removal.

Feed water

The plant treats raw municipal sewage of about 15% industrial contribution from food and textile industries. The waste activated sludge has a sludge age of about 15 days. The suspended solids concentration varies between 4 000 $mg.\ell^{-1}$ and 7 000 $mg.\ell^{-1}$, with an average of 450 $mg.\ell^{-1}$. Excellent bioflocculation is reported which obviates the need for chemical treatment.

Flotation tank

There is 1 circular flotation tank with a fixed peripheral overflow weir. The recycle is introduced through an adjustable plug valve and blended with the feed at the bottom of the reaction zone. 4 radial scrapers are operated at 8 $r.min^{-1}$ and a tilt valve desludges the bottom at a rate of 1 m^3 every 30 minutes.

Recirculation system

Saturation takes place in a vertical, unpacked saturator. The recycle is sprayed into the saturator by means of a number of nozzles. Originally the saturator was packed, but it then blocked within a few hours. The liquid level is controlled by modulation of the air inlet rate. The recycle rate can be manually adjusted, depending on the solids loading rate.

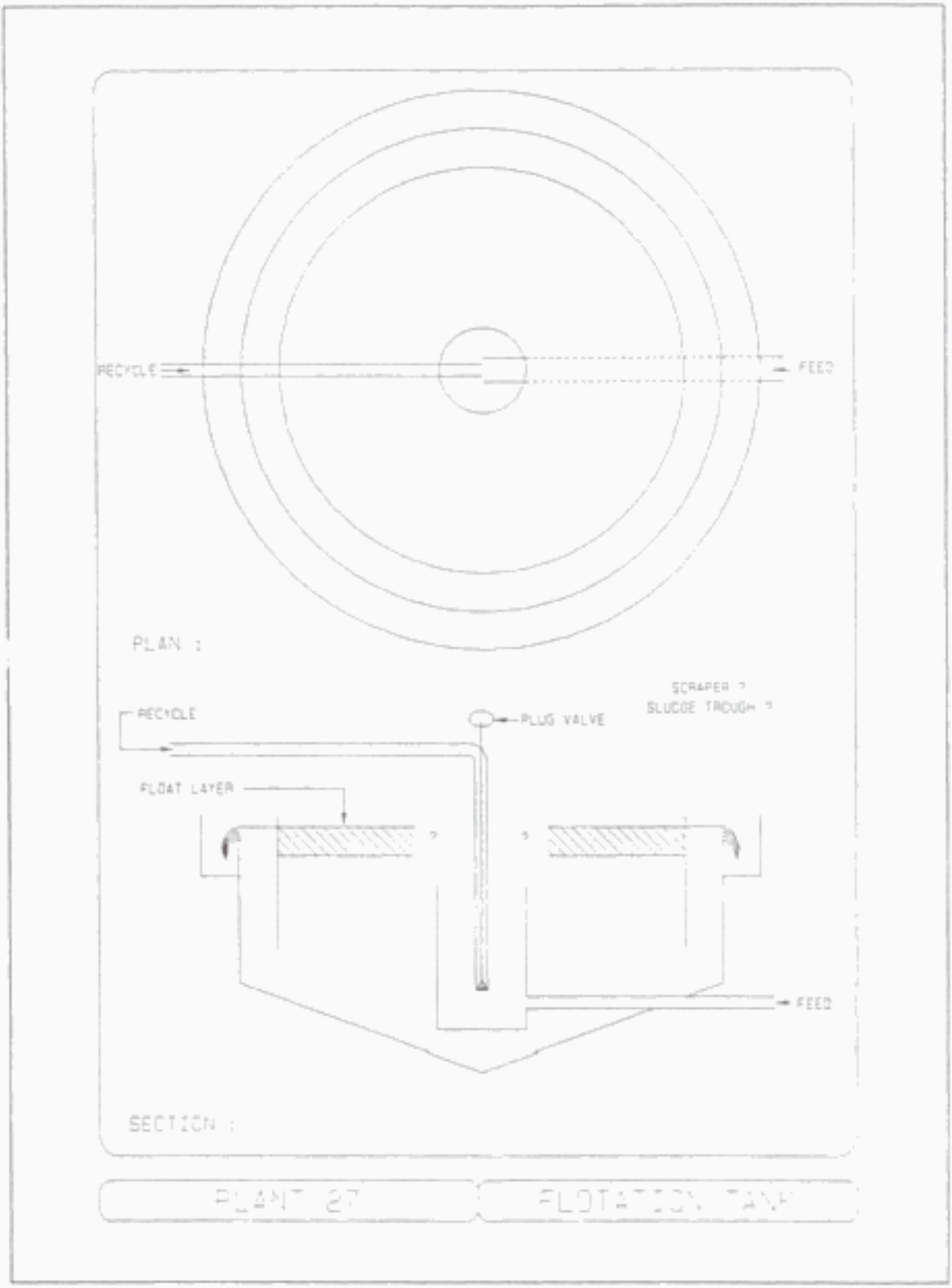
Performance

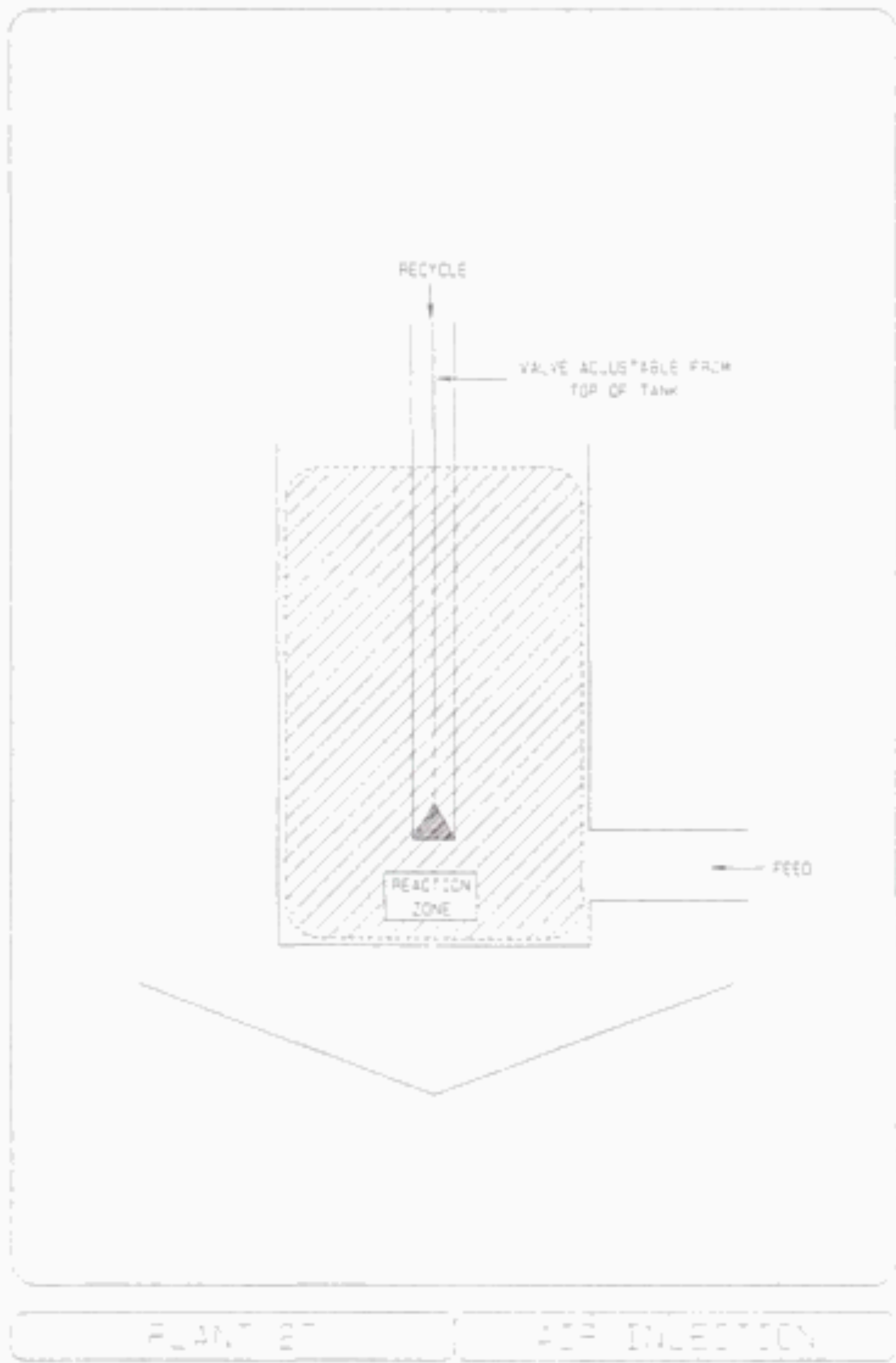
The float layer solids concentration varies between 3% and 4%. The underflow solids concentration is about 50 $mg.\ell^{-1}$ on the average. The general performance is satisfactory with minor problems such as water in the solenoid valve.

PLANT NUMBER # 27

*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	10.4	
Inner diameter	D	m	1.1	
Depth	D	m	3.2	
Number flotation cells			1	
Flotation area	A	m^2	84.0	
Reactor area	A	m^2	1.0	
Reactor volume	V	m^3	4.2	
Crossflow area	A	m^2	1.0	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	125	71
Recycle rate/cell	q	$k\ell.h^{-1}$	137	115
Saturator pressure	P	kPa	500	600
Concentration in	C	SS		4500
Concentration out	C_o	SS		50
*** Saturation ***				
Position (H/V)			V	
Diameter	D_s	m	1.60	
Length	L_s	m	3.3	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D_p		n/a	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Number of scrapers		no	4.0	
Speed scrapers		$r.h^{-1}$	8.2	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***


RECYCLE
FEED
PLAN :
SCRAPER ?
SLUDGE TROUGH ?
RECYCLE
PLUG VALVE
FLOAT LAYER
FEED
SECTION :
PLANT 27
FLOTATION


RECYCLE
VALVE ADJUSTABLE FROM TOP OF TANK
FEED
REACTION ZONE

DESCRIPTION OF FLOTATION PLANT #28

Objective

The flotation plant thickens waste activated sludge from a waste-water treatment plant designed for biological nutrient removal.

Feed water

The treatment plant treats predominantly domestic wastewater. The solids concentration of the waste activated sludge varies between 4 000 $mg.\ell^{-1}$ and 6 500 $mg.\ell^{-1}$ with an average of 5 000 $mg.\ell^{-1}$. The sludge passes through a mechanically raked bar screen with 5 mm openings between bars before flotation - without this, numerous problems were earlier encountered. No treatment chemicals are added

Flotation tank

There are 2 circular flotation tanks with fixed peripheral overflow weirs. The water flows from the central reaction zone through a number of crossflow ports into the flotation zone. The recycle is released into the reaction zone through an adjustable plug valve. Provision for both surface and bottom scrapers is made. Four radial arms on the surface, at a speed of 5 $r.min^{-1}$ discharge continuously into a radial trough. Bottom desludging occurs routinely once per day.

Recirculation system

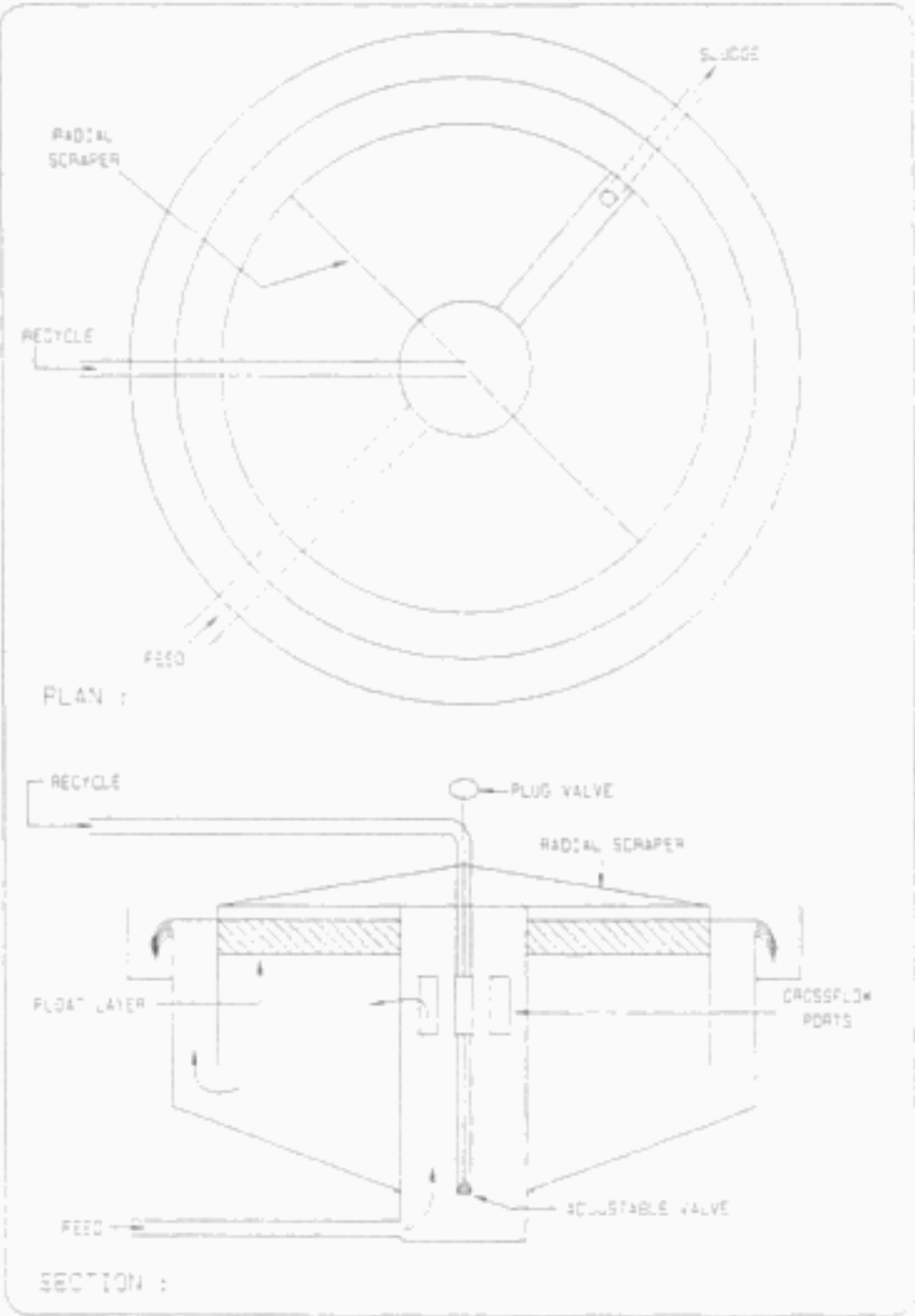
A single vertical unpacked saturator serves both flotation tanks. The recycle is sprayed into the saturator. A magnetic switch on the air inlet controls the water level.

Performance

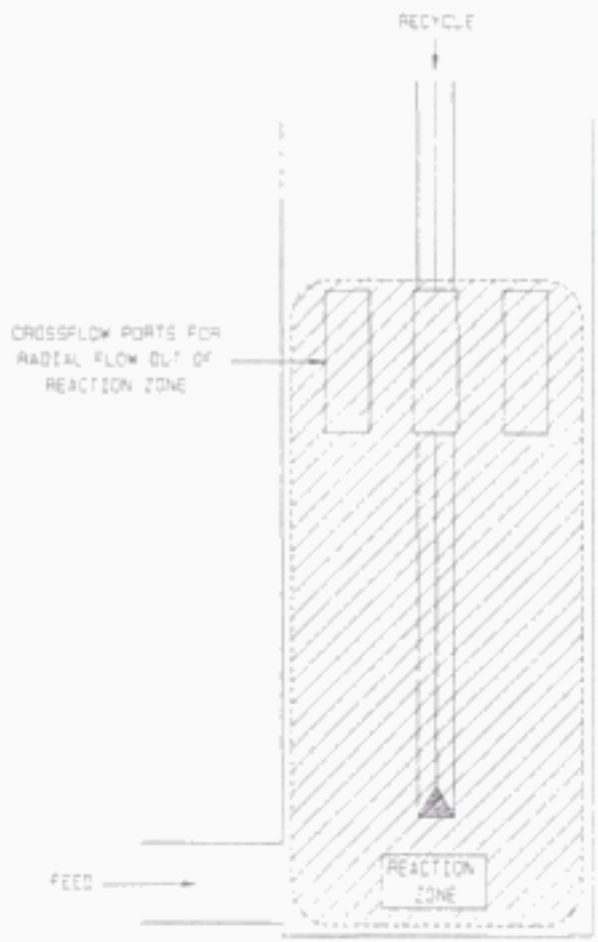
An average float solids concentration of 3,5% is achieved, with the underflow solids concentration at about 300 $mg.\ell^{-1}$. Problems are encountered with the air compressors due to the high switching frequency, as well as with the scraper gearbox. Generally the plant operates satisfactorily.

PLANT NUMBER			# 28	
*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	10.0	
Inner diameter	D_i	m	1.0	
Depth	D_d	m	2.6	
Number flotation cells			2	
Flotation area	A_f	m^2	78.6	
Reactor area	A_r	m^2	0.8	
Reactor volume	V_r	m^3	2.6	
Crossflow area	A_c	m^2	ID	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	108	83
Recycle rate/cell	q	$k\ell.h^{-1}$	128	128
Saturator pressure	P	kPa	600	450
Concentration in	C_i	SS	4500	5000
Concentration out	C_o	SS		300
*** Saturation ***				
Position (H/V)			V	
Diameter	D_s	m	1.80	
Length	L_s	m	2.0	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D_p		n/a	
Number of saturators			2	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Number of scrapers		no	4.0	
Speed scrapers		$r.h^{-1}$	5.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***



PLANT 29 | FLOTATION TANK



AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #29

Objective

The flotation plant thickens waste activated sludge from a waste-water treatment plant designed for biological nutrient removal. Ferric chloride is used to achieve special phosphate standards in a sensitive catchment area.

Feed water

The solids concentration of the waste activated sludge varies between 3 300 $mg.\ell^{-1}$ and 4 200 $mg.\ell^{-1}$, with an average of 4 000 $mg.\ell^{-1}$. The sludge is prescreened through a raked bar screen with 5 mm openings between bars.

Flotation tank

There are 2 circular flotation tanks with fixed peripheral overflow weirs. The recycle enters the reaction zone through a plug valve which is adjustable from the outside of the tank. There are 2 radial surface scraper arms per tank which operate continuously at about 3 revolutions per hour. The bottom is also scraped, and desludged once per day. Approximately 70 m^3 of sludge is produced daily from a feed of 545 $m^3.d^{-1}$.

Recirculation system

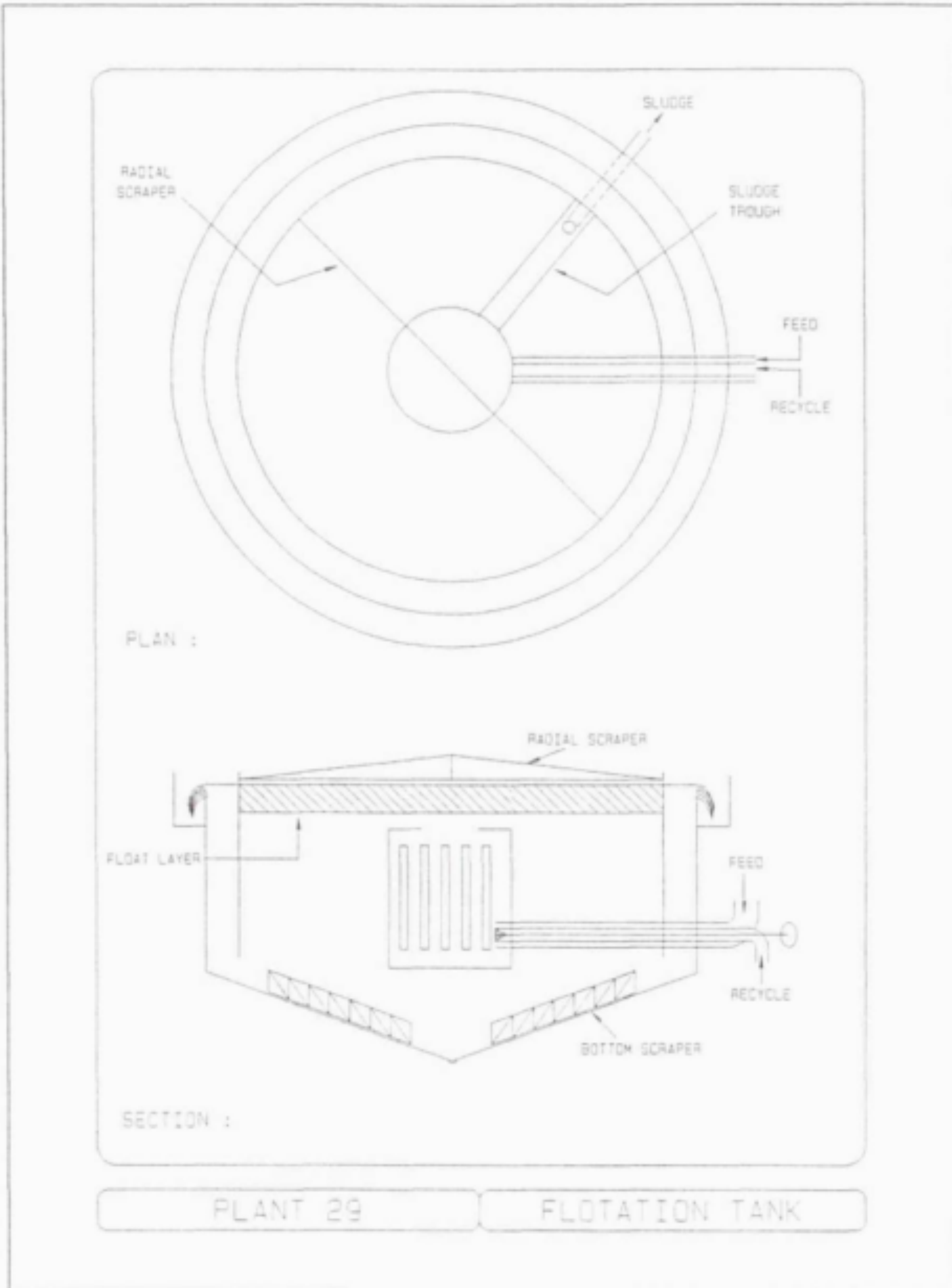
A single vertical unpacked saturator serves both flotation tanks. The recycle is sprayed into the saturator by means of an inverted splash plate. The water level is maintained by controlling the air supply via a float switch in the saturator.

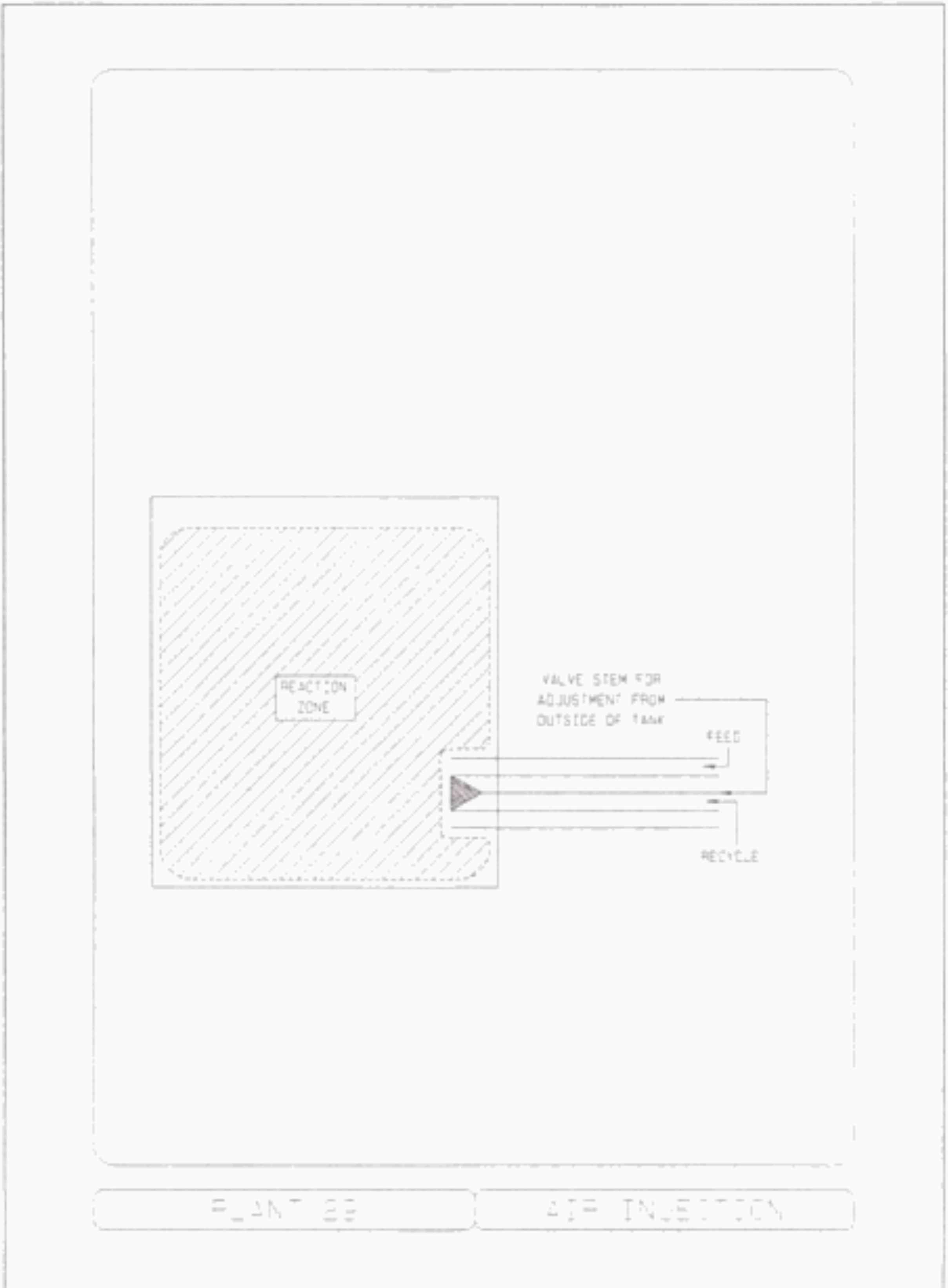
Performance

The float solids concentration varies between 3,8% and 4,7% with a solids concentration in the underflow of 50 $mg.\ell^{-1}$ to 60 $mg.\ell^{-1}$. The general performance of the plant is highly satisfactory, although there are typical problems such as high wear rate on the mono-type sludge pumps, which can be minimized by effective screening of the waste activated sludge stream, and the fouling of the float switches in the saturator. The plug valve controlling the recycle rate and system pressure requires a weekly flush of 30 seconds to avoid build-up of hair, plastic and other waste in the valve aperture.

PLANT NUMBER			# 29	
*** Tank geometry ***				
Shape (Round/Rect)			Round	
Length	L		n/a	
Width	W		n/a	
Outer diameter	D_o	m	6.0	
Inner diameter	D_i	m	0.5	
Depth	D	m	2.2	
Number flotation cells			2	
Flotation area	A_f	m^2	28.1	
Reactor area	A_r	m^2	0.2	
Reactor volume	V_r	m^3	0.2	
Crossflow area	A_c	m^2	0.7	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	22	10
Recycle rate/cell	q	$k\ell.h^{-1}$	22	22
Saturator pressure	P	kPa		550
Concentration in	C_i	SS	3150	3700
Concentration out	C_o	SS	315	50
*** Saturation ***				
Position (H/V)			V	
Diameter	D_s	m	0.90	
Length	L_s	m	2.2	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D_p		n/a	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		no	2.0	
Speed scrapers		$r.h^{-1}$	3.3	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***


SLUDGE
RADIAL SCRAPER
SLUDGE TROUGH
FEED
RECYCLE
PLAN :
RADIAL SCRAPER
FLOAT LAYER
FEED
RECYCLE
BOTTOM SCRAPER
SECTION :
PLANT 29
FLOTATION TANK


REACTION
ZONE
VALVE STEM FOR
ADJUSTMENT FROM
OUTSIDE OF TANK
FEED
RECYCLE
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #30

Objective

Flotation is used to thicken waste activated sludge before it is passed on to primary and secondary anaerobic digestion

Feed

The solids concentration varies between 5 000 $mg.\ell^{-1}$ and 6 000 $mg.\ell^{-1}$. No chemicals are added.

Flotation tank

There is 1 rectangular flotation tank with multiple hoppers at the bottom. The water level is controlled by an adjustable overflow weir. The recycle is drawn from the subnatant and used to be passed through a filter, but the filter is not being used any more. The recycle is injected through a diaphragm valve into the feed line and blending therefore takes place before the flotation tank. The blended stream is introduced near the top of tank into the flotation zone with a distribution "umbrella" which directs the flow upwards. A chain driven surface scraper with rubber blades operates intermittently. Sludge is withdrawn from the bottom hoppers at weekly intervals. About 100 m^3 of sludge is produced daily from 500 m^3 feed.

Recirculation system

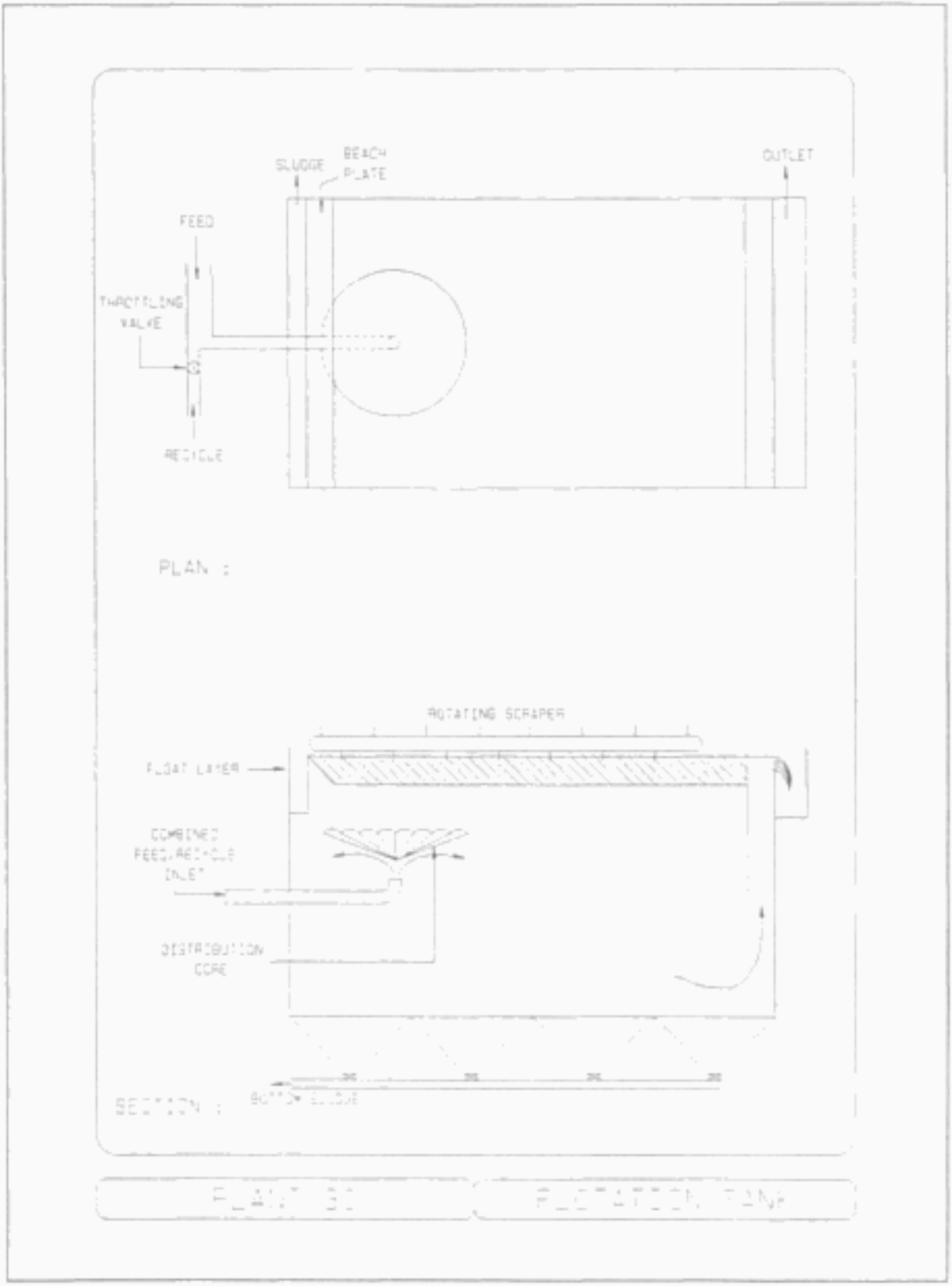
One horizontal, unpacked saturator with internal reaeration is used. The liquid level is measured with level probes and the level is maintained by controlling the air flow rate into the saturator

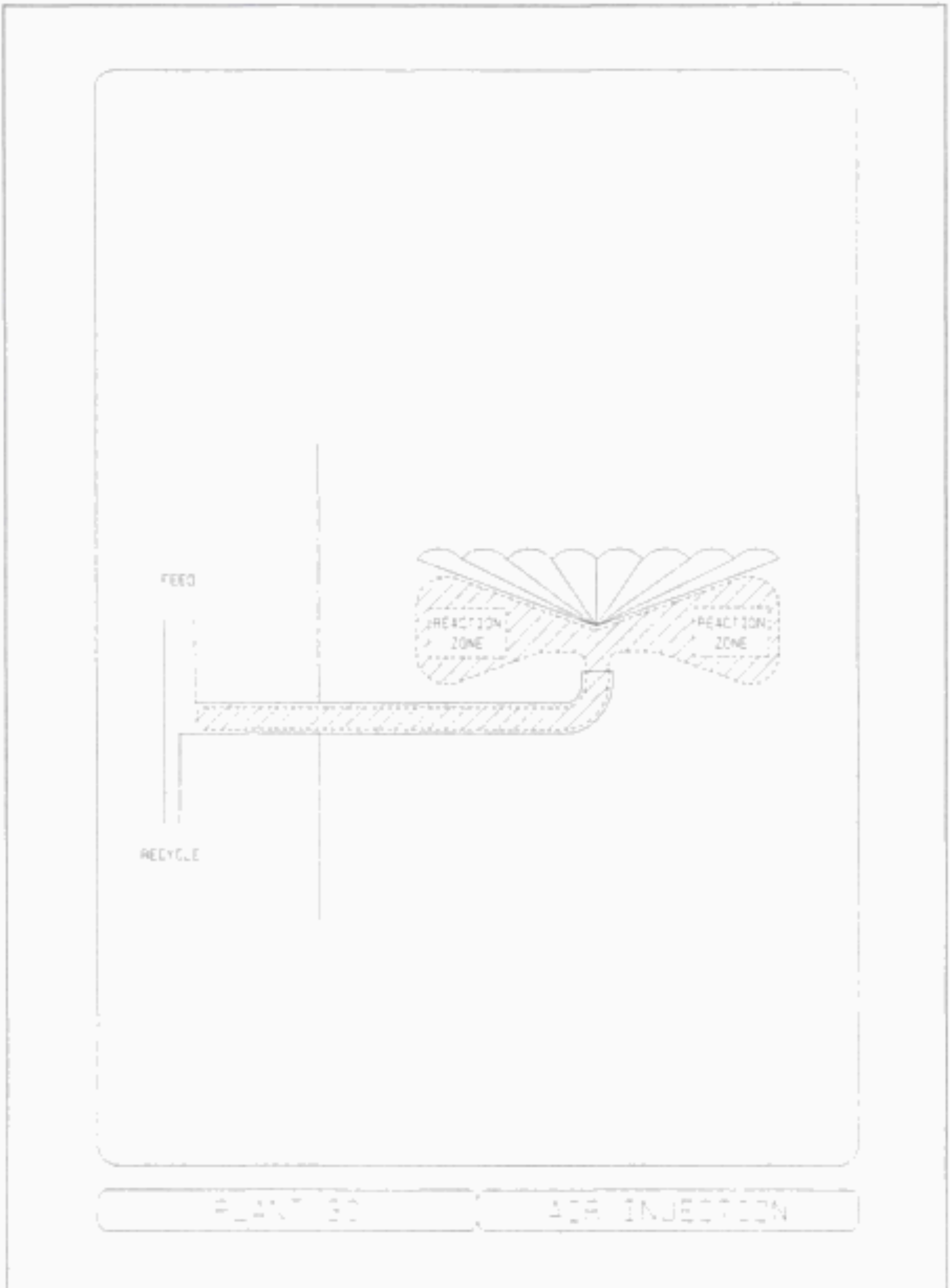
Performance

The float solids concentration varies between 3,5% and 4,0% with the solids concentration in the underflow about 200 $mg.\ell^{-1}$. The problems encountered are mainly related to corrosion. Rollers and chains were replaced after 7 years. Level probes and sight glasses must be cleaned regularly. In general, the plant operation is highly satisfactory and claimed to be superior to gravity thickeners for the thickening of waste activated sludge

PLANT NUMBER			# 30	
*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	11.0	
Width	W	m	3.6	
Outer diameter	D_o		n/a	
Inner diameter	D		n/a	
Depth	D_t	m	2.3	
Number flotation cells			1	
Flotation area	A_f	m^2	39.6	
Reactor area	A_r	m^2	ID	
Reactor volume	V_r	m^3	ID	
Crossflow area	A_c	m^2	ID	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	50	21
Recycle rate/cell	q	$k\ell.h^{-1}$	50	50
Saturator pressure	P	kPa		480
Concentration in	C	SS	7000	5500
Concentration out	C_e	SS	350	200
*** Saturation ***				
Position (H/V)			H	
Diameter	D_s	m	1.00	
Length	L_s	m	2.5	
Packing (Y/N)			N	
Recirculation (Y/N)			Y	
Depth packing	D_p		n/a	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		m	2.0	
Speed scrapers		$m.h^{-1}$	90.0	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			Y	

*** Notes ***


SLUDGE
BEACH PLATE
OUTLET
FEED
THROTTLING VALVE
RECYCLE
PLAN :
ROTATING SCRAPER
FLOAT LAYER
COMBINED FEED/RECYCLE INLET
DISTRIBUTION CONE
SECTION :


FEED
REACTION ZONE
REACTION ZONE
RECYCLE
AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #31

Objective

Flotation is used for pollution control purposes treating effluents discharged from a waste paper mill.

Feed water

The solids concentration varies between 2 500 mg.ℓ^{-1} and 5 000 mg.ℓ^{-1}, with an average of 3 000 mg.ℓ^{-1}. It contains alum residuals from the paper manufacturing process and associated organic constituents. Polymer dosages from 10 mg.ℓ^{-1} to 50 mg.ℓ^{-1} are injected in-line.

Flotation tank

The tank is of rectangular, proprietary design with surface scraping only. The water level is controlled by height adjustment of telescopic weirs on the outlet pipes. There is no flow equalization and the flow rate through the flotation unit is variable, depending on the production of the paper plant. The recycle is injected through a pressure reducing valve into the feed pipe before the blend is introduced into the tank.

Recirculation system

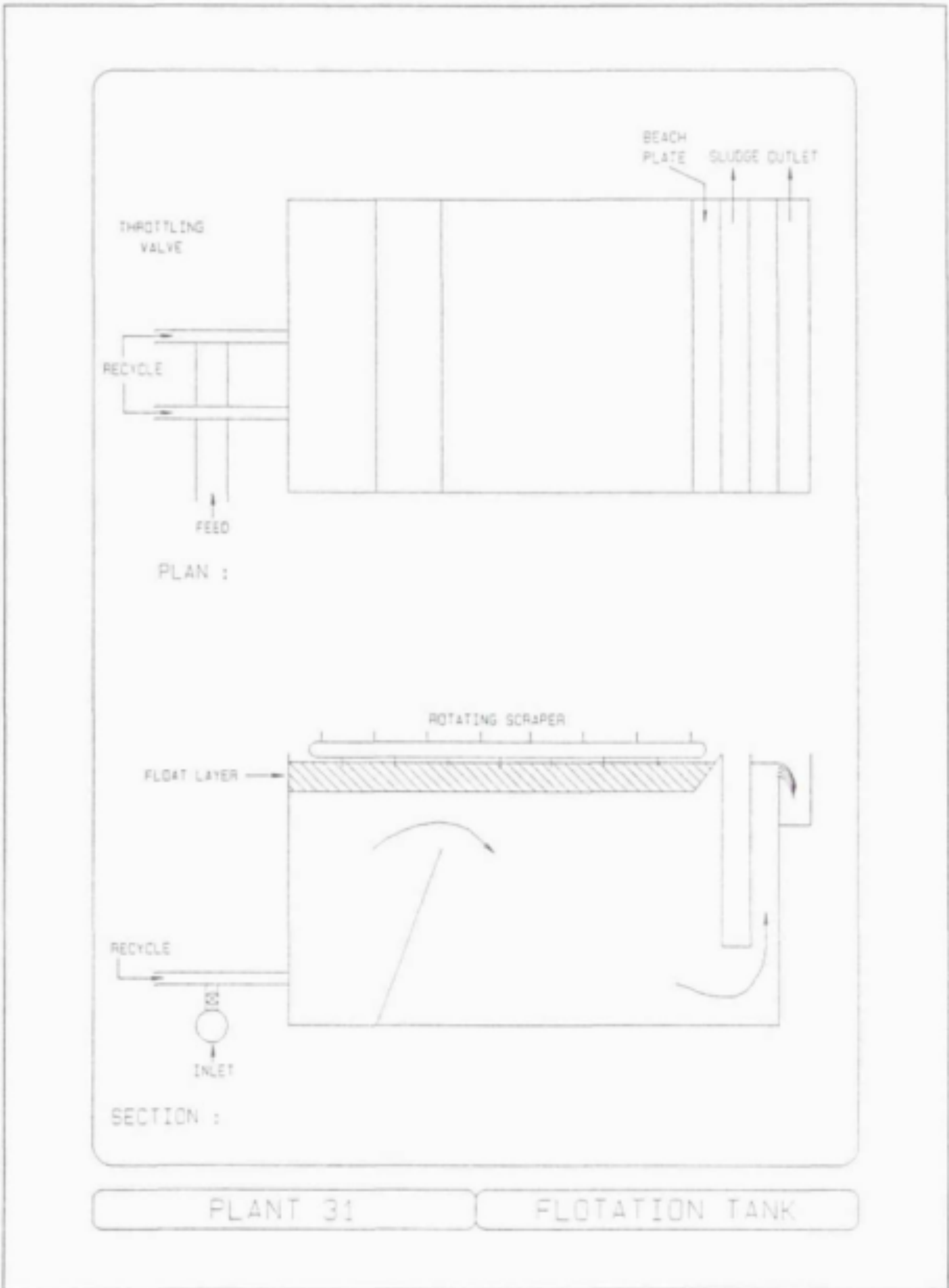
There is 1 unpacked, vertically mounted saturator. The water level is controlled by a float valve which controls the recycle inlet.

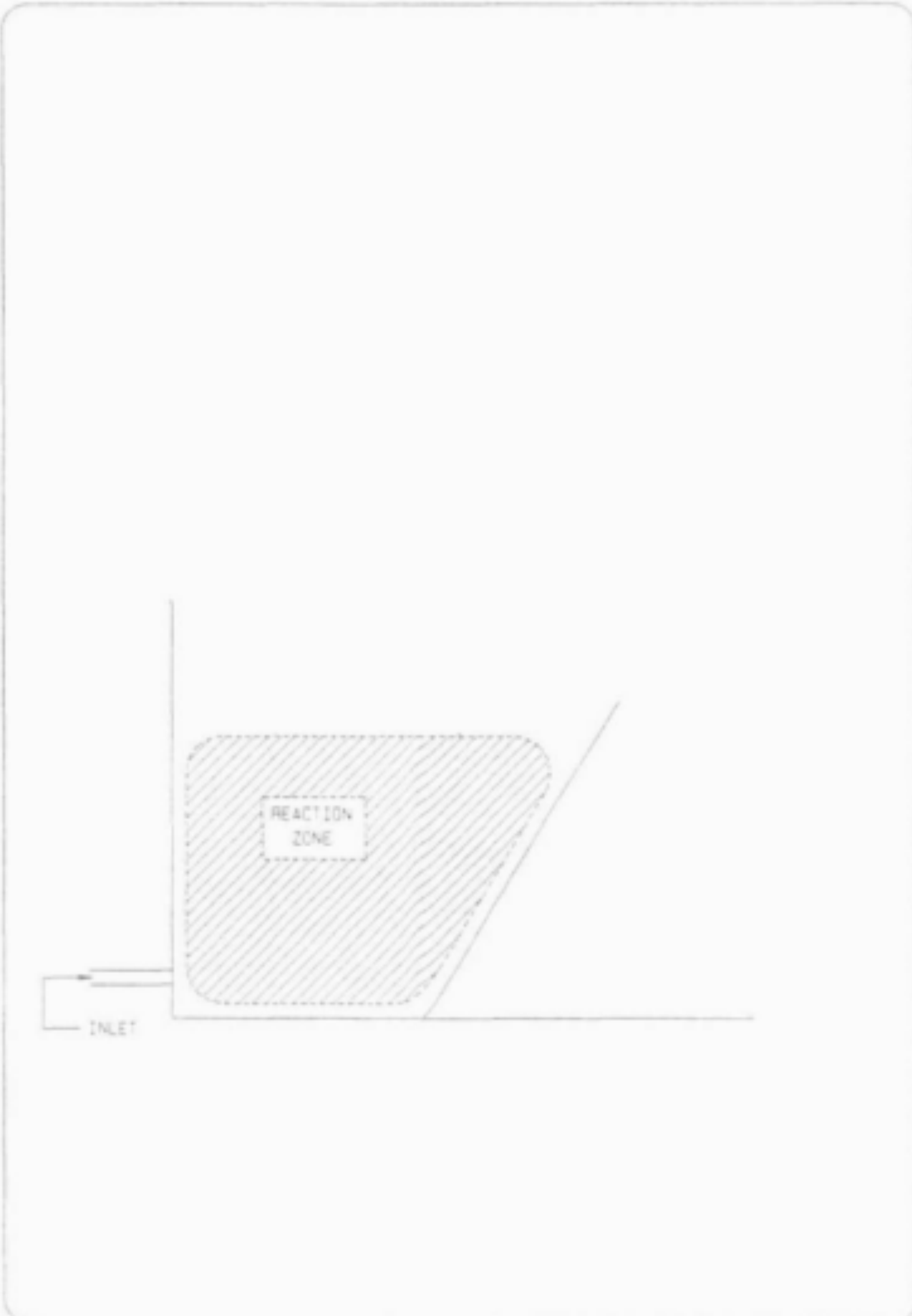
Performance

The float solids concentration varies between 1,0% and 1,5%. The solids concentration in the underflow varies from 75 mg.ℓ^{-1} to 2 000 mg.ℓ^{-1}, with an average of 120 mg.ℓ^{-1}. Maintenance requirements are very low, with scraper chain snaps about once per year the most serious. Too little available air had been sporadically reported.

PLANT NUMBER			# 31	
*** Tank geometry ***				
Shape (Round/Rect)			Rect	
Length	L	m	11.0	
Width	W	m	3.2	
Outer diameter	D_o		n/a	
Inner diameter	D		n/a	
Depth	D_t	m	1.5	
Number flotation cells			1	
Flotation area	A_f	m^2	19.2	
Reactor area	A_r	m^2	9.6	
Reactor volume	V_r	m^3	14.4	
Crossflow area	A_c	m^2	0.8	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	25	41
Recycle rate/cell	q	$k\ell.h^{-1}$	5	8
Saturator pressure	P	kPa		450
Concentration in	C	SS		3000
Concentration out	C_e	SS		120
*** Saturation ***				
Position (H/V)			V	
Diameter	D_s	m	0.80	
Length	L_s	m	1.6	
Packing (Y/N)			N	
Recirculation (Y/N)			N	
Depth packing	D_p		n/a	
Number of saturators			1	
*** Float removal ***				
Grid (Y/N)			N	
Scraper (Y/N)			Y	
Spacing scrapers		m	?	
Speed scrapers		$m.h^{-1}$	?	
Continuous scraping (Y/N)			Y	
Bottom desludging (Y/N)			N	

*** Notes ***


BEACH
PLATE
SLUDGE OUTLET
THROTTLING
VALVE
RECYCLE
FEED
PLAN :
ROTATING SCRAPER
FLOAT LAYER
RECYCLE
INLET
SECTION :
PLANT 31
FLOTATION TANK



PLANT 31 | AIR INJECTION

DESCRIPTION OF FLOTATION PLANT #32

Objective

Flotation is used to treat effluent from a paper factory. Waste fibres and pulp are reclaimed advantageously and the underflow is partly returned to complement the process water intake.

Feed water

The concentration of suspended solids is subject to wide variations (from 800 mg.ℓ^{-1} to 14 000 mg.ℓ^{-1}) with an average of about 2 800 mg.ℓ^{-1}. The pH varies between pH 4,5 and pH 11,0 with an average of pH 5,2. The presence of organic colouring matter also shows extreme short-term fluctuations. Aluminium salts are present on account of the manufacturing process. A stirred flow equalization tank precedes flotation.

Chemical conditioning

The feed is treated with 4 to 6 mg.ℓ^{-1} polymer through an in-line mixer. Lime is automatically dosed to that portion of the effluent which is wasted to maintain a constant pH, in order to meet municipal specifications.

Flotation tank

The flotation tank is rectangular with a flat bottom, except for 2 hoppers near the inlet for the capture of heavy solids. The recycle is blended with the feed outside the flotation tank and injected through 7 short pipe sections into the bottom of the flotation tank. A chain-driven scraper with 32 blades is operated continuously. The sludge concentration is about 4%.

Recirculation system

A single vertical unpacked saturator is operated at moderately low pressure of 200 to 300 kPa. It is fitted with 2 internal impellers which are shaft driven from the top and with a sparge plate above the air inlet at the bottom.

Performance

Excellent performance has been recorded over more than 15 years of operation. The performance is probably enhanced by the inherent buoyancy of the solids. An underflow solids concentration of 1 mg.ℓ^{-1} to 400 mg.ℓ^{-1}, average 8 mg.ℓ^{-1}, substantiates this claim. Variable quality and blockages are problematic at times. During recent years a tendency towards increased solids in the feed with subsequent higher polymer requirements were evident. This is ascribed to increased leakages and ageing of equipment in the manufacturing process.

PLANT NUMBER # 32

*** Tank geometry ***

Shape (Round/Rect)			Rect	
Length	L	m	15.0	
Width	W	m	4.0	
Outer diameter	D_o		n/a	
Inner diameter	D_i		n/a	
Depth	D_f	m	2.0	
Number flotation cells			1	
Flotation area	A_f	m^2	60.0	
Reactor area	A_r	m^2	4.0	
Reactor volume	V_r	m^3	13.6	
Crossflow area	A_c	m^2	2.4	
*** Variable ranges ***			Design	Average
Feed rate/cell	Q	$k\ell.h^{-1}$	180	240
Recycle rate/cell	q	$k\ell.h^{-1}$	137	206
Saturator pressure	P	kPa	270	290
Concentration in	C_i	SS		2800
Concentration out	C_o	SS		80

*** Saturation ***

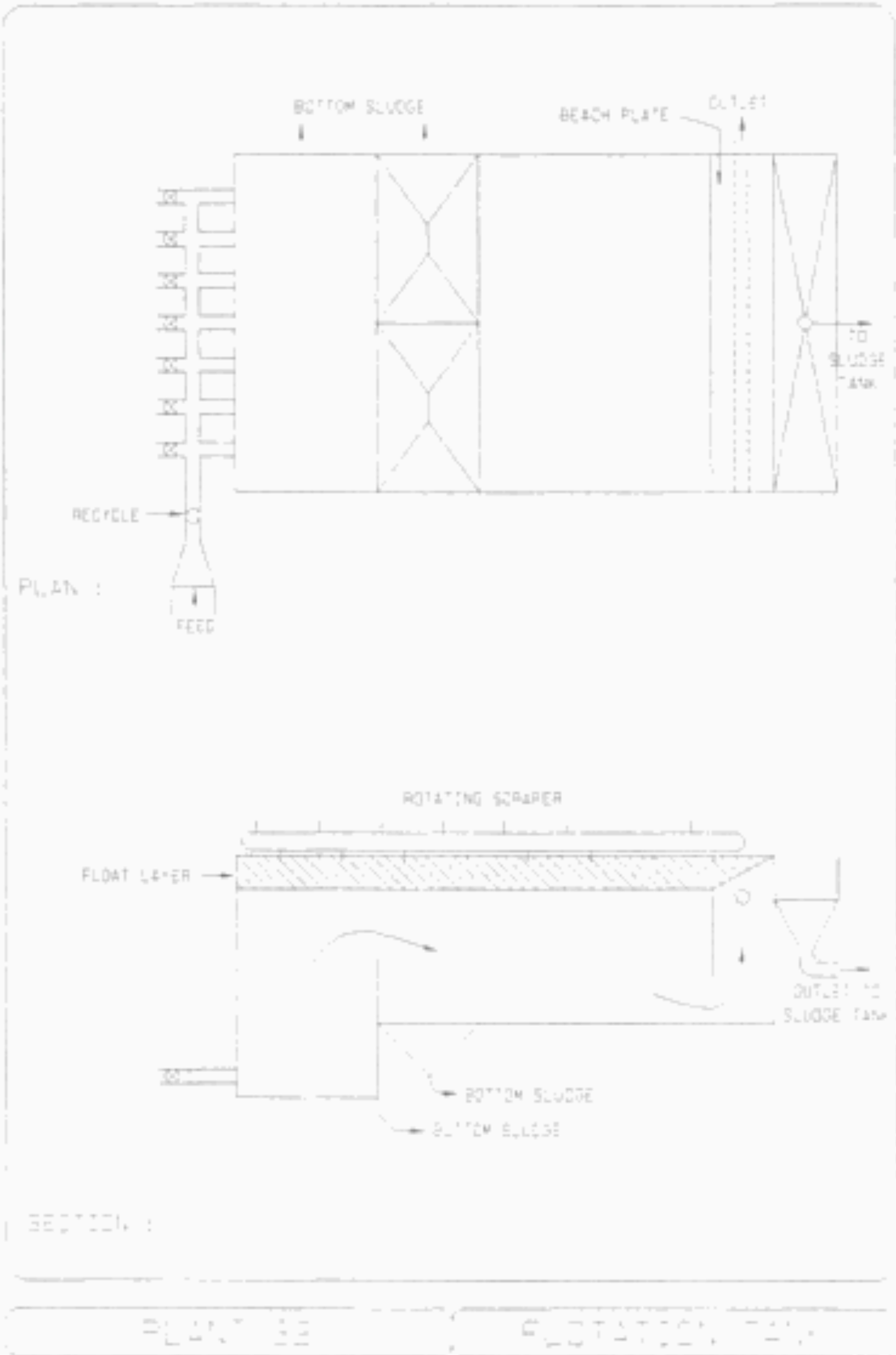
Position (H/V)			V
Diameter	D_s	m	1.90
Length	L_s	m	4.3
Packing (Y/N)			N
Recirculation (Y/N)			N
Depth packing	D_p		n/a
Number of saturators			1

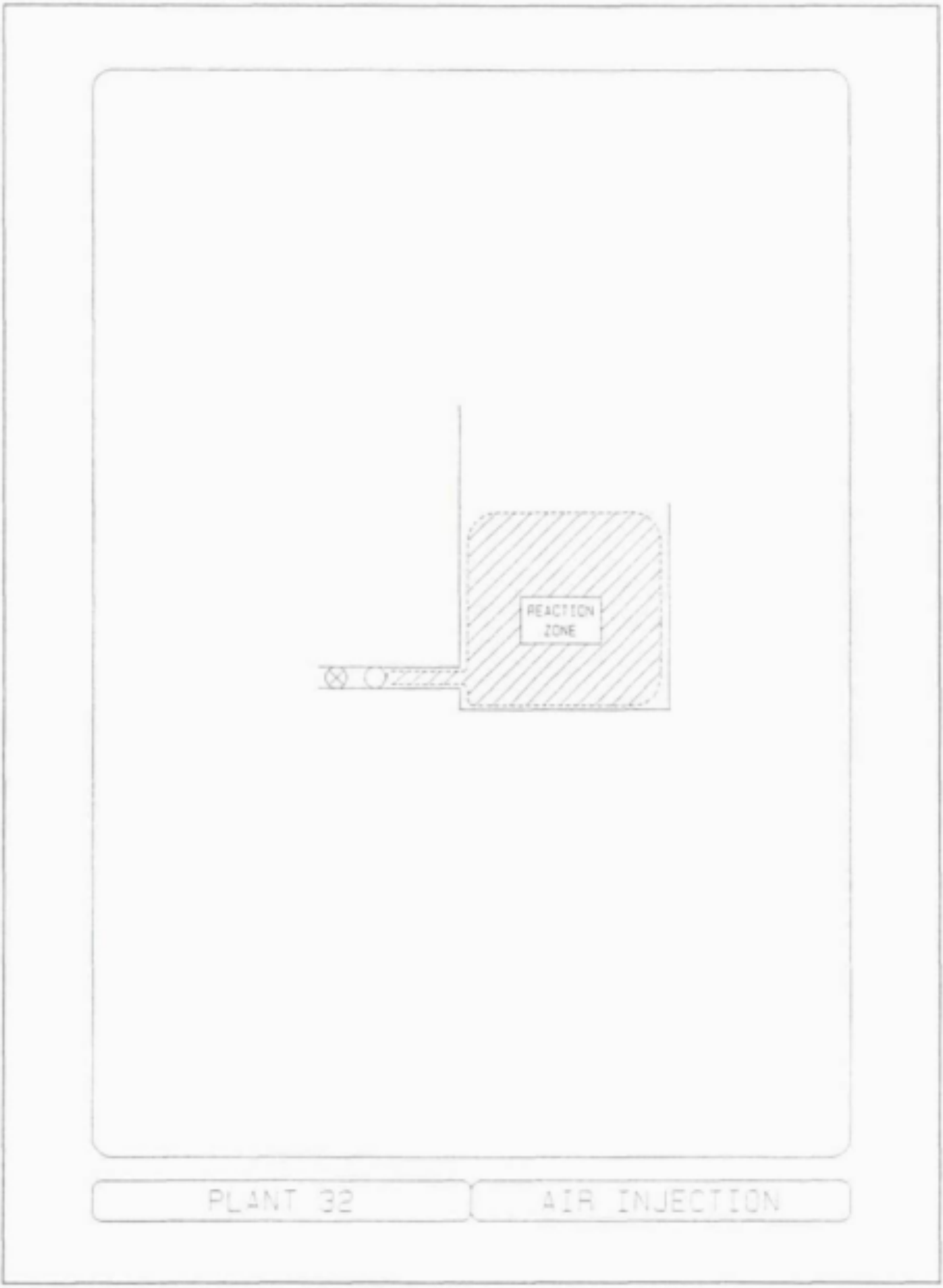
*** Float removal ***

Grid (Y/N)		N
Scraper (Y/N)		Y
Spacing scrapers	m	0.6
Speed scrapers	$m.h^{-1}$	72
Continuous scraping (Y/N)		Y
Bottom desludging (Y/N)		Y

*** Notes ***

Bottom desludging at weekly intervals.


BOTTOM SLUDGE
BEACH PLATE
OUTLET
TO SLUDGE TANK
RECYCLE
PLAN :
FEED
ROTATING SCRAPER
FLOAT LAYER
OUTLET TO SLUDGE TANK
BOTTOM SLUDGE
BOTTOM SLUDGE
SECTION :
FLOTATION TANK


REACTION
ZONE
PLANT 32
AIR INJECTION

APPENDIX C

BIBLIOGRAPHY OF SOUTH AFRICAN PUBLICATIONS ON DISSOLVED AIR FLOTATION

ANON (1983) Eimco equipment at Athlone sewage works. *Water Sewage and Effluent*, **13** 52.

ANON (1984) Effluent management : UF System demonstrated. *SA Waterbulletin*, August 1984 4-5.

BERNSTEIN, M, VAN VUUREN, LRJ & BOTES, W (1985) Pilot and full-scale applications of DAF technology to treat eutrophic waters from Lake Nsese. In : *Conference Institute of Water Pollution Control*, 1985, Parhurst : IWPC, 1-9.

BINNIE & PARTNERS (1987) Guide to Water and Waste-water Management in the Fruit and Vegetable Processing Industry. WRC Project no. 96, TT 30/87. PRETORIA.

BINNIE & PARTNERS (1987) Research on and an investigation into the use of physical/chemical techniques of water and waste-water management in the meat processing industry. WRC Project no. 127.

BOTES, V & VAN VUUREN, LRJ (1990) Dissolved air flotation for the removal of algae and inorganic turbidity on large scale. *Water Supply*, **8** 133-139.

BRABYN, NK, SMITH, EJ & DAVIES, CJ, CINDOFF, AL, GROVES, GR & OFFRINGA, G (1985) Technical performance evaluation of treatment processes for the recycling of paper machine wastewater. In : *Conference Institute of Water Pollution Control*, 1985, Pretoria : CSIR, 1-24.

BRATBY, JR (1976) Dissolved air flotation in water and wastewater treatment. M.Sc. Thesis, University of Cape Town.

BRATBY, JR (1978) Aspects of sludge thickening by dissolved-air flotation. *Water Pollution Control*, *77(3)* 421-432.

BRATBY, JR (1982) Treatment of raw wastewater overflows by dissolved-air flotation. *Journal of the Water Pollution Control Federation*, **54***(12) 1558-1565.*

BRATBY, JR & MARAIS, GvR (1973) Dissolved air flotation. *Filtration and Separation*, Nov/Des 1974 614-624.

BRATBY, JR & MARAIS, GvR (1975a) Dissolved air (pressure) flotation - an evaluation of inter-relationships between process variables and their optimisation for design. *Water SA*, **1**(2) 57-69.

BRATBY, JR & MARAIS, GvR (1975b) Saturator performance in dissolved-air (pressure) flotation. *Water Research*, **9** 929-936.

BRATBY, JR & MARAIS, GvR (1976a) A guide for the design of dissolved-air (pressure) flotation systems for activated sludge processes. *Water SA*, **2**(2) 87-100.

BRATBY, JR & MARAIS, GvR (1976b) Thickening of brown water sludges by dissolved-air (pressure) flotation. University of Cape Town, Dept of Civil Engineering, Research Report No. W 16.

BRATBY, JR & MARAIS, GvR (1976) Dissolved air flotation in water and wastewater treatment. University of Cape Town. Department of Civil Engineering, Research Report No. W 17.

BRATBY, JR & MARAIS, GvR (1976) Dissolved-air flotation in activated sludge. In : *Conference International Association Water Pollution Research*, 1976, Sydney : IAWPR. 311-322

CILLIE, GG, VAN VUUREN, LRJ, STANDER, GJ & KOLBE, FF (1966) The reclamation of sewage effluents for domestic use. In : *3rd International Conference on Water Pollution Research*, 1966, Munich, **2** 1-19.

COERTZE, JA (1978) Dissolved air flotation treatment of paper mill effluent. *Progress in Water Technology*, *10(1,2)* 449-457.

COOPER, DR, SHUTTLEWORTH, SG & SLABBERT, NP (1983) The treatment of wastewater from the leather industry. In : *Conference Institute of Water Pollution Control, SA Branch. Proceedings*. 1983, Thohoyandou : IWPC. 12/01-12/09.

DAVIS, C (1977) Dissolved air flotation and belt filter pressing of activated sludge in Johannesburg. In : *Symposium on the Treatment and Disposal of Sludges Deriving from Municipal Sewage*, 1977, Pretoria : IWPC. 1-9.

DE WET, FJ (1980) Flotation of algal waters. *Paper Presented at Meeting of the Institute of Water Pollution Control*, 1986, SA, Pretoria, 1-28.

DE WET, FJ & VAN VUUREN, LRJ (1980) Investigation into the reclamation of sewage effluent for reuse at the Kloof mine. Pretoria : CSIR, NIWR, 1-42.

DE WET, FJ & VAN VUUREN, LRJ (1982) Investigation into the quality improvement of reclaimed sewage effluent by dissolved air flotation-filtration (DAFF) at SAPPI Adamas. Pretoria : CSIR, NIWR, 1-31 (Contract report)

EVERETT, P (1983) Rooiwal sewage purification works - dissolved air flotation system. *Water Sewage and Effluent*, **13** 47, 49

GEHR, R, SWARTZ, CD & OFFRINGA, G (1991) Dissolved air flotation for the pretreatment of eutrophied surface water for potable use. WRC. Final Report No 202/1/91.

GRABOW, WOK, GRABOW, NA & BURGER, JS (1969) The bactericidal effect of lime flocculation/ flotation as a primary unit process in a multiple system for the advanced purification of sewage works effluent. *Water Research*, **3** 943-953.

GROVES, GR (1977) Preliminary investigation into the treatment of wool scouring effluents by flotation for grease removal. *Water Management and Effluent Treatment in the Textile Industry*, Report WS2. Water Research Commission Group.

HAARHOFF, J & FOUCHE, L (1989) Evaluation of combined flotation/filtration of Rietvlei water treatment plant. In : *WISA 1st Biennial Conference and Exhibition*, 1989, Cape Town, 1-13.

HEBDEN, D (1981) The use of dissolved air flotation in the pretreatment of seawater for reverse osmosis. M.Sc. Thesis, Potchefstroom University for CHE.

HEBDEN, D & BOTHA, GR (1978) A pretreatment method for seawater desalination by reverse osmosis. In : *6th International Symposium on Fresh Water from the Sea*, 1978, Las Palmas, **4** 209-214

HOWELL, HO (1982) Johannesburg municipality's northern sewage disposal works operates the world's largest dissolved air flotation plant for activated sludge thickening. *IMIESA*, **7**(6) 53, 55.

JACOBS, HJ & SMITH, L (1973) Advanced treatment of purified sewage for the production of high-brightness pulp and paper. *Tappi*, **56** (12) 170-174.

KEGEL, T (1987) Sewage effluent treatment to supplement mine service water supply. *Mine Service Water Supply*, 1-22

LANGENEGGER, O (1984) Separation of suspended solids by dissolved air flotation. In : *The South African Chapter of the International Filtration Society*, 1-18.

LANGENEGGER, O (1985) New developments in flotation for drinking water. In : *Symposium on Recent Developments in Drinking Water Purification*, 1985, Pretoria, 1-20

LANGENEGGER, O (1976) Flottasie van geaktiveerde slykmengsels. Finale vorderingsverslag aan die WNNR.

LANGENEGGER, O & VIVIERS, JMP (1978) Thickening of waste activated sludge with a dissolved air flotation unit. *Water Pollution Control*, **77**(1) 79-86.

LEACH, GW, SOUTAR, BN, PULLEN, AW & VAN VUUREN, LRJ (1985) Sewage effluent reclamation for potable industrial mine service water. (NIWR Reprint RW 096), 1-43.

OFFRINGA, G (1986) Dissolved air flotation treatment of final effluent from three South African board mills. *Paper Southern Africa*, **6**(1) 16-23.

OFFRINGA, G & COOPMANS, EJA (1986) Phosphate removal from Johannesburg northern sewage works secondary effluent by dissolved air flotation. *Municipal Engineer*, May 1986 19-25.

OFFRINGA, G & SCHEEPERS, JA (1985) Removal of algae by dissolved air flotation at Swartwater dam, Qwaqwa Pretoria : CSIR, NIWR, 1-18.

OFFRINGA, G & VAN VUUREN, LRJ (1987) Treatment of eutrophic waters. Paper Presented at Symposium Hartbeespoortdam 'Quo vadis', 1987, Pretoria : CSIR, 1-9

OFFRINGA, G, VENNEKENS, MJA & DE VILLIERS, HA & BOTHA, A (1987) *Dissolved Air Flotation Studies in the Pulp and Paper Industry*. 1-11.

ROETS, SD & HEUNIS, EA (1984) The application of an induced air flotation process in the treatment of proteinaceous waste. *Water Sewage and Effluent*, **19** 7, 9-11

ROSS, WR, MOCKE, FAJ & ALBERTS, PS (1977) Flotation of spent wine anaerobic sludges. *Progress Report for presentation to the 2nd Steering Committee Meeting for the Technological Application of the Anaerobic Digestion*, 1977, Belville, 17-19.

RYKAART, EM (1991) Die modellering van lugversadigers vir opgelostelugflottasie. Undergraduate thesis presented to the Rand Afrikaans University.

SANDBANK, E (1983) Algae Harvesting. In : *Proceedings of a Conference of the Institute of Water Pollution Control*, 1983. Thohoyandou : IWPC, 1-16.

SANDBANK, E & VAN VUUREN, LRJ (1983) Algae harvesting. *Paper presented by the Environmental Engineering and Water Resources Centre*, 1983, Technion Haifa. 1-10.

SCOTT, PD & SQUIRES, RC (1983) Protein recovery from abattoir effluent. *Chemsa*, **9**(6), 100-103

STANDER, GJ & VAN VUUREN, LRJ (1969) Flotation of sewage and waste solids. *Paper presented at the 3rd Water Resources Symposium*, 1969, Austin : University of Texas, 190-199.

SWART, JS DE K & TOLMAY, AT (1973) Air flotation of humus tank effluent - a systematic approach. University of Pretoria, Department of Chemical Engineering. Research Report No. W103.

VAN VUUREN, LRJ (1969) Die ontwikkeling en toepassing van flottasie as 'n watersuiweringsproseseenheid. Verhandeling voorgelê vir die graad M.Sc. (Ind. Chem) aan die Potchefstroomse Universiteit vir Christelike Hoër Onderwys

VAN VUUREN, LRJ (1985) Water reuse technology. *Seminar on Technology Transfer in Water Supply and Sanitation in Developing Areas*, 1985, King Williams Town, **2** 1-11.

VAN VUUREN, LRJ, CLAYTON, AJ & VAN DER POST, DC (1980) Current status of water reclamation at Windhoek. *Journal Water Pollution Control Federation*, **52** 661-671

VAN VUUREN, LRJ & DE WET, FJ (1981) The application of the flotation/filtration process to water and effluent treatment. In : *Symposium on Industrial Effluent - Control and Treatment*, 1981. Pretoria, 1-20

VAN VUUREN, LRJ, DE WET, FJ & CILLIE, GG (1983) Treatment of water from eutrophied impoundments. *Water Supply*, *1(1)* 145-156

VAN VUUREN, LRJ, DE WET, FJ & PRINSLOO, J (1982) Preliminary report on pilot plant runs at Empangeni waterworks. Pretoria : CSIR, NIWR, 1-36.

VAN VUUREN, LRJ, FUNKE, JW & SMITH, L (1972) The full-scale refinement of purified sewage for unrestricted industrial use in the manufacture of fully-bleached craft-pulp and fine paper. In : *6th International Conference on Water Pollution Research*, 1972, Jerusalem, 18-24

VAN VUUREN, LRJ, HENZEN, MR, STANDER, GJ & CLAYTON, AJ (1970) The full-scale reclamation of purified sewage effluent for the augmentation of the domestic supplies of the city of Windhoek. In : *5th International Conference on Water Pollution Research*. 1970, San Francisco, 1-32/1.

VAN VUUREN, LRJ, MEIRING, PGJ & HENZEN, MR (1965) The flotation of algae in water reclamation *Int J Air Wat Poll*, **9** 823-832.

VAN VUUREN, LRJ & OFFRINGA, G (1983) Effluent treatment and water reuse in the pulp and paper industry *Paper to be presented at a Seminar on Water Conservation*, 1983. Pretoria. 1-16.

VAN VUUREN, LRJ & OFFRINGA, G (1985) Production of potable supplies from algae-laden waters. In : *Proceedings of the Symposium on the impact of phosphate on South African waters*, 1985, Pretoria CSIR, 1-17.

VAN VUUREN, LRJ & OFFRINGA, G (1989) History of dissolved air flotation in Southern Africa. *Paper presented at a short course on algae in water - problems and treatment*, 1989, Pretoria, 1-18.

VAN VUUREN, LRJ, OFFRINGA, G & WILLIAMS, PG (1985) Development and application of DAF technology in South Africa. In : *The Institute of Water Pollution Control Year Book and Buyers Guide, 1985-1986*, 75-89.

VAN VUUREN, LRJ, PRINSLOO, J & DE WET, FJ (1983) Pilot-scale studies for the treatment of eutrophied water by dissolved air flotation filtration. In : *10th Federal Convention of the Australian Water and Wastewater Association*, 1983, Sydney : AWWA, 14/1-14/10

VAN VUUREN, LRJ & ROSS, WR (1975) Integrated physical chemical and biological treatment systems. *Paper presented at Symposium on Water Research : Selected in-depth Studies (Part II)*, 1975, Pretoria.

VAN VUUREN, LRJ, STANDER, GJ, HENZEN, MR, MEIRING, PGJ & VAN BLERK, SHV (1967) Advanced purification of sewage works effluent using a combined system of lime softening and flotation. *Water Research*, **1**(7) 463-474.

VAN VUUREN, LRJ, STANDER, GJ & HENZEN, MR (1968) Dispersed air flocculation/flotation for stripping of organic pollutants from effluents. *Water Research*, **2** 177-183

VAN VUUREN, LRJ & VAN DER MERWE, PJ (1989) Thickening of sludges by dissolved air flotation. *Water Science Technology*, **21** 1771-1774.

VAN VUUREN, LRJ & VAN DUUREN, FA (1965) Removal of algae from wastewater maturation pond effluent. *Water Pollution Control Federation Journal*, **39**(9) 1256-1262

VIVIERS, JMP & VAN VUUREN, LRJ (1980) Harvesting and processing of algae *Paper presented at a Workshop on Biological Production System and Waste Treatment*, 1980 Bloemfontein, 49-51.

VOSLOO, PBB & LANGENEGGER, O (1979) Possible methods for dealing with phytoplankton at water purification works. *Paper Presented at Monthly Meeting of the Institute of Water Pollution Control*, 1979, SA - IWPC, 1-14.

VOSLOO, PBB, WILLIAMS, PG & RADEMAN, RG (1985) Pilot and full-scale investigations on the use of combined dissolved air flotation and filtration (DAFF) for water treatment. *Biennial Conference and Exhibition of the Institute of Water Pollution Control*, 1985, SA Branch, Durban : IWPC, 1 1-12.

WATER RESEARCH COMMISSION (WRC) (1984) Theory, design and operation of nutrient removal activated sludge process. Water Research Commission of South Africa, P O Box 824, 0001 Pretoria.

WILLIAMS, PG (1983) DAFF pilot plant runs at Mzingazi water purification plant Richards Bay. Pretoria : CSIR, NIWR, 1-8.

WILLIAMS, PG & VAN VUUREN, LRJ (1984) Windhoek water reclamation plant process design for dissolved air flotation. Pretoria : CSIR, NIWR, 1-20.

WILLIAMS, PG, VAN VUUREN, LRJ & VAN DER MERWE, PJ (1985) Dissolved air flotation upgrades a conventional plant treating eutrophic water. In : *Proceedings of the 11th International Convention of the Australian Water and Wastewater Association*, 1985, Melbourne : AWWA, 189-196