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# INCORPORATING A RULE-BASED MODEL OF JUDGEMENT INTO A WASTEWATER TREATMENT PLANT DESIGN OPTIMIZATION MODEL

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## ABSTRACT

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The use of a rule-based modeling technique for the formal consideration of poorly modeled issues in a water quality management problem is illustrated in the context of wastewater treatment plant design. Sludge bulking is a poorly understood problem in activated sludge wastewater treatment plants. An engineer must use judgement gained from experience when he designs an activated sludge plant to prevent bulking from causing the plant to fail. An attempt was made to use fuzzy logic in order to model that judgement. Results from research were taken from the literature and used independently as constraints to an activated sludge wastewater plant design optimization model to see their effect on the optimal design. Some of the research results were then formulated as rules in a rule-based system which relates design variable values to the likelihood of a design experiencing bulking problems. The weights of association of those rules to the conclusion that a given design would experience bulking problems and the logical interaction of those rules were calibrated using an experienced engineer's evaluation of a set of 15 plant designs. The consistency of the engineer's and the judgement model's evaluations were then checked with a second set of 15 designs. The model of judgement could be used to evaluate the bulking potential of any design. In the particular example developed, the judgement model was incorporated into a wastewater treatment plant design optimization model so that the cost-effectiveness of constraint combinations could be examined. The tradeoff between cost and the likelihood of experiencing bulking problems was examined for a typical plant design problem.

Keywords: Wastewater Treatment, Mathematical Models, Optimization, Expert Systems

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# CHAPTER 1

## INTRODUCTION

A rule-based modeling technique is presented to illustrate the use of such an approach for a water-quality management problem. The illustration is a model developed to capture the judgement of an experienced engineer in evaluating the potential for sludge bulking in various designs of an activated sludge system. In the particular example developed, it is shown that the judgement model can be used to evaluate the bulking potential of any design or can be incorporated into an optimization model to determine the added cost associated with reducing the likelihood of bulking.

The activated sludge wastewater treatment process is characterized as suspended growth biological treatment. Although there are many variations to the process, the main feature is the existence of a tank where a high concentration of active biomass is mixed with wastewater and the substrate in the wastewater is consumed by the biomass in the presence of oxygen. As the microorganisms feed on the wastewater, they grow and are subsequently settled in the final clarifier. A portion of the sludge that is removed is returned to the aeration tank in order to maintain the high biomass concentration.

A typical treatment plant contains not only the activated sludge biological treatment process, but also may contain primary sedimentation and sludge handling and disposal facilities. Such a process train is shown in Figure 1. The design of such a plant consists of sizing the various treatment units so as to meet the plant effluent requirements. The sizing of each process is determined, however, by the various recycle and supernatant flows which it might receive, and by the efficiency of the preceding unit processes. Finding a feasible, cost-effective design is most definitely a challenge.

Because the design of a plant is difficult, and because a poorly designed plant could violate water quality standards, many state regulatory agencies set forth design standards for treatment works. However, even within the constraints set by the states, there are many possible design combinations. Tang, et al.[1] developed a model of the activated sludge process which considers the

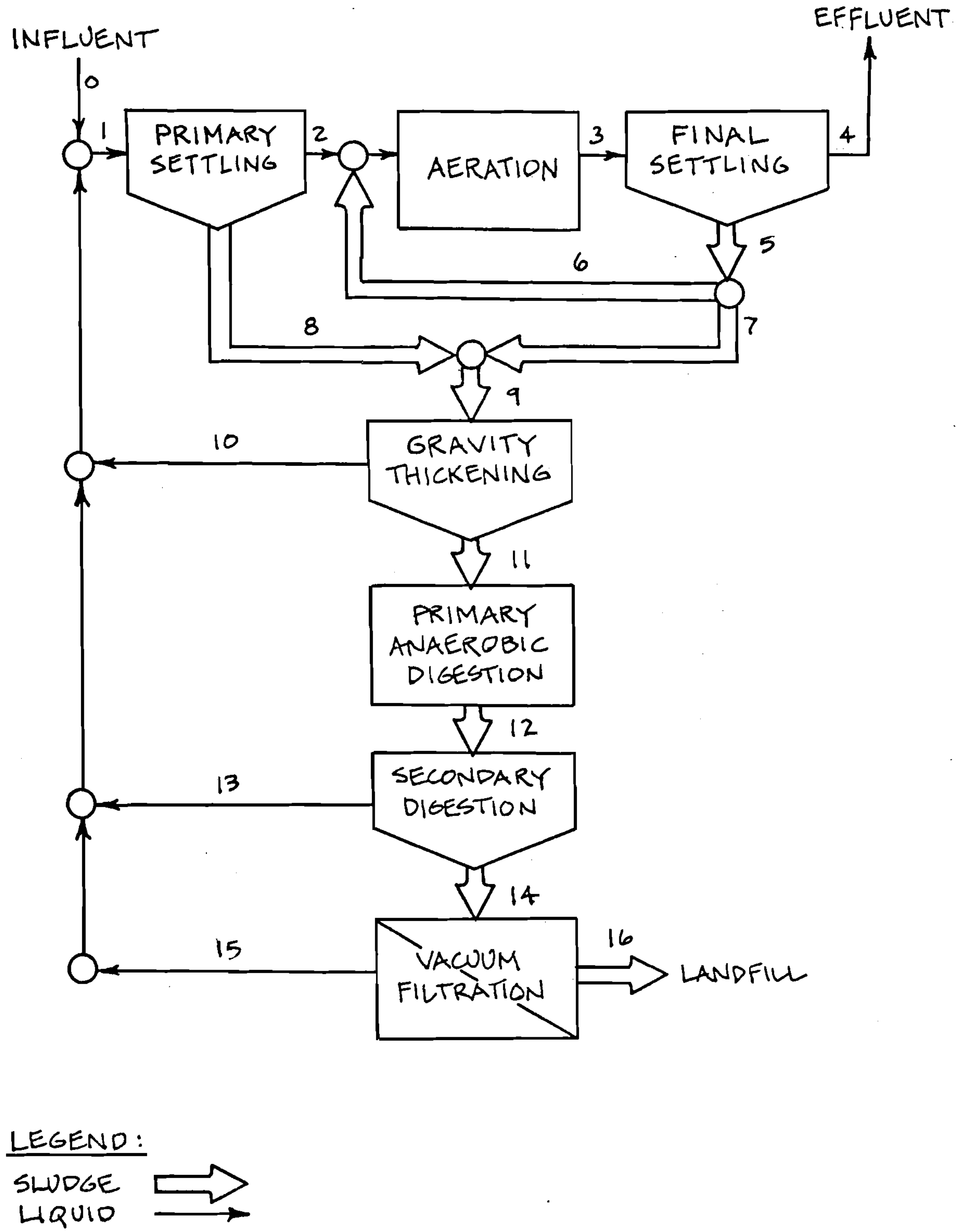



Figure 1--Activated Sludge Process Train

interaction of the various unit processes found in a conventional plant. That model allows one to find a system design which is cost-optimal subject to the constraints which are imposed on the solution.

While an optimization model is a powerful tool for the engineer, it does not relieve him of the challenge of finding a good, cost-efficient design. Many of the relationships which Tang used to describe the functions of the unit processes are empirical and were developed for limited conditions. For example, the Chapman[2] model is used to estimate the effluent solids concentration of the final clarifier. Chapman based his model on studies which he performed on a pilot-scale clarifier at a full-scale treatment plant. He kept close control over the sludge recycle rate so that the sludge coming from the aeration basin would be of a consistently good quality. Because the primary focus of his work was the physical nature of clarification, he did not include the sludge characteristics as variables in his model. He recognized the omission and felt that "...factors which influence the settling and clarification properties of the floc must also be considered in designing and operating plants."

In addition, there may be unmodeled issues which the designer might consider important. For instance, objectives such as minimizing the sensitivity of the microbial population to changes in the influent conditions are not considered in Tang's model and are ignored when the designer seeks only to minimize cost.

This paper presents a modeling technique by which the judgement of an experienced engineer relative to these unmodeled issues can be formally considered. To illustrate this technique, a problem common to activated sludge plants, sludge bulking, was selected. This periodic loss of solids over the final clarifier effluent weir is a problem that the design engineer would want to avoid. Tang's design optimization model does not consider the problem of sludge bulking and the cost-optimal solutions may be such that they are likely to develop sludge bulking problems. This technique allows for the evaluation of a plant design with respect to its potential for developing bulking problems. In this particular case, it was determined that the judgement model could be incorporated into an optimization model so that solutions may be found which are good with respect to both cost and the likelihood of a bulking problem occuring.

A rule-based inference system was constructed in a first attempt to model the judgement which an experienced engineer might use in evaluating a given plant design for its potential for developing bulking problems. Such a judgement might be inferred from the values of several different design parameters which have been associated with bulking problems. The associations which have been reported in the literature are initially reviewed to identify some general trends between variable values and bulking problems, and some proposed variable boundaries. The effect of constraining the design with such boundaries are then investigated using Tang's model. Next, some of those trends and boundaries are used in a rule-based inference model which determines the overall likelihood of bulking for a given design. That model is calibrated to an experienced engineer's evaluation of a set of plant designs. The consistencies of both the engineer and the model are then checked with the engineer's evaluation of a second set of designs. Finally, the inference model is incorporated into Tang's optimization model to identify the tradeoff between cost and the likelihood of developing bulking problems.

# CHAPTER 2

## ACTIVATED SLUDGE BULKING

Activated sludge bulking is a common problem in activated sludge wastewater treatment plants. Tomlinson[3] reports on a 1976 study of plants in the U.K. where 52% had experienced excessive loss of solids into their effluent. While there are many problems associated with activated sludge seperation in the final settling tank, a bulking sludge is considered to settle and compact poorly. When the activated sludge settles poorly, it may become difficult to maintain a high concentration of biomass in the aeration tank which could lead to a break-down in the operation of the plant. Activated sludge contains a diverse population of microorganisms and its properties are controlled by the relative numbers of the various species present. The conditions in the plant and the makeup of the wastewater influent seem to cause the relative numbers of microorganisms to change.

There has been no good model developed which predicts the settleability of a sludge given the conditions of the plant and which works for a wide variety of plants. Experience has shown that bulking has a wide variety of possible causes. Table 1 summarizes some of them, and divides them into those which Tang's model considers, and those which it does not.

Table 1
Factors Related to Sludge Bulking

| Considered in Design Model | Not Considered |
|---|---|
| mixing characteristics | pH |
| feed pattern | waste type |
| D.O. concentration | micro-nutrients |
| Sludge Loading | fats, starch, carbohydrates in influent |
| primary sedimentation | septic sewage |

When designing a wastewater treatment plant, the designer would like to minimize the chance of encountering bulking problems. While the sewage make-up may indicate the potential for a bulking problem, it normally cannot be altered beyond pH and micro-nutrient adjustment. Rather, the plant must be designed to avoid bulking problems. The following sections examine the plant design variables which the designer should evaluate in considering the potential for bulking problems.

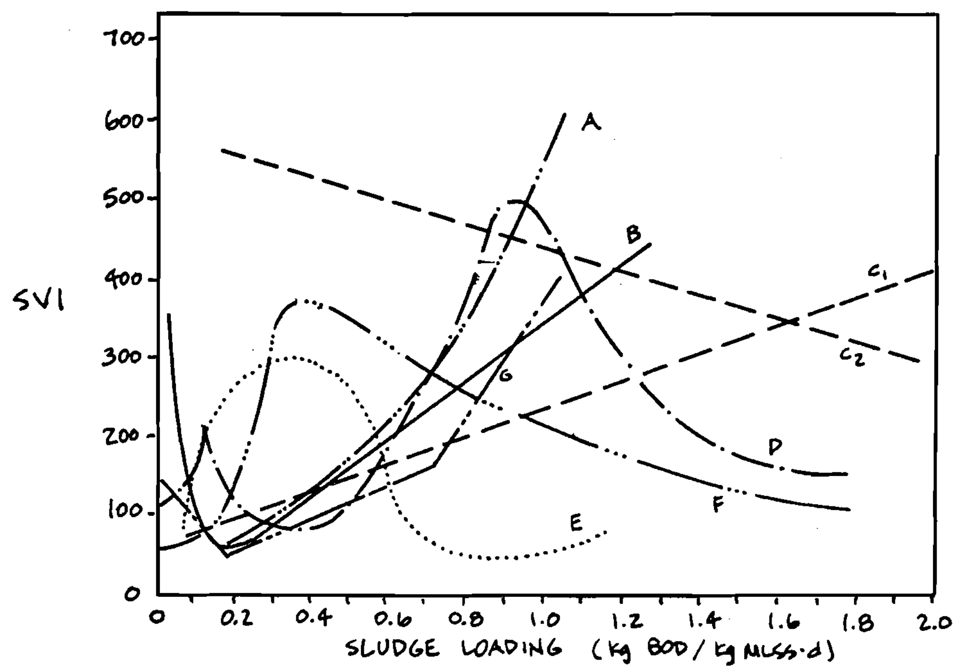
## 2.1 SLUDGE LOADING

### 2.1.1 Background

Sludge loading is a measure of the food to micro-organism ratio (F/M) in the aeration tank. Pipes[4] related the fat sludge hypothesis as an explanation of how sludge loading is important. Microorganisms which live in a high F/M environment are like pigs which are fed too much corn; they become fat and lazy and move slowly whereas organisms living under starved conditions are spartan and settle well. While a correlation between sludge loading and settleability has been made, the numerous investigators have found sometimes conflicting results. Figure 2 shows six published correlations between sludge loading and settleability.

Orford, et al.[5] studied the effect of sludge loading on the completely mixed activated sludge process. The results of their laboratory experiments showed a maximum sludge density index (minimum sludge volume index) at a sludge loading of 0.17 lb $BOD_5$/lb MLVSS•d (pounds of 5 day biochemical oxygen demand per pound of mixed liquor volatile suspended solids per day). At loadings above that they found a nearly linear decrease in the sludge density index. Manipulation of their results yields a predicted sludge volume index (SVI) of 108 at a sludge loading of 0.3 lb $BOD_5$/lb MLVSS•day and an SVI of 150 (an approximate boundary for bulking sludge) at a loading of 0.42 lb $BOD_5$/lb MLVSS•day.

Stewart[6] presented a typical relationship between SVI and sludge loading. He noted that for conventional plants, loadings in the range of 0.5 to 2.0 lb $BOD_5$/ lb MLVSS•day were unstable and that normally attempts are made to maintain a loading factor of about 0.3 lb $BOD_5$/lb MLVSS•day. Ten years later, Rensink[7] found in pilot plant studies that completely mixed units resulted in a filamentous bulking of *Sphaerotilus natans* when loaded above 0.3 kg $BOD_5$/kg MLSS•day. Below 0.3, no bulking problems were noted. MLSS (mixed liquor suspended solids) includes the inert solids concentration with the volatile solids concentration as a measure of the microorganisms present. Also, Chudoba[8]




A. Kalbskopf, K.H.[13]
B. Goodman, B.L.[30]
C1. Chudoba et al.,[8]
C2. Chudoba et al.,[8]
D. Rideau, J.P., et al.,[30]
E. W.P.R.L.[30]
F. Stewart[6]
G. Rensink[7]

Most of this figure is from Chambers and Tomlinson[30]

Figure 2--Relationships Between Sludge Loading and SVI

had found in pilot plant studies that at loadings above 0.5 kg $BOD_5$/kg MLVSS• day, SVI values were high. Kiff[9] found that in a laboratory plant operated on settled sewage, SVI values were greater than 150 when the biomass loading was above about 0.35 kg/kg•day (measure of microorganisms not specified).

Metcalf and Eddy[10] suggested that a completely mixed activated sludge plant should have a sludge loading of between 0.2 and 0.6 kg BOD/kg MLVSS•day and a volumetric loading of between 0.8 and 2.0 kg BOD/$m^3$• day. Escritt[11], in a text on International Sewage treatment practice, reports that plants, with average aeration basin detention times, should not be loaded above 0.03 kg BOD/kg MLVSS•d. The Illinois Recommended Standards for Sewage Works[12] states that the organic loading density shall not exceed 35 lbs/day of $BOD_5$ per 1000 cubic feet of usable tank volume (0.56 kg $BOD_5$/$m^3$• day).

While most investigators found an increasing SVI with increased sludge loading, Chuboda[8] found that in completely mixed laboratory systems and with loadings in the range of 0.5 to 1.6 kg $BOD_5$/kg MLVSS•d, the SVI decreased with increased sludge loadings. The SVI was greater than 400 ml/g in all cases. His findings for plug-flow systems concurred with the other investigators.

Kalbskopf[13] reported of studies on bulking in Germany. In extended aeration plants, SVI values of less than 100 ml/g could only be maintained when the sludge loading rate was less than 0.05 kg $BOD_5$/kg MLSS•day. While the data are scattered, the results showed that it was necessary to load below 0.07 kg $BOD_5$/kg MLSS•day to remain below an SVI of 150. He also reported on a pilot plant fed with chemical and steel-producing industrial wastewater where it was necessary to maintain a loading of less than 0.2 kg $BOD_5$/kg MLSS•day to keep an SVI of less than 150 ml/g.

Palm, et al.[14] found that at high bulk dissolved oxygen concentrations in the aeration basin, high substrate removal rates are possible while maintaining acceptable SVI values. More on this is given in a later section on dissolved oxygen. He also proposed that at low loadings (below 0.2 kg COD/kg VSS•day, where COD refers to the chemical oxygen demand), problems with sludge bulking would be found. Such a case of low loading bulking was observed by Ganczarczyk[15] when he studied the operation of a full scale plant treating effluent from a paper pulp mill. He observed a minimum SVI at sludge loadings of from 0.3 to 0.5 g $BOD_5$/g·day, and a sharp increase in SVI on both sides of this range.

Wagner[16], in his studies of bulking in Germany, found a relationship between volumetric loading and the SVI. He found a maximum SVI occuring at sludge loadings between 0.4 and 0.7 kg $BOD_5/m^3$· day. He found that sludge loadings below 0.3 kg/$m^3$· day gave the best SVI values and that loading above 1.0 kg /$m^3$· day gave moderately high, but fairly constant values.

Sludge loading and sludge age are inversely related variables that define the aeration basin loading. Bisgoni[17] studied the effect of sludge age on the settling characteristics of activated sludge in bench scale reactors with a synthetic feed. The SVI was at a maximum of 600 ml/g at a sludge age of about 3 days. Based on the total biomass in the effluent, the best overall solids removal occurred at sludge ages from 4 to 9 days. Effluent from reactors with a short sludge age contained solids with dispersed growth while effluent from reactors with a long sludge age contained pin-point floc and small deflocculated particles. Mulbarger et.al.[18] reported that the NorthEastern Ohio Regional Sewer District's Easterly plant experienced uncontrolled bulking when the sludge age was dropped below 1.5 to 1.2 days.

Eikelboom[19] used the concept of floc loading as a level of loading during the mixing stage. In contrast to the sludge loading rate which is averaged over the entire aeration tank, floc loading is an instantaneous value which decreases after mixing due to biosorption by the floc. He defined the floc loading (mg COD/g MLSS) as:

$$FL = \frac{(COD_I - COD_{RS})Q_I}{MLSS_{RS}\, Q_{RS}} \qquad (2.1)$$

$I$=influent

$RS$=return sludge.

### 2.1.2 Results from Optimization Model

The wastewater treatment plant design optimization model typically yields optimal solutions with sludge loadings higher than recommended values. Design 1 in Table 2 shows such an optimal design. Note that the sludge loading is higher and the volumetric loading is lower than recommended by Metcalf and Eddy (M&E). If those loadings are constrained to meet the M&E guidelines, design 2 is found to be optimal. However this design still has a sludge loading at which several investigators have found bulking. Further constraining the design to the limits of sludge loading suggested by various investigators gives the other designs in Table 2. Note that design 6, conforming to Wagner's volumetric loading constraint, also satisfies the sludge loading constraints based on F/M ratio.

The floc loading of the designs may be evaluated with a minor manipulation of Eikelboom's expression. Since substrate utilized in the activated sludge process is made up of degradable solids and soluble BOD, and assuming that the effluent soluble BOD is equal to the return sludge soluble BOD and that no degradable solids leave the aeration tank (as Tang originally assumed), the numerator represents the rate of the substrate COD utilized in the process. Since COD removal and ultimate BOD removal are approximately equivalent in the activated sludge process, the following expression holds true:

$$S(gBOD_5)\left[\frac{1.5gBOD_L}{1gBOD_5}\right]\left[\frac{1gCOD}{1gBOD_L}\right]=COD_I-COD_{RS} \qquad (2.2)$$

By the definition of the recycle rate,

$$Q_{RS}=rQ_I \qquad (2.3)$$

Substituting (2.2) and (2.3) into (2.1), floc loading may be determined for designs produced by Tang's model as:

$$FL=\frac{1.5S}{rM_{t_6}} \qquad (2.4)$$

$r$=recycle ratio

Table 2
Cost Optimal Designs
Subject to Various Loading Constraints

| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| Cost (\$/yr) | 500,398 | 509,170 | 544,830 | 526,230 | 546,410 | 546,730 |
| Eff. $BOD_5$ (mg/l) | 30.0* | 26.954 | 19.789 | 22.080 | 18.286 | 19.029 |
| Eff. TSS (mg/l) | 30.0* | 30.0* | 30.0* | 30.0* | 30.0* | 30.0* |
| ORPST (m/hr) | 6.0* | 6.0* | 6.0* | 6.0* | 6.0* | 6.0* |
| Recycle Rate (%) | 12.50 | 18.36 | 37.41 | 12.72 | 15.27 | 10.0* |
| Sludge Age (days) | 2.19 | 2.53 | 4.21 | 3.44 | 4.94 | 4.55 |
| Volume,A.T. (cu.m) | 5696 | 5397 | 6267 | 8967 | 11,582 | 13,371 |
| Area,F.S.T. (sq.m) | 684 | 755 | 993 | 687 | 717 | 653 |
| MLVSS (mg/l) | 1087 | 1333 | 1917 | 1099 | 1210 | 968 |
| Sludge Loading[a] | 0.66 | 0.60* | 0.42 | 0.42 | 0.30* | 0.31 |
| Sludge Loading[b] | 0.47 | 0.43 | 0.30* | 0.30* | 0.22 | 0.22 |
| Volumetric Loading[c] | 0.72 | 0.80* | 0.80* | 0.46 | 0.36 | 0.30* |
| Floc Loading[d] | 130 | 102 | 63 | 136 | 123 | 163 |

Design
1 No additional constraints
2 M&E constraints
3 Rensink & M&E constraints
4 Rensink constraint
5 Stewart constraint
6 Wagner constraint

Notes
[a] kg $BOD_5$/kg MLVSS·day
[b] kg $BOD_5$/kg MLSS·day
[c] kg $BOD_5$/cu.m·day
[d] mg $BOD_L$/g MLSS
* Binding constraint

$S$=substrate utilized in aeration tank ($BOD_5$)
$M_{t_6}$=total solids concentration, return sludge

Eikelboom[19] recommended floc loadings between 50 and 150 mg COD/g MLSS. Most of the designs given in Table 2, including the original optimal design, satisfy his criteria.

## 2.2 MIXING

### 2.2.1 Background

While sludge loading has been found to play an important role in determining which bacteria are dominant in the mixed cultures of an aeration basin, a number of investigators have found that the degree of longitudinal mixing, and the consequent development of a substrate or loading gradient, is also important.

The most intensive research along this line has been done by Chudoba[20]. He found that the degree of mixing influenced the selection of microorganisms in the culture and the lower the dispersion number, the lower the SVI.

Van den Eynde[21] explained the selection by the existence of two phases of microbial activity. During the exogenous phase, the organism removes substrate from solution and stores it for later use in its endogenous phase, where there no longer is substrate left in solution. Different organisms will remove substrate at different rates while in the exogenous phase and those with the greater removal rates will be able to continue their growth into the endogenous phase. Van den Eynde found that the substrate uptake rate of *Sphaerotilus natans*, a filamentous bacterium, was lower than that of floc-forming *Arthrobacter.* He also reported on the findings of Mulder and Krul. Mulder had found that filamentous bacteria were outgrown by the floc-forming bacteria because of a less economical metabolism of their stored substrate. Krul found that *Haliscomenobacterhydrossis*, a filamentous bacterium, could not produce reserve substances. This theory could explain the reason that a reactor with a substrate concentration gradient favors the selection of floc-forming microorganisms. There appears to be a limit to that selection, however, as Chudoba[8] reports cases of high sludge loading (5.0 kg BOD/kg MLSS•d) in which plug-flow reactors produced bulking sludge.

While it is well known that, for a given species and substrate and a first order kinetic model, a plug-flow reactor yields a higher conversion then a completely mixed stirred tank reactor (CSTR), experiences with sewage treatment facilities have shown that completely mixed aeration basins have higher conversions than plug flow basins. The reason for this can be explained by the selection of different dominant microorganisms, and thus different kinetic constants, using different systems. The microorganisms seem to be selected primarily by the substrate concentration at the inlet end of the basin.[20]

Many investigators have confirmed the work of Chudoba. Rensink[7] showed with synthetic wastewater in laboratory units that at a loading of 0.3 kg BOD/kg MLSS•d the batch and plug flow reactors had SVI values below 100 while the completely mixed reactor had bulking problems. At 0.5 kg BOD/kg MLSS•d, the batch reactor had a stable sludge, while the plug-flow and completely mixed reactors produced a bulking sludge.

Kroiss[22] reported on a pilot plant constructed at the Vienna, Austria treatment plant. Two parallel systems were set up with one using a completely-mixed basin and the other using a series of 6 to 8 seperated tank segments. The pilot plant was operated for 6 months and, while the less dispersed plant did experience bulking, it occured over a shorter period than it did in the completely-mixed plant.

Chambers[23] performed studies on pilot plants using aeration basins with from 1 to 24 compartments in series. He found that at a hydraulic residence time of eight hours, the degree of mixing did not seem to effect the SSVI (stirred sludge volume index), while at lower hydraulic retention times (3.3 and 5.0 hrs) the SSVI decreased with decreasing longitudinal dispersion. He also investigated the use of an anoxic mixing zone ahead of the aeration basin and found its use beneficial. While a seperate anoxic mixing zone will decrease the longitudinal mixing, there may be other factors involved which could have helped increase the sludge settleability.

Waller[24] reported on modifications to the Lambourn Division Sewage Works (UK) which were intended to control bulking. The plant's problems seemed to arise from the summertime addition of wastes from a fruit and vegetable cannery, especially the wastes from the processing of potatoes. The system was modified to a two stage aeration process where the first stage was high rate (0.72 kg $BOD_5$/kg MLSS•d) and the second stage was low rate (0.14 kg $BOD_5$/kg MLSS•d). An immediate improvement was seen in sludge settleability (from SSVI=260 ml/g to SSVI=100 ml/g) and the plant has since been converted to permanently run as a ''plug-flow'' system.

Rachwal[25] tried different feed arrangements in a Carrousel activated sludge plant. He found that arrangements increasing the plug-flow nature of the plant resulted in the best settling sludges. The best settling sludges were found associated with a single point feed into an anoxic zone.

Wheeler[26] reported on the modification to the Hamilton, Ohio Water Pollution Control Facility. The plant had been constructed in two phases with the original plant's aeration basin having less longitudinal dispersion and typical SVI values of 50-100 ml/g. The new plant's aeration tank was nearly completely mixed and had typical SVI values of 300-500 ml/g. An aerated selector channel was installed in the newer basin to allow for approximately four minutes of plug flow of combined return activated sludge and influent before they were mixed with the rest of the basin. Approximately 25-30% of the soluble COD was removed in the selector channel. The dissolved oxygen uptake, however, did not occur simultaneously. This suggests a storage of COD which is consistent with the theory of Van den Eynde presented earlier. The modification raised the settleability of the sludge from the newer aeration basin to that of the original basin.

### 2.2.2 Results from Optimization Model

Tang's Wastewater Treatment Plant model considers the aeration tank to be completely mixed. It was modified to represent the aeration basin as three CSTR tanks in series. In this modification, the degradable solids were included as substrate which is then utilized through the three tanks. It was assumed that no degradable solids exist in the effluent of the aeration basin. Inert solids include the decayed cells based on the concentration of active biomass in each tank.

Table 3 compares the cost-optimal designs of plants with a single CSTR and three CSTRs in series. The cost of the plant with less longitudinal mixing is shown to be only 2.08% greater and is constrained by the sludge age being at its lower bound of 2.0 days. It is possible that these optimally designed plants do not reflect the differences in completely mixed and less mixed aeration basins because the kinetic constants are kept unchanged between cases, and it has been previously pointed out that the microbial culture could be completely different. The hydraulic retention time of the cost-optimal plant is about four hours which, according to Chambers[23] , should show an increase in settleability with a decrease in the longitudinal mixing.

## 2.3 PRIMARY SEDIMENTATION

### 2.3.1 Background

Wagner[16] reported that, in his study of plants in Germany, those plants which had primary sedimentation had the worst SVI values. Plants without primary sedimentation usually had considerably fewer filamentous microorganisms. He also showed an increase in SVI with the residence time in the primary clarifier.

Wagner proposed that filamentous microorganisms have a competetive advantage over the floc-forming microorganisms when most of the substrate is in solution because of their greater surface area to volume ratio. Therefore, in plants without primary sedimentation, where a higher percentage of substrate is

Table 3
Cost-Optimal Designs
for Different Degrees of Mixing in Aeration Basin

| Aeration Tank | 1-CSTR | 3-CSTR |
|---|---|---|
| Cost ($/yr) | 500,394 | 510,784 |
| Eff. $BOD_5$ (mg/l) | 30.0 | 15.4 |
| Eff. TSS (mg/l) | 30.0 | 30.0 |
| ORPST (m/hr) | 6.00 | 5.21 |
| Recycle Rate (%) | 12.5 | 10.75 |
| Sludge Age (days) | 2.19 | 2.00 |
| Volume, A.T.(cu.m) | 5696 | 6207 |
| Area, F.S.T. (sq.m) | 684 | 664 |
| $Q_{in}$, Grav. Thick. (cu.m/hr) | 11.75 | 12.50 |
| Soluble $BOD_5$ (mg/l), Tank #1 | - | 57.2 |
| , Tank #2 | - | 15.2 |
| , Tank #3 | 19.8 | 4.0 |
| Active Biomass (mg/l), Tank #1 | - | 713 |
| , Tank #2 | - | 728 |
| , Tank #3 | 707 | 731 |
| MLVSS (mg/l), Tank #1 | - | 1040.4 |
| , Tank #2 | - | 1045.0 |
| , Tank #3 | 1061 | 1048 |
| Sludge Loading, Tank #1 | - | 1.88 |
| (kg $BOD_5$/kg MLVSS·d), Tank #2 | - | 1.06 |
| , Tank #3 | 0.66 | 0.28 |
| Volumetric Loading, Tank #1 | - | 1.95 |
| (kg $BOD_5$/cu.m·d), Tank #2 | - | 1.11 |
| , Tank #3 | 0.72 | 0.30 |

contained in particulate matter, filamentous microrganisms enjoy less of an advantage over the floc-forming bacteria.

2.3.2 Results from Optimization Model

Cost-optimal plants resulting from Tang's model consistently show designs with primary clarifier overflow rates at the upper bound of 6.0 m/hr, suggesting that the cost-optimal design eliminates the primary clarifier. However, that result depends on the influent conditions. Tang found that the primary clarifier is cost-effective when there is a high concentration of suspended solids in the influent.

While the primary clarifier may be shown not to be cost-effective in many cases, it is included in many plants because of historical circumstances or for reasons of reliability. Clark, Viesmann, and Hammer[27] stated that completely mixed activated sludge processes without primary sedimentation are generally used only in small municipalities because of the costs involved in sludge disposal and operation. This, however, runs contrary to the results of Tang's model which considers those sludge handling costs. While reliability constraints may call for the inclusion of the primary clarifier, cost and incidence of sludge bulking call for small or no primary clarifiers.

## 2.4 DISSOLVED OXYGEN

2.4.1 Background

Low dissolved oxygen (D.O.) concentrations in the aeration basin are quite often felt to be the cause of sludge bulking in activated sludge plants. Cameron, et al.[28] listed the D.O. concentration as one of the things to check if bulking occurs. The New York State Department of Health[29] warned operators that dissolved oxygen must always be present in the sewage in the final clarifiers and a remedial step to take after bulking occurs is to increase the time and rate of aeration. It was the opinion of David Jenkins[30] that bulking problems in Europe were caused by low loadings while problems in the USA were usually caused by low D.O. concentrations.

Orford[5] found that the sludge density index (SDI) increased only slightly with increasing mixed liquor dissolved oxygen concentrations. He found that a good correlation existed between the SDI and sludge loading and that he could not get a better correlation by including dissolved oxygen in the expression.

Bhatla[31] studied a full-scale plant treating a pulp and paper industrial waste. He initially ran the plant at normal aeration rates where the SVI was about 100 ml/g. After three days, he increased the aeration levels in the tanks and found that the sludge bulked (SVI values greater than 230). Initially, D.O. levels in the aeration basin ranged from 0 to 2.2 mg/l, whereas in the second phase the range was from 0 mg/l at the head end of the tanks to 6.3 mg/l at the outlet. He found that the filamentous sludge produced a more purified effluent and that there existed a tradeoff between a less filamentous sludge at low dissolved oxygen levels (below 2.5 mg/l) which would have better settling characteristics but poor BOD removal and a more filamentous sludge at higher oxygen levels which settled poorly but showed good BOD removal. Bosman[32] also found that poor settling rates were the result of over aeration. He studied extended aeration plants treating mine wastes.

Bhatla was studying a waste known for its tendency toward bulking. The loading rate, estimated using several of the plant parameters given, was about 0.6 kg $BOD_5$/kg MLVSS day. His results ran contrary to those of other investigators who found bulking caused by insufficient rather than excessive dissolved oxygen.

It is interesting to note that the reactor which Bhatla had studied was plug flow and that in all cases the dissolved oxygen concentration was less than 0.5 mg/l at the head end of the basin. If the microorganisms which predominate are selected by the initial loading as has been previously proposed, then they would have been selected under low dissolved oxygen conditions.

In the discussion following a paper presented by Tomlinson and Chambers[33], H.B. Tench related his experience controlling bulking. Where surface aerators are used, shutting off aeration for a period could temporarily more than double the surface area available for settling, reduce the sludge loading on the final clarifier and effectively transfer sludge from the final tanks to the aeration tank.

Palm, et al.[14] performed a detailed study of the relationship between dissolved oxygen concentrations and bulking using laboratory-scale completely mixed activated sludge units treating settled domestic wastewater. By varying loading rates and dissolved oxygen concentrations, they determined an empirical relationship between dissolved oxygen concentration and the maximum COD removal rate above which bulking would occur. Generally they found that the D.O. required in the basin increases as the loading increases and that bulking was caused by D.O. deficiencies (see Figures 3 and 4). In these experiments, when bulking occured, *Sphaerotilus* was the responsible microorganism. They felt that this would generally be true for low D.O. bulking. In cases of low sludge loading bulking, other microorganisms may be involved.

It is interesting to note that, while Bhatla's plant was plug-flow, the loading rate/dissolved oxygen concentration combination he found to be limiting for bulking was close to that which Palm's relationship predicts. However, Bhatla found that bulking occured with higher levels of oxygen rather than lower.

Pitman[34] observed that for plants treating domestic sewage, high D.O. levels promoted good settling rates and sludge densities; furthermore the higher the D.O., the lower the sludge age (and thus the higher the sludge loading) at which endogenous deflocculation occurs.

Starkey and Karr[35] studied the effect of low dissolved oxygen concentration on effluent turbidity. Using a bench scale plant fed with a synthetic dextrose substrate solution, they concluded that increased effluent turbidity at low D.O. concentrations was due to the inhibition of exocellular polymer production and the reduction in the number of eucaryotic microorganisms. They found that the SVI was not affected by lowering the D.O. concentration and that filamentous microorganisms were only observed after aeration was stopped for several days.

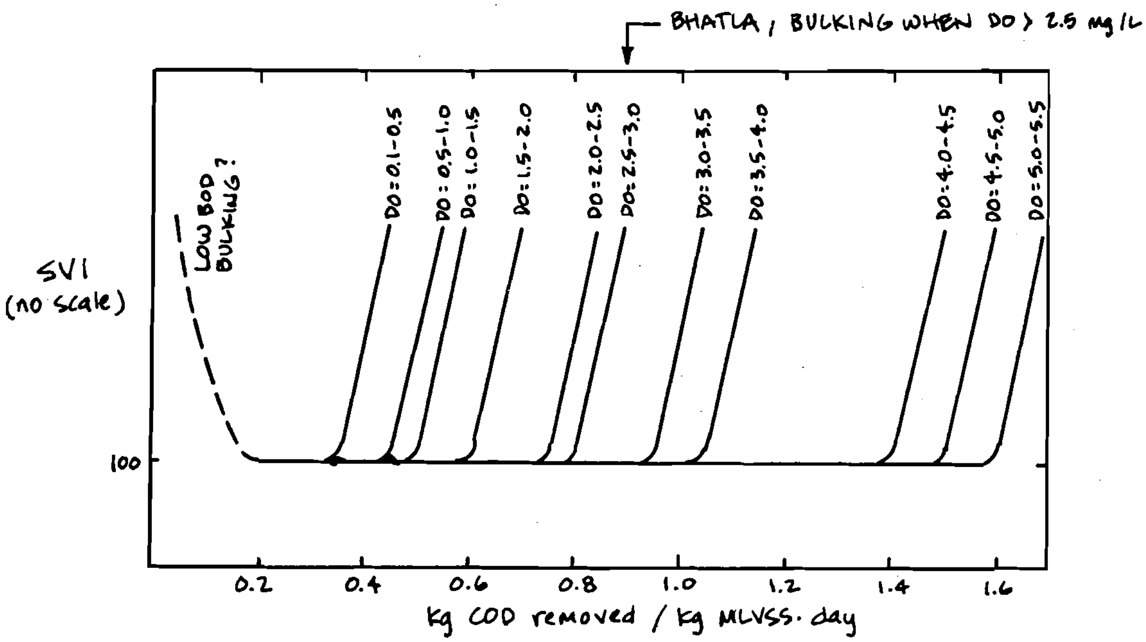



Figure 3--Proposed Relationship Between SVI and COD Removal Rate (After Palm, et al.[14])

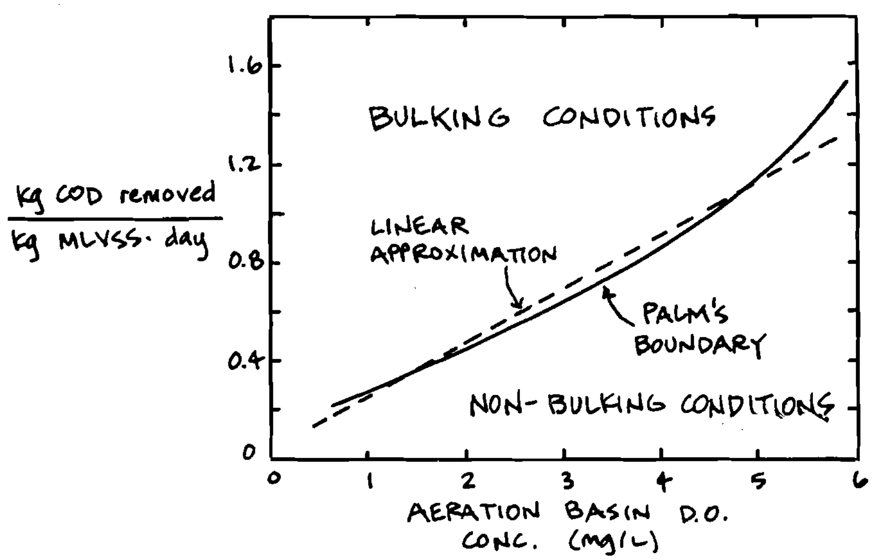



Figure 4--Proposed Boundary Between Bulking and Non-Bulking Conditions (After Palm, et al.[14])

It should be noted however that the duration of low D.O. concentrations during the test was 50 hours, possibly too short of a time for the filamentous organisms to develop.

2.4.2 Results from Optimization Model

Tang's wastewater treatment plant model is rather insensitive to the DO concentration maintained in the aeration basin. While the cost rises with increasing D.O. concentration, the cost-optimal design is essentially the same through the range of dissolved oxygen levels. If the model is constrained so that the BOD removal rate is lower than the bound suggested by Palm for a given D.O. concentration, the cost optimal solution for low D.O. levels changes. Table 4 summarizes the designs. Note that because of the upper bound of 6.0 days placed on the sludge age and the lower bound of 10% on the recycle ratio, a BOD removal rate below 0.216 kg $BOD_5$/ kg MLVSS·day is infeasible. It is also worth noting that Palm's experiments were carried out at a MLSS concentration of 1100 mg/l whereas the optimal designs showed MLSS concentrations of about 1500 mg/l. Although this concentration difference is minor, his bounds may not be applicable at that higher MLSS concentration.

The results in Table 4 show that, when the BOD removal rate is constrained as Palm's results suggest is necessary to eliminate sludge bulking, the least cost design no longer occurs where the dissolved oxygen concentration is at a minimum. While increasing the D.O. level in the aeration basin increases the cost of a plant, Palm's constraint requires lower BOD removal rates (and thus larger aeration tanks) at lower D.O. levels. The result of these trends is a minimum cost, for the influent conditions studied, at a D.O. level of 3.7 mg/l.

## 2.5 ASL/MPSL FINAL CLARIFIER

2.5.1 Background

In establishing a strategy for controlling a bulking sludge in an existing plant, Tomlinson and Chambers [33] suggested comparing the applied solids loading

Table 4
Cost-Optimal Designs for Varied D.O. Levels

| D.O. (mg/l) | 5.0 | 4.0 | 3.7 | 3.0 | 2.0 | 1.0 |
|---|---|---|---|---|---|---|
| **Palm Constraint on BOD Removal** | | | | | | |
| Cost ($/yr) | 556,240 | 533,540 | 528,530 | 534,110 | 545,550 | 646,930 |
| Eff. $BOD_5$ (mg/l) | 30.0 | 30.0 | 30.0 | 24.589[b] | 19.301[b] | 11.364 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 | 14.596 |
| Sludge Age (days) | 2.1924 | 2.1921 | 2.1926 | 2.8916 | 4.4222 | 6.0 |
| Recycle Ratio (%) | 12.466 | 12.544 | 10.0 | 11.033 | 14.064 | 10.0 |
| Volume,A.T. (cu.m) | 5704 | 5686 | 6409 | 8095 | 10,858 | 13,263 |
| Area,F.S.T. (sq.m) | 683 | 684 | 653 | 666 | 703 | 1572 |
| MLSS (mg/l) | 1514 | 1514 | 1347 | 1420 | 1614 | 1917 |
| MLVSS (mg/l) | 1085 | 1085 | 966 | 1021 | 1160 | 1329 |
| Sludge Loading[a] | 0.663 | 0.663 | 0.649 | 0.49 | 0.330 | 0.232 |
| $BOD_5$ Removed[a] | 0.533[b] | 0.533[b] | .521 | 0.420 | 0.30 | 0.216[c] |
| **No Additional Constraints** | | | | | | |
| Cost ($/yr) | 556,240 | 533,540 | 528,090 | 517,470 | 505,440 | 497,050 |
| Eff. $BOD_5$ (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| Sludge Age (days) | 2.1924 | 2.1921 | 2.1925 | 2.1923 | 2.1923 | 2.1923 |
| Recycle Ratio (%) | 12.466 | 12.544 | 12.534 | 12.539 | 12.662 | 12.525 |
| Volume,A.T. (cu.m) | 5704 | 5686 | 5687 | 5687 | 5658 | 5690 |
| Area, F.S.T. (sq.m) | 683 | 684 | 684 | 684 | 686 | 684 |
| MLSS (mg/l) | 1519 | 1519 | 1519 | 1519 | 1519 | 1519 |
| MLVSS (mg/l) | 1088 | 1088 | 1088 | 1088 | 1088 | 1088 |
| Sludge Loading[a] | 0.663 | 0.663 | 0.663 | 0.663 | 0.663 | 0.663 |
| $BOD_5$ Removed[a] | 0.533 | 0.533 | 0.533 | 0.533 | 0.533 | 0.533 |

[a] kg $BOD_5$/kg MLVSS•day
[b] Constraint not binding
[c] Design infeasible at 0.19 kg $BOD_5$/kg MLVSS•day

(ASL) on the final clarifier to the maximum permissible solids loading (MPSL) and determining the most cost-effective way of insuring that the ASL is less than the MPSL. The applied solids loading can be calculated as:

$$ASL = \frac{Q_2(1 + r)MLSS}{A_f} \tag{2.5}$$

$r$=Recycle Rate
$Q_2$=Flow into Aeration Basin
$A_f$=Surface Area of Final Clarifier
$MLSS$=Mixed Liquor Suspended Solids Conc.

while the maximum permissible solids loading is a function of the sludge settleability and the final clarifier underflow (recycle rate). This approach can also be applied at the design stage if a bulking problem is expected to develop. While the previous sections have dealt with ways of controlling the settleability, and thus lowering the SVI one need assume during design (and thus raising the MPSL), a more cost-effective approach might be to design for higher sludge loadings (e.g. smaller aeration tank), assume a higher design SVI value, and lower the ASL by increasing the size of the final clarifier or decreasing the MLSS, or increase the MPSL by increasing the recycle rate (which will also increase the ASL, but not necessarily to the same extent). A plant that has a lower ASL might be considered to be able to effectively thicken a sludge with a lower settleability and so will not be as likely to develop bulking problems.

2.5.2 Results From Optimization Model

As the settleability of the sludge decreases, the thickening action of the secondary clarifier degrades. Thickening is modeled in Tang's wastewater treatment plant model according to Dick's equation:

$$M_{t_5} = \left[a_w(n_w - 1)\right]^{\frac{1}{n_w}} \left(\frac{n_w}{n_w - 1}\right)\left(\frac{A_f}{Q_5}\right)^{\frac{1}{n_w}} \tag{2.6}$$

$A_f$ = Area of Final Clarifier
$Q_5$ = Final Clarifier Underflow
$a_w$ and $n_w$ are sludge thickening constants.

$M_{t_5}$ = Underflow total solids concentration

The thickening parameters are highly empirical and have been studied only on a limited basis. For sludges with normal settling properties that were studied however, $n_w$ varied only slightly while $a_w$ varied over a wide range.[36]

It should be possible to link the sludge settling constants to the SVI or some other settleability measure. If $n_w$ is a measure of interference between the sludge flocs and $a_w$ is a measure of the settling velocity of the sludge floc, one could imagine that as a sludge began to bulk (as the SVI increased), $n_w$ would increase (as the filamentous microorganisms interfere with thickening to an extent much greater than their concentration would predict) and $a_w$ would decrease (as the filamentous microorganisms would form a mat which would settle more slowly). While the direction of change may be intuitive, the magnitude of the change is not.

Table 5 shows the results from the optimization model when the settling constants are perturbed by 10% in each direction. It is interesting that the control strategy which Tomlinson and Chambers recommend for an existing plant -- increasing the recycle rate and decreasing the MLSS concentration -- is reflected in the cost optimal solutions for perturbations in the directions suggesting bulking. This seems to imply that the cost optimal approach to designing for a poorly settling sludge is to increase the maximum permissible solids loading by increasing the sludge recycle rate and decreasing the applied solids loading by decreasing the MLSS concentration rather than simply increasing the area of clarification. The identification of the range of sludge settleability over which this conclusion holds might be more easily understood with the use of a more familiar expression of settleability, i.e. the SVI.

Daigger and Roper[37] reported on data obtained at pilot plants operated by the Milwaukee Metropolitan Sewerage District to correlate the SVI to the batch settling velocity of the activated sludge. Sludges with a wide SVI range were studied and the following equation was proposed:

$$V_i = 7.80\, e^{-[0.148+0.00210(SVI)]\, C_i} \qquad (2.7)$$

Table 5
Cost-Optimal Designs
for Perturbed Final Clarifier Settleability Constants

| | $a_w=26.66$ | $a_w=21.82$ | $a_w=24.24$ | | $a_w=24.24$ |
|---|---|---|---|---|---|
| | $n_w=2.3747$ | | $n_w=2.6122$ | $n_w=2.1372$ | $n_w=2.3747$ |
| Cost ($/yr) | 498,170 | 502,960 | 519,630 | 482,100 | 500,394 |
| Eff.BOD (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| Eff.TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |
| Sludge Age (days) | 2.1924 | 2.1923 | 2.1806 | 2.1978 | 2.19 |
| Recycle Rate (%) | 12.167 | 12.924 | 14.772 | 10.004 | 12.50 |
| Volume A.T. (cu.m) | 5541.3 | 5860.3 | 6330.2 | 5031.4 | 5696 |
| Area F.S.T. (sq.m) | 685.47 | 682.73 | 686.06 | 685.18 | 684 |
| MLSS (mg/l) | 1558 | 1474 | 1377 | 1707 | 1516 |
| $Q_{in}$ G.T. (cu.m/hr) | 11.215 | 12.379 | 14.946 | 8.8129 | |

Table 6
Cost-Optimal Designs
for Sludges with Varied SVI Values

| SVI (ml/g) | 60 | 100 | 140 | 180 | 220 | 260 |
|---|---|---|---|---|---|---|
| Cost ($/yr) | 491,120 | 498,222 | 504,510 | 510,400 | 516,010 | 521,700 |
| Eff. BOD (mg/l) | 30 | 30 | 30 | 30 | 30 | 30 |
| Eff. TSS (mg/l) | 30 | 30 | 30 | 30 | 30 | 30 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |
| Sludge Age (days) | 2.1925 | 2.1923 | 2.1922 | 2.1919 | 2.1918 | 2.1910 |
| Recycle Rate (%) | 10.036 | 13.657 | 15.624 | 17.301 | 18.703 | 20.039 |
| Volume, A.T. (cu.m) | 4714 | 5023 | 5705 | 6381 | 7061 | 7713 |
| Area, F.S.T. (sq.m) | 697 | 710 | 704 | 700 | 698 | 698 |
| MLSS (mg/l) | 1831 | 1719 | 1514 | 1355 | 1225 | 1122 |
| $Q_{in}$ G.T. ($m^3$/hr) | 8.4 | 11.2 | 14.0 | 16.8 | 19.8 | 22.8 |
| Settling Vel. (m/hr) | 4.72 | 4.22 | 3.99 | 3.82 | 3.70 | 3.58 |
| ORFST (m/hr) | 2.15 | 2.11 | 2.13 | 2.14 | 2.15 | 2.15 |

$V_i$=batch settling velocity (m/hr)
$C_i$=influent solids concentration (g/l)

This equation takes the form of the settling equation proposed by Vesilind[1]:

$$u_i = a' e^{-b' C_i} \tag{2.8}$$

as opposed to the form proposed by Duncan and Kawata[1] and used by Dick and Suidan to derive the expression used in Tang's model.

Using the expression developed by Berthouex and Polkowski[1] for the limiting flux:

$$G_L = a' b' C_u^2 e^{-b' C_u} \tag{2.9}$$

$C_u$=underflow solids concentration
$a'$ and $b'$ are the same as in (2.8)

and the definition of limiting flux:

$$G_L = \frac{Q_u C_u}{A} \tag{2.10}$$

$Q_u$ = final clarifier underflow
A = surface area of final clarifier

then substituting (2.10) into (2.9) and taking the values of a′ and b′ from (2.7) the underflow concentration can be found to be directly related to the SVI:

$$C_u = \frac{Q_u e^{\left[0.148+0.00210(SVI)\right]C_u}}{7.80\left\{0.148+0.00210(SVI)\right\}A} \tag{2.11}$$

Substituting this equation into Tang's model allows a closer look at how cost-optimal designs change as the design sludge settleability varies. Of course, the change in sludge characteristics is only reflected in the thickening equation. Clarification, modeled according to Chapman's equation, still does not consider the settleability of the sludge.

The designs summarized in Table 6 are consistent with what was found by perturbing the sludge constants in Dick and Suidan's equation. As the expected SVI increases, the cost-optimal plant should be designed with a larger aeration tank and a lower mixed liquor suspended solids concentration which effectively reduces the ASL. The size of the final clarifier remains constant while increasing the recycle rate dampens the decrease in the maximum permissible solids loading caused by the reduced settleability.

Keefer[38] saw that as the SVI of a sludge rises, better removal levels are seen in the final clarifier until the point where the settling velocity of the sludge blanket is less than the overflow rate of the final clarifier. At that point, the sludge blanket would be lost over the final clarifier weir. The better removal levels were thought to be due to the increased contact time between the sludge flocs and the wastewater as the floc settled more slowly. Table 6 shows that the sludge settling velocities, as predicted by Daigger & Roper's equation, are greater than the final clarifier overflow rates for the cost-optimal designs. One might expect, following Keefer's observations, that the effluent would show better removal levels than Chapman's equation would predict in determining these solutions. Considering both thickening and clarification then, it would seem that simply increasing the size of the final clarifier is not a cost-efficient approach to designing for a potentially bulking sludge.

## 2.6 SUMMARY

Although much work has been done pertaining to the study of activated sludge bulking, there still is no all-encompassing model of its occurence. The design engineer must deal with the conflict and uncertainty which exists regarding the problem of activated sludge bulking. In many cases the engineer deals with the problem by conservatively oversizing the final clarifier so as to meet the effluent requirements during periods when the sludge might settle poorly. However, this approach may not consider the efficient design of the total system. It might be more efficient to prevent the formation of a sludge with poor settleability by using a lower sludge loading, decreasing the longitudinal mixing, or increasing the dissolved oxygen concentration in the aeration tank.

Eliminating or decreasing the size of the primary clarifier might decrease system cost while a better settling sludge is produced. It would be helpful if a design optimization model could incorporate a model of the sludge bulking problem so that the overall model could consider the combined effect of these different approaches and then determine the cost-effectiveness of designing to prevent poor settleability.

## CHAPTER 3

## MODELING THE JUDGEMENT OF AN EXPERIENCED ENGINEER

### 3.1 INTRODUCTION

While the previous section showed cost-optimal designs subject to the various single constraints proposed, it could not examine possible tradeoffs between the effectiveness of controlling bulking and system cost for constraint combinations. This section attempts to deal with those tradeoffs by creating a model which logically infers, from the values of some design variables, the likelihood of a given plant design experiencing bulking problems. The structure of the model is patterned as a rule-based system and the logical structure and the relative truth-value of each rule are fit to one engineer's evaluation of a set of designs. The consistency of both the engineer and the model are then checked with a second set of designs. Finally, the model is incorporated into Tang's optimization model so that some trends in the tradeoff between cost and likelihood of bulking problems can be examined.

### 3.2 BACKGROUND

Rule-based systems are one way to manipulate a complex pathway of rules which an expert might implicitly use to come to a decision regarding a problem which is inherently fuzzy (i.e., there is a poorly understood relationship between pieces of evidence and a conclusion) and for which someone with a good deal of experience is needed. A number of rule-based systems have been set up for problems as diverse as medical diagnosis, civil evacuation plans, and structural analysis[39]. Blockley[40] presented a fuzzy rule-based system for the subjective assessment of the safety of a structure before, during, and after construction. Ishizuka, et al.[41] presented a method of rule-based inference for structural damage assessment.

In the environmental engineering field, Flanagan[42] showed how such a system could be used to control the activated sludge process by interpretation of the dissolved oxygen profile in the aeration tank. Tong, et al.[43] also looked at the problem of automatic control of an activated sludge plant and presented 20 fuzzy rules which consider effluent water quality and several process operation parameters (such as MLSS concentration) to determine what changes are to be made to the D.O. set point, the recycle rate, and the sludge wastage rate. Johnston[44] set up a rule-based system to diagnose problems with the wastewater treatment process based on the judgement of a plant operator.

The concept of fuzzy association is used to deal with the uncertainty of information with which to evaluate a rule, and the uncertainty of the rules themselves. A piece of evidence may only have partial correlation to a given conclusion, and so rather than throwing out this incomplete knowledge and looking for more consistent rules, it is said to be fuzzily associated with that conclusion.

The problem of determining why a given plant is experiencing bulking problems, or the problem of determining if a given plant will experience a bulking problem, may be reduced to several sub-problems as shown in Figure 5. The problem may arise because of a troublesome wastewater influent, because of a troublesome plant design, or because of some combination of the two.

Each sub-problem may be reduced again to its inter-related evidence. Influent characteristics which are associated with bulking problems can be related to types of industrial dischargers, levels of nutrients, carbohydrates, and fats, and septicity. Design parameters which have been found to influence the development of bulking problems include BOD removal, sludge loading, dissolved oxygen, volumeteric loading, mass loading on the final clarifier, and primary clarifier size.

One significant problem in the modeling of the fuzzy interaction of pieces of evidence and their association to the overall conclusion is determining the weights of association of a particular piece of evidence to a hypothesis. Another significant problem is determining the propositional operator which might best describe the interaction of several pieces of evidence to a hypothesis.

In many cases, the weights of association are found by asking an expert for an

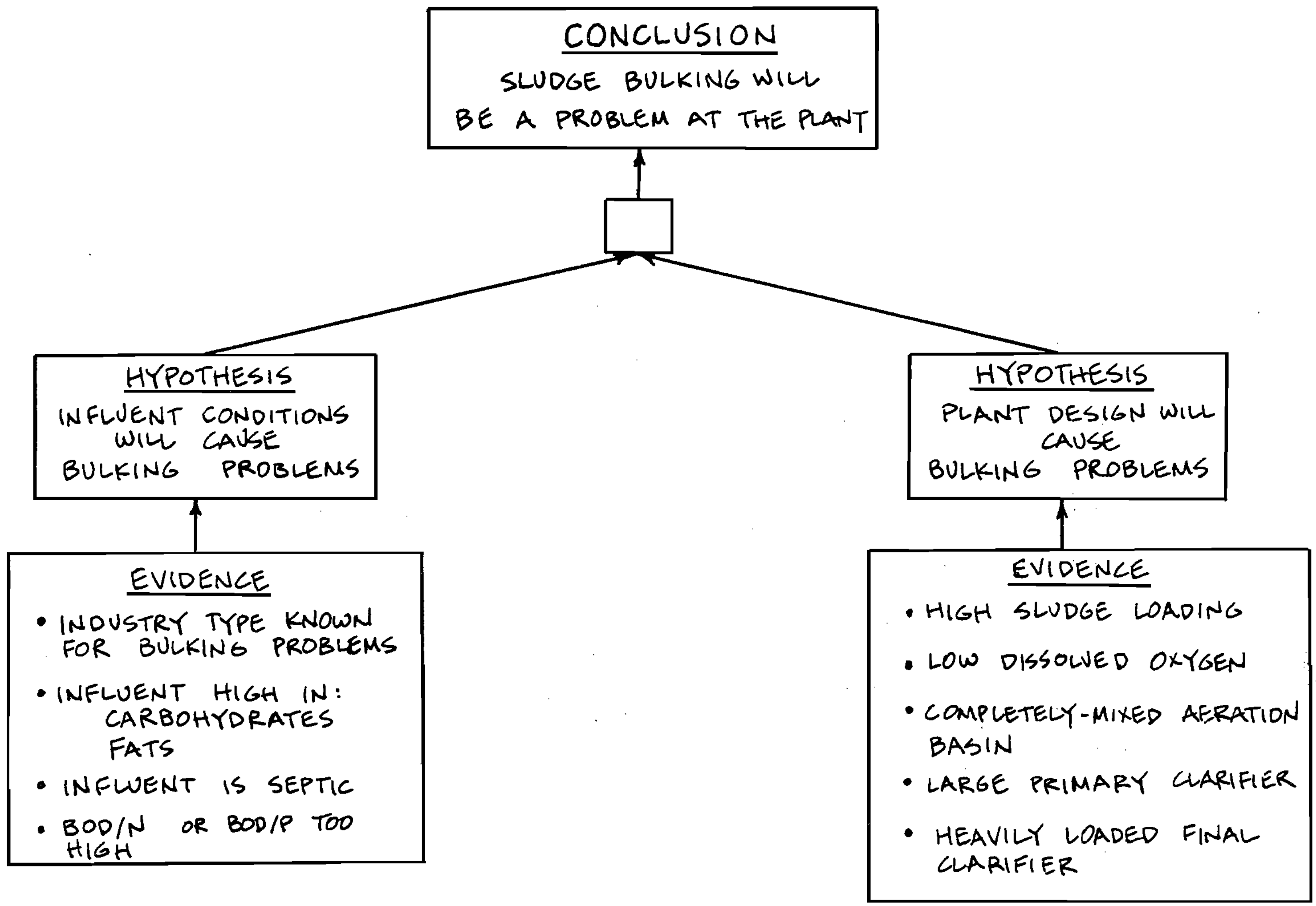



Figure 5--Conceptual Inference Tree

opinion as to their values. It seems, however, that such a method assumes that the expert explicitly knows the relationship of the model variables to the conclusion and could, if pressed, develop an empirical relationship. In other cases, the weights are obtained statistically by observing a number of events where the piece of evidence held true and determining the fraction for which the conclusion also held true. This is a good approach, but one which requires a good deal of historical data.

Chu, et al.[45] for example, estimated the relative association of different countries to the fuzzy set of important trading partners with Taiwan by looking at export and import volumes between Taiwan and the other countries and Taiwan's total trade.

While one may determine weights of association by statistical or interview techniques, the way in which pieces of evidence are combined is also important. While a single piece of evidence pointing toward a bulking problem may not be significant, the existence of several pieces of evidence might increase the likelihood of a problem developing. Some form of fuzzy reasoning might be effective in modeling such an interaction.

While boolean algebra allows for rule interaction by using **AND** and **OR** nodes into which the rules branch, it cannot deal with rules which are only sometimes true. Fuzzy logic is a more general form of boolean logic which allows the truth value at a node to vary depending on its level of belief. The overall level of belief of a rule branch is found as:

$$T_{i,j} = W_i A_{i,j} \tag{3.1}$$

$T_{i,j}$ = truth value of rule branch $i$ for decision $j$
$W_i$ = weight of association of rule $i$ to higher node
$A_{i,j}$ = truth of rule statement $i$ for decision $j$

A fuzzy propositional operator considers the truth values of the rules branching into it and passes a representative truth value to the next higher node. The **OR** operator passes the maximum and the **AND** operator passes the minimum of its branch values. The **WAND** operator, proposed by Harandi[46], passes the average of the branch values.

If the **OR** operator is used, the most heavily weighted piece of evidence which is satisfied would control the truth value of the conclusion no matter what the value of the other weighted branch values. The **WAND** operator on the other hand passes a value which is influenced by the satisfaction of a piece of evidence which might be inconsequential in the face of other evidence. It might make sense to use a fuzzy operator termed **XOR** which would pass the average of the **X** greatest branch values.

Blockley[40] used a formulation similar to **WAND** in considering the imminence of failure of a structure. He considered the sum of the weighted truth values of 24 parameter statements for each of 23 past structural failures. The structure which had the greatest sum was considered to be the most inevitable failure.

## 3.3 MODEL OF JUDGEMENT

Rather than depend on an expert to know explicitly (and be able to communicate) the relevant pieces of evidence, their weights of association to the conclusion, and the logical operation used to combine different pieces of evidence, a different calibration technique was used in this work, as outlined below.

### 3.3.1 Expert Judgement

The problem, presented in Figure 5, was simplified as an attempt at creating an accurate portrayal of the weighing of evidence which the expert might go through to make a judgement regarding the potential bulking problems in a plant. A concentrated effort was made at modeling the sub-problem of determining if a given plant design, under normal domestic wastewater influent conditions, was likely to experience sludge bulking problems. Fifteen plant designs, different with respect to process parameters, yet all subject to the influent conditions shown in Table 7, were given to Dr. J.T. Pfeffer, Professor of Sanitary Engineering at the University of Illinois (referred to as the expert or the experienced engineer). Those designs are shown in Table 8.

| Table 7<br>Influent Characteristics | |
|---|---|
| Flow | 1500 $m^3$/hr |
| Soluble $BOD_5$ | 100 mg/L |
| Active Biomass | 5 mg/L |
| Biodegradable Volatile Solids | 100 mg/L |
| Inert Volatile Solids | 45 mg/L |
| Fixed Solids | 50 mg/L |

The designs were chosen so that, while they are still within the bounds placed on the design variables in Tang's model (e.g. sludge age of less than 6 days), they represented a wide range of possible designs. The sludge loading varied from 0.22 to 0.72 kg $BOD_5$/kg MLVSS•day, the volumetric loading varied from 0.29 to 0.96 kg $BOD_5/m^3$•day, the D.O. concentration varied from 1.5 to 6.0 mg/L, the ORPST varied from 1.5 to 6.0 m/hr, and the MLFST varied from 2.01 to 5.53 kg/$m^2$•hr.

The expert was asked to assign a likelihood that the hypothesis is true for each plant design. The likelihood is a value from 0 to 1, with 0 meaning that the plant would definitely not experience sludge bulking problems, and with 1 meaning that it definitely would. The expert was not informed of the rules used to model his judgement so that his decisions would not be influenced.

3.3.2 Model Rules & Rule Structure

The evidence, in the form of rules which were thought to be important in judging a design, involved the values of the following parameters:

**1. MLFST** -- Since sludge bulking is a settleability problem, plants with a high mass loading on the final settling tank (MLFST) might be considered likely to experience bulking problems. The rule that

IF "MLFST $>$ 97.7 kg/$m^2$•day" THEN "there might be bulking problems " comes from the Illinois Recommended Standards for Sewage Works.[12]

Table 8
Designs for Survey #1

| Design Parameter | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| PST overflow rate (m/hr) | 6.0 | 3.0 | 6.0 | 6.0 | 6.0 |
| ($gpd/ft^2$) | (3534) | (1767) | (3534) | (3534) | (3534) |
| Sludge Age (days) | 6.0 | 4.0 | 2.2 | 2.9 | 4.55 |
| Recycle Rate (%) | 10.0 | 30.7 | 12.5 | 11.0 | 10.0 |
| Vol. A.T. ($m^3$) | 13,263 | 5,000 | 5,690 | 8,095 | 13,371 |
| Area, FST ($m^2$) | 1572 | 1500 | 684 | 666 | 653 |
| MLVSS (mg/l) | 1329 | 2145 | 1088 | 1021 | 968 |
| MLSS (mg/l) | 1917 | 2967 | 1519 | 1420 | 1344 |
| DO Conc. (mg/l) | 2.0 | 2.0 | 6.0 | 3.0 | 1.5 |
| Sludge Loading[a] | 0.23 | 0.45 | 0.66 | 0.49 | 0.31 |
| Vol. Loading[b] | 0.31 | 0.96 | 0.72 | 0.50 | 0.30 |
| | (19.3) | (59.9) | (44.9) | (31.2) | (18.7) |
| FST Solids Loading[c] | 2.01 | 3.88 | 3.75 | 3.55 | 3.40 |
| | (9.8) | (19.1) | (18.4) | (17.4) | (16.7) |
| **Design Parameter** | **6** | **7** | **8** | **9** | **10** |
| PST Overflow rate (m/hr) | 6.0 | 6.0 | 6.0 | 3.0 | 3.0 |
| ($gpd/ft^2$) | (3534) | (3534) | (3534) | (1767) | (1767) |
| Sludge Age (days) | 4.2 | 2.2 | 2.2 | 2.2 | 3.75 |
| Recycle Ratio (%) | 37.4 | 10.0 | 20.0 | 9.5 | 10.3 |
| Vol. A.T. ($m^3$) | 6267 | 4714 | 7713 | 6000 | 9200 |
| Area, FST ($m^2$) | 993 | 697 | 698 | 690 | 850 |
| MLVSS (mg/l) | 1917 | 1318 | 808 | 967 | 1093 |
| MLSS (mg/l) | 2663 | 1831 | 1122 | 1341 | 1511 |
| DO Conc. (mg/l) | 1.5 | 2.5 | 1.5 | 1.5 | 1.5 |
| Sludge Loading[a] | 0.42 | 0.64 | 0.69 | 0.72 | 0.40 |
| Vol. Loading[b] | 0.8 | 0.84 | 0.56 | 0.67 | 0.44 |
| | (49.9) | (52.4) | (34.9) | (41.8) | (27.5) |
| FST Solids Loading[c] | 5.53 | 4.33 | 2.89 | 3.15 | 2.92 |
| | (27.2) | (21.3) | (14.2) | (15.5) | (14.4) |
| **Design Parameter** | **11** | **12** | **13** | **14** | **15** |
| PST Overflow rate (m/hr) | 1.5 | 1.5 | 4.5 | 3.0 | 3.0 |
| ($gpd/ft^2$) | (884) | (884) | (2650) | (1767) | (1767) |
| Sludge Age (days) | 4.6 | 4.2 | 4.2 | 4.6 | 4.0 |
| Recycle Ratio (%) | 7.1 | 22.8 | 34.7 | 13.8 | 6.6 |
| Vol. AT ($m^3$) | 13371 | 6267 | 6267 | 10000 | 13000 |
| Area, FST ($m^2$) | 653 | 993 | 993 | 800 | 700 |
| MLVSS (mg/l) | 837 | 1658 | 1869 | 1214 | 824 |
| MLSS (mg/l) | 1136 | 2250 | 2605 | 1679 | 1138 |
| DO Conc. (mg/l) | 6.0 | 1.5 | 1.5 | 1.5 | 1.5 |
| Sludge Loading[a] | 0.35 | 0.43 | 0.42 | 0.22 | 0.30 |
| Vol. Loading[b] | 0.29 | 0.72 | 0.78 | 0.42 | 0.36 |
| | (18.1) | (44.9) | (48.7) | (26.2) | (22.5) |
| FST Solids Loading[c] | 2.74 | 4.16 | 5.3 | 3.55 | 2.6 |
| | (13.5) | (20.4) | (26.0) | (17.5) | (12.8) |

Notes:
[a]Aeration Tank Sludge Loading, kg $BOD_5$/kg MLVSS•day
[b]Aeration Tank Volumetric Loading, kg $BOD_5/m^3$•day, (lb $BOD_5$/1000 $ft^3$•day)
[c]kg solids/$m^2$• hr, (lb solids/$ft^2$• day)

**2. Sludge Loading --** Plants with a high sludge loading (F/M ratio) have been associated with bulking problems. Two variations of the sludge loading parameter, SL1 (kg $BOD_5$/kg MLSS·d) and SL2 (kg $BOD_5$/kg MLVSS·d) were used in four rules. SL1$>$0.30 was proposed by Rensink[7], SL1$>$0.35 was proposed by Kiff[9], and SL2$>$0.30 was proposed by Stewart[6] as dividing points for bulking and non-bulking conditions in the activated sludge process. SL1$>$0.5 was used to add further delineation for highly loaded processes. The three SL1 rules might have a relationship distinct from that of the other rules since these rules depend on a common parameter (e.g. SL1$>$0.5 cannot be satisfied while SL1$>$0.3 is not).

**3. Volumetric Loading --** Wagner[16] found high SVI levels in plants with intermediate volumetric loadings and good, stable values for the SVI in plants with high or low volumetric loadings. The rule that 0.3$<$VolLoad (kg $BOD_5/m^3$· day)$<$0.8 was used to reflect possible bulking found at intermediate loadings. The Illinois Recommended Standards for Sewage Works[12] requires a volumetric loading of less than 0.56 kg $BOD_5/m^3$· day. This rule is imposed, in part, to reduce the risk of bulking problems. While these two rules are not mutually exclusive, they are conflicting as to whether a bulking problem will exist at volumetric loadings above 0.8 kg/$m^3$· day.

**4. ORPST --** Wagner[16] found that plants with small or no primary clarifiers had a lower incidence of bulking problems. An overflow rate of 3.0 m/hr was chosen as the dividing line between large clarifiers which may be associated with bulking problems and small clarifiers which may not.

**5. Dissolved Oxygen--** Palm, et al.[14] found a relationship between the BOD removal rate and the minimum dissolved oxygen level needed in the aeration tank so as not to experience bulking. Their relationship was linearized into the following rule:

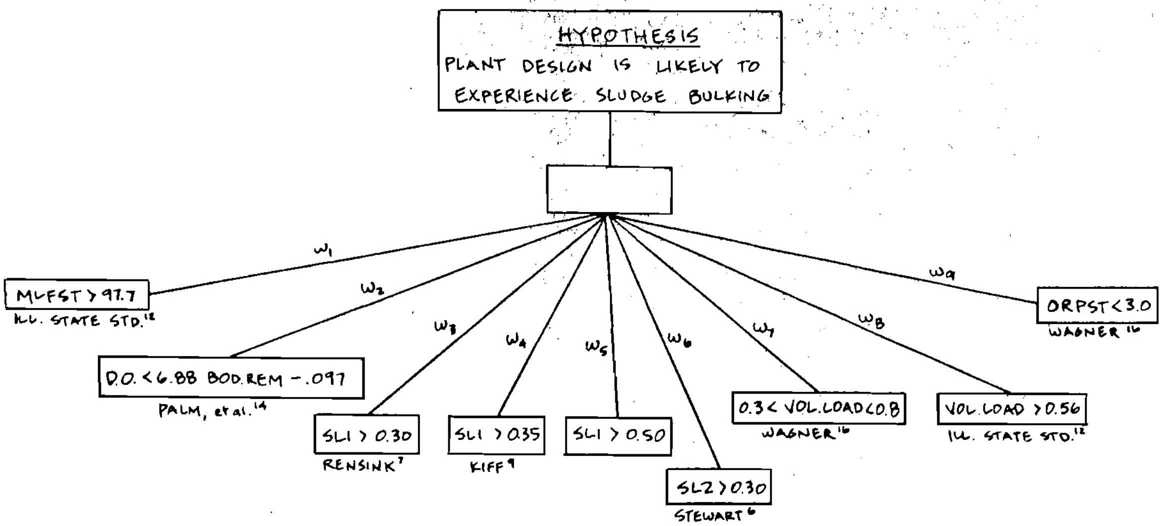
IF "D.O. $<$ 6.88 BOD REM - .097"

THEN "the plant will likely experience bulking problems"

Eleven rule structures were investigated to find how well each could be fit to the expert's judgement as to each plant's likelihood of experience bulking problems. For each rule structure, the weights were varied to find combinations which would minimize the variance of the deviation of the model's prediction from that of the expert. Since minimizing variance insures a minimal spread of deviations and not minimal deviations, the model's prediction is adjusted by the mean deviation from the expert's judgement in order to obtain the best fit.

Figure 6 presents the general rule structure which was used for five of the cases. The blank box is the propositional operator node which contained, for the different cases, **OR**, **2OR**, **3OR**, **4OR**, or **WAND (9OR)**. Additionally, six hybrid logic structures were tried. Hybrid 1 is shown in Figure 7 and passes an averaged truth value of the four sludge loading rules to be combined with the remaining rules into an **OR** node. Hybrid 2, shown in Figure 8, is similar, but combines only the three SL1 rules into the **WAND** node. Hybrid 3, shown in Figure 9, considers the greatest violation of the four sludge loading rules, and then averages that value with the branch values of the other rules. The Hybrid 4 through Hybrid 6 formulations are shown in Figure 10 and they consider the greatest violation of the three SL1 rules and compare that value with the other rules at the next higher operator. That next operator is an **OR** node for Hybrid 4, a **2OR** node for Hybrid 5, and a **3OR** node for Hybrid 6.

The non-linear optimization code GRG[47] was used on a Harris 800 computer to find the best-fit weights and approximately 1.5 CPU seconds were required for each case. Table 9 presents the weights which gave the best fit for each logic structure investigated. Additionally, alternative combinations of the weights were found in order to establish the sensitivity of the model to the weights placed on the different rules. Brill[48] proposed the use of an optimization model to explore alternative solutions to a problem by maximizing an objective function which is a measure of difference from a previously optimal solution. To find the alternative weight combinations, the square of the deviation of the weights from the set of weights found to give the minimal variance was maximized subject to an allowable 10% increase on the variance. Such alternative solutions are labeled in Table 9 by ''mga''.




D.O. = Dissolved Oxygen Conc., Aeration Tank (mg/l)

MLFST = Mass Loading, Final Settling Tank (kg/m$^2$ day)

$$= \frac{(MLSS)\ Q_2\,(1+r)}{AFST}$$

BOD REM = BOD$_5$ Removal

$$= \frac{Q_2\,(1+r)(S_2 - S_3)}{(MLVSS)\ (VAT)}$$

ORPST = Overflow Rate, Primary Settling Tank (m/hr)

SL1 = Sludge Loading Aeration Tank (kg BOD$_5$/kg MLSS day)

SL2 = Sludge Loading Aeration Tank (kg BOD$_5$/kg MLVSS day)

VolLoad = Volumetric Sludge Loading (kg BOD$_5$/m$^3$ day)

Figure 6--General Rule Structure

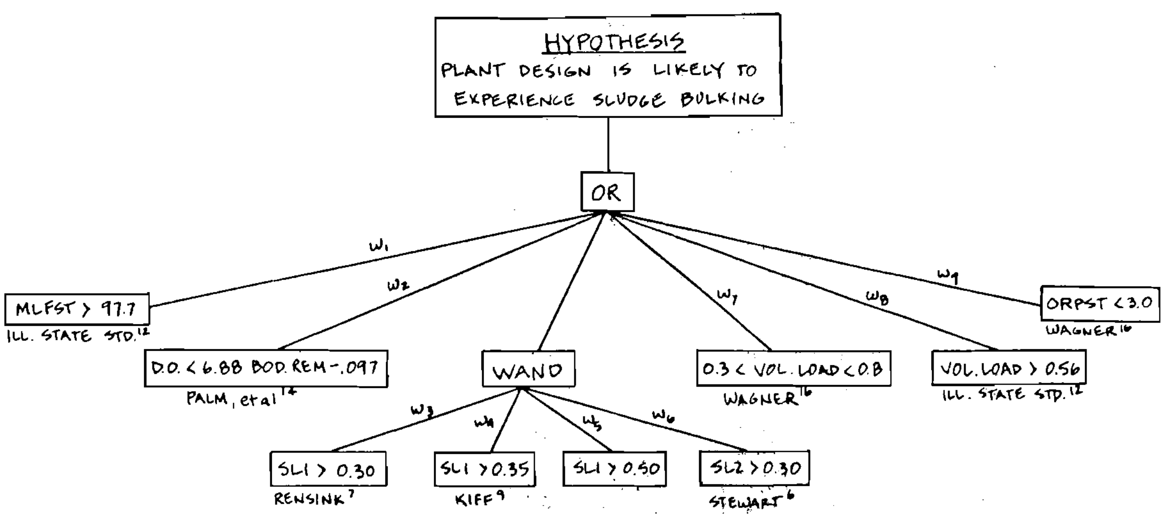



Figure 7--Hybrid 1 Rule Structure

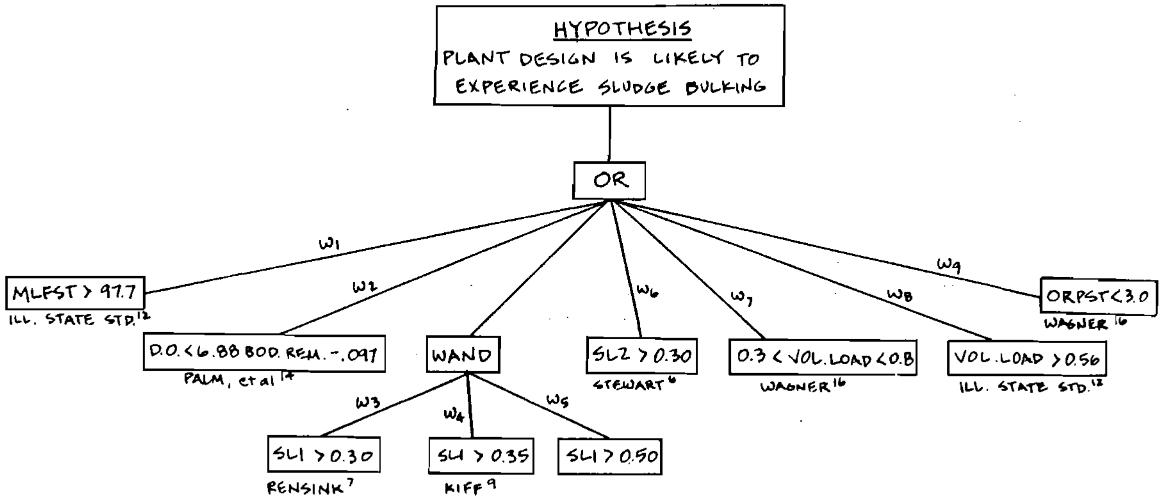



Figure 8--Hybrid 2 Rule Structure

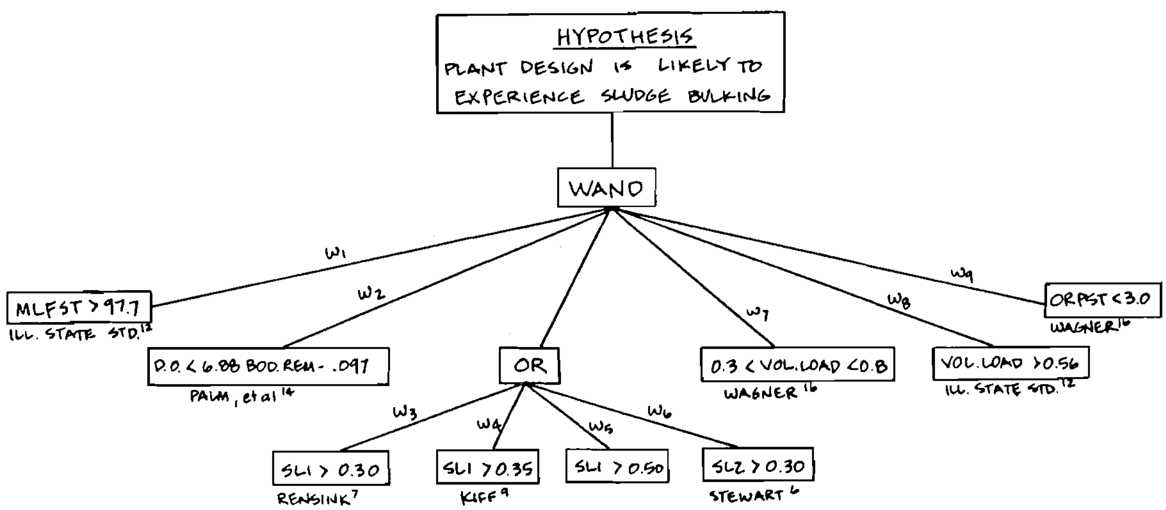



Figure 9--Hybrid 3 Rule Structure

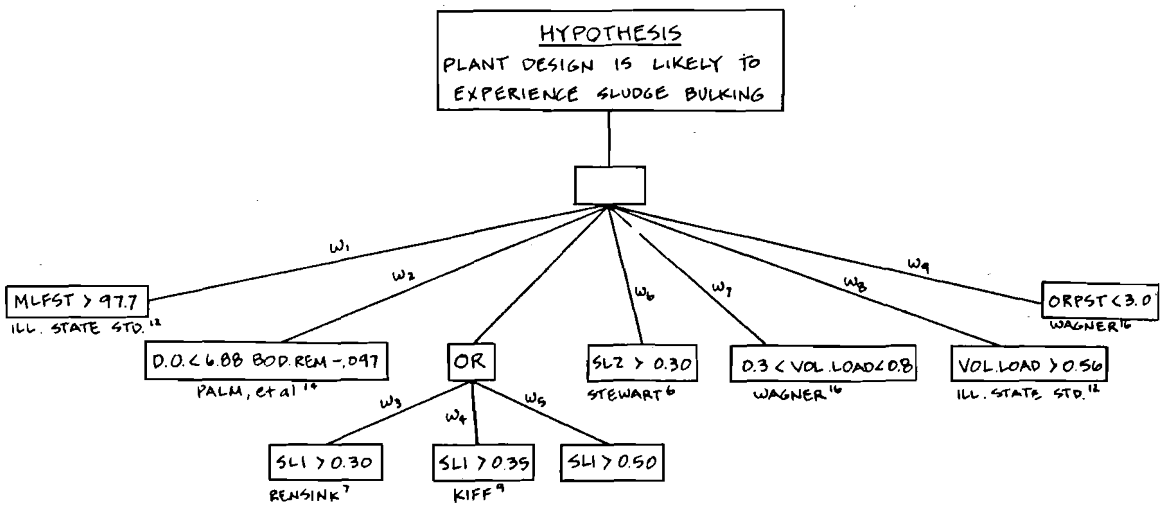



Figure 10--Rule Structures for Hybrids 4, 5, and 6

| Table 9<br>Weights of Association for Best Fit | | | | | | |
|---|---|---|---|---|---|---|
| | OR | OR mga | 2OR | 2OR mga | 3OR | 3OR mga |
| Var | .00936 | .01032 | .00633 | .00698 | .00766 | .00843 |
| Mean | .0066 | .144 | -.075 | -.010 | -.093 | -.102 |
| MLFST>97.7 | .517 | .517 | .819 | .748 | .747 | .734 |
| DO<6.88*BODR-.097 | .218 | .075 | 0.0 | 0.0 | .126 | .348 |
| SL1>0.3 | .593 | .418 | .847 | .748 | .838 | .902 |
| SL1>0.35 | .877 | .730 | 1.0 | 1.0 | 1.0 | 1.0 |
| SL1>0.5 | .943 | .779 | 1.0 | .933 | 1.0 | 1.0 |
| SL2>0.3 | .093 | 0.0 | .424 | .299 | .747 | .734 |
| 0.8>VL>0.3 | .043 | 0.0 | .474 | .378 | .609 | .461 |
| VL>0.56 | .731 | .588 | .819 | .748 | 1.0 | 1.0 |
| ORPST<3.0 | .093 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| diff | | .139 | | .049 | | .076 |
| | 4OR | 4OR mga | WAND | WAND mga | Hyb 1 | Hyb 2 |
| Var | .00992 | .0109 | .0333 | .0366 | .0132 | .0716 |
| Mean | -.116 | -.116 | .182 | .162 | .056 | -.35 |
| MLFST>97.7 | .671 | .853 | 1.0 | 1.0 | .614 | .614 |
| DO<6.88*BODR-.097 | .671 | .757 | 1.0 | .718 | .119 | .127 |
| SL1>0.3 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| SL1>0.35 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | .861 |
| SL1>0.5 | 1.0 | 1.0 | 1.0 | 0.0 | .575 | .267 |
| SL2>0.3 | .922 | .757 | 1.0 | 1.0 | 1.0 | 1.0 |
| 0.8>VL>0.3 | .648 | .670 | .320 | 1.0 | 0.0 | .483 |
| VL>0.56 | 1.0 | 1.0 | 1.0 | 1.0 | .681 | .900 |
| ORPST<3.0 | 0.0 | .395 | 0.0 | 0.0 | .159 | .300 |
| diff | | .224 | | 1.542 | | |
| | Hyb 3 | Hyb 4 | Hyb 5 | Hyb 6 | | |
| Var | .0545 | .0325 | .0087 | .0093 | | |
| Mean | .161 | .102 | -.109 | -.114 | | |
| MLFST>97.7 | .656 | .588 | .706 | .716 | | |
| DO<6.88*BODR-.097 | .663 | .704 | .034 | 0.0 | | |
| SL1>0.3 | 1.0 | .743 | .683 | .716 | | |
| SL1>0.35 | .861 | 1.0 | 1.0 | 1.0 | | |
| SL1>0.5 | .267 | 1.0 | 1.0 | 1.0 | | |
| SL2>0.3 | 1.0 | 1.0 | .889 | .445 | | |
| 0.8>VL>0.3 | .578 | .468 | .706 | .652 | | |
| VL>0.56 | 1.0 | 1.0 | 1.0 | 1.0 | | |
| ORPST<3.0 | 0.0 | 0.0 | 0.0 | 0.0 | | |

While a non-linear optimization code was used to find the "best" weight values, they may not be truly optimal points. In many cases, GRG terminates its search at local optima and it may take many different starting points to find what might be considered a close approximation to the global optimum. In each case presented in Table 9, three to five different starting points were tried, and the termination points are considered good.

From an analysis of the data presented in Table 9 it seems that there may be many logical structures which are acceptable. The **2OR** and **3OR** formulations give the minimum variances (0.00633 and 0.00766 respectively) and thus a standard deviation (square root of the variance) of the deviation from the expert's rating of $\pm$ 0.08 (8%). Six out of the 11 formulations could be fit to a standard deviation of less than $\pm$ 0.10 (10%). The **WAND** operator gives a very poor fit. This result could point to the existence of "model noise" which must be filtered out by considering only major associations to the conclusion.

At the same time, the "mga" solutions show that alternate weight combinations may be found for a given logical formulation, with only a minor impact on the model prediction. While, for example, most formulations give the rule concerning the ORPST a weight of 0.0 (meaning that the overflow rate of the primary clarifier being greater than 3.0 m/hr was not associated with the conclusion), it is possible to give the rule some weight without changing the model's performance significantly.

While the weights could be interpreted as the expert's opinion as to the truth values of the rules with respect to the conclusion that the plant would experience bulking problems, those weights are dependent on the logic structure and the set of rules which are being used. This is clearly shown by the different weighting values which are found for the different logic structures. This result points to the possible danger of assigning weights of association for rules without considering the interaction in the rule-base.

Using the models to evaluate the likelihood that a plant will experience bulking problems is straightforward. First, one of the rule structures presented in

Figures 6-10 is chosen. Next, the rule-branch truth values are obtained using equation (3.1); the truth values of the pieces of evidence evaluated based on the plant design parameters are multiplied by the weights presented in Table 9. Next, the appropriate propositional operation (e.g. averaging the two greatest rule-branch values) determines the overall likelihood. Finally, the result is adjusted by the mean deviation of the model's evaluation from the expert's evaluation of the design in the first survey (also given in Table 9). Such a procedure was implemented on an NEC PC-8801 personal computer in the BASIC language and was used to evaluate each of the plant designs in the survey. The results of those evaluations, along with the expert's evaluations, are presented in Table 10 and discussed in the following section.

## 3.4 CHECK OF CONSISTENCY

To check both the models' and the expert's consistency in judging plant designs, a second set of fifteen plant designs (shown in Table 11), satisfying the same influent conditions as the first set, were given to the expert. These plant designs, unlike the first set, had the common characteristic of being cost-optimal designs for different combinations of rule satisfaction which correspond to the range of possible likelihoods. Additionally, these plants all had dissolved oxygen levels of 1.5 mg/l. The expert's and each model's evaluation of each of the designs in the second set is presented in Table 12.

The **3OR** and **2OR** formulations accurately predict the expert's judgement for both the first and second set of designs when one considers the variance of the deviation from the expert's judgement. In both cases, however, they produce a judgement regarding the second set of designs that is on the average 0.1 units (10%) high. This deviation does not detract from the results since the rating scale is somewhat arbitrary and it is plausible that the expert's rating scale was shifted by 0.1 units from the first survey. What is more important, however, is the variance of the deviation. With a variance of 0.00358, the **3OR** model has predicted the expert's rating with a fairly consistent error and so the ordinal rankings were predicted quite well.

Table 10
Likelihood of Bulking for First Set of Designs

| Design | Pfeffer 1 | OR | 2OR | 3OR | 4OR | WAND | Hyb 5 | Hyb 6 |
|---|---|---|---|---|---|---|---|---|
| 1 | .05 | .050 | .162 | .110 | .046 | .218 | .126 | .212 |
| 11 | .10 | .100 | .137 | .156 | .114 | .293 | .187 | .108 |
| 5 | .10 | .225 | .137 | .198 | .282 | .404 | .198 | .108 |
| 15 | .15 | .225 | .162 | .152 | .214 | .329 | .137 | .212 |
| 14 | .20 | .225 | .162 | .152 | .214 | .329 | .137 | .212 |
| 10 | .45 | .225 | .374 | .401 | .444 | .440 | .434 | .434 |
| 4 | .60 | .600 | .586 | .638 | .526 | .440 | .650 | .570 |
| 12 | .60 | .738 | .758 | .769 | .782 | .773 | .756 | .744 |
| 13 | .65 | .738 | .744 | .738 | .700 | .662 | .756 | .744 |
| 2 | .80 | .738 | .758 | .769 | .782 | .626 | .748 | .744 |
| 3 | .85 | .884 | .849 | .853 | .864 | .662 | .854 | .886 |
| 6 | .90 | .738 | .744 | .738 | .700 | .626 | .756 | .744 |
| 8 | .90 | .884 | .849 | .769 | .782 | .662 | .756 | .712 |
| 7 | .90 | .884 | .849 | .853 | .864 | .849 | .854 | .886 |
| 9 | .95 | .950 | .925 | .907 | .884 | .884 | .854 | .886 |
| Mean dev | | -.0003 | .0003 | -.0002 | .0001 | .0002 | 0.0 | 0.0 |
| Mean \|dev\| | | .064 | .060 | .068 | .070 | .155 | .077 | .070 |
| Var. | | .00936 | .00633 | .00766 | .00992 | .0333 | .00865 | .00925 |

Table 11
Designs for Survey #2

| Design # | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ORPST | 3.0 | 6.0 | 6.0 | 3.0 | 3.0 |
| Sludge Age (days) | 6.0 | 2.942 | 4.729 | 5.663 | 2.225 |
| Recycle (%) | 10.0 | 12.23 | 10.0 | 10.58 | 13.21 |
| Vol AT ($m^3$) | 13,510 | 7,309 | 13,369 | 13,421 | 4838 |
| Area FST ($m^2$) | 1681 | 690 | 658 | 855 | 703 |
| MLVSS (mg/l) | 1204 | 1151 | 1005 | 1111 | 1216 |
| MLSS (mg/l) | 1685 | 1600 | 1399 | 1530 | 1675 |
| BOD rem | 0.232 | 0.419 | 0.273 | 0.250 | 0.567 |
| SL 1 | 0.178 | 0.35 | 0.214 | 0.196 | 0.509 |
| SL 2 | 0.249 | 0.486 | 0.299 | 0.270 | 0.702 |
| Vol Load | 0.299 | 0.56 | 0.30 | 0.30 | 0.854 |
| ML FST | 39.91 | 94.28 | 84.71 | 71.53 | 97.70 |
| Design # | 6 | 7 | 8 | 9 | 10 |
| ORPST | 3.0 | 6.0 | 3.0 | 6.0 | 6.0 |
| Sludge Age (days) | 2.248 | 5.895 | 2.226 | 2.195 | 3.456 |
| Recycle (%) | 12.32 | 13.136 | 13.21 | 13.13 | 13.12 |
| Vol AT ($m^3$) | 5120 | 13,775 | 4833 | 5159 | 8,209 |
| Area FST ($m^2$) | 690 | 703 | 703 | 703 | 703 |
| MLVSS (mg/l) | 1162 | 1200 | 1217 | 1201 | 1205 |
| MLSS (mg/l) | 1600 | 1675 | 1675 | 1675 | 1675 |
| BOD rem | 0.558 | 0.232 | 0.566 | 0.535 | 0.368 |
| SL 1 | 0.50 | 0.179 | 0.510 | 0.477 | 0.300 |
| SL 2 | 0.689 | 0.250 | 0.702 | 0.666 | 0.417 |
| Vol Load | 0.80 | 0.299 | 0.854 | 0.800 | 0.503 |
| ML FST | 94.28 | 97.70 | 97.70 | 97.70 | 97.70 |
| Design # | 11 | 12 | 13 | 14 | 15 |
| ORPST | 3.0 | 3.0 | 6.0 | 3.0 | 6.0 |
| Sludge Age (days) | 3.737 | 3.176 | 2.193 | 5.188 | 4.847 |
| Recycle (%) | 13.20 | 12.31 | 13.13 | 13.21 | 13.13 |
| Vol AT ($m^3$) | 8,212 | 7,314 | 5,152 | 11,298 | 11,427 |
| Area FST ($m^2$) | 703 | 690 | 703 | 703 | 703 |
| MLVSS (mg/l) | 1221 | 1166 | 1201 | 1218 | 1203 |
| MLSS (mg/l) | 1675 | 1600 | 1675 | 1675 | 1675 |
| BOD rem | 0.367 | 0.419 | 0.536 | 0.277 | 0.275 |
| SL 1 | 0.30 | 0.35 | 0.478 | 0.218 | 0.215 |
| SL 2 | 0.412 | 0.480 | 0.667 | 0.30 | 0.30 |
| Vol Load | 0.503 | 0.56 | 0.801 | 0.365 | 0.361 |
| ML FST | 97.70 | 94.28 | 97.7 | 97.70 | 97.70 |

**Notes:**
ORPST : Overflow Rate, Primary Settling Tank, m/hr
BOD rem : $BOD_5$ removal in Aeration Tank, kg $BOD_5$/kg MLVSS•day
SL 1 : Aeration Tank Sludge Loading, kg $BOD_5$/kg MLSS•day
SL 2 : Aeration Tank Sludge Loading, kg $BOD_5$/kg MLVSS•day
Vol Load : Aeration Tank Volumetric Loading, kg $BOD_5/m^3$•day
ML FST : Mass Loading, Final Settling Tank, kg MLSS/$m^2$•day
All designs have an aeration tank D.O. conc = 1.5 mg/l

Table 12
Likelihood of Bulking for Second Set of Designs

| Design | Pfeffer 2 | OR | 2OR | 3OR | 4OR | WAND | Hyb 6 | Hyb 5 |
|---|---|---|---|---|---|---|---|---|
| 7 | .05 | .007 | -.075 | -.093 | -.116 | .182 | -.114 | -.109 |
| 1 | .05 | .007 | -.075 | -.093 | -.116 | .182 | -.114 | -.109 |
| 3 | .10 | .225 | -.075 | -.051 | .052 | .293 | -.114 | -.098 |
| 4 | .10 | .225 | -.075 | -.051 | .052 | .293 | -.114 | -.098 |
| 15 | .20 | .225 | .162 | .152 | .214 | .329 | .212 | .137 |
| 14 | .20 | .225 | .162 | .152 | .214 | .329 | .212 | .137 |
| 11 | .57 | .225 | .374 | .401 | .444 | .440 | .434 | .434 |
| 10 | .60 | .225 | .374 | .401 | .444 | .440 | .434 | .434 |
| 12 | .65 | .600 | .586 | .638 | .694 | .551 | .570 | .650 |
| 2 | .70 | .600 | .586 | .638 | .694 | .551 | .570 | .650 |
| 13 | .90 | .884 | .849 | .853 | .864 | .738 | .886 | .853 |
| 9 | .90 | .884 | .849 | .853 | .864 | .738 | .886 | .853 |
| 6 | .92 | .884 | .849 | .853 | .864 | .738 | .886 | .853 |
| 8 | .95 | .950 | .925 | .907 | .884 | .849 | .886 | .853 |
| 5 | .95 | .950 | .925 | .907 | .884 | .849 | .886 | .853 |
| Mean dev | | .0483 | .0999 | .0915 | .0603 | .0225 | .0956 | .1031 |
| Mean \|dev\| | | .0883 | .0999 | .0915 | .0699 | .1436 | .0988 | .1031 |
| Var. | | .01966 | .00455 | .00358 | .00445 | .0225 | .00626 | .00383 |

The prediction of the different models for the designs of the first survey (Table 10), and the same information for the designs of the second survey (Table 12) show the abilities of the models to rank the designs as compared with the expert's rankings. Since the rating scale is arbitrary, the proper ranking rather than an actual rating of the likelihood of bulking might be the proper objective of this work. Both Table 10 and Table 12 show several cases where the models do not indicate differences between designs which the expert felt to be different. This is caused by the limited number of rules that were used in the rule base. For example, designs 14 and 15 of the first survey have different volumetric loadings (0.42 and 0.36 $kg/m^3 \cdot day$ respectively), but are the same with respect to the volumetric loading rules used. The problem could probably be alleviated by the use of more rules, but the ranking error is at most two positions and does not appear serious.

# CHAPTER 4

## EXPLORATION OF THE MODEL RESULTS

Tang's wastewater treatment plant design optimization model does not include a means of determining the potential for activated sludge bulking problems. If the judgement model developed in the previous section is incorporated into the optimization model, a tradeoff between optimal cost and the likelihood of experiencing bulking problems can be obtained for a given set of design conditions.

A rule-based system utilizing fuzzy logic can deal with not only the fuzzy rules relating a piece of evidence to a conclusion, but also with the fuzziness of whether a piece of evidence has or has not been satisfied. For example, the engineer might design for a sludge loading of 0.3 kg $BOD_5$/MLSS•day. Whether that loading is actually maintained depends on the variation in the influent $BOD_5$, the MLSS concentration maintained in the aeration tank, and the operator's response to changing plant conditions. For this problem, however, the fuzziness of whether the evidence regarding the design of the plant was not considered.

If only absolutely yes or no truths of the rules are given, the judgement model necessarily yields discrete levels of the likelihood of bulking. Each likelihood value can be generated as one of many possible combinations of rule satisfaction. For example, for the **2OR** logic structure there are 18 possible values of the likelihood of bulking. Presented in Table 13 are four combinations of rule satisfaction which will give a likelihood of 0.3345:

Table 13
Rule Combinations

| Rule | Truth Value | | | |
|---|---|---|---|---|
| MLFST>97.7 | 1 | 1 | 0 | 0 |
| D.O.<6.88*BOD.REM-.097 | 0 | 0 | 0 | 0 |
| SL1>0.3 | 0 | 0 | 0 | 0 |
| SL1>0.35 | 0 | 0 | 0 | 0 |
| SL1>0.5 | 0 | 0 | 0 | 0 |
| SL2>0.3 | 0 | 0 | 0 | 0 |
| 0.3<VolLoad<0.8 | 0 | 0 | 0 | 0 |
| VolLoad>0.56 | 0 | 0 | 1 | 1 |
| ORPST<3.0 | 0 | 1 | 0 | 1 |

Since Tang's model has 9 degrees of freedom (there are 64 variables and 55 equality constraints), it is possible to think of the cost function as a surface in a

9-dimensional space. The 9 rules related to bulking would be multi-dimensional borders of that space which define regions of equal likelihood of bulking. In a 9 rule structure, there are 512 possible rule combinations. Many of those combinations, however, may be infeasible and many of the regions delineated by the rules may have the same likelihood of bulking. In the case shown in Table 13, the four regions yield the same likelihood.

During the optimization process, the GRG algorithm determines the 9-dimensional slope of the surface and searches for the low point in the constrained region. Determining the cost-optimal design for a given likelihood of bulking involves searching the feasible regions which have that likelihood, and finding the region that has the lowest cost. It typically takes from 5 to 10 CPU minutes, depending on the feasibility and quality of the initial starting point, to solve the constrained design optimization program on the Harris 800 computer.

Table 14 gives the optimal design for the 11 feasible values for the **2OR** formulation, and Table 15 gives the optimal design for the 15 feasible values for the **3OR** formulation. In both cases the cost-optimal solution without bulking constraints has a high likelihood of bulking (about 0.85 in each case). Optimal designs with a high likelihood of bulking differ from those with a low likelihood of bulking mainly in the size of the aeration tank. Plant designs with a low likelihood of bulking are more expensive because they have a larger aeration tank.

Figure 11 shows a plot of the likelihood of bulking and the optimal cost for the two formulations. Note that while the general trend is that there is an increase in cost associated with decreasing the likelihood of bulking, this does not hold when comparing discrete values of the likelihood. Looking at the **3OR** formulation, it is not desirable to design for a likelihood of bulking of 0.2823 (point A in Figure 11) since one can decrease both the likelihood and the cost by designing for a plant with a likelihood of 0.152 (point B in Figure 11). The stepwise decreasing lines are representations of the non-inferior set for both formulations.

Table 14
2OR Formulation--Cost Optimal Designs

| | | | | | | |
|---|---|---|---|---|---|---|
| Likelihood | -.075 | .137 | .162 | .3345 | .374 | |
| Cost ($/yr) | 547,136 | 548,662 | 542,696 | 555,646 | 522,494 | |
| Eff.$BOD_5$ (mg/l) | 18.68 | 18.65 | 18.46 | 16.91 | 22.04 | |
| Eff.TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 | |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | |
| Sludge Age (days) | 4.729 | 4.706 | 4.847 | 5.881 | 3.456 | |
| Recycle (%) | 10.0 | 10.0 | 13.13 | 13.14 | 13.12 | |
| VAT ($m^3$) | 13,369 | 13,390 | 11,427 | 13,750 | 8209 | |
| AFST ($m^2$) | 658 | 658 | 703 | 703 | 703 | |
| MLVSS (mg/l) | 1005 | 1000 | 1203 | 1200 | 1205 | |
| MLSS (mg/l) | 1399 | 1400 | 1675 | 1675 | 1675 | |
| BOD Rem. | 0.273 | .274 | .275 | .233 | .368 | |
| SL 1 | .214 | .214 | .215 | .179 | .300 | |
| SL 2 | .299 | .30 | .30 | .250 | .417 | |
| Vol Load | .300 | .300 | .361 | .300 | .503 | |
| MLFST | 84.71 | 84.78 | 97.7 | 97.7 | 97.7 | |
| Likelihood | .5715 | .5855 | .744 | .758 | .8485 | .925 |
| Cost ($/yr) | 522,360 | 514,210 | 522,474 | 514,089 | 498,227 | 499,535 |
| Eff. $BOD_5$ (mg/l) | 21.90 | 24.32 | 21.77 | 24.195 | 30.0 | 30.0 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 5.797 |
| Sludge Age (days) | 3.492 | 2.942 | 3.529 | 2.964 | 2.193 | 2.194 |
| Recycle (%) | 14.32 | 12.23 | 15.50 | 13.126 | 13.640 | 17.994 |
| Vol. AT ($m^3$) | 7837 | 7309 | 7523 | 7036 | 5027 | 4196 |
| Area FST ($m^2$) | 720 | 690 | 737 | 703 | 710 | 774 |
| MLVSS (mg/l) | 1276 | 1151 | 1343 | 1205 | 1231 | 1471 |
| MLSS (mg/l) | 1774 | 1600 | 1867 | 1676 | 1717 | 2052 |
| BOD Rem. | .369 | .418 | .369 | .419 | .538 | .560 |
| SL 1 | .300 | .350 | .300 | .350 | .480 | .500 |
| SL 2 | .417 | .486 | .417 | .487 | .670 | .697 |
| Vol Load | .532 | .56 | .56 | .586 | .825 | 1.026 |
| MLFST | 102.01 | 94.28 | 105.95 | 97.71 | 99.56 | 113.38 |

**Notes:**
ORPST: Overflow Rate, Primary Settling Tank, m/hr
BOD Rem.: $BOD_5$ Removal rate in Aeration Tank, kg $BOD_5$/kg MLVSS·day
SL1: Sludge Loading, Aeration Tank, kg $BOD_5$/kg MLSS·day
SL2: Sludge Loading, Aeration Tank, kg $BOD_5$/kg MLVSS·day
Vol Load: Aeration Tank Volumetric Loading, kg $BOD_5/m^3$·day
MLFST: Mass Loading Final Settling Tank, kg $MLSS/m^2$·day

Table 15
3OR Formulation--Cost-Optimal Designs

| | | | | | |
|---|---|---|---|---|---|
| Likelihood | -.093 | -.051 | .11 | .152 | .156 |
| Cost ($/yr) | 555,807 | 547,136 | 564,580 | 542,696 | 555,794 |
| Eff. $BOD_5$ (mg/l) | 16.89 | 18.68 | 16.89 | 18.46 | 16.88 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |
| Sludge Age (days) | 5.895 | 4.729 | 5.865 | 4.847 | 5.898 |
| Recycle (%) | 13.14 | 10.0 | 13.13 | 13.13 | 13.19 |
| Vol. AT ($m^3$) | 13,775 | 13,369 | 13,781 | 11,427 | 13,756 |
| Area FST ($m^2$) | 703 | 658 | 703 | 703 | 703 |
| MLVSS (mg/l) | 1200 | 1005 | 1200 | 1203 | 1203 |
| MLSS (mg/l) | 1675 | 1399 | 1676 | 1675 | 1680 |
| BOD Rem. | .232 | .273 | .232 | .275 | .232 |
| SL 1 | .179 | .214 | .179 | .215 | .179 |
| SL 2 | .250 | .299 | .250 | .300 | .250 |
| Vol Load | .299 | .300 | .300 | .361 | .300 |
| MLFST | 97.70 | 84.71 | 97.70 | 97.70 | 97.91 |
| Likelihood | .198 | .2823 | .5313 | .608 | .6383 |
| Cost ($/yr) | 548,662 | 588,782 | 530,722 | 522,360 | 514,195 |
| Eff. $BOD_5$ (mg/l) | 18.65 | 14.31 | 20.60 | 21.90 | 24.32 |
| Eff. TSS (mg/l) | 30.0 | 22.34 | 30.0 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |
| Sludge Age (days) | 4.706 | 5.939 | 3.899 | 3.492 | 2.941 |
| Recycle (%) | 10.0 | 36.97 | 27.49 | 14.32 | 12.23 |
| Vol AT ($m^3$) | 13,390 | 6246 | 5813 | 7837 | 7310 |
| Area FST ($m^2$) | 658 | 1894 | 919 | 720 | 690 |
| MLVSS (mg/l) | 1000 | 2667 | 1918 | 1276 | 1151 |
| MLSS (mg/l) | 1400 | 3728 | 2667 | 1774 | 1600 |
| BOD Rem. | .274 | .279 | .374 | .369 | .419 |
| SL 1 | .214 | .215 | .300 | .300 | .350 |
| SL 2 | .300 | .300 | .417 | .417 | .487 |
| Vol Load | .300 | .800 | .800 | .532 | .560 |
| MLFST | 84.78 | 97.70 | 134.04 | 102.01 | 94.28 |
| Likelihood | .6843 | .7383 | .7687 | .853 | .907 |
| Cost ($/yr) | 516,562 | 522,474 | 507,795 | 498,227 | 499,535 |
| Eff. $BOD_5$ (mg/l) | 23.51 | 21.77 | 26.93 | 30.0 | 30.0 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 30.0 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 6.0 | 5.797 |
| Sludge Age (days) | 3.103 | 3.529 | 2.531 | 2.193 | 2.194 |
| Recycle (%) | 13.12 | 15.50 | 10.0 | 13.64 | 17.99 |
| Vol. AT ($m^3$) | 7367 | 7523 | 7165 | 5027 | 4196 |
| Area FST ($m^2$) | 703 | 737 | 658 | 710 | 774 |
| MLVSS (mg/l) | 1206 | 1343 | 1006 | 1231 | 1471 |
| MLSS (mg/l) | 1675 | 1867 | 1400 | 1717 | 2052 |
| BOD Rem. | .403 | .369 | .465 | .538 | .560 |
| SL 1 | .334 | .300 | .400 | .480 | .500 |
| SL 2 | .464 | .417 | .557 | .670 | .697 |
| Vol Load | .560 | .560 | .560 | .825 | 1.026 |
| MLFST | 97.7 | 105.95 | 84.76 | 99.56 | 113.38 |

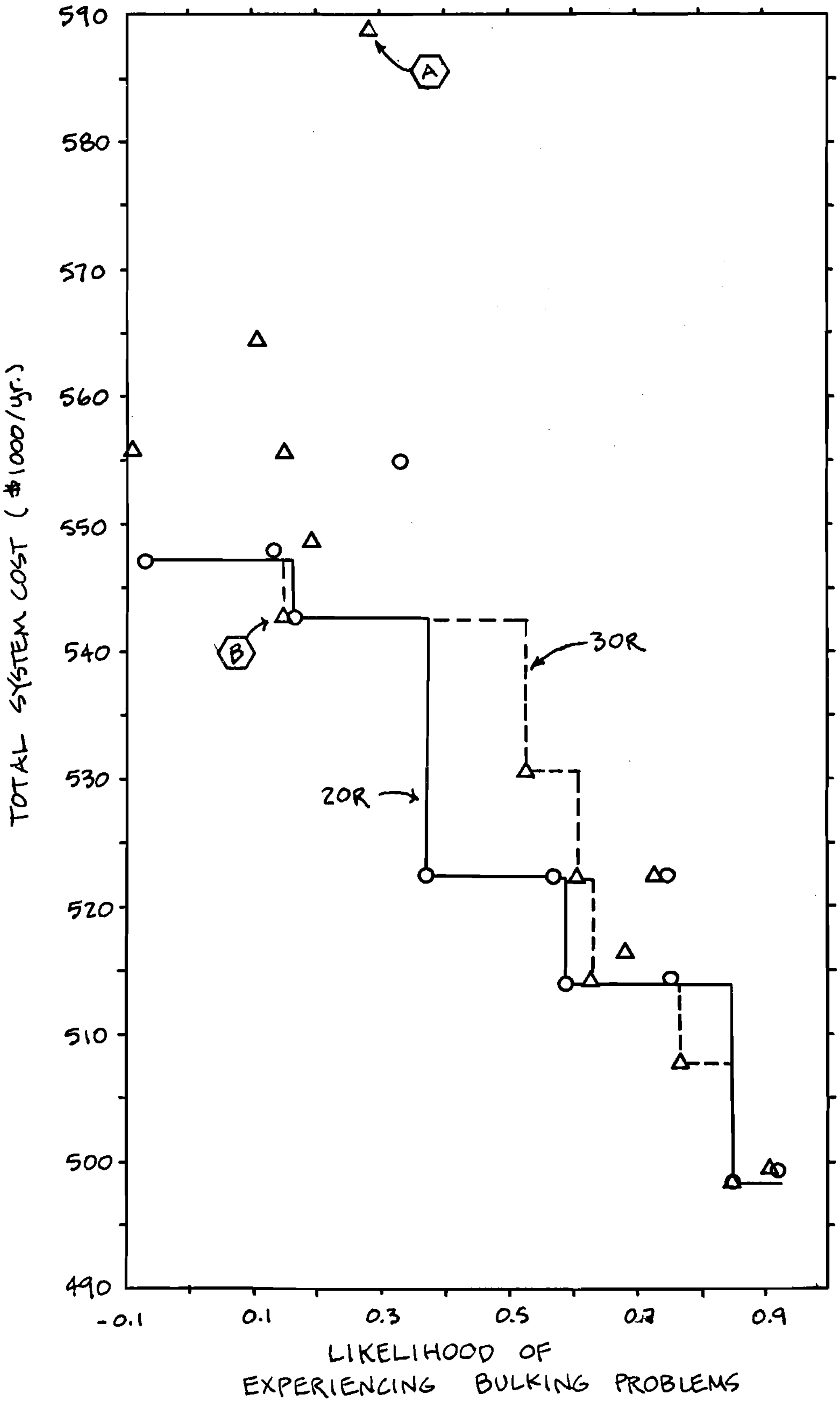



Figure 11--Approximation of the Non-Inferior Set

It is important to consider what such a tradeoff represents. The optimization model considers thickening in the final clarifier as a function of the SVI (which is held constant at 100) and considers clarification to be independent of the sludge settleability. The design shown for a low likelihood of bulking was determined with the same SVI as was used for the design which exhibits a high likelihood of bulking. Since the SVI is a measure of sludge settleability, one would expect a lower SVI for plants with a low likelihood of bulking and thus the cost of such plants should be lower than what the model predicts (better settling and less recycle pumping needed). Conversely, plants with a high likelihood of bulking can be expected to have a higher cost than what the model predicts.

However, the settleability of the activated sludge at any plant does not remain constant. Rather it varies with changes in the influent and plant conditions. The optimization model, being a steady-state model, cannot capture such variation. Instead, it examines what might be considered as average operating conditions at the plant. The plant is optimally designed for such conditions and the likelihood of bulking is an indication of the amount of time that the plant is operating at conditions significantly different than what is assumed.

Exploring the range of designs which are assigned the same likelihood of bulking is useful in two ways. First, it is interesting to consider the range of alternatives available to the designer restricted to one level of the likelihood of bulking. Second, it may lend insights into how the model can be refined. Assuming an acceptable likelihood of about 0.15, one can see from Tables 14 and 15 that both the **2OR** and **3OR** formulations give the same design. It should be noted that at that likelihood, a discrepency between the expert's judgement and the model exists. Table 11 shows that the expert saw a difference in designs 14 and 15 to which the models assigned the same likelihood. If the model is to be fine tuned so that it may determine the same differences as the expert, designs which are sufficiently different, but still constrained to the same likelihood of bulking, should be examined.

Table 16 shows alternative designs subject to an allowable increase in cost of

Table 16
Alternative Designs for Likelihood = .15

| | Optimal | Alt 1. | Alt 2. | Alt 3. | Alt 4. |
|---|---|---|---|---|---|
| Cost ($/yr) | 542,696 | 548,120 | 548,120 | 548,120 | 548,120 |
| Eff. $BOD_5$ (mg/l) | 18.46 | 18.46 | 17.07 | 18.30 | 18.71 |
| Eff. TSS (mg/l) | 30.0 | 30.0 | 26.62 | 30.0 | 30.0 |
| ORPST (m/hr) | 6.0 | 6.0 | 6.0 | 3.74 | 6.0 |
| Sludge Age (days) | 4.847 | 4.839 | 4.992 | 5.068 | 4.699 |
| Recycle (%) | 13.13 | 10.30 | 16.33 | 13.18 | 10.00 |
| Vol AT ($m^3$) | 11,427 | 13,415 | 9856 | 11,344 | 13,313 |
| Area FST ($m^2$) | 703 | 662 | 863 | 703 | 658 |
| MLVSS (mg/l) | 1203 | 1024 | 1435 | 1212 | 1005 |
| MLSS (mg/l) | 1675 | 1427 | 2000 | 1675 | 1399 |
| BOD Rem. | .275 | .268 | .276 | .276 | .274 |
| SL 1 | .215 | .210 | .215 | .217 | .215 |
| SL 2 | .30 | .293 | .30 | .30 | .30 |
| Vol Load. | .361 | .300 | .430 | .364 | .301 |
| MLFST | 97.7 | 86.1 | 97.7 | 97.7 | 84.7 |

**Notes:**
ORPST: Overflow Rate, Primary Settling Tank, m/hr
BOD Rem: $BOD_5$ Removal Rate, Aeration Tank, kg $BOD_5$/kg MLVSS•day
SL1: Sludge Loading, Aeration Tank, kg $BOD_5$/kg MLSS•day
SL2: Sludge Loading, Aeration Tank, kg $BOD_5$/kg MLVSS•day
Vol Load: Aeration Tank Volumetric Loading, kg $BOD_5/m^3$•day
MLFST: Mass Loading on the Final Settling Tank, kg $MLSS/m^2$•day

1% above the optimal value. Alternatives 1 and 2 represent design approaches different than a least cost approach. Alternative 1 was generated by finding the design with the maximum aeration tank volume, while alternative 2 was found by maximizing the size of the final clarifier. While both the flow rate and the solids concentration of the flow from the aeration tank to the final clarifier have increased, the larger clarifier size gives a better clarified effluent which exceeds the suspended solids standard. Such alternatives can be examined readily by a design engineer using models such as those used here.

# CHAPTER 5

## CONCLUSION

One purpose of integrating a model of judgement about a fuzzy problem into an exact mathematical optimization model is to learn more about the problem. Considering the tradeoff analysis presented for the example problem, a plant which will have a low likelihood of experiencing bulking problems could be built at only about a 10% increase in cost over the optimal design when bulking is not considered. On the other hand, the optimal design obtained without considering bulking would have a high likelihood of bulking. While the change in clarification efficiency with settleability has not been incorporated in the model, it seems that the most cost-efficient design strategy for lowering the potential for experiencing bulking problems is to design a larger aeration tank. While that conclusion is based on the specific influent conditions studied, the model developed here could prove to be a useful starting point for helping the design engineer identify cost-effective ways of reducing the likelihood of bulking for any influent conditions.

The results of this work should not be interpreted as the way to alleviate bulking problems when designing an activated sludge plant. Rather it is a representation of one engineer's opinion. As there are many differing opinions regarding the bulking problem, it might be expected that the same approach would give different results if used to model another experienced engineer's judgement. The approach is useful, however, in that the judgement model helps to screen the results of the optimization model so that the range of choice may be considered by the designer.

Another result that has come from this work is a feeling for the nature of the inter-relationship of rules and their association to a conclusion in a rule-based system. The weights of association seem to depend on the way in which the rules are logically combined. Methods which solicit weights of association independent of the logic structure in which they will be used or the use of weights in a structure expanded beyond that in which the weights were originally assigned should be cautiously evaluated.

While the calibrated rule-base presented here matches the expert's judgement fairly well, it can be improved. If more rules are incorporated into the rule base, it might be able to distinguish between designs in a manner closer to that of the expert. Besides increasing the number of breakpoints used for a specific variable, more process variables might be considered. For example, while the sludge loading rules implicitly considered the MLSS and MLVSS concentrations, it might be helpful to use rules which would link these variables directly to the conclusion. Additionally, mixing was not considered in the presentation of the designs to the expert or in the rule base. Values of dispersion number are generally not given for plants, and may not be closely controlled during the design process. Research has shown that mixing is very much associated with bulking problems and it should be incorporated.

A closer approximation of the non-inferior set of solutions and a clearer picture of the most cost-effective approach to reaching a specific likelihood of bulking can be found if the discrete nature of the rules is avoided. It might be worth investigating the use of a continuous association function as used by Soula and Sanchez[49] in setting up a model for medical diagnosis. Rather than using variable ranges to formulate rules, a generic function (possibly linear) could specify the association given the variable value. The attributes of the function (e.g. the slope and the y-intercept) could be found in a manner similar to that followed in the work here.

The problem that was considered in this work dealt only with plant designs for normal domestic wastewater. The larger problem of identifying if a plant, considering both the design and the influent conditions, will experience bulking problems should be pursued. A complete model of the judgement used in evaluating the problem shown in Figure 5 might be a powerful tool for diagnosing and predicting the potential for bulking problems at a specific plant. Then, in combination with an optimization model, the most cost-effective remedy to the problem may be found.

The ability to manipulate rule-bases efficiently is a powerful tool which the environmental engineer could use to help insure the proper operation of the plant he designs. Formalizing expert fault diagnosis techniques into a rule-based system could tremendously aid plant operators determine possible sources and solutions to problems which arise in their plants An expert system could make the information currently available in a plant operation and maintenance manual quickly accessible by giving the operator only information which might be relevant to the problem at hand. The work presented here is one step toward such a goal.

The rule-based approach may, of course, be helpful for other types of pollution control problems. It may be especially important to develop such models to assist in the design or management of environmental systems where the number of experts is limited. It may also be desirable to incorporate results from rule-based analyses into other optimization models, as illustrated by this work.

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