APPROPRIATE TECHNOLOGY FOR WASTEWATER TREATMENT IN COASTAL TOURIST AREAS

Hallvard Ødegaard

The Norwegian Institute of Technology, N-7034 Trondheim-NTH, Norway

ABSTRACT

The composition of wastewater from tourist areas is discussed. Since such a significant part of the contaminants in wastewater is associated with particles, a good particle removal should be the first step in the treatment of wastewater. In coastal areas it is recommended that coagulation with lime should be used. Special attention is drawn to the lime/seawater process where 2-5 vol-% of seawater is added to the influent resulting in several process improvements. For low-cost treatment the use of the lime/seawater process in a pond system is discussed.

KEYWORDS

Wastewater treatment, coastal tourist areas, lime coagulation.

INTRODUCTION

The choice of technology to be used for treatment of wastewater from coastal tourist areas should be based upon:

a. Effluent quality requirements before discharge to the sea.
b. Characteristics of wastewater from tourist areas.
c. Suitability of treatment method in relation to a) and b).

It is beyond the scope of this paper to discuss the various effluent criteria one may meet for discharge to coastal waters in various locations around the world. Generally, however, the effluent criteria are less stringent that those for discharge to inland waterways. In Norway one typically aims at a good removal of suspended solids, a moderately high removal of organic matter, removal of nutrients if the recipient is a eutrophic fjord and preferably a certain hygienization of the effluent if discharged to publically much used coastal zones, like beaches.

The characteristics of wastewater coming from a tourist area will of course depend upon several factors, like:

a. Standard of tourist establishment (hotel, camping etc.).
b. Geographical location of the establishment.
c. Other activities besides tourism in the area.

Generally it may be stated, however, that the most characteristic feature about flow and composition of wastewater from tourist areas, is that it varies a lot.

It varies both over the season and the day, it varies with the standard of the establishment and with the type of activity which takes place within the area. Our experience is, however, that wastewater from heavily exploited tourist areas is like municipal wastewater, only being:
- more concentrated in terms of typical pollutant parameters
- more variable in terms of flow variation and contaminant variation
- less disintegrated in terms of particulate matter as a result of a short sewerage system.

What we are looking for, therefore, is a technology which:
- removes particles well and organic matter pretty well
- improves the hygienic property of the wastewater significantly
- can stand seasonal and daily variations both in flow and wastewater composition well
- is easy to operate and economically acceptable.

In Norway we think that chemical coagulation plants, especially with lime as the coagulant, are particularly appropriate for this situation, and this will be discussed in this paper. It will, however, be convenient first to discuss the occurrence of contaminants in wastewater because this gives the basis for determining the technology to be used.

CLASSIFICATION OF CONTAMINANTS IN WASTEWATER

Surprisingly enough, very few studies have been performed in order to characterize the contaminants in municipal wastewater in terms of the particle size that they appear in. Very early, however, Imhoff (1939) presented in his classic pocket-book, tables showing the distribution between settleable and not-settleable suspended matter and soluble matter in wastewater, demonstrating that more than half of the organic matter was associated with the suspended solids.

In some early American studies (Balmat 1957, Heukelekian and Balmat 1959 and Richert and Hunter 1971), the organic contaminants in wastewater were separated into four size fractions by successive sedimentation, centrifugation and filtration. The fractions were classified by size range as settleable, supracolloidal, colloidal and soluble (table 1).

<table>
<thead>
<tr>
<th>Classification</th>
<th>Soluble</th>
<th>Colloidal</th>
<th>Supra-colloidal</th>
<th>Settleable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size range</td>
<td>&lt;0.08 μm</td>
<td>0.08-1.0 μm</td>
<td>1-100 μm</td>
<td>&gt;100 μm</td>
</tr>
<tr>
<td>COD ( % of total)</td>
<td>25</td>
<td>15</td>
<td>26</td>
<td>34</td>
</tr>
<tr>
<td>TOC ( %)</td>
<td>24</td>
<td>14</td>
<td>24</td>
<td>31</td>
</tr>
<tr>
<td>Org. constit. ( % of tot. solids)</td>
<td>12</td>
<td>51</td>
<td>24</td>
<td>19</td>
</tr>
<tr>
<td>Grease</td>
<td>12</td>
<td>51</td>
<td>24</td>
<td>19</td>
</tr>
<tr>
<td>Protein</td>
<td>4</td>
<td>25</td>
<td>45</td>
<td>25</td>
</tr>
<tr>
<td>Carbohydrates</td>
<td>58</td>
<td>7</td>
<td>11</td>
<td>24</td>
</tr>
<tr>
<td>Biochemical oxid. rate, k, d⁻¹</td>
<td>0.39</td>
<td>0.22</td>
<td>0.09</td>
<td>0.08</td>
</tr>
</tbody>
</table>

It is demonstrated that only 25-30% of the COD/TOC was found to be truly soluble by the definition used. The soluble organic matter smaller than 1 μm, was found to degrade biochemically at a much more rapid rate than the organics in particles larger than 1 μm.

A more recent study by Munck et al. (1980) (table 2) confirmed the results of the earlier investigations, even though Munck used a slightly different definition of the size ranges and used only filtration for classification.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Soluble</th>
<th>Colloidal</th>
<th>Supra-colloidal</th>
<th>Settleable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size range</td>
<td>&lt;0.025 μm</td>
<td>0.025-3 μm</td>
<td>3-106 μm</td>
<td>&gt;106 μm</td>
</tr>
<tr>
<td>BOD ( % of total)</td>
<td>19</td>
<td>16</td>
<td>46</td>
<td>21</td>
</tr>
<tr>
<td>COD ( - &quot; - )</td>
<td>12</td>
<td>15</td>
<td>30</td>
<td>43</td>
</tr>
<tr>
<td>TOC ( - &quot; - )</td>
<td>22</td>
<td>6</td>
<td>36</td>
<td>36</td>
</tr>
<tr>
<td>Tot P ( - &quot; - )</td>
<td>63</td>
<td>3</td>
<td>12</td>
<td>22</td>
</tr>
<tr>
<td>Org. N ( - &quot; - )</td>
<td>27</td>
<td>15</td>
<td>38</td>
<td>20</td>
</tr>
</tbody>
</table>
As demonstrated in table 2, Munck et al. (1980), also investigated the distribution of total phosphorous and organic nitrogen. Even if most of the phosphorous in wastewater appeared as soluble phosphates, the amount of phosphorous associated with particles was found to be significant, and even more so was the amount of organic nitrogen.

Levine et al. (1985) performed a thorough study of particle size distribution in several wastewaters of different quality using several classification techniques, and concluded that the organic contaminants in municipal wastewater were effectively classified as either greater than or less than 0.1 \( \mu m \).

One should always analyze contaminants in and out of the different steps in a treatment plant both on unfiltered and filtered samples in order to evaluate whether the contaminants are associated with particles or not. In the day to day operation much information is gathered by using the traditional GF/C-filter (pore opening about 1 \( \mu m \)), but in optimization and choice of particle separation methods, one should consider using serial filtration based on a series of polycarbonate membrane filters.

From the big kitchens that one may find in tourist areas, the composition of wastewater may be expected to be similar to that of the food-processing industries. Ødegård (1985) showed that also in such wastewaters, 40-70 % of the total COD may be associated with the suspended solids (table 3).

<table>
<thead>
<tr>
<th>Type of industry</th>
<th>COD(_{\text{Total}}) g m(^{-3})</th>
<th>COD(_{\text{Filtered}}) g m(^{-3})</th>
<th>COD(_{\text{Suspended}})%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cheese processing</td>
<td>3036</td>
<td>1452</td>
<td>52</td>
</tr>
<tr>
<td>Potato processing</td>
<td>6735</td>
<td>1575</td>
<td>76</td>
</tr>
<tr>
<td>Slaughterhouse</td>
<td>1833</td>
<td>718</td>
<td>61</td>
</tr>
<tr>
<td>Vegetable processing</td>
<td>1588</td>
<td>977</td>
<td>47</td>
</tr>
<tr>
<td>Dairy</td>
<td>971</td>
<td>539</td>
<td>45</td>
</tr>
</tbody>
</table>

1) 1 \( \mu m \) glass fibre filter.

Also other contaminants such as metals, bacteria and viruses and organic micropollutants are strongly associated with particles. Table 4 shows that the percentage of total metal content associated to suspended matter in the raw water of a Swedish wastewater treatment plant, was in the range of 50-75 %, with the exception of nickel (SWEP, 1985).

<table>
<thead>
<tr>
<th>Metal</th>
<th>Zn</th>
<th>Cu</th>
<th>Ni</th>
<th>Cr</th>
<th>Pb</th>
<th>Cd</th>
</tr>
</thead>
<tbody>
<tr>
<td>%-suspended</td>
<td>51</td>
<td>48</td>
<td>71</td>
<td>71</td>
<td>82</td>
<td></td>
</tr>
</tbody>
</table>

Bacteria and viruses are particles in the size range of about 0.01-10 \( \mu m \), and even if some of the viruses are very small (0.01-0.1 \( \mu m \)), several investigations (Lewis et al. 1986, Wellings et al. 1976, Vasl and Kott 1982), have demonstrated that 60-100 % of the viruses in sewage are adsorbed to particles. It is also well known that some of the organic micropollutants as PCB and PAH have a high affinity to particulate matter (SWEP 1985, Luin et al. 1984).

Since such a significant part of the pollutants in wastewater appear as, or is associated with, particles, one would think it most rational to start sewage treatment with a thorough removal of particles. This is not, however, what we normally do. Normally we start directly with biological treatment, sometimes after presettling, which gives, however, a very modest particle removal. Let us therefore shortly consider the fate of particles in biological treatment.

**THE FATE OF PARTICLES IN BIOLOGICAL TREATMENT.**

Principally a biological treatment process consists of a bioreactor and a particle separation reactor. In the bioreactor soluble material (organics, ammonia etc.) is transferred from its soluble state to a particulate state, (as bacteria, protoza, algae, etc.), and the particles have to be separated from the water in the separation reactor. Much more effort has been performed to study the fate of the soluble matter than the fate of the particulate matter in the bioreactor.
Even if it can be demonstrated than an activated sludge process has a great ability to separate particles from the water by flocculation, it may be questioned whether it is economically sound to use the bioreactor as a captor of particles. Gujer (1980) demonstrated that particulate organic material in primary effluents is rapidly taken up into the activated sludge, but its degradation rate is very slow. And, as it was demonstrated in table 1 of this paper, the biochemical oxidation rate is lower the bigger the particles are. Barløs (1957). Likewise, there are strong indications in the work of Sørensen (1980, 1981, 1985) and Rusten (1983, 1984), that the removal of particulate and dissolved organics in biofilms interact, and that particle adsorption on biofilm surfaces decreases the removal rate of dissolved organics. Based on these findings, it seems that the particles should be removed ahead of the bioreactors, both in order to reduce the overall load and to promote rapid biodegradation.

In conclusion, therefore, it may be stated that based on the composition of wastewater, the most rational way of wastewater treatment would be first to remove particles thoroughly, for instance by coagulation, and thereafter, if necessary, use biological treatment to remove soluble compounds that can be biodegraded.

**PARTICLE SEPARATION FROM RAW WASTEWATER**

Particles may be separated from raw wastewater by straining, sedimentation, flotation and filtration, sedimentation being most commonly used. Sedimentation is, however, only effective for the removal of particles larger than about 50 μm on a practical basis. Based on Stokes' law and a practical overflow rate in the range of 1-2 m h⁻¹, the minimum particle size that would be removed by sedimentation is in the range of 50-70 μm assuming an average density of particles of 1.2 g m⁻³ and a temperature of 15°C. A major part of the particles larger than 0.1 μm may, however, be separated by sedimentation subsequent to coagulation/flocculation.

Flotation may be used to separate particles in a much broader size range than sedimentation. Flotation as such is seldom used for pretreatment, but excellent pretreatment results may be obtained by flotation subsequent to coagulation/flocculation (Ødegaard 1987). Filtration is also normally applied as the last separation step in a treatment plant, even though filtration through granular medium filters has been used with success both directly on primary effluent (Matsumoto 1982, Tchobanoglous 1983) and subsequent to coagulation of raw wastewater (Peterson 1985). In order, therefore, to be able to remove particles down to the size of 0.1 μm, coagulation has to be applied prior to the floc separation method in question.

**Coagulation of wastewater.**

The train of unit processes shown in fig. 1, is probably the most commonly used treatment in order to achieve good particle separation both in drinking water and wastewater treatment. When used in wastewater treatment, aluminium salts, iron salts or lime are used as coagulants resulting in phosphate precipitation as well as particle coagulation.

The process consists of a reaction step where particles are coagulated and solubles, like phosphates, are precipitated, followed by a flocculation step where the coagulated/precipitated particles are aggregated to separable flocs, to be removed by sedimentation or flotation and/or filtration. The floc separation process to be used, depends upon the particle separation requirement. In wastewater treatment filtration is normally not included.

**Fig. 1.** The coagulation/precipitation - flocculation - floc separation treatment process.
The reaction part of the process, the coagulation/precipitation, is very rapid and is terminated within seconds while the flocculation and floc separation processes are far slower. Investment cost is consequently almost entirely linked to the particle separation part of the process. Both the reaction part and the particle separation part of the total process are, however, equally important for the total treatment result. We shall therefore, shortly discuss the factors that govern optimization of particle removal by the different unit processes.

Optimization of the reaction step. The two most important factors influencing the reaction step are the dosage of coagulant and the pH of coagulation. Since all the commonly used coagulants in wastewater treatment influence the pH of the water upon addition, the alkalinity of the water is indirectly a very important parameter. When aluminium- and iron-salts are used, a minimum dosage is required to obtain the coagulation/precipitation aimed at. When lime is used, the dosage for effective precipitation is primarily a matter of pH and hence of the alkalinity of the water.

In many coagulation/precipitation plants with iron- or aluminium-coagulation, the alkalinity of the raw water has an indirect impact on the necessary dosage too, since it is common to dose more chemical than is theoretically needed to coagulate particles and precipitate phosphate, in order to bring pH down to the wanted level. The pH of coagulation/precipitation is consequently the most important process parameter. Typical dependencies between effluent suspended solids and pH for alum- and ferri-chloride-coagulation are shown as examples in fig. 2 (Ødegaard, 1982).

![Fig. 2. Suspended solids in effluent from a coagulation/flocculation/sedimentation system (jar-test) versus coagulation - pH (Ødegaard 1982).](image1)

The removal of organics will obviously not only be dependent upon dosage and pH, but also upon how much of the organics that are associated with particles. This is demonstrated in fig. 3 (Ødegaard 1982).

![Fig. 3. Treatment efficiency with respect to total COD in a coagulation/flocculation/ sedimentation-system (jar-test) versus coagulation-pH at different levels of raw water COD and total COD/soluble COD ratios (Ødegaard 1982).](image2)
The pH for optimal coagulation of particles is about the same as that for optimal phosphate removal, even though it seems that the pH-range for acceptable removal of COD by coagulation is a little broader than that for acceptable phosphate precipitation (Ødegård 1982, Balmer et al. 1975). Table 5 shows typical optimum pH-values and dosages found for wastewater coagulation.

<table>
<thead>
<tr>
<th>Coagulant</th>
<th>pH</th>
<th>Typical dosage, g m⁻³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al-sulphate</td>
<td>5.5-6.5</td>
<td>150-250</td>
</tr>
<tr>
<td>Fe⁺⁺⁺-chloride</td>
<td>5.0-6.0</td>
<td>200-300</td>
</tr>
<tr>
<td>Fe⁺⁺-sulphate + lime</td>
<td>8.0-9.0</td>
<td>150-250 + 100-150</td>
</tr>
<tr>
<td>Lime (Ca(OH)₂)</td>
<td>11.0-12.0</td>
<td>200-400</td>
</tr>
</tbody>
</table>

Since the reaction is so rapid, the initial mixing of chemicals has a bearing both on the optimization of the perikinetic flocculation and on the minimization of coagulant consumption. Several authors (Vrále et al. 1971, Stenquist et al. 1972, Ødegård 1975, Klute 1985, Francois et al. 1985) have demonstrated plug flow mixing devices to be more effective than complete mixing devices. The importance of optimal mixing seems to increase with decreasing particle concentration in the water to be coagulated.

Optimization of the flocculation step. The purpose of the flocculation part of the process, is to aggregate the tiny, newly coagulated/precipitated particles (the primary particles) into aggregates of such particles (flocs), which are separable by some kind of floc separation process (e.g. sedimentation, flotation or filtration). In other words, a high flocculation performance is wanted, flocculation performance being defined as the ratio between the concentration of primary particles going into (n₀) and out of (n) the flocculation reactor.

It is out of the scope of this paper to go into any detail about flocculation kinetics which, however, is thoroughly dealt with in the literature (Harris et al. 1966, Argaman et al. 1970, Parker et al. 1972, Ødegård 1979, 1985). Fig. 4 may, however, be used to explain how the different flocculation variables influence flocculation performance.

Fig. 4 demonstrates that the flocculation performance increases (at constant T and ϕ) with the intensity of mixing (expressed as G-value) up to a maximum point after which the performance decrease as G increases. This is well known from practice and is caused by the fact that collision frequency between flocs and particles increases with increasing mixing intensity up to a maximum where floc breakup begins to dominate the result. It is also demonstrated that performance increases with increasing mixing efficiency (number of compartments) at constant overall residence time, and that the G-value for optimum performance decreases when the number of compartments (mixing efficiency) for a given flow is increased.
Fig. 4b demonstrates that the performance increases with increasing residence time (at constant G and y) up to a certain residence time, beyond which further increase in residence time will give no increase in performance. This residence time for optimum performance decreases when the number of compartments (mixing efficiency) increases.

Fig. 4b also demonstrates that the higher the mixing efficiency (number of compartments), the lower overall residence time is needed in order to obtain a given performance. In other words, the more plug-flow-like the flow in a flocculator is, the shorter is the residence time needed to achieve optimum performance at a given velocity gradient.

Finally, fig. 4c demonstrates that the higher the floc volume fraction at a given G, T and m, the higher will the performance be. And the higher the floc volume fraction is, the shorter residence time is needed (at a constant G) to obtain a given performance.

Optimization of flocculation with respect to floc separation. A great many scientists have studied the optimization of flocculation performance independent of which floc separation process is to be used. This may, however, result in misleading conclusions. One simple definition of the purpose of flocculation might be: To treat the primary particles, formed in the reaction step, in such a way that they are most easily removed by the floc separation process in question. Then, it becomes quite obvious that the optimum flocculation conditions for, for instance sedimentation, are hardly the same as those for flotation or filtration.

When sedimentation is used for floc separation, the floc separation performance is primarily influenced by the size of the floc. For a given coagulant, the size of the floc is primarily influenced by the intensity of mixing. The lower the intensity is, the better is the settleability of the flocs. But as we have seen, we need a certain mixing intensity to optimize flocculation performance.

This leads us then to the practice of tapering the intensity of mixing towards the outlet of the flocculator. The intensity of mixing just ahead of the inlet to the settling tank should be as low as possible, and transport of flocs into the settling tank as open and smooth as possible in order to avoid floc breakup.

The flocculation/sedimentation performance may be improved considerably by the use of polymers as flocculation aids both because polymers may strengthen the flocs so they can withstand higher shearing forces and because they increase the size of the flocs (Øyegaard 1978, 1985, 1987).

When the flocs are to be separated by flotation, the flocculation variables should not be as they should in sedimentation. Øyegaard (1987) demonstrated that the optimum G-value in flocculation prior to flotation was much higher (G = 70-90 sec⁻¹) in flotation than in sedimentation, see fig. 5. For the conventional system with stirred reactors in series, it was found, when coagulating presettted wastewater, that the performance increased with residence time, as shown in fig. 5. It was demonstrated that compartmentalization of the flocculator in order to enhance plug flow distribution, was also favourable in a flocculation/flotation system, but that tapering of the G-value from one compartment to the other resulted in poorer performance.

![Fig. 5. Influence of flocculation variables on flocculation/flotation performance (Øyegaard 1987).](https://iwaponline.com/wst/article-pdf/21/1/1/99677/1.pdf)
In table 6 are given tentative design recommendations for flocculation/floc separation in chemical wastewater treatment.

**TABLE 6** Tentative design recommendations for flocculation/floc separation of biologically treated wastewater at daily design flow.

<table>
<thead>
<tr>
<th>Floc separation process</th>
<th>Precipitant</th>
<th>G-value, sec⁻¹</th>
<th>T_floc (min)</th>
<th>Overflow rate ( v_F ) (m/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>First comp., Internal comp., Last comp.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flocculation/</td>
<td>Al&lt;sub&gt;III&lt;/sub&gt;/Fe&lt;sub&gt;III&lt;/sub&gt;</td>
<td>40-50</td>
<td>15-20</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>Ca</td>
<td>30-40</td>
<td>15-20</td>
<td>&lt;10</td>
</tr>
<tr>
<td></td>
<td>Fe&lt;sub&gt;III&lt;/sub&gt;</td>
<td>25-35</td>
<td>10-20</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Al/Fe&lt;sub&gt;III&lt;/sub&gt;polymer</td>
<td>50-70</td>
<td>30-40</td>
<td>10-15</td>
<td>1.5-2.0</td>
</tr>
<tr>
<td>Flocculation/</td>
<td>Al&lt;sub&gt;III&lt;/sub&gt;/Fe&lt;sub&gt;III&lt;/sub&gt;</td>
<td>70-90</td>
<td>70-90</td>
<td>70-90</td>
</tr>
<tr>
<td>Flotation</td>
<td>Al&lt;sub&gt;III&lt;/sub&gt;/Fe&lt;sub&gt;III&lt;/sub&gt;</td>
<td>&gt;100</td>
<td>&gt;100</td>
<td>&gt;100</td>
</tr>
</tbody>
</table>

1) In the treatment of raw or presettled wastewaters, the necessary residence time \( T_{floc} \) tends to be lower and the necessary overflow rate higher in flocculation/sedimentation systems.
2) If polymers are added, the necessary residence time is probably lower.
3) Depends heavily upon filter design.
4) The recommendations are valid as long as peak flow is equal to or less than twice the daily design flow.

**EXPERIENCES WITH CHEMICAL TREATMENT OF RAW WASTEWATER IN NORWAY**

In Norway we have well over 100 treatment plants as those shown in fig. 6. Earlier these plants were normally built as secondary precipitation plants (see fig. 6), but as it became evident that almost equally good results were obtained in primary precipitation plants, the latter is now more often chosen because of lower cost.

Screening and grit removal → Pre-settling add. → Coag. Flocc. Post-settling → Sludge treatment → Primary precipitation (PP) — without presettling → Secondary precipitation (SP) — with presettling

**Fig. 6.** Schematic of non-biological, chemical treatment plants.

Table 7 demonstrates that good particle removal and generally good treatment results (in terms of suspended solids, organics and phosphates), may be obtained by this simple treatment. The effluent is very clear with turbidities in the range of 1-10 NTU. The SS in the effluent consists of unsettled flocs. Both aluminium-sulphate, ferrichloride, ferrosulphate plus lime and lime are used even if aluminium sulphate is dominating.

When relating this treatment to coastal discharges, the use of lime as coagulant has two very distinct advantages:

1. The high pH necessary for lime coagulation results in a good, yet not complete disinfection.
2. Since the treatment normally will be located close to the sea, one will have the possibility to use the lime/seawater process.
TABLE 7. 1985 yearly average treatment results obtained in 56 chemical, non-biological treatment plants in Norway, ranging in size from 2000 - 750 000 PE (Ødegaard 1987).

<table>
<thead>
<tr>
<th>Process</th>
<th>Number of plants</th>
<th>S S</th>
<th>B O D</th>
<th>C O D</th>
<th>T O T. P</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>n</td>
<td>in</td>
<td>out</td>
<td>%</td>
</tr>
<tr>
<td>PP1)</td>
<td>23</td>
<td>294</td>
<td>172</td>
<td>27</td>
<td>84,3</td>
</tr>
<tr>
<td>SP2)</td>
<td>33</td>
<td>371</td>
<td>218</td>
<td>22</td>
<td>89,9</td>
</tr>
<tr>
<td>PP &amp; SP</td>
<td>56</td>
<td>665</td>
<td>387</td>
<td>24</td>
<td>87,6</td>
</tr>
</tbody>
</table>

1) Primary precipitation  
2) Secondary precipitation  
3) Number of samples that the average values are based upon.

In Al- and Fe- precipitation one will normally achieve a 2 - 3 log reduction in E. coli, (typically from 10^7 E.coli/100 ml to 10^5 E.coli/100 ml Ødegaard et al. 1978). In lime-precipitation, however, a 4 - 5 log reduction may be expected, leaving less than 1000 E.coli/100 ml (Ødegaard et al. 1978, Vrale 1978).

In a recent full-scale investigation the results obtained in table 8 were found, showing an almost complete disinfection when the plant was operated with lime treatment (Myrre 1986).

TABLE 8 Average effluent E. coli counts at Buskerud Hospital primary precipitation plant (Myrre 1986).

<table>
<thead>
<tr>
<th>Precipitant</th>
<th>Number of samples</th>
<th>E. coli per 100 ml</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Influent</td>
</tr>
<tr>
<td>Al-sulphate</td>
<td>16</td>
<td>245 · 10^6</td>
</tr>
<tr>
<td>Lime</td>
<td>17</td>
<td>50 · 10^6</td>
</tr>
</tbody>
</table>

For discharge to inland waters lime is not so suitable because of the high effluent pH. When discharging to the sea, however, the high effluent pH will be adequately neutralized very rapidly. A diffusor with an initial dilution greater than 50, will produce a pH well below 9.

The lime/seawater process.

The schematic of a lime/seawater treatment plant is shown in fig. 7. It is a primary precipitation plant with lime as coagulant and with a few percent of seawater blended continuously to the raw wastewater.

![Fig. 7. Schematic of a lime/seawater treatment plant.](image-url)

For discharge to inland waters lime is not so suitable because of the high effluent pH. When discharging to the sea, however, the high effluent pH will be adequately neutralized very rapidly. A diffusor with an initial dilution greater than 50, will produce a pH well below 9.

Typical design criteria for Norwegian lime/seawater plants are given in the figure. The sludge handling commonly includes sludge thickening, dewatering by centrifugation or filter presses and disposal on land.

In Norway the objective of treatment is often very oriented towards phosphate removal in order to prevent eutrophication of the fjords, and therefore the process is optimized for phosphate removal. However, in all cases if total phosphate is low, the suspended solids are also low. Table 9 shows some operational data from the plant that has been most thoroughly investigated (Vrale 1979).

In table 10 is included data from shorter, but more intense investigation periods from various plants.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean flow, m³d⁻¹</td>
<td>491</td>
<td>903</td>
<td>1629</td>
<td>1818</td>
</tr>
<tr>
<td>Lime dosage, gCa(OH)₂m⁻³</td>
<td>236</td>
<td>226</td>
<td>233</td>
<td>228</td>
</tr>
<tr>
<td>Seawater addition, %</td>
<td>5</td>
<td>5</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>COD-reduction, %</td>
<td>66</td>
<td>71</td>
<td>67</td>
<td>56</td>
</tr>
<tr>
<td>Effluent, tot P, gm⁻³</td>
<td>0,74</td>
<td>0,38</td>
<td>0,22</td>
<td>0,36</td>
</tr>
<tr>
<td>Tot P-reduction, %</td>
<td>83</td>
<td>93</td>
<td>95</td>
<td>92</td>
</tr>
<tr>
<td>Specific sludge prod., %</td>
<td>0,50</td>
<td>0,56</td>
<td>0,48</td>
<td>0,41</td>
</tr>
</tbody>
</table>

Spec. sludge prod. = \( 100 \% \times \frac{m³}{m³} \) thickened sludge transported away per year of treated effluent per year

TABLE 10 Operational data from four lime/seawater treatment plants.

<table>
<thead>
<tr>
<th></th>
<th>Sandvika</th>
<th>VEAS</th>
<th>Korsvikfjorden</th>
<th>Seattle</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>11-12</td>
<td>10.8-11.2</td>
<td>11.0-11.2</td>
<td>10-11</td>
</tr>
<tr>
<td>SS, mg₁⁻¹ (%-red)</td>
<td>16 (96)</td>
<td>9 (93)</td>
<td>16 (90)</td>
<td>22 (92)</td>
</tr>
<tr>
<td>Tot P, mg₁⁻¹ (%-red)</td>
<td>0.31 (92)</td>
<td>0.34 (92)</td>
<td>0.38 (94)</td>
<td>-</td>
</tr>
<tr>
<td>BOD, mg₁⁻¹ (%-red)</td>
<td>-</td>
<td>-</td>
<td>57 (70)</td>
<td>34 (79)</td>
</tr>
<tr>
<td>COD, mg₁⁻¹ (%-red)</td>
<td>40 (76)</td>
<td>-</td>
<td>129 (66)</td>
<td>-</td>
</tr>
<tr>
<td>Lime dosage</td>
<td>240</td>
<td>158</td>
<td>300</td>
<td>220-230</td>
</tr>
<tr>
<td>Seawater addition (%)</td>
<td>3-5</td>
<td>4,5</td>
<td>2-3</td>
<td>10</td>
</tr>
</tbody>
</table>


The effects of seawater addition. The beneficial effect of the added seawater is generally: a better effluent quality at a reduced lime dosage. The effect of 5 % seawater addition on total phosphorus residual is clearly shown in fig. 8. The phosphorus residual at a given lime dose is 50-90 % lower when 5 % seawater (from Oslofjord, 24 o/o0 salinity) is mixed before lime addition than when lime is added without seawater.

As pointed out by Ferguson and Vråle (1984), one might suspect the improved tot P-reduction to be a result of improved ortho-P-reduction. However, fig. 9 shows that ortho-P is with 5 % seawater added, effectively removed even at dosages that are ineffective in removing total phosphorus. This means that the effect of seawater is that it improves the removal of particulate matter, including precipitated phosphate. The beneficial effects of seawater are particle destabilization, aggregation and improved settling.

Fig. 8. Tot P-residual for increasing lime dosage with 5 % seawater (Vråle 1978).

Fig. 9. Total P, particulate P and ortho-P residuals for lime dosages with 5 % seawater (Vråle 1978).
The theory behind these effects of seawater addition, is, however, not fully understood. One may imagine that both of the two following effects have an impact:

a. An effect of increased magnesium concentration leading to increased precipitation of the very flocculant Mg(OH)₂.

b. An influence of increased electrolyte-concentration leading to a compaction of the colloidal double layer resulting in improved coagulation.

A jar-test study by Haugan (1977) demonstrated that magnesium by itself could duplicate the effect of seawater addition. In fig. 10 the total phosphorus residuals after settling are plotted as a function of lime dosage for wastewater with from 5 to 80 mg/l magnesium addition and for addition of 5 % seawater (31 % salinity) which added approximately 60 mg/l magnesium. The results for 5 % seawater and for 20, 60 and 80 mg/l magnesium are almost indistinguishable, while results for lower magnesium concentrations are clearly inferior.

Fig. 10. Total phosphorus residuals for varying magnesium concentrations and seawater percentages (Haugen 1977).

Ferguson and Vrål (1984) demonstrated that the phosphate (or suspended solids) removals obtained were consistent with enhanced coagulation resulting from magnesium precipitate formation. They concluded that seawater adds enough magnesium to precipitate at least 0.6 millimoles l⁻¹ of magnesium hydroxide. The amount of seawater needed depends, therefore, on the solubility of magnesium hydroxide, which is a function of pH. As little as 1 - 2 % seawater is needed if pH values above 11 are used. If pH values are below 10.5, 10 % or more seawater will be required according to a theoretical evaluation made by Ferguson and Vrål (1984).

The reason why it is felt that magnesium precipitation may not alone explain totally the beneficial effects of the seawater addition is:

a. Surprisingly good results have been obtained at relative low pH values (10-10.5) simultaneous to low seawater addition (2 - 3 %).

b. The addition of seawater has also proven to be beneficial at plants using coagulation with ferric chloride as precipitant at a pH of 6-6.5. At the biggest treatment plant in Norway, VEAS (750 000 PE) treating wastewater from the Oslo area, it is claimed that the addition of 1 - 2 % seawater has made it possible to reduce the ferric chloride dosage by 20 - 30 % (Berge and Sether 1985).

There is consequently still a lot to be investigated before we really know all the mechanisms behind the lime/seawater process. From a practical point of view, however, the process is known well enough and may be used favorably in coastal areas.

Necessary lime and seawater dosages. The practical benefits of reduced lime consumption are very well shown in the diagram in fig. 11 given by Vrål (1978) which shows the changes in pH and phosphorus residuals at different lime dosages and seawater additions. A very similar diagram could be constructed for suspended solids or any parameter related to particulate matter.

Sludge production. One of the drawbacks of the process is the relatively high sludge production. Reduced lime dosages give reduced sludge production, but on the other hand will increased magnesium precipitation result in higher sludge production. The sludge settles and thickens very well, however, resulting in a more moderate sludge volume production.

The full scale experiences in Norway show a sludge solids production in the range of 230 - 250 gTS·m⁻³, and a specific sludge volume production after sludge thickening of 0.4 - 0.6 % (specific sludge production = 100 % · m⁻³ thickened sludge/m³ raw water treated).
Gravity thickening will typically give 5 - 6 % solids in the thickened sludge and the thickened sludge is typically dewatered to 20 - 30 % TS without the addition of polymers or more lime when filter presses are used for sludge dewatering. Because of the high lime-content and high pH of the sludge, it is chemically "stabilized". It does not decompose rapidly and causes little odour problems. In some areas in Norway more lime is needed in the soil and the sludge is used on corn production areas after having been stored for some months.

**Hydraulic loading.** The flocs in the lime/seawater process flocculate and settle very well. Probably lime/seawater plants can be more heavily loaded hydraulically than primary precipitation plants operating with aluminum, iron or lime alone as coagulant. This has, however, not yet been scientifically demonstrated.

**Cost.** In an evaluation of the total operation cost at the VEAS-plant (Vråle 1987), it was found that the lime/seawater process would be 28 % cheaper in operation than would coagulation with ferric chloride as used in the plant at that time.

**Hygienization.** The E.coli reduction is probably primarily a matter of pH. The results shown in fig. 12 may, however, indicate that increasing saltwater addition results in increasingly lower E.coli residuals even if pH is lowered by the seawater dilution for a given lime dosage (Vråle 1978).
**Sludge recirculation.** Ferguson and Vråle (1984) studied in full scale the effect of sludge recirculation from the sludge thickeners to the inlet of the flocculation basins. The idea was to reduce lime consumption and improve clarification by improved coagulation and flocculation.

And indeed it was found that sludge recycle (1.3 - 3.8 % of plant flow) improved the steady-state of the process. At lime dosages of 165 and 240 mg/l, effluent turbidity was reduced by 30 - 50 % by sludge recirculation, down to values in the range of 1.6 - 3.4 FTU. It was also found that sludge recycle improved process performance during transient failures of seawater or lime dosing.

**Dosing of lime.** One of the practical difficulties encountered in the use of lime as coagulant is problems with the lime dosing equipment. Normally hydrated lime, Ca(OH)₂, is used. Since the lime dissolves so poorly in water, a lime slurry of 5 - 20 % is prepared, and the slurry is dosed to the main water stream. The way the slurry is added to the water is, however, quite important for the effectiveness, because if the lime is not totally dissolved relatively rapidly, it will settle in channels, in the flocculation basins or in the settling basin without having acted as coagulant.

The loss of lime in this way may be prevented by:

a. Using as low slurry concentration and fine dry lime grading as possible.

b. Ensuring sufficient mixing intensity and residence time (15 min) in the mixing tank.

The cost of using a low slurry concentration (0.5 - 1 %) could be acceptable if treated effluent was used as dilution water when preparing the slurry, or seawater was used as such. Laboratory experiments seem, however, to indicate that using the seawater gives suboptimal results.

**Recirculation of sludge water.** The sludge water coming from thickening and dewatering of sludge has a very high pH and a great coagulating capacity. This water should therefore be returned via a balancing tank back to the plant inlet in a controlled way.

### LOW-COST CHEMICAL TREATMENT USING PRECIPITATION PONDS IN TOURIST AREAS

When appropriate technology in tourist areas is discussed, the use of stabilization ponds should be mentioned. Conventional ponds require, however, enormous areas and it will normally be a goal to reduce the area requirement.

In Scandinavia we have some experience with low cost chemical treatment in tourist areas based upon the so-called precipitation ponds. This is presented in detail elsewhere (Ødegaard et al. 1987), but the general experiences may be included here. Our experiences with precipitation ponds are from mountain tourist areas, but they should be even more applicable in coastal areas.

**Chemical treatment in combination with ponds.**

Pond systems may be combined with chemical treatment in three ways, see fig. 13.

**Fig. 13.** Three ways of combining chemical treatment and stabilization ponds

For coastal tourist areas the pre-pond and in-pond precipitation modes are most relevant. In the pre-pond precipitation system, the pond will act as an effluent polisher, primarily for improved soluble BOD removal. The advantage with the pre-precipitation step is that the organic loading on the pond system will be reduced by 70 - 80 % which necessitates a much smaller pond than without pre-treatment.
It is, however, the in-pond precipitation mode that can be referred to as a low-cost system. The Scandinavian full-scale experiences show that these plants are primarily chemical treatment plants and to a much lesser extent biological treatment plants. The loadings on these ponds are therefore quite high, typically 5 m² per person connected. In coastal areas the lime/seawater process may be implemented.

The high load results, however, in two distinct disadvantages:

a. The necessity of frequent desludging.
b. The possibility of anaerobiasis and odour as a consequence of this.

The process of desludging is normally carried out as follows:

1. The pond to be desludged is drained of water.
2. The bottom sludge is allowed to dry for some time.
3. The sludge is moved by frontloaders to an area where it is allowed to dry.

The problem of anaerobiasis may be solved either by the use of lime- and lime/seawater process resulting in a pH too high for anaerobic biodegradation, or by pond aeration. This is done at the Nordseter plant in Norway, built for a tourist area and now treating the wastewater from 1300 PE.

Results from the Nordseter plant in Norway. The plant has three ponds with a total surface area of 8000 m², see fig. 2. The precipitant, a technical grade alum, was at the start added to the pipe between the first and the second pond (fig. 14).

![Fig. 14. The lay-out of the Nordseter plant.](https://iwaponline.com/wst/article-pdf/21/1/1/99677/1.pdf)
Technology for wastewater treatment

Since the recipient is a sensitive and small mountain brook, the environmental authorities set very strict effluent criteria when they allowed the load to be raised to 1300 PE in 1986. The average yearly effluent concentrations shall not be higher than 10 mg/l BOD$_7$ and 0.4 mg/l P and the maximum concentration shall not exceed 20 mg/l BOD$_7$ and 0.8 mg/l P.

In order to meet these very strict criteria, both the chemical and the biological process had to be optimized. The changes now carried out are as follows:

a. The chemicals are now added flow-proportional to the inlet of the first pond.

b. The bottom of the half of the first pond closest to the inlet has been asphalted and the two halves of the first pond has been divided in order to be able to desludge the first part separately.

c. Aerators have been installed at the outlet of the first pond, as well as those already at the outlet of ponds 2 and 3.

Precipitation ponds in coastal areas. In coastal areas the in-pond precipitation system could well use the lime/seawater process. If the plant is located in places with warm and sunny climate, the two first ponds in a system of preferably three or more ponds should be about 2 m deep, asphalted and run in parallel. In this way, one of the two parallel ponds could be drained and the sludge allowed to dry without closing down the plant.

CONCLUSIONS

1. The appropriate technology for coastal tourist areas is a technology that:

a. Can meet the effluent quality requirement, typically being good SS and E.coli removal, and moderately good BOD removal.

b. Can deal with the extreme variations in flow and composition of the wastewater that one must expect from these areas.

c. Is relatively simple to operate and is economically acceptable.
2. In this paper it is shown that since such a significant part of the contaminants in wastewater appear as, or is associated with, particles, a good particle removal should be the first step of wastewater treatment. Such particle removal may be obtained by wastewater coagulation.

3. Since hygienization of the wastewater is wanted when discharging it to coastal waters, lime is most suitable as coagulant, since the high coagulation pH (10.5 – 12) results in considerable disinfection.

4. Especially appropriate in coastal areas is the use of the lime/seawater process where 2 – 5 vol-% of seawater is added to the treatment plant inlet, resulting in improved clarification, reduced lime consumption, reduced sludge volume production, and a chemically stabilized sludge.

5. A low cost chemical treatment, very useful in tourists areas, may be obtained by the use of precipitation ponds where the precipitant is added ahead of a pond system. The lime/seawater process could well be implemented also in this system.

REFERENCES

Odegaard, H., Thorvaldsen, G., Storebråten, B. and Skjefstad, J. (1978). Reduction of E coli by chemical precipitation of wastewater. VANN nr. 3 (In Norwegian)