TECHNICAL REPORT 136

Date: 9 September 1974
Prepared by: Jorge Arbeleda, Adviser in Water Treatment
Subject: Georgetown Water Treatment Plant

1. WATER SOURCE

The water source for the Georgetown Water Treatment Plant comes from two entirely different systems:

a) Surface water – from East Demerara Water Conservancy (storage reservoir), conveyed to Shalter Belt by an 8-mile long unlined open channel, called Lumsah Canal.

b) Ground water – seven deep wells. Wells 2 and 6 tap water from aquifer A (700 ft deep), and wells 1, 3, 5 and 7 from aquifer B (1500 ft deep).

The water quality of each source is different.

Surface water has very low alkalinity (2.0-7.0 mg/l), and turbidity, but extremely high color (600-800 CD) and sometimes iron (0.2-2.2 mg/l). It is subject to a certain degree of contamination due to the fact that the channel crosses through extensive sugarcane plantations.

The aquifer A water has a high temperature (95°F), low dissolved solids and anaerobic iron content (1.6 mg/l). The aquifer B water also has high temperature but moderate amounts of dissolved solids and no iron.

The alkalinity and hardness content of both waters is not clear. In the analysis presented by ESJ, carbonate hardness is 8.9, 2.2 mg/l and bicarbonate alkalinity 36, 203 mg/l, which is not possible because these values do not permit the balance of cations with anions.
2. WATER CONSUMPTION

The total flow supply for the city is as follows:

a) From wells 2 and 6: 2.1 MGD (110.1 lt/sec)

b) From the treatment plant (including 80% of the supply from wells 1, 3, 5 and 7): 8.0 MGD (420 lt/sec)

c) From four other wells (including 0.9 MGD from wells 1, 3, 5 and 7): 5.0 MGD (262 lt/sec)

Total: 15.1 MGD (791 lt/sec)

The per capita consumption for 170,000 inhabitants is 88.8 g/d (403 lt/d) which is high but not unusual in tropical climates in South America. Under these circumstances it has seldom been possible to reduce the consumption to less than 60 g(pc/d) (270 lt/pc/d) including domestic and nonresidential consumption and permissible leakage, which in very few places in this continent is less than 20%.

3. DESCRIPTION OF THE PLANT

The plant consists of:

a) Inlet and mixing chamber
b) Six flocculation basins
c) Three horizontal settling tanks (one of these tanks is used as storage reservoir for well water)
d) Ten rapid gravity filters
e) Chemical dosing and chlorination facilities
f) One storage canal
g) One clear water reservoir

This plant was built in 1951 for a capacity of 6 MGD and it was expanded in 1956 (adding four more filters) to a capacity of 10 MGD (325 lt/sec).

Since then the system has been modified several times. Settling basin 1 was converted into a storage reservoir, to hold the 2.1 MGD (110 lt/sec) pumped from wells 2 and 6. Water is cooled there and pumped directly to the city without treatment. The settled water channel was changed in order to be able to convey the effluent to the 40' wide 1.220' long storage canal adjacent to the treatment plant.

A pumping station was also built to take the water from this canal and pump it into the filters.
All these modifications have made the operation of the treatment plant very complex, as follows (see figure 1):

The 4.7 MGD (246 lt/sec) ground water from Lamaha Canal are pumped to the mixing chamber and from there flow by gravity into settling tanks 2 and 3. The effluent of these tanks is transferred to the canal where it undergoes further settling and is mixed with the 3.8 MGD (200 lt/sec) from wells 1, 5, 6 and 7. Both flows (the 4.7 MGD surface water plus the 3.8 MGD ground water) are pumped back from the canal into the filters and, after chlorination, stored in the filtered water reservoir. Therefore, settled water does not have the same characteristics as filtered water.

A further complication is caused by the lack of storage capacity in the clear water reservoir which makes it necessary to change the plant output constantly and to adjust all processes to these variations. Average water production at the moment is around 8 MGD (420 lt/sec), but it fluctuates from 11.65 MGD down to 4.32 MGD or even zero, according to the hour of the day.

4. INLET AND MIXING CHAMBER

The 24" main from the pumping station in Lamaha Canal enters into an 11' x 11' concrete box where the mixing of the coagulant with water is done by means of a water jet produced by an auxiliary pump, which recirculates the flow into the tank.

Lime is added at the entrance point of the inlet main, and a mixture of aluminum sulphate and sodium aluminate (in proportion 8:1 or 10:1) is applied near the water jet. Alum dosages of 30 to 60 mg/ltr are commonly used, which seem to be low for waters of such high color.

It should be borne in mind that:

a) Optimum pH for color removal is between 5 and 5.5.
b) Optimum pH becomes more critical as the concentration of color-producing substances increases.
c) The residual color that remains in the water after coagulation is also dependent on pH. The higher the pH the higher the residual color.

For all these reasons the lime dosage applied to the water must be good enough to neutralise the acidity induced by the alum without actually increasing the pH above 5.5.

Jar tests should be routinely performed to assess the right combination of lime, aluminum and sodium aluminate. The latter could perhaps be replaced advantageously by soda ash (Na₂CO₃).

Many colored waters, however, do not need any lime for coagulation, especially when an iron salt is used. High pH's are very detrimental to color removal.
Detention time in the mixing chamber is more than one minute, which is ample.

Due to the very rapid reaction of the coagulant with water, there is no need for prolonged mixing periods. According with Hahn and Stumm, the hydrolysis reaction takes place in $10^{-10}$ to $10^{-3}$ sec, and the polymerization reaction in $10^{-2}$ to 1 sec. After one second very little can be achieved to improve the coagulant dispersion. That is why the tendency today is to use piston flow reactors instead of the old backmix reactors, too slow to avoid coagulant segregation.

5. **Flocculation Basins**

Each settling tank has two 20' x 20' x 7' flocculation basins. Three flow conditions will be considered:

a) The actual situation in which two settling basins are working and the flow is 4.7 MGD. In this case each flocculator is taking 1.175 MGD (5334 m$^3$/d).

b) The plant is treating the designed capacity, that is 10 MGD, and all three settling units are working. In this case each flocculator would receive 1.667 MGD (7568 m$^3$/d).

c) The plant capacity is expanded up to 15 MGD (68,100 m$^3$/d) and all settling units are working. In this case each flocculator would receive 2.5 MGD (11,350 m$^3$/d).

Detention times for these flows are as follows:

<table>
<thead>
<tr>
<th>Flow (MGD)</th>
<th>Detention Time (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.175</td>
<td>5.334</td>
</tr>
<tr>
<td>1.667</td>
<td>7.568</td>
</tr>
<tr>
<td>2.500</td>
<td>11.350</td>
</tr>
</tbody>
</table>

The optimum theoretical detention time for flocculation has to be determined by jar test experiments, but it normally falls (for turbid water) between 16 and 25 min. If water is not completely flocculated before entering the sedimentation basins, the residual turbidity or color will be high, no matter what coagulant dosage and settling period are used.

For this reason, flocculation in the shelter belt plant is very inefficient. Not only the theoretical detention times are short but, having only one chamber, most of the flow is passing through the basin in much less than the theoretical detention time. Furthermore, the energy communicated to the water mass by the agitators seems to be low.
All these problems together render it almost impossible to get a complete removal of fine color particles, which remain in suspension throughout the settling process.

6. SETTLING BASINS

There are three 40 ft wide, 149 ft long and 7 ft deep horizontal settling tanks. Surface loads for the three flow conditions previously considered are as follows:

<table>
<thead>
<tr>
<th>Flow for settling</th>
<th>Surface load</th>
<th>detention time</th>
<th>Horizontal velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>MGD</td>
<td>m³/d</td>
<td>m²/m²·d</td>
<td>g/min/ft²</td>
</tr>
<tr>
<td>4.7</td>
<td>21,338</td>
<td>19.26</td>
<td>16.37</td>
</tr>
<tr>
<td>10</td>
<td>45,400</td>
<td>27.32</td>
<td>23.22</td>
</tr>
<tr>
<td>15</td>
<td>68,100</td>
<td>40.97</td>
<td>34.83</td>
</tr>
</tbody>
</table>

Horizontal velocity, surface load and detention time are optimal at the moment. This confirms the idea that the problem lies not in the settling process (although this could be improved) but in the flocculation process. For 10 MGD condition the process characteristics will not be as good as they are now, but with the help of polyelectrolytes, good performance could still be achieved.

For 15 MGD, high-rate sedimentation has to be introduced, or a very careful control of the processes should be maintained.

The above comments are made for well-flocculated water. At the moment, this is not the case.

If new flocculators are built inside the present sedimentation tanks, the length of these tanks will be reduced to 129 ft (assuming 29 ft more are used for flocculation) and surface loads are as follows:

<table>
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<th>detention time</th>
<th>Horizontal velocity</th>
</tr>
</thead>
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<tr>
<td>MGD</td>
<td>m³/d</td>
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<tr>
<td>4.7</td>
<td>21,338</td>
<td>22.22</td>
<td>18.88</td>
</tr>
<tr>
<td>10</td>
<td>45,400</td>
<td>31.53</td>
<td>26.80</td>
</tr>
<tr>
<td>15</td>
<td>68,100</td>
<td>47.22</td>
<td>40.19</td>
</tr>
</tbody>
</table>
If this alternative is accepted, high-rate settling should be immediately considered. As the sedimentation basin depth is only 7', the introduction of the modules is going to be difficult because not enough space can be left below them. Two solutions for this problem could be considered:

a) To raise the level of the walls from 3' to 4', if the structure of the tanks permits it, changing the sludge draw-off system.

b) To build the flocculation basins outside the present tank structure, increasing its length by 40 ft and changing the flow direction as shown in figure 3. This will provide an extra 15.4% increase in settling time and similar decrease in surface load that would become 23.66 m³/m²/d for 10 MGD.

Whatever alternative is selected, the take-off section should be modified. There is only one outlet weir per tank, with a load of 10 cm/sec/m for 4.7 MGD, 14.3 cm/sec/m for 10 MGD and 21.5 cm/sec/m for 15 MGD. The maximum load recommended is 3 cm/sec for heavy floc. For light floc, as the one produced in Shelter Belt, it is preferable to have a load of 2 ft/sec. This means that an increase of 7 to 10 times should be introduced in the outlet weirs length.

7. FILTERS

7.1 Surface loads

There are 10 filters at Shelter Belt with a length of 29.5 ft and a width of 13.5 ft. The filtering area per filter is 398 ft² (37.04 m²) and 3980 ft² (370 m²) for the ten filters. Surface load for the present flow (8.5 MGD), the design capacity flow (10 MGD), and the maximum flow (15 MGD), are as follows:

<table>
<thead>
<tr>
<th>Flows</th>
<th>Surface load</th>
</tr>
</thead>
<tbody>
<tr>
<td>MGD</td>
<td>m³/d</td>
</tr>
<tr>
<td>8.5</td>
<td>38,590</td>
</tr>
<tr>
<td>10</td>
<td>45,400</td>
</tr>
<tr>
<td>15</td>
<td>66,100</td>
</tr>
</tbody>
</table>

These loads are not excessive for sand beds, provided water pretreatment is adequate.

However, larger floc storage capacity and longer runs could be obtained, changing the sand bed to sand–anthracite, and modifying the backwashing system and the bottom drains.

7.2 Backwashing

There are two main filter backwashing systems. Both produce good results when they are properly designed: low-rate and high-rate backwashing.
The first one is used in combination with air scouring and there is no expansion of the granular bed during the process. The second is aided by surface wash and there is 15 to 25% bed expansion throughout the cleaning operation.

Backwashing at Shelter Belt is done with air during 3 min, followed by water during 3 to 5 min. Normally there is a time lag between air and water injection. This sequence is repeated three to four times.

Washwater rise measured in one filter was 8.5" per min (22 cm/min). This low upflow velocity produces no expansion of the sand bed, and as the filter has only one washwater trough at its far end, 50% of the flow has to travel a considerable horizontal distance (more than 4.5 m).

During this process, a good proportion of particles that are lifted during the air scour settle out again on the surface of the bed, before reaching the outlet weir, due to the low horizontal velocity which is not enough to transport the sediments and which increases only as the flow approaches the washwater trough. This is shown in figure 4.

Scouring velocities can be calculated with the well-known Camp formula:

\[ v_c = \sqrt{\frac{g}{f}} \left( \frac{S_o}{S_p} - 1 \right) d \]

Given \( K = 0.8 \) (as recommended by Shields for effective scouring), \( f = 0.03 \) and varying \( S_o \) between 1.1 (for heavy floc) to 1.01 (for very light floc), horizontal velocities needed for efficient sediment transportation range from 32, 22, 14.5 cm/sec, for a floc size of \( d = 0.8 \) mm, to 14, 10, 4.6 cm/sec for that of \( d = 1 \) mm. The percentage of solids carried on by the flow will depend on the density of the floc, the smoothness of the surface, the water temperature, and the particle size distribution.

Since the average horizontal flow velocity that drags the particles to the weir ranges from zero (theoretically) to 0.10 m/sec, only the very fine and light sediments (< 0.1 mm in diameter) have the chance to be removed, even near the washwater trough where the velocity is at a maximum.

It is important to notice that the horizontal velocity is a function of the water depth on top of the sand. When the sand is below the edge of the effluent weir, the horizontal velocity decreases proportionally to the increasing of the cross section area, according with the continuity equation \( V = \frac{Q}{A} \).

The time for a particle to travel with an average velocity of 0.05 m/sec, along the length of the filter (that is 9 m), is about 3 min, time enough so settle 33 cm and come to rest on the surface of the sand bed.
In practice this time is even longer due to the time lag between the closing of the air scouring and the opening of the washwater valve, that could be as long as 1 min.

The above clearly explains why the sand bed in the filters appears completely covered with mud and full of cracks, no matter how long the backwashing is applied. The problem is so serious that a special machine for washing the sand has been constructed to facilitate the bed cleaning, because the media has to be periodically pumped out, at least every two years, washed and then put back into the filter box.

The writer has found similar problems in other plants of the same make in Africa, where filter beds look equally dirty.

9. DOSING FACILITIES

The dosing facilities seem adequate at the moment, and could be used for some time in the future.

9. CHLORINATION FACILITIES

They are located in two 11' x 14' and 11' x 11' rooms. The first is used for chlorine storage and the second for the chlorine dosing equipment. The space is very limited and does not permit good operation of the equipment.

10. RECOMMENDATIONS

10.1 Location of the plant

The first question to be solved in connection with the water production system for Georgetown is whether the existing treatment plant can be improved and used in the future or whether it has to be abandoned and a new plant constructed instead.

Before resolving this question the following information has to be gathered and analyzed:

10.1.1 Complete statistical data on the contamination of the Lamine Canal, taken during a period of one year, that include no less than 500 samples taken at two different points along the canal, and tested for coliforms, BOD, COD, and OD.

10.1.2 Sufficiently detailed cost figures of a new and complete water treatment plant, and those related to modifying and upgrading the existing treatment plant, including preliminary drawings of both possibilities and operational costs.

10.1.3 Economical and technical study of the modifications to be introduced in the city’s pipeline network, if the location of the treatment plant is moved up along the canal.
10.1.4 Reliability of the supply in each location of the treatment plant (present and new site) and cost of modifications to the canal, if any, in order to make both sites equally reliable.

10.1.5 Complete soil analyses of the present site, as well as the selected new one.

Once this information is available, a detailed economic analysis must be made of the alternatives. The decision criterion to be applied is that of cost minimization, that is, the minimum cost alternative should be adopted.

10.1.6 The presentation of costs for the purpose of comparison requires that all costs be expressed as present values, utilizing the same period of analysis for each alternative (in this case the period of analysis should probably be 25 years) and the same discount rate, and making use of constant monetary units for the same base year (e.g. 1974 dollars).

10.1.7 The costs should include both capital costs and costs of operation, maintenance and replacement (OMR costs). Capital expenditures could include such costs as engineering design, land, water rights, construction, administration and financing during construction, amortization, etc. OMR costs include labor, materials, administration and overhead, periodic replacement of equipment, chemicals, power, and so on.

10.1.8 In general, costs of all components of the water supply system should be considered, including the costs of developing different water sources, intakes, transmission lines and pumping stations, treatment, storage and the distribution system.

10.1.9 Points 10.1.7 and 10.1.8 above indicate in general what types of costs and the cost elements to be included in an economic comparison. However, when comparing specific alternatives the procedure can be considerably simplified since it will only be necessary to include those costs which differ for the different alternatives. If some cost element is unaffected by the alternatives proposed, it need not be considered when choosing between the alternatives. In the case of the Georgetown water treatment alternatives, the capital and OMR costs of new designs and all proposed modifications of the existing plant must be included, as well as cost differences for intakes, transmission, storage and distribution.

10.1.10 Since it has been indicated that there could be some risk factors involved, such as subsoil conditions, raw water quality and flow reliability, each one of these factors should be studied in detail and if it appears warranted the risks should be quantified monetarily and included in the economic analysis. This could be accomplished through some Bayesian decision process or similar techniques.
10.1.11 In summary, the only valid method for choosing between the alternatives proposed is by means of a comparison of the present value of the total affected costs of all the alternatives to find the one having minimum costs. With regard to accepting the sunk costs of abandoning existing facilities, this is only justified if and when it can be demonstrated to be anti-economic to continue their use.

10.2 Alternatives that can be adopted to upgrade the existing treatment plant

10.2.1 Alternative 1 - The existing treatment plant was designed for 10 MGD, and can be easily recuperated to treat this flow efficiently. In order to achieve that (see figure 5), all well water, including that from wells 2 and 6, could be pumped into the existing storage canal for aeration and cooling. Both flows (2.1 MGD + 3.8 MGD) could be pumped back to the treatment plant inlet chamber, together with the surface flow (4.7 MGD). The amount of water treated in the plant, in this case, would be around 10.6 MGD. The settling and filtering units must be modified as it will be explained later, and a new storage reservoir must be constructed to be able to operate the treatment plant with constant flow, without the need for varying the flow according to the demand, as is done presently. The advantages of this alternative are:

a) It simplifies the operation of the treatment plant, which at present is too complicated.

b) Mixing the alkaline well water with very acid surface water could eliminate the need to use lime for coagulation decreasing the operational cost. Tests should be conducted to determine the benefit of this mixture.

c) The initial raw water color will be less.

d) Few modifications in the treatment plant have to be made.

The production of 10 MGD in the plant, plus 3 MGD from other sources, will give a total supply of 15 MGD for the city, which, according to the ESI Report, will be enough until 1990 if condition 1 is met. From then on, increased sedimentation area or high-rate settling must be provided together with dual media filters, or the construction of a new treatment plant in some other site could be considered, to complete the 20 MGD to 25 MGD needed for the year 2000.

10.2.2 Alternative 2 - Consists of (see figure 6):

a) Constructing an independent storage reservoir for the water extracted from wells 2 and 6. This water should be chlorinated and pumped directly to the city, as it is done today.
b) From Lamaha Canal, 10.4 MGD would be pumped into the mixing chamber, where the coagulant would be applied, and from there this water would flow into the coagulation and sedimentation basin.

c) Water from wells 1, 3, 5 and 7, after receiving an injection of potassium permanganate for iron/destabilization, would be stored in the canal.

d) Water pumped from this canal (3.8 MGD) would go to a chamber where it would blend with the 10 MGD of settled water coming from the clarifiers. Therefore, the filters will be loaded with 13.8 MGD. To maintain long runs the present sand filters should probably be converted to sand-anthracite.

If this alternative is adopted the treatment plant would produce 13.5 MGD which, together with the 2.1 MGD from wells 2 and 6 and the 5 MGD from other sources, will give a total supply of 20.6 MGD which, according to NSF, will be enough until year 2000 if condition 1 is met. This alternative has the advantage that the output of the Shelter Belt system, which at the moment is only 10.6 MGD, could be upgraded to 15.6 without major modifications in the treatment plant. Filtered water storage must be increased as in alternative 1.

10.2.3 Alternative 3 - Alternatives 1 and 2 could be combined introducing a connection between the pipeline that conveys the water from the storage canal into the filters and the pipeline that comes from Lamaha Canal carrying the surface water (see figure 6). A sluice gate could also be left open to communicate the storage for wells 1, 3, 5 and 7 and that for wells 2 and 6.

This way all the flows (surface and ground water) could be pumped together to the mixing chamber at the plant, to be subject to the same processes (coagulation, sedimentation and filtration), or the flows coming from each source could be treated separately as in alternative 2.

This alternative may have the advantage that when the demand is low, the simple solution outlined in alternative 1 could be adopted, and when demand grows the more complicated alternative 2 could be used, extending the life of the plant until year 2000.

An increase in filtered water storage is also essential in this case.

10.3 Modification of the processes

In order to implement any of the alternatives previously described, the following modifications to the treatment units should be considered:

10.3.1 Mixing chamber - It is adequate and no specific modifications are suggested.
10.3.2 Coagulation - The main difficulty of the Shelter Belt Treatment Plant is the lack of good coagulation, as it was previously pointed out. The chemistry of the process has to be completely restudied to find out: the type of coagulant needed (iron or aluminum coagulants), the optimal dosages, detention time, velocity gradient and pH for color removal, the type and dose of flocculant aid, the effect of blending surface and ground water and the settling velocities obtained with the different combinations of coagulants and waters.

10.3.3 Flocculation basins - The existing flocculation basins are inadequate due to short-circuiting, low average velocity gradient and low theoretical detention time. The best solution would probably be to construct four new flocculation chambers in each settling basin working in series, provided with vertical paddle agitators. The system used to transfer the flow from one chamber to the next one must be carefully studied so that a minimum of 60% piston flow can be obtained.

Probably the best location for the new settling basin is at the end of the existing sedimentation basins, changing the flow direction, because in this way the settling area is increased, and the structure could be easily built without interrupting the operation of the clarifiers.

10.3.4 Sedimentation basins - The sedimentation basins have enough area to treat 10 MGD with 23.66 m³/m²/d (20 g/min/ft²) (if the flocculators are moved away), which is a conservative and acceptable surface load. No modifications are suggested, except at the inlet and outlet sections.

At the inlet, a very well calculated and perforated trough should be constructed to distribute the flow that comes from the flocculator evenly, all along the cross section area of the tank. A perforated wall should also be designed to aid in the uniform distribution of the water.

At the outlet zone, several outlet troughs with "v" notched edges should be introduced in order to decrease the longitudinal load to at least 2 lt/sec per meter of weir length. The sludge removal equipment should be overhauled and put back into efficient operation.

10.3.5 Filters - The filter load for 10 MGD is 126 m³/m²/d (104.3 g/min/ft²) and new modified sand beds are the only thing required. For 13.5 MGD the load would be 169 m³/m²/d (144 g/min/ft²). In this case, it is probably better to have sand-anthracite beds so as to increase the length of filter runs, although there is no basic objection to keeping the sand beds.

Filtering material in all filters should be changed as follows:

a) For sand alone: 27" of 0.8 mm effective size with a uniformity coefficient of 1.1 in order to avoid increasing the porosity of the bed in the flow direction.
b) For sand-anthracite: 27" of anthracite of 1.0 mm effective size and a uniformity coefficient of 1.3 on top of 10" of sand of 0.35 mm effective size and a uniformity coefficient of 1.65.

The backwashing system should be modified in all cases as follows:

a) For sand beds: Increase the upflow velocity to 10" per min, construct two washer water troughs longitudinally in the filter to collect particles as soon as they rise, and establish a grid of perforated pipes 1" on top of the sand to be able to bubble air during the water wash. Pipes could be spaced 1 ft center to center and fixed at the washer water troughs to hold them in position. Walls of the existing washer water weir should be raised to force all water to be carried by the washer water troughs.

b) For sand-anthracite beds: Modifications are similar to the one previously described but provision must be made for backwashing with 36" per min, in order to re-stratify the sand and anthracite layers. This implies not only increasing the backwashing pumps but changing all filter bottoms. Porous plate or Leopold-block-type filter bottoms, which do not need thick gravel layers, could be used.

The filter control system requires substantial modifications. The PCE module (flow regulator) should be abandoned and a sluice valve located instead, with extended spindles and headstocks to be operated on the top floor. In order to avoid de-aerating of the bed during the initial part of the run, the walls of the filtered water box should be raised and a weir inserted, so that at least 6" of water remain on top of the bed at the beginning of the run. This way the water level in the filter will rise during its working period; backwashing should be done when this level reaches a maximum, drawing the filter. With this system, there is no need for headloss gauges, which could also be scrapped.

The filter inlet should also be modified in case there is need to work with constant flow. Weirs, as high as possible in the filter box, should be installed in all filters at the same level, in order to divide the flow equally in all units.

10.3.6 Chlorination facilities - Better chlorination facilities should be designed. It is necessary to increase the size of the storage area and the chlorines dosing room, providing it with adequate ventilation and safety equipment.