D15: Technical Considerations Part 2 - MYRIAFLOW and HYDROBARRIER

Work package No. 15:	**Technical considerations**
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1 EXECUTIVE SUMMARY

This part of the report is the phase 4 of a study relative to technical solutions applicable to medium size lakes such as those studied in the EUROLAKES project.

It is shown that the behaviour of a lake can be modified with low power consumption systems. In the first three reports, we have shown that other countries consider artificial actions on lakes as possible means of remediation. This is mostly the case of Canadians which have applied a number of techniques for curing and evaluation.

We have shown that it is possible to use systems to increase the oxygen content of the hypolimnion. The AIRLAC permits to increase significantly the oxygen content of lakes as large as the Leman lake with limited investment and operating expanses. One AIRLAC is enough to treat a lake like the Bourget Lake.

We have also studied the use of oil skimmers in case of limited pollution or major pollution problems of oil or floating liquids of any kind. Several techniques can be used, including the Euroskim which is a very high velocity skimmer applicable on lakes and at sea.

In this report, we give some examples of application of two extra techniques, the MYRIAFLOW and the HYDROBARRIER.

The MYRIAFLOW is aimed to displace large quantities of water in the lake to enhance its global or local quality. Global application concern the vertical mixing of different water layers. It is shown that a limited number of vertical tubes permit to do that at the scale of the lake. Another application is to reduce the level of the lake by pumping of large volumes.

Finally, an application of depollution of a beach is presented. It is shown that the same system can lower the concentration in different species and be designed in such a way that they constitute an attraction which permit water games. This idea to improve the water quality and to use the equipment for amusement purposes seems interesting for the local authorities. In fact, the financing of this equipment can be paid by an increase of tourism and an augmentation of the sea shore length.

In order to complete the set of means applicable to lakes, we have made a study of HYDROBARRIER. They can be used to prevent the feeding of the lake by underground pollution which may last for years. It is shown that the equipment to install is very light whatever the type of ground and the slope of the aquifer. This type of equipment might solve the problem of high nutrients levels coming from underground.

Finally, the interest all these "technical considerations" might be better assessed if their actions was included in the very powerful calculation codes developed in the frame of the EUROLAKES project. Our aim was to show that artificial actions are possible at low prices. Our hope is to find practical cases of lakes having major problems where the combination of theoretical calculations of flows, thermodynamics and life in the lake and of technical solutions might lead to real remediation or better utilisation of the lake.

2 INTRODUCTION

This report constitutes the part 2 of the study relative to technical aspects applicable to lakes. It two aspects of artificial means used to fight lake pollution.

The first system presented concerns the applicability of low head small propellers to create flow acceleration or displacement of large quantities of water. This system is called MYRIAFLOW and some possible applications are presented.

The second system is a method of pumping polluted underground water coming from the catchment area around the lake to prevent it to come to the lake without treatment. It is called "HYDROBARRIER" and it is shown that efficient protection can be made with limited investments and operating costs.

The idea is to show that limited investment can produce interesting effects at the scale of the lake. It is understood that the examples given are not limitative and that these systems can be used to treat other situations.

3 THE MYRIAFLOW AND ITS APPLICATIONS

3.1 SOME ORDERS OF MAGNITUDE

Flow rates.

Most pump makers tend to use hydraulic machinery as large as possible to increase the efficiency. When large quantities of water have to be processed, typical diameters of the runner reach several meters.

In the case of lakes, typical characteristic volumes are of some km^3. The panel of lakes under study in the frame of EUROLAKES varies between 3 and 1000 km^3 (3 10^9 to 10^{11} m^3.)

A system of pumps used to process these large quantities of water will be said to have an appreciable effect if a volume equivalent to the lake volume passes through it within a year.

For a medium size lake of 3 km^3, this flow rate will be of 100 m^3/s for the lake of 100 km^3, it will be of 3000 m^3/s.

Given a water flow velocity through the pump impellers of 10 m/s, one can conclude that a pump of 10 m^2 (Diameter 3.6 m) will produce a significant effect on a medium size lake. To obtain the same effect on a very large lake, the cumulated surface of the pumps will be of 300 m^2, which can be produced by a set of 10 pumps each having a diameter of 6.2 m.

We can note that, if the head of the pump is of 50 000 Pa (5 mCE), the power installed will vary between 5 and 150 MW, and these powers are relatively small if it is divided by the number of people living in the catchment area. (100 W for the Leman lake and 30 W for the Bourget lake.).

This shows that it is relatively simple to install pumping installations which are likely to change the lake ecology.

Some possible uses of pumps in a lake and its catchment area.

The list of possible applications given below is obviously limited and other uses are likely to appear, depending on specific problems.

Among others, we have selected the following list :

- Pumping or siphoning of hypolimnion,
- Pumping of epilimnion
- Horizontal displacement of water,

- Vertical displacement of water (mixing)
- Limitation of floods.

Most of these techniques have already been cited in the first part of our study and have been applied to some lakes in north America and Europe.

What we would like to enhance is the interest of using MYRIAFLOW techniques to enhance the quality of the treatment and reduce costs.

3.2 BASIC PRINCIPLES

The MYRIAFLOW concept.

The MYRIAFLOW concept has been historically proposed to take rid of polluted atmospheric layers in the absence of wind. The basic idea was to use a very great number of small fans distributed in the polluted area to push horizontally a part of the stratified layers limited in height.

To remove a 50 meter layer of air in a city like Paris at a mean velocity of 1 m/s (4 km/h), calculation showed that it is possible to use 100 000 fans distributed in the streets. Each fan would have a diameter of 500 mm and a power of 200 W.

The total power of the installation would be of 20 MW, which makes a power of some Watts per person living in the city.

Equivalent results could be obtained with giant fans alike wind generators, but it is obvious that the environmental impact would be much higher. Calculations also show that the investment would have been much higher. The energy consumption would also be higher.

This principles of using small fans or hydraulic machines instead of big ones has also been applied in wind tunnels or in cavitation tunnels. Also some innovative projects of ship propulsion have been studied which use similar approaches.

It is easy to show that the dimensions of the machines are simply calculated by application of known scaling laws (The Rateau invariants). From this, the weight of components can be calculated as well as the other operating parameters.

It is of course obvious that in most cases, the installation of a great number of small machines is much simpler than the one of bigger ones.

Scaling relationships.

These scaling techniques are utilised in the theory of models.

Considering two homothetic pumps, with a geometric scale ratio of λ, the following relation ships are readily obtained if the two pumps are working at a given reduced flow rate π_2 :

$$\pi_2 = \frac{Q}{\omega R^3}$$

Where Q is the volumetric flow rate, ω the angular velocity and R the radius of the runner.

We can define the Reynolds number of the machine :

$$R_e = \pi_3 = \frac{\omega R^2}{\nu}$$

Where ν is the cinematic viscosity of the fluid.

Provided the Reynolds number Re is high enough, the two homothetic pumps are characterised by a single curve giving the reduced energy parameter π_1 as a function of the flow parameter. This curve is called the characteristic of the pump.

$$\pi_1 = \frac{\Delta p}{\rho \ \omega^2 R^2}$$

Δp is the pressure difference (Pa) produced by the pump and ρ is the specific mass of the fluid.

As it is well known, a typical representation of this non dimensional characteristic curve is given by Figure 3-1.

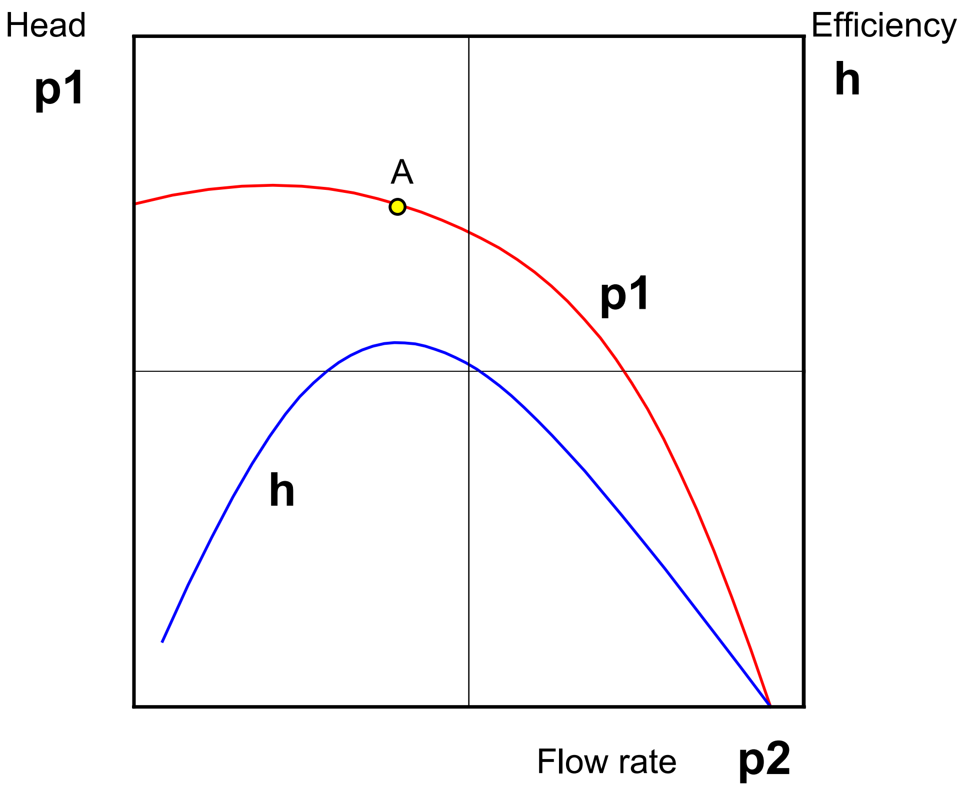

Figure 3-1: Typical non dimensional performance curve of a pump and its efficiency. A is an operating point.

From the Rateau invariant parameters, it is easy to derive the following rules:

Two geometrically similar pumps working at same non dimensional flow rate π_2 will generate the same pressure difference provided the tip velocity of the rudder be the same.

The Rateau head parameter is written :

$$\pi_1 = \frac{\Delta p}{\rho\ \omega^2 R^2}$$

and :

$$\Delta p = \pi_1 \rho\ \omega^2 R^2$$

The tip velocity is proportional to $R\omega$. Consequently, for the same fluid, the head depends only of this tip velocity and the value of π_1 at a given π_2 value.

Given two pumps (1 and 2) working at same reduced flow rate and same tip velocity, the ratio of the two flow rates is simply proportional to the square of the scale ratio.

This is also simply deduced from the equality of the Rateau parameters :

$$\pi_2 = \frac{Q}{\omega R^3}$$

From which one can write :

$$\frac{Q_2}{Q_1} = \frac{\omega_2 R_2^3}{\omega_1 R_1^3} = \frac{U_{T2}}{U_{T1}} \frac{R_2^2}{R_1^2} = \lambda^2 \frac{U_{T2}}{U_{T1}}$$

If the tip velocities are equal, the flow rate is proportional to the surface of the pump.

In those cases, the efficiency of the two pumps is the same.

3.3 PRACTICAL CONSEQUENCES

The number of pumps and their performances.

1. A pump of given diameter can be replaced by N pumps of smaller diameter. If the scale ratio between the two pumps equals λ, the number of smaller pumps giving the same flow rate is equal to λ^2.

For example if a large pump is replaced by a set of small pumps 10 times smaller, the necessary number of small pumps to obtain the same performance is of 100.

For example, a large pump of 5 meters will be replaced by 100 pumps of 500 mm or 400 pumps of 250 mm.

2. The power of each pump as well as its flow rate is inversely proportional to the number N of pumps.

In the above example, if the flow rate of the large pump is of 200 m^3/s, the flow rate in each 500 mm pump will be of 2 m3/s. It would be of 0.5 m3/s in a 250 mm pump.

If the power of the large pump was of 10 MW, the corresponding powers would be of 100 kW for each 500 mm pump and of 25 kW for the 250 mm pump.

3. The diameter of each pump is proportional to the scale λ. Consequently, providing the same technology is used, the mass of each pump is proportional to λ^3. If M_2 is the mass of the large pump, the mass M_1 of each small pump will be such that :

$$\frac{M_2}{M_1} = \lambda^3$$

4. The total mass of the whole set of n pumps is proportional to :

$$M_{tot1} = n\frac{M_{tot2}}{\lambda^3}$$

Or :

$$M_{tot1} = \lambda^2\frac{M_{tot2}}{\lambda^3} = \frac{M_{tot2}}{\lambda}$$

The conclusion is that the mass of the set of small pumps is much smaller than the mass of the big one.

This is a major advantage for the installation. It can be shown that civil engineering works can be very much reduced if a large hydraulic machine is replaced by a number of small ones.

Moreover, as in general small pumps are produced in large quantities and at low price, the price of the set of pumps will be lower than the price of the large pump.

Summary.

The summary of pumps characteristics is given in the following figure.

The example compares pumps of 5 m, 0.5 m and 0.25 m in diameter.

Scale	n	N rpm	Q	□p	P (1 pump)	P_{tot}	M	M_{tot}
□	1	1	$\square^{\square}$	□	$\square^{\square}$	$\square^{\square}$	$\square^{\square}$	□
1	$\square^{\square}$	□	1	1	1	$\square^{\square}$	1	1
5 m	1	150	200	50000	10 MW	10 MW	50 T	50 T
0.5 m	100	1500	2	50000	100 kW	10 MW	50 kg	5000 kg
0.25 m	400	3000	0.5	50000	25 kW	10 MW	6.25 kg	2500 kg

Table 3-1: Comparison of a large pump and a set of n pumps with the same overall performances.

These three sizes of pump have been chosen because they correspond to different civil engineering approaches.

Obviously, the large pump implies the use of large lifting machines such as cranes or bridges. Installing such a machine will require the construction of large structures and the time to put them in operation will be in the range of 1 year.

For a 500 mm pump the mass of which is of 50 kg, light lifting means are enough and, provided the equipment is available, it can be installed within a typical period of 1 week.

The smallest pump of 250 mm weights only 6 kg. It can be installed manually even though it may be necessary to use a large number of such individual machines to solve a given problem. The installation can take only one day or even less if necessary.

3.4 THE JET PRODUCED BY A PUMP OR A PROPELLER

The conservation of momentum.

When a jet is issued from a propeller or a pump, it is well known the axial momentum which it produces is conserved in the absence of external forces.

One can imagine a jet issued from such a machine with a mean velocity U_J. The surface of the jet is S_J. The corresponding momentum is q_J :

$$q_J = \rho \, S_J U_J$$

It is also known that the jet has a diverging angle which is constant and of about 12°. If α is this half angle of diffusion, the surface of the jet at any location distant of x from the jet exit is easily calculated.

D_x is the diameter of the jet :

$$D_x = D_J + 2x\mathrm{tg}\alpha$$

And :

$$S_x = \frac{\pi}{4}(D_J + 2x\mathrm{tg}\alpha)^2$$

If one considers a "top hat" velocity distribution, which means that the velocity is constant in the jet and equal t 0 outside, one calculate this mean velocity at x :

$$q_J = \rho \, S_J U_J^2 = \rho \, S_x U_x^2$$

$$U_x = U_J \sqrt{\frac{S_J}{S_x}}$$

or :

$$S_x = \frac{\pi}{4}(D_J + 2x\mathrm{tg}\alpha)^2$$

and :

$$U_x = U_J \frac{D_J}{D_J + 2\ x\ \mathrm{tg}\alpha}$$

As the velocity decreases, the flow rate increases. It is simply given by :

$$Q_x = \frac{\pi}{4} U_J\ D_J\ \left(D_J + 2\ x\ \mathrm{tg}\alpha\right)$$

We give on Figure 3-2 the flow rate and the mean velocity in such a jet versus the distance. The jet considered has a diameter of 500 mm and an initial velocity of 10 m/s. The initial flow rate is of 2 m^3/s. The semi angle of diffusion is of 6°.

It can be seen from the equation and the curve that the flow rate increases linearly with distance. At a reduced distance of 100, the flow rate is increased by 20 and the velocity is reduced by also 20 as a result of the conservation of initial momentum.

It can be said tat such a jet will be efficient as long as its velocity is higher than the natural adverse velocity of the flow. This velocity is in general due to the wind action and it is always quite low. This shows that a small propeller is able to displace very high quantities of water in natural fluid environment.

Moreover, as far as transport of water is concerned, it may occur at a very large distance in the absence of external forces such as wind. In the case of wind, a jet will be efficient until the velocity it induces is lower than the velocity due to wind.

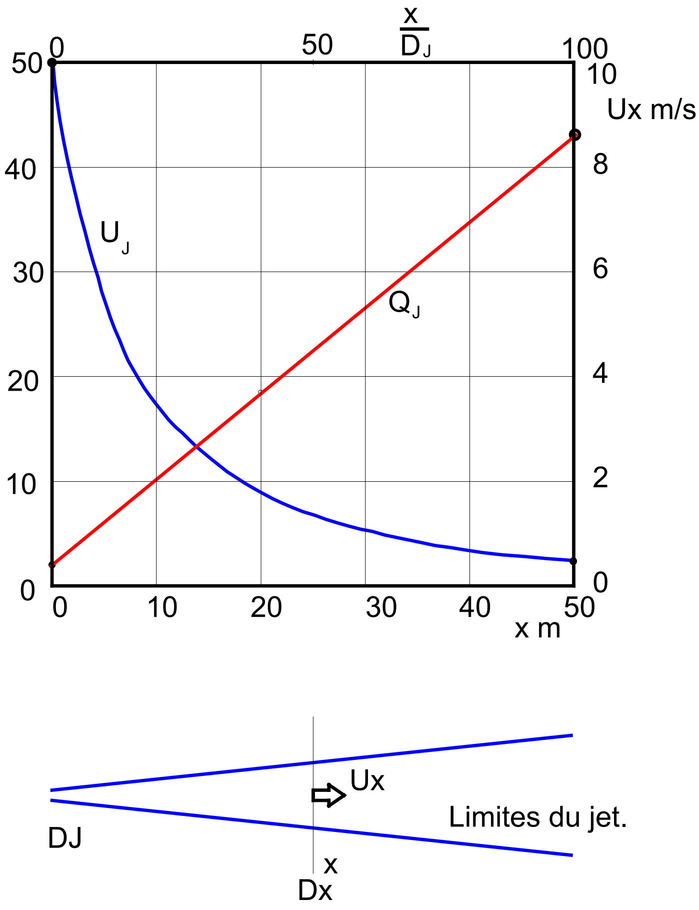

Figure 3-2: Characteristics of a jet versus the abscissa.

Of course, the efficiency of the jet will be better if the initial velocity U_J is lower. The power of such a free jet is equal to :

$$P = \eta\, Q_J\, \frac{1}{2} \rho U_J^2$$

Where η is the efficiency of the machine.

Installation.

Small propellers or pumps permit to make arrangements to move the water. These arrangements can be as follows :

- Point pumping.
- Line pumping,
- Surface pumping.

The best arrangement will be used to solve problems concerning at least a higher order in the bulk of fluid.

Some examples of application are given below.

3.5 APPLICATIONS AND POSSIBLE INSTALLATIONS

3.5.1 Horizontal displacement of water

Case of a harbour.

It may happen that a pollution or a lack of some dissolved species like oxygen appear in certain zones of a lake. This may be the case in the vicinity of a fish farm or in a harbour where concentrations of pollutants can become too high. Such a situation can be critical in a lake where there are no tides to mix the trapped water and the main water body. It can be made even more critical in summer in the absence of wind. For example, the result of such situations is a bad smell in some zones.

In general, the volume of water in such protected zones is smaller by orders of magnitude to the total volume of the lake, so that the impact of this mixing will be negligible.

For example, one can consider the size of a harbour of 200 m x 100 m x 3 m. This leads to a volume of 60 000 m^3. This volume is only 50 000 times lower than the volume of the smaller lake of the EUROLAKES project. Even a large harbour has a negligible volume compared to the volume of the lake where it is located.

In the case of a harbour, which concerns only the lighter water of the epilimnion, one can consider that a complete renewal of water within a day is enough. In our example, this means that 60 000 m^3/day must be pumped and dispersed in the lake. This gives a mean flow rate of 0.69 m^3/s, which will be rounded at 1 m^3/s.

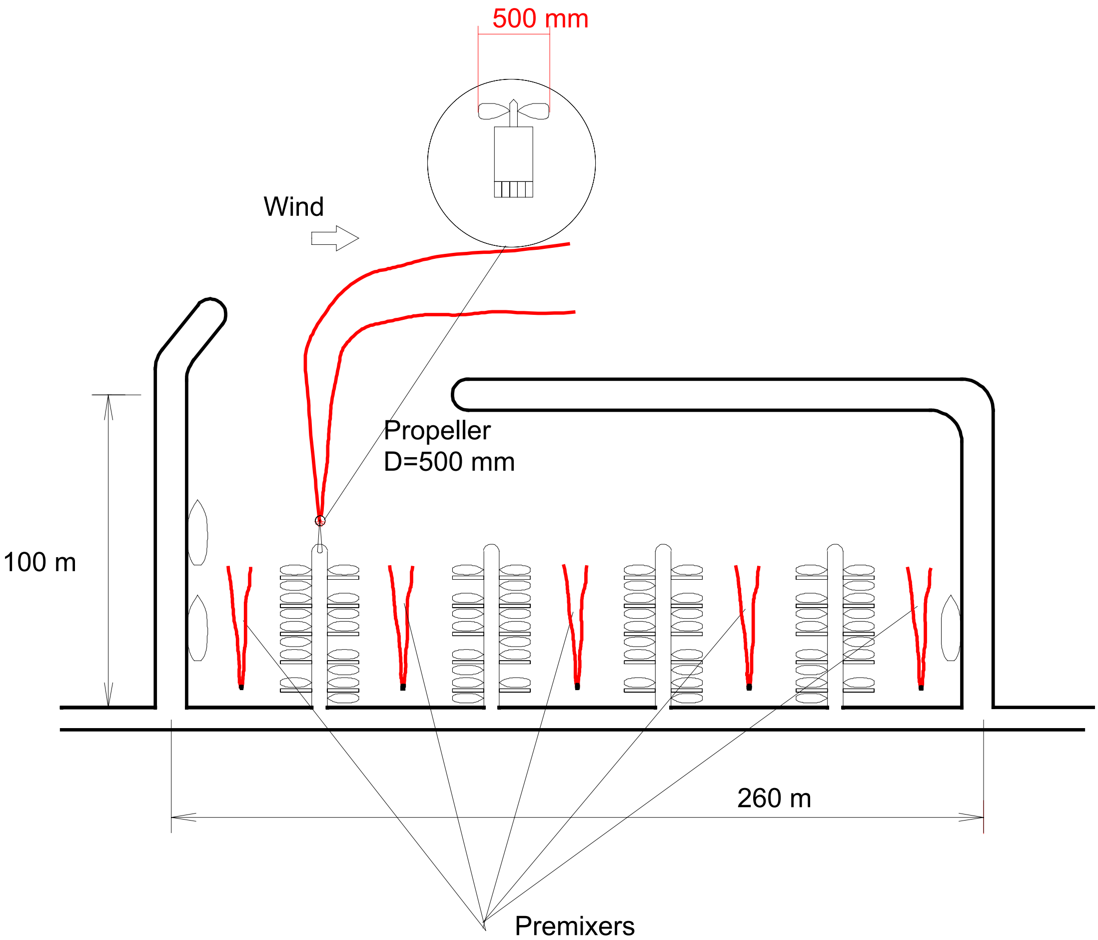

Figure 3-3: Depollution of a harbour.

In the design proposed on Figure 3-3 , the harbour has a surface of about 30 000 m^2. Its volume is of 100 000 m^3.

First of all, secondary propellers called "premixers" have been installed. They are used to mix the water contained in the harbour and to make its concentration in diluted species homogeneous. If floating species are in the harbour, the installation of these premixers should be near the surface.

In our example, 5 premixers have been prepared and their diameter is of 250 mm. The flow rate they produce is of 0.5 m^3/s. It increases in the jet as shown above.

The main propeller has a diameter of 500 mm and an initial flow rate of 2 m^3/s. When the jet leaves the harbour, its flow rate increases. As the exit of the harbour is at 40 meters from the propeller, the flow rate entrained is of 35 m^3/s. The water contained in this jet is mainly made of the homogeneously polluted water contained in the harbour.

This equipment permits a complete renewal of the water contained in the harbour in less than one hour. This means that the mean concentration in dissolved species is lowered to the half of its initial value.

Such an installation is extremely efficient to reduce dissolved pollution in short periods of time. It is of course very easy to installed due to the small size of the propellers. The large 500 mm propeller can be replaced by 4 small 250 mm propellers which can simply be moved by hand.

The power of each small propeller is of about 30 kW. Considering a total number of 10 small propellers sufficient to protect any harbour, the total installed power will be of 300 kW.

The force developed by each small propeller is of 5000 N. They can simply be installed on masts put in the bottom of the harbour.

Such an installation can be used to remove pollution from a harbour or any closed region in the lake. It can also be installed to prevent oxygen depletion in special regions, like fish farms for example. In all cases, the interest of such systems is that the time needed to remove the pollution is extremely short.

In case of accidents, the specific propellers can be simply replaced by the propellers of small boats anchored in such a way that the jets are in the good direction. This permits to have a very fast action in case of crisis.

Case of a beach.

Pollution due to an overuse of a beach can happen in summer in a lake in the absence of wind and the same technique can be utilised to remove dissolved matter or microorganisms.

In all cases, the water of the beach is light and it comes from the epilimnion in summer.

Three solutions are possible:

- First solution consists in removing the shore water and to send it to the core of the lake.
- Second solution will consist to do the contrary, which means that the water will be sent from the lake to the beach.
- The third solution will be to push the polluted water in a direction parallel to the coast.

There is no best solution, because this depends on each configuration. What is important is that the water near the beach can be renewed within a period of one day or less with very simple and cheap means.

An example of such an arrangement is given on Figure 3-4. The 5 propellers are fixed under a wharf mounted on pillars so that it is transparent to the flow. The characteristic of each propeller is as follows :

Q=0.5 m3/s,
D=0.25 mm,
U_J=10 m/s
F=5000 N.
P=30 kW
Electric motor.

The Total force acting on the flow is of 25 000 N and the propellers have to be arranged in such a way that friction on the lake bottom be low enough to prevent erosion and loss of momentum.

In the absence of friction, the flow rate entrained at a velocity of 0.5 m/s which is already a high velocity would be of 50 m^3/s. Considering a total volume of the water in the vicinity of the beach of 500 000 m^3, the time required to entrain this volume and lower the concentration by 2 will be of 10 000 s or 3 hours.

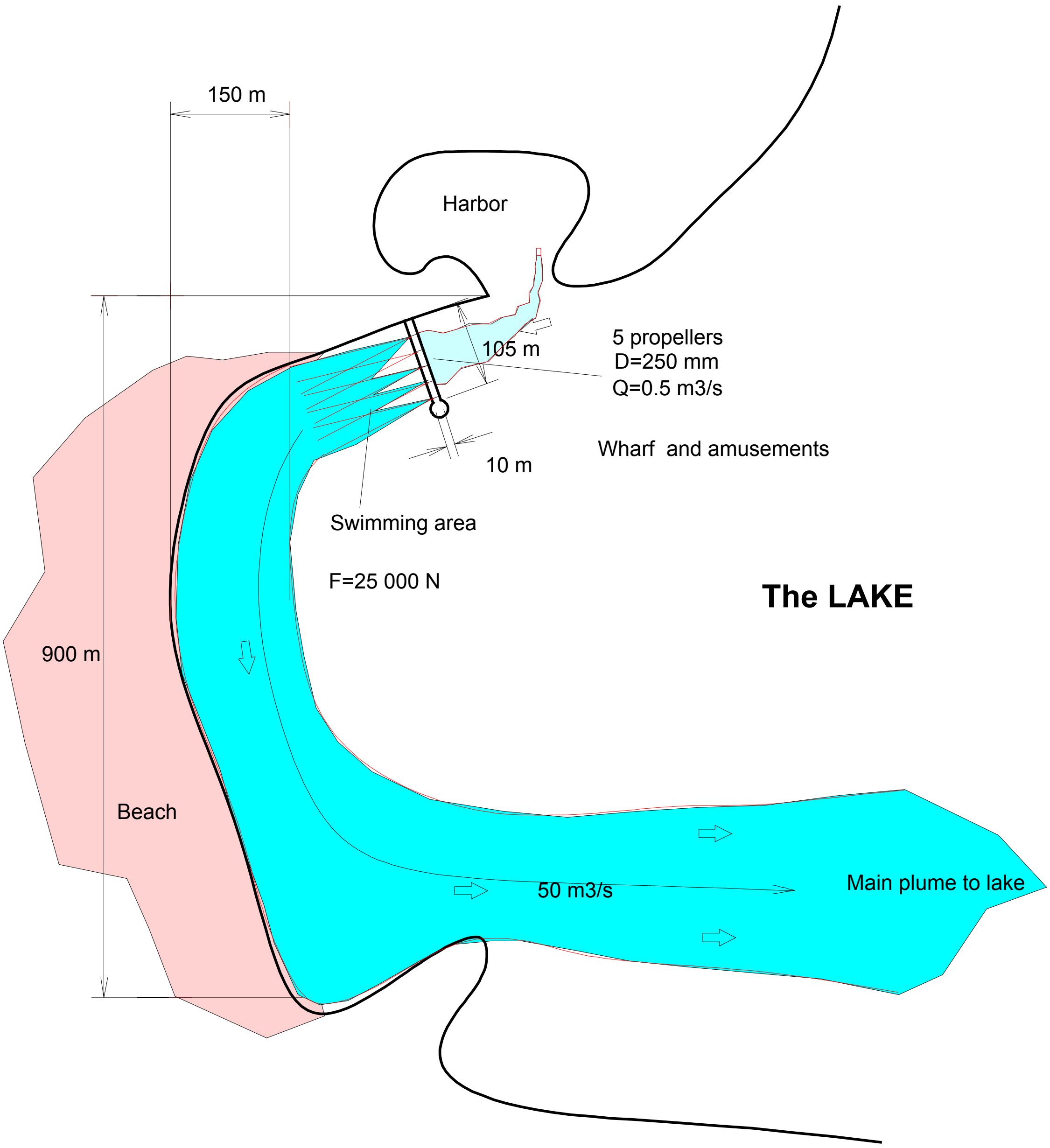

Figure 3-4: Example of beach equipment.

This means that the possible pollution is destroyed or at least highly decreased within less than 1 day.

Such an equipment can be used for other purposes.

For example, it is possible to install a swimming or boating area at the exit of the pump wall where surface flow velocity can be high enough to train or play. The supporting structure of propellers can be equipped as a promenade wharf where shops, parking lots or amusements are likely to be installed.

This would increase the available place in the lake and could be attractive to tourists and possibly sportsmen.

In any case, the impact of such installations on lake ecology should be studied, but it is obvious that the MYRIAFLOW system will reduce dramatically the local concentrations of dissolved and possibly floating matter. This could be studied in some detail and for each location in the lake by use of the calculations developed during the EUROLAKES project.

The financing of the MYRIAFLOW capital and operating expenses would of course be a part of the users charges during the vacation periods. For the local authorities concerned with localised pollution problems, this type of integrated installation would not only solve the problems but also create situations likely to increase the tourism capacity and improve the amusement offer.

3.5.2 Pumping of epilimnion

There are circumstances where the water level in the lake is too high due to the weather. There are also circumstances where it mould be desirable to lower the level of some tens of centimetres. This has been used in Canada to freeze shore sediments and remove certain makrophytes.

If one considers that a lowering of 50 centimetres is needed, the volume to process is equal to the surface of the lake in square meters time this 0.5 m. For example, in the case of the Bourget Lake, the volume to transfer would be of about $30\ 10^6\ m^3$.

A typical time to do this could be of 1 month. This implies that a flow rate of 11 m^3/s be pumped out of the lake during one month.

The problem is not to look at the way the water could be evacuated. A canal could be built to do this and the water sent to the Rhône river. What we would like to show is that a large pump of 11 m^3/s can be used, but that many small pump of the same head will be much simpler to install.

For example, for such a process, small pumps of 5 mCW head and 100 l/s flow rate could be used. This means that 110 such pumps would have to be installed. The diameter of the impeller of such machines could be of 125 mm. and the size of the pipes at inlet and outlet could be of 300 mm. The power of each pump would be of around 8 kW, taking into account the efficiency of the machine. The total power of the installation would consequently be of 660 kW.

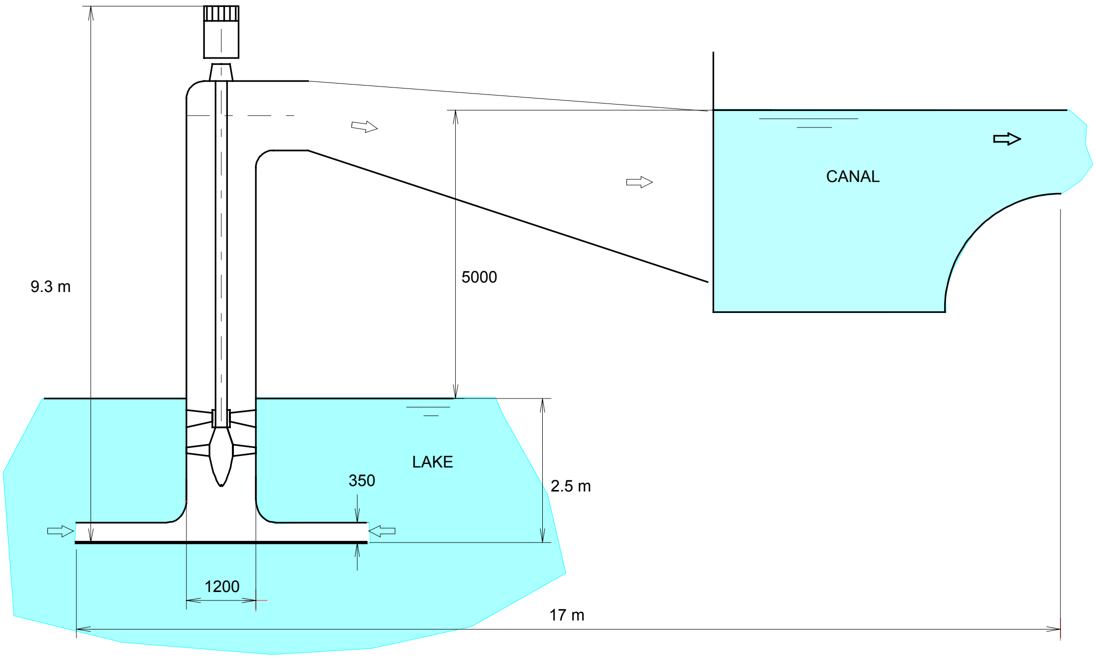

Figure 3-5: Installation of a large pump with a capacity of 11 m^3/s.

Figure 3-6 shows a sketch of a pumping station using such small pumps. It is compared on Figure 3-5 to a pumping station using a large pump having the same overall capacity, but a runner diameter of 1.2 m instead of 250 mm.

For that large pump, civil engineering works will be necessary. For the set of 110 small pumps, the installation is much lighter, even though the plant has to be extended in length.

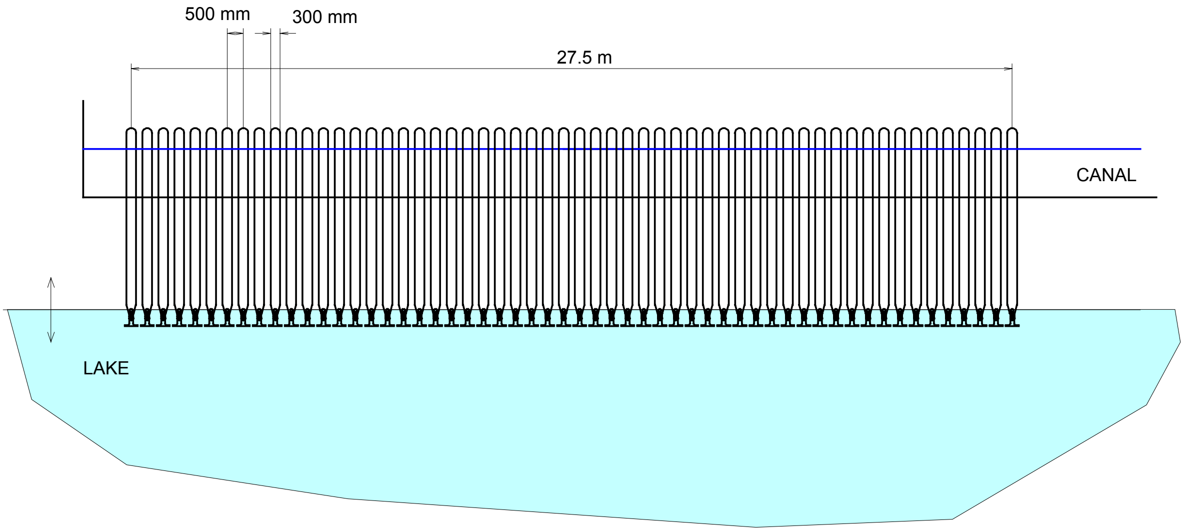

Figure 3-6: Installation of a set of 110 pumps (2 lanes) with a capacity of 11 m^3/s.

For such an application, it is clear that the second solution needs much less civil engineering than the former and that it will cause a lighter impact on environment.

The efficiency and performances of both solutions are identical.

This solution can of course be applied to other situations such as the filling of a lake from the neighbouring river or the transfer of water from a channel to another.

The solution is always interesting when complex civil works are required. In terms of safety, it is also very interesting because some pumps can be stopped for repair without major effect on the global performance.

The same concept can also be applied to turbines and very simple solutions can be designed if the unit is smaller.

3.6 VERTICAL DISPLACEMENT OF WATER

Principle : The air column.

This type of technique has also been applied in Canada for certain purposes which have been described in report N°1. These purposes regard the chemistry and biochemistry of lakes.

Again our aim is to show that such actions can be made at the scale of the lake in such a way that it can modify its natural behaviour.

The idea is to mix the epilimnion with the hypolimnion. To achieve this, it is necessary to cross the thermocline and to mix the deep water to the upper water or to do the contrary.

It may be interesting to use a simple pump or a set of small pumps. The depth at which the intake should be is of 10 to 30 meters. We can consider an average depth of 20 meters for the thermocline.

The theoretical pressure necessary to move the hypolimnion water and transport it to the top of the epilimnion is equal to :

$$\Delta p = \Delta \rho g H$$

Where $\Delta\rho$ is the density difference of the water contained in the hypolimnion and the surface water g the acceleration of gravity and H the height of the epilimnion.

If one considers a maximum temperature of 32° in surface and a temperature of the hypolimnion of 4°C, the maximum density difference is of :

$$\Delta \rho = 1000 - 995 = 5\ \mathrm{kg/m^3}$$

Considering an epilimnion height of 20 m, the pressure difference to create is of

$$\Delta \mathrm{p} = 5\ 9.81\ 20 = 1000\ \mathrm{Pa}$$

This corresponds to an head of 0.1 mCW.

As the water has to be transported on a length of 30 meters, the friction head in a pipe of 1 m will be of about 1 times the dynamic pressure.

Considering a pipe of 1 meter and a flow velocity in this tube of 2 m/s, the total head loss would be of 0.3 mCW.

To achieve this, a propeller pump might be used, but it would have to run at a very low tip velocity and its price would be very high.

Another solution consists to create an air injection in the bottom of the pipe. The movement of water will be induced by the pressure difference between the two phase flow and the surrounding water.

Precise calculations could be made by use of the procedure which we have developed for the Airlac.

An order of magnitude can be readily given by supposing that the mean volumetric mass difference between the two phase flow is constant and equal to $\Delta\rho'$.

The system can work provided the following equation holds :

$$\Delta\rho g H + \Lambda \frac{L}{D}\frac{1}{2}\rho U^2 + \frac{1}{2}\rho U^2 = \Delta\rho' g H'$$

With the values proposed above and H'=30 m, one finds:

$$\Delta\rho g H + \left(\Lambda \frac{L}{D} + 1\right)\frac{1}{2}\rho U^2 = \Delta\rho' g H'$$

or :

$$\Delta\rho' = \frac{\Delta\rho g H}{g H'} + \frac{+\left(\Lambda \frac{L}{D} + 1\right)\frac{1}{2}\rho U^2}{g H'}$$

$$\Delta\rho' = \frac{5\ 20}{30} + \frac{2\frac{1000}{2}4^2}{10\ 30} = 57$$

To produce the necessary flow, the mean volumetric mass of the two phase flow has to be of 6%

Practically, a flow rate of gas of about 50 l/s at a depth of 30 m must be used to entrain a water flow rate of 2 m^3/s.

The theoretical isothermal power is of

$$P_{th} = q\ p_{inj}\ Ln\frac{p_{inj}}{p_a}$$

Where p_{inj} is the pressure at injection and p_a the atmospheric pressure.

One calculate :

$$P_{th} = 60\,10^{-3}\ 4\,10^{5} p_{inj}\ Ln4 = 33000W$$

Taking into account the efficiency of the compressor, a power of 50 kW is necessary to move 2 m^3/s.

If a pump was used, the calculations shows that this power would be reduced to about 10 kW for the same 2 m^3/s.

The choice between the 2 will be done by comparing OPEX and CAPEX.

It may be noted that a certain quantity of oxygen will be transferred if the bubbles were used, but the efficiency would not be very good as we have shown in the study of oxygenators.

Example.

If one considers that the height of the epilimnion is of 20 m, such a treatment will have an effect on concentration and temperature if This epilimnion is mixed to a quantity of water coming from below equal to its volume in a typical period of 100 days (3 months)

The volume of water to process is of 20 10^6 m3 for each km^2. The corresponding flow rate is of 2.3 m^3/s.

This calculation shows that column pumps of 1 m^2 scattered in the lake each km^2 can produce a sensible effect.

As in the case of oxygenators, it may be interesting to install larger equipment, a good compromise being for example columns of 3 m. In that case, the efficiency is somewhat better and the surface covered by each column would be of 10 km^2.

Figure 3-7 shows a sketch of such a single mixing column installed in a lake.

Mooring is not presented in this figure.

As the thermocline tends to be horizontal in the lake, it is not necessary to perfectly scatter the columns in the lake. It is practically enough to install some columns in regions which are morphologically different, like basins.

For a given area, the columns can be arranged in clusters and this permits a single compressor or limitations of electric cables.

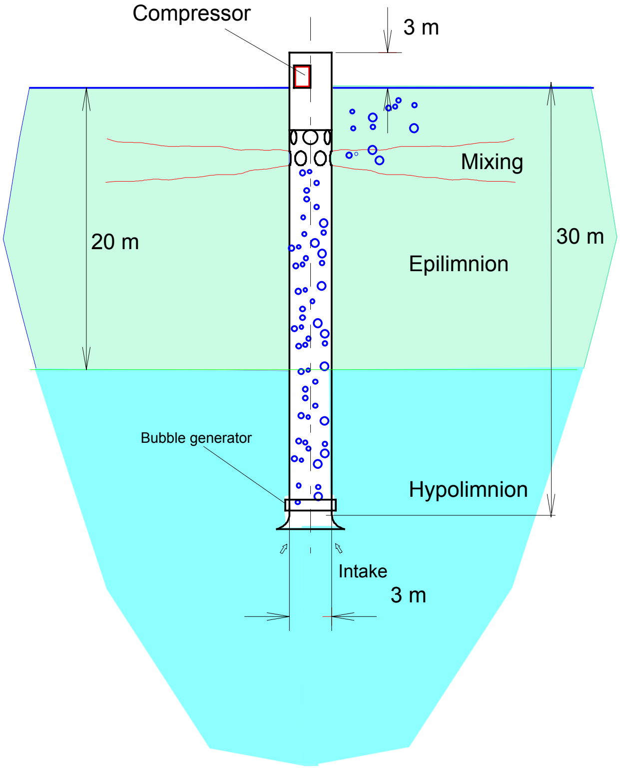

Figure 3-7: Water column used for vertical mixing on a surface of 10 km^2.

In some circumstances, mixing has been made by a simple injection of air. We have made some calculations on this process and the result is that the efficiency is much lower than the efficiency of water columns.

4 LIMITATION OF FLOODS.

Foreword.

Flooding of regions situated at the inlet or outlet of lakes is likely to occur due to heavy rain, underground surges or snow melt.

There are few means to prevent this flooding, but this may be done in some situations, especially if the slope of the rivers entering or leaving the lake is very small.

There is also a possibility to lower the level of the lake, but this takes too much time.

Another solution is to inject momentum in the rivers to increase their flow rate by creation of an artificial slope.

Free surface permanent flows.

The normal regime of a river is characterised by a "normal" depth which depends on the slope, the hydraulic radius and the Chézy coefficient C which is itself a function of the hydraulic radius and the roughness of the bottom[1].

This equation is simply the equality of the projection of the weight of water on the river direction and the forces due to friction.

- Projection of the weight of water on a length L :

$$F_1 = \rho\, g\, L\, S\, i$$

- Friction :

$$F_2 = C_f \frac{1}{2} \rho U^2 \chi L$$

χ is the wetted perimeter and U the mean flow velocity.

The Chézy constant is defined by :

$$C = \sqrt{\frac{2g}{C_f}}$$

It is a function of the shape of the river, characterised by the hydraulic radius R_H and the roughness :

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[1] R. COMOLET. Mécanique expérimentale des fluides. Tome 2. MASSON.

$$R_H = \frac{S}{\chi}$$

Figure 4-1 represents the situation:

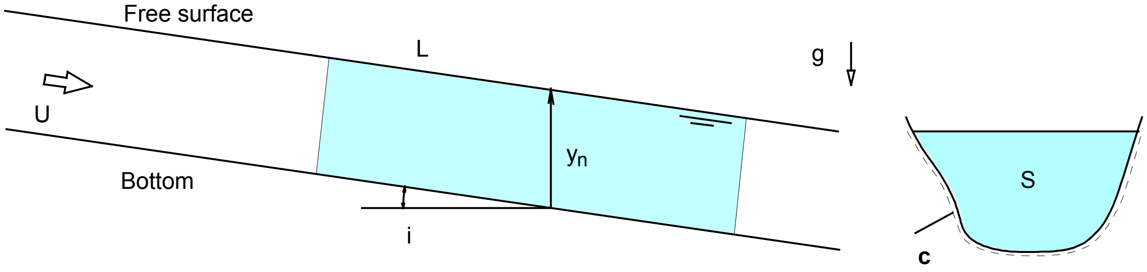

Figure 4-1: Notations.

The mean flow velocity is given by :

$$U = C\sqrt{R_H i}$$

And the flow rate can be written :

$$Q = SC\sqrt{R_H i}$$

The Chézy constant C can be calculated by application of the Manning formula[2].

$$C = \frac{1}{n} R_H^{\frac{1}{6}}$$

n depends on the bottom roughness.

One finds :

$$Q = \frac{1}{n} S R_H^{\frac{2}{3}} \sqrt{i}$$

In the case of a rectangular channel of width b and normal depth y, which is the maximum height of the river before flooding, the flow rate can be written:

---

[2] Ven Te Chow, Open Channel Hydraulics, Mac Graw Hill Classic Text Book reissue. 1959.

$$Q = \frac{1}{n} by \left(\frac{by}{b+2y} \right)^{\frac{2}{3}} \sqrt{i}$$

As an example, a river of width b of 30 m with a depth of 4 m, a typical flow rate can be of 100 m^3/s.

The corresponding Manning coefficient with a slope of 5%₀ is calculated :

$$n = \frac{1}{Q} by \left(\frac{by}{b+2y} \right)^{\frac{2}{3}} \sqrt{i} = 0.058$$

We can calculate K, flow coefficient of the river :

$$Q = \frac{1}{n} by \left(\frac{by}{b+2y} \right)^{\frac{2}{3}} \sqrt{i} = K\sqrt{i}$$

K is given on Figure 4-2:

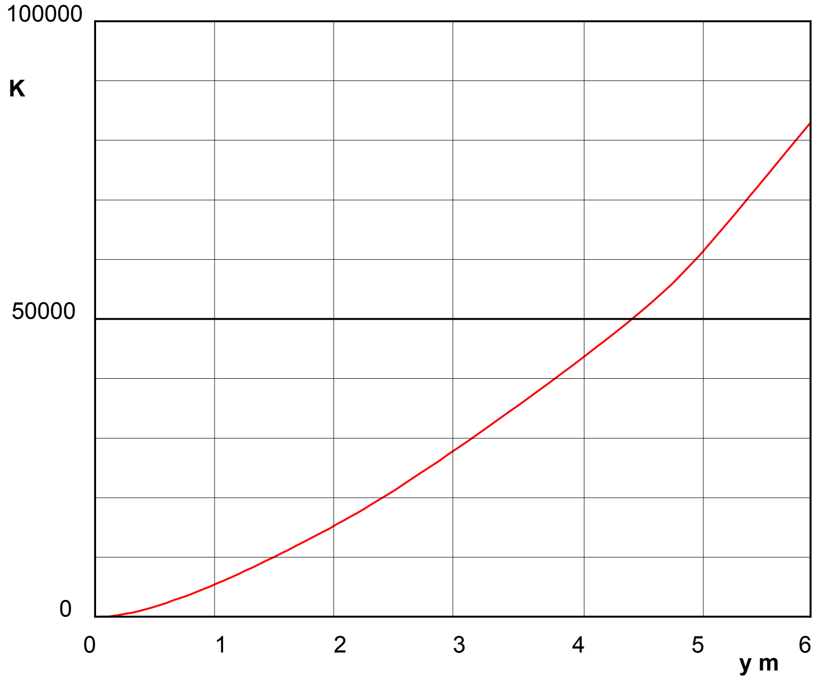

Figure 4-2: Flow coefficient (m^3/s/$rad^{1/2}$) of a rectangular channel of width 30 m with a Manning coefficient of 0.058.

For example, it is shown that slope of 5%₀ gives a flow rate of 100 m^3/s.

If the slope was doubled, the flow rate would reach 142 m^3/s.

4.1 INCREASE THE SLOPE

To increase the flow rate by 40% the slope has to be increased by a factor of 2. This is of course difficult to realise practically.

An equivalent solution consists to push the flow with a force equal to the projection on its bottom of the weight of the river.

With the above values, and a depth of 4 meters, the weight of water on a length of 100 meters is of :

$$P = Mg = \rho LbSg$$

The mass of water is of 12 10^6 kg for 100 m. The weight is therefore of 120 10^6 N.

The component of this force on the river bottom is of :

$$F_1 = Pi = \rho LbSgi$$

And this gives a force of 6000 N. It can be considered as low to increase the flow rate of the river of 40%.

Possible solution.

To create that force of 6000 N, a solution is to put into operation a propeller or a set of propellers which cumulated thrust is of 6000 N for each length of 100 m.

This can be done with motor boats at rest. One can suppose that the speed of the jet escaping the propeller is of 10 m/s.

The cumulated necessary areas of propellers would be given by :

$$F_1 = \rho Q U_j = \rho S U_j^2$$

To create the force of 6000 N, the cumulated surface should be of 0.06 m^2, The equivalent diameter would be of 0.276 m.

The theoretical power would be given by :

$$P = F_1 S = \rho S^2 U_j^2$$

For the example cited, a power of 60 000 W would be necessary for each river length of pour 100 m.

Energy expenses.

Taking into account the low efficiency of propellers, the necessary power would be of about 120 kW for each 100 m. In general, the rivers entering or leaving the lake do not extend on very long distances with a low slope. For example, a maximum of 10 km seems reasonable.

This means that the total power will be of 12 000 kW to solve the reference problem.

These values are based on high velocity jets of 10 m/s. The efficiency would increase with slower jets, bur the number of propellers would also increase.

Number of boats.

In such a process, it is always interesting to increase the number of points where the forces are applied. This permits to avoid too high a velocity at the bottom of the river, which would increase friction. The use of small propellers permits the mixing of the jets very near from the point of application.

For example, a solution would be to install 4 boats of 30 kW each at a distance of 25 meters? Such boats are available in most lakes. For a river length of 10 km, 400 such boats would be enough to limit the flood.

Figure 4-3 shows how boats could be moored to avoid a great number of pilots.

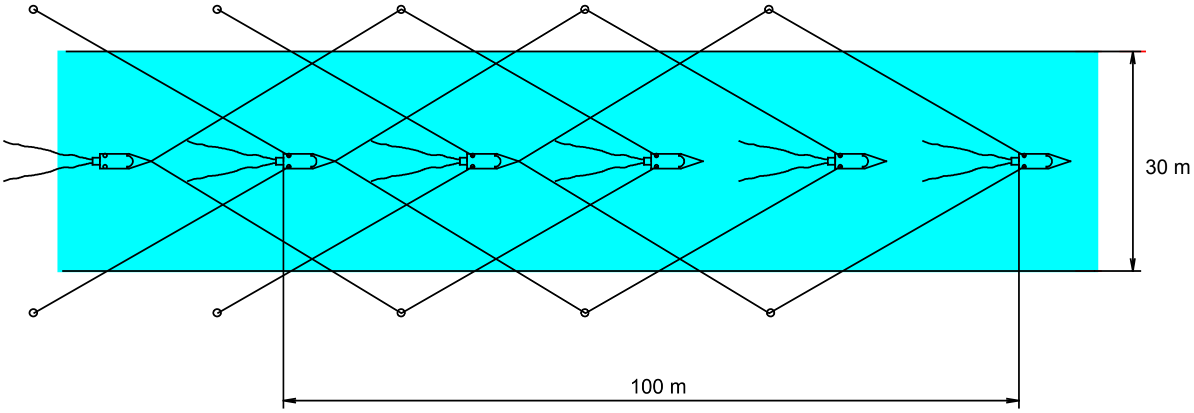

Figure 4-3: Mooring of boats.

Any boat available on the lake could be used in this operation.

Conclusion.

This last example shows that there exist solutions based on the use of scattered propellers which permit to solve problems at the scale of the lake.

We are convinced that a lot of such problems could find practical solutions based on low energy consumption compared to the volume of water which has to be moved.

5 THE HYDROBARRIER

5.1 FOREWORD

Principle.

Accidental or permanent liquid pollution may be produced in the vicinity of a lake and the liquids released penetrate the underground water.

This can be the case in case of an accident of truck or train or in case of a pipe line breakage. This can also be the case when a plant or a gas station produces badly controlled liquid effluents which permanently pollute the water table.

Finally, diffuse pollution is produced by agriculture and it heavily charges the water with nutrients like nitrates or phosphorus and this can have an effect on the lake ecology.

The HYDROBARRIER is a process by which the water coming from the catchment area is pumped before it reaches the lake. This water can then be treated before it is re-injected in the lake.

This type of pollution is typical of a line pollution.

Some orders of magnitude.

The mean residence time (MRT) of the lakes under study in the EUROLAKES is typically of 5 years. This means that, if evaporation is neglected, the mean flow rate of water entering the lake is equal to :

$$Q = \frac{V}{MRT}$$

We have calculated the mean flow entering the lakes under study.

	Volume m^3 10^{-6}	Max depth (m)	Catchment area km^2	Mean residence time (Year)	Mean flow rate m^3/s
Bodensee (Lac de Constance)	48 530	252	10 900	4.5	340
Lake Geneva	88 900	310	7 975	11.8	240
Loch Lomond	2 600	190	696	1.9	44
Bourget Lake	3 600	145	560	7.0	16

Table 5-1: Some figures of EUROLAKES.

Most of the lake under study are elongated with a typical ratio of length L to width l of 5. A rough estimation of their mean depth is of 0.3 times their maximum depth. We can estimate the perimeter of the lake by :

$$V = 1.5\, l^2\ h_{max}$$

or :

$$l = \sqrt{\frac{V}{1.5 h_{max}}}$$

and

$$\chi = 12\, l = 12\sqrt{\frac{V}{1.5 h_{max}}}$$

	Volume m^3 10^{-6}	Max depth (m)	Mean flow rate m^3/s	□ m	Specific flow rate $m^3/s/km$
Bodensee (Lac de Constance)	48 530	252	340	136	2.5
Lake Geneva	88 900	310	240	166	1.44
Loch Lomond	2 600	190	44	36	1.22
Bourget Lake	3 600	145	16	48	0.33

Table 5-2: Specific linear flow rate for EUROLAKES.

It appears that the mean specific flow decreases when the lake is larger. The order of magnitude is of 1 l/s/m.

Of course, not all the water coming to the lake comes from groundwater. Rivers and streams also participate to this process, but our only goal was to give orders of magnitude.

Considering the fact that the flatter lands have a larger population and are more likely to pollute, the specific flows given above are a good estimate of groundwater flows going to the lake.

This means that a typical supply of 1 m^3/s/km of groundwater might have been an estimate for the following. In fact, as will be shown below, the flow rates through aquifers are much lower than this value and depend mainly on the type of soil and the slope of the water table.

5.2 THE HYDROBARRIER - THE PRINCIPLE

The HYDROBARRIER consists of a number of wells distributed along the shoreline. These wells pump the water in the ground in such a way that there is a line along which the height of underground water is slightly lower than the height in the lake. This line is the shore line.

If the wells are evenly distributed, and the ground is supposed to be homogeneous, the points at which the flow coming from the lake must be zero are the point A_i on Figure 5-1.

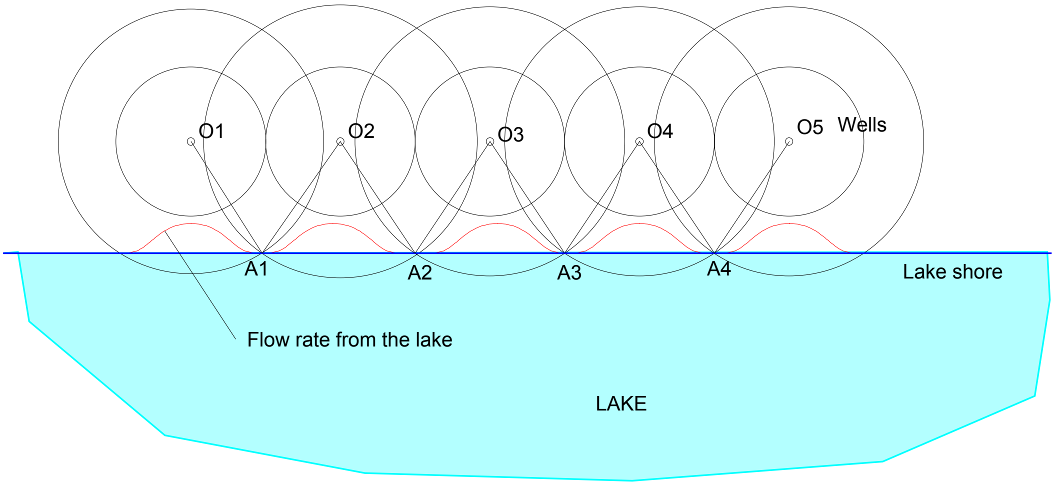

Figure 5-1: Principle of the HYDROBARRIER and optimal flow distribution at the lake shore.

The wells are noted O_1, O_2 etc…

If the flow pumped at each well is high enough, it is obvious that there exists a certain drawdown for which the condition of zero flow at points A_i can be reached. This is the condition for which the lake is protected from any groundwater pollution.

The flow rate of extracted water can be treated by conventional means to take rid of pollutants such as floating pollutants like gasoline or dissolved pollutants like nitrates.

The Darcy law.

We found it useful to recall some concepts derived by Darcy, concerning the flow of liquids in porous media.

We also give some orders of magnitude of the parameters involved.

The filtration velocity of a fluid through a porous medium is the ratio of the flow rate divided by the surface of the soil. It is the flow per unit surface of flow which is different from the actual velocity of the fluid which is always higher due to the porosity of the medium.

In the case of a flow of cold water, this relationship is simply called the Darcy law :

$$U = K\frac{\Delta h}{L} = Ki$$

h is the height of the aquifer and L a length perpendicular to the flow direction.

The dimension of K is in m/s.

The intrinsic permeability k of a medium takes into account the viscosity of the fluid. Some calculations give :

$$K = k\frac{g}{\nu} \qquad \text{or} \qquad k = K\frac{\nu}{g}$$

□ is the cinematic viscosity of the fluid.

The dimension of k is m^2.

In the case of water at 20°C, calculations give :

$$K = 10^7 k$$

A classical way to present the intrinsic permeability is to use the Darcy[3].

$$1 \text{ Darcy} = 10^{-12} \text{ m}^2$$

The dimension of Darcy is as follows :

---

[3] H. Darcy. Les fontaines publiques de la ville de Dijon. V.Dalmont Paris. 1856.

$$1\ \text{Darcy} = \frac{1\text{cm}/\text{s x 1centipoise}}{1\text{atm}/\text{cm}}$$

$$U = \frac{k}{\mu}\frac{\Delta p}{L}$$

An interesting (although approximate) relationship concerns an evaluation of the permeability. it is admitted that K is mainly a function of d_{10}. where d_{10} is the diameter of the size distribution of grains for which 10% of remaining grains are smaller.

$$K = N\, d_{10}^2$$

K is very often given in cm/s.
N is a shape coefficient the order of magnitude of which is of 100. It is of about 50 for clayey sands and of 140 for sands.
d_{10} is given in cm.
The temperature is of 20°C ($\mu=10^{-3}$ Pa.s)

Some typical values are given below. G. Castany[4] (page 89). They are compared to calculated values with N=100.

The results show that there are large discrepancies between the calculated and the actual values. The consequence is that it is always necessary to make hydraulic tests to characterise the permeability of underground soils.

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[4] G. Castany. Principes et méthodes de l'hydrogéologie. DUNOD 1982.

Sediment	d_{10} mm	K measured m/s	K calculated m/s.
Gravel.	2.5	$3\ 10^{-1}$	$6.25\ 10^{-2}$
Sand.	0.25	$2\ 10^{-3}$	$6.25\ 10^{-4}$
Medium size sand.	0.125	$6\ 10^{-4}$	$1.56\ 10^{-4}$
Fine sand	0.09	$7\ 10^{-4}$	$8\ 10^{-5}$
Very fine sand.	0.045	$2\ 10^{-5}$	$2\ 10^{-5}$
Sand + Silt.	0.005	$1\ 10^{-9}$	$2.5\ 10^{-7}$
Silt.	0.003	$3\ 10^{-8}$	$9\ 10^{-8}$
Clay+Silt.	0.001	$1\ 10^{-9}$	10^{-8}
Clay.	0.0002	$5\ 10^{-10}$	$4\ 10^{-10}$

Table 5-3: Permeability measured and calculated for various soils.

We give also below the corresponding values of permeability coefficient and intrinsic permeability for the calculated values given above.

Sediment	d_{10} mm	K_0 m^2	K_0 cm^2	K_0 Darcy	K calculated m/s
Gravel.	2.5	$6.25\ 10^{-9}$	$6.25\ 10^{-5}$	$6.25\ 10^{3}$	$6.25\ 10^{-2}$
Sand.	0.25	$6.25\ 10^{-11}$	$6.25\ 10^{-7}$	62.5	$6.25\ 10^{-4}$
Medium size sand.	0.125	$1.56\ 10^{-11}$	$1.56\ 10^{-7}$	15.6	$1.56\ 10^{-4}$
Fine sand	0.09	$8\ 10^{-12}$	$8\ 10^{-8}$	8	$8\ 10^{-5}$
Very fine sand.	0.045	$2\ 10^{-12}$	$2\ 10^{-8}$	2	$2\ 10^{-5}$
Sand + Silt.	0.005	$2.5\ 10^{-14}$	$2.5\ 10^{-10}$	$2.5\ 10^{-2}$	$2.5\ 10^{-7}$
Silt.	0.003	$9\ 10^{-15}$	$9\ 10^{-11}$	$9\ 10^{-3}$	$9\ 10^{-8}$
Clay+Silt.	0.001	10^{-15}	10^{-11}	10^{-3}	10^{-8}
Clay.	0.0002	$4\ 10^{-17}$	$4\ 10^{-13}$	$4\ 10^{-5}$	$4\ 10^{-10}$

Table 5-4: Intrinsic permeability and permeability values given with various units.

Calculation of flow rates.

As already said above, the flow rate at any point A must be zero. This condition is reached if the specific flow rate coming from the groundwater in the absence of pumping equals the flow rate pumped by the combination of two neighbouring wells. This is a simplification in which the influence of non neighbouring wells is neglected.

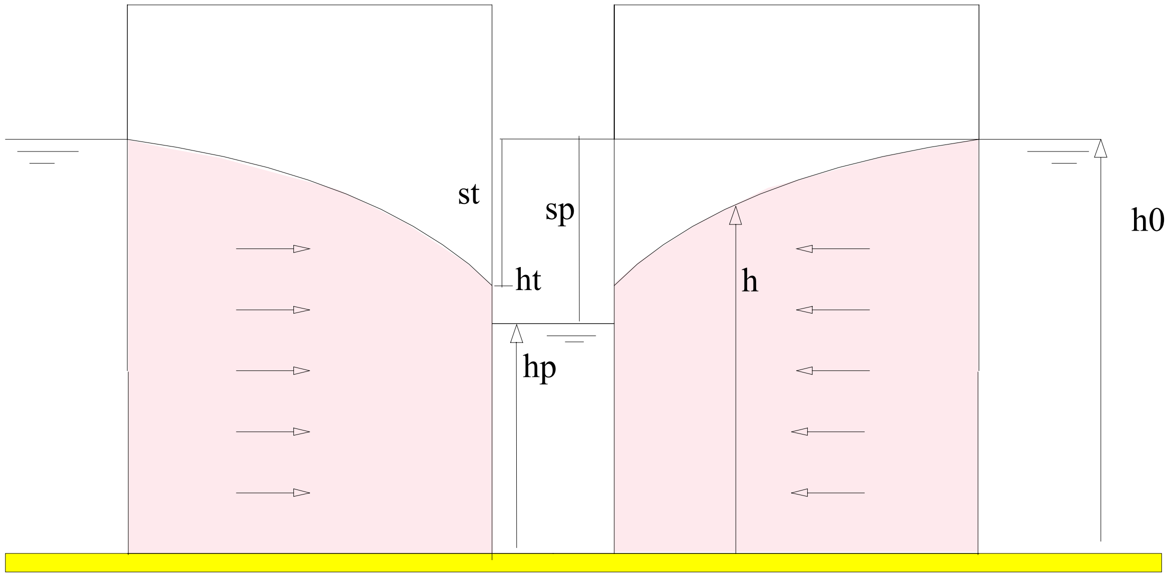

Figure 5-2: Underground water table.

If one considers a water table like that illustrated on Figure 5-2, with an height equal to h_0, the flow rate is given by :

$$Q = \pi \rho g \frac{k}{\mu} \frac{(h_0 + h_t) s_t}{\mathrm{Log} \frac{r_0}{r_p}}$$

Where h_t is the drawdown at the well periphery. k is the intrinsic permeability of the soil, μ the viscosity of water and r_p the radius of the well.

In general, a good estimation is to fix the drawdown at half the height h_0. The flow rate becomes :

$$Q = \pi \rho g \frac{k}{\mu} \frac{(h_0 + 0.5\, h_0) 0.5\, h_0}{\mathrm{Log} \frac{r_0}{r_p}} = \pi \rho g \frac{k}{\mu} \frac{0.75\, h_0^2}{\mathrm{Log} \frac{r_0}{r_p}}$$

The flow per unit length naturally induced by the pumping at any radius r far from the well is given by :

$$q_r = \frac{1}{2\pi r}\pi\rho g \frac{k}{\mu}\frac{0.75\, h_0^2}{\mathrm{Log}\frac{r_0}{r_p}} = \frac{1}{2r}\rho g \frac{k}{\mu}\frac{0.75\, h_0^2}{\mathrm{Log}\frac{r_0}{r_p}}$$

The natural flow rate of the water table in the absence of pumping is simply given by the Darcy law for each meter width q :

$$q = \frac{k}{\mu}\frac{\Delta p}{L} h_0 = \frac{k}{\mu}\frac{\rho g \Delta h}{L} h_0 = \frac{k}{\mu}\rho\, g\, i\, h_0$$

Δp is the pressure difference over the length L. i is the natural slope of the soil or of the water table.

A possible installation of the wells is given in figure where the distance of two wells is equal to the distance of each well to the shore r_0.

The distance r_1 of any point A_i to any well is equal to :

$$r_1 = r_0\sqrt{2}$$

Combining the velocity induced by the natural slope (q)and the velocity induced by pumping of two neighbouring wells (q_1)gives the schema of Figure 5-3.

The solution giving zero flow at any point A_i is such that :

$$\frac{k}{\mu}\rho\, g\, i\, h_0 = \frac{1}{2r_1}\rho g \frac{k}{\mu}\frac{0.75\, h_0^2}{\mathrm{Log}\frac{r_1}{r_p}} = \frac{1}{2\sqrt{2}r_0}\rho g \frac{k}{\mu}\frac{0.75\, h_0^2}{\mathrm{Log}\frac{r_1}{r_p}}$$

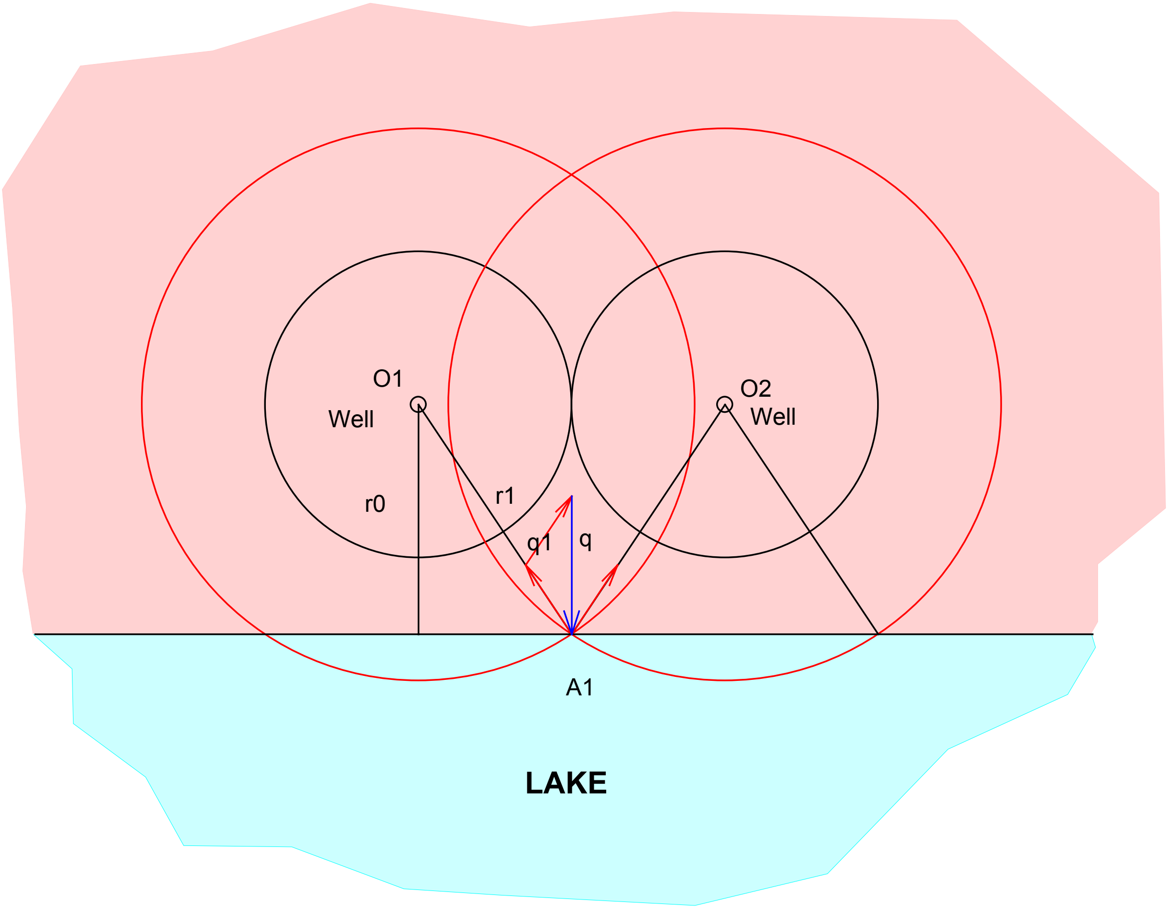

Figure 5-3: Combination of natural flow and the pumping by wells.

This can also be written :

$$i = \frac{1}{2\sqrt{2}r_0} \frac{0.75\ h_0}{\text{Log}\frac{\sqrt{2}r_0}{r_p}}$$

This shows that, if the well radius is known, R0, which is half the distance between two wells depends only of the initial slope of the aquifer. The more i is high and the smaller the distance between wells have to be.

Table 5-5 represents the distance between two wells as a function of the slope. We have developed a small EXCEL calculation to solve the equation which has no analytical solution.

r_p m	r_0	h_0 m	i	k m^2	mu Pa.s	ρ kg/m^3	g m2/s	q m^3/s/m	Q puits m^3/s	U m/s
0.04	15	4	0.011271	1.00E-12	0.001	1000	10	4.4E-07	4.2E-05	1.1E-07
0.04	30	4	0.005075	1.00E-12	0.001	1000	10	2.0E-07	3.8E-05	5.0E-08
0.04	50	4	0.002837	1.00E-12	0.001	1000	10	1.1E-07	3.5E-05	2.8E-08
0.04	10	4	0.018075	1.00E-12	0.001	1000	10	7.1E-07	4.5E-05	1.8E-07
0.04	5	4	0.040993	1.00E-12	0.001	1000	10	1.6E-06	5.1E-05	4.0E-07
0.04	15	4	0.011271	1.00E-13	0.001	1000	10	4.4E-08	4.2E-06	1.1E-08
0.04	30	4	0.005075	1.00E-13	0.001	1000	10	2.0E-08	3.8E-06	5.0E-09
0.04	50	4	0.002837	1.00E-13	0.001	1000	10	1.1E-08	3.5E-06	2.8E-09
0.04	10	4	0.018075	1.00E-13	0.001	1000	10	7.1E-08	4.5E-06	1.8E-08
0.04	5	4	0.040993	1.00E-13	0.001	1000	10	1.6E-07	5.1E-06	4.0E-08
0.04	15	4	0.011271	1.00E-11	0.001	1000	10	4.4E-06	4.2E-04	1.1E-06
0.04	30	4	0.005075	1.00E-11	0.001	1000	10	2.0E-06	3.8E-04	5.0E-07
0.04	50	4	0.002837	1.00E-11	0.001	1000	10	1.1E-06	3.5E-04	2.8E-07
0.04	10	4	0.018075	1.00E-11	0.001	1000	10	7.1E-06	4.5E-04	1.8E-06
0.04	5	4	0.040993	1.00E-11	0.001	1000	10	1.6E-05	5.1E-04	4.0E-06

Table 5-5: Relationship between the slope of the aquifer and the distance between two neighbouring wells. Calculation of the flow rates and filtration velocity.

Some other useful relationships can be derived :

If q and the permeability of the ground is known, this can be written as follows :

$$\frac{q}{\frac{k}{\mu}\rho\, g\, h_0} = \frac{1}{2\sqrt{2}r_0}\frac{0.75\, h_0}{\mathrm{Log}\frac{\sqrt{2}r_0}{r_p}}$$

and :

$$\mathrm{Log}\frac{r_0\sqrt{2}}{r_p} = \frac{1}{2\sqrt{2}r_0}\frac{k}{\mu}\rho\, g\frac{0.75\, h_0^2}{q}$$

or :

$$r_0 = \frac{1}{\sqrt{2}} r_p \exp\left(\frac{1}{2\sqrt{2}r_0}\frac{k}{\mu}\rho\, g\frac{0.75\, h_0^2}{q}\right)$$

Practically speaking, the radius of the wells will be of about 0.04 m.

It is possible to calculate the radius r_0, which is half the distance between two neighbouring wells. the above formula shows that it is only a function of i, given a certain height h_0 of the aquifer.

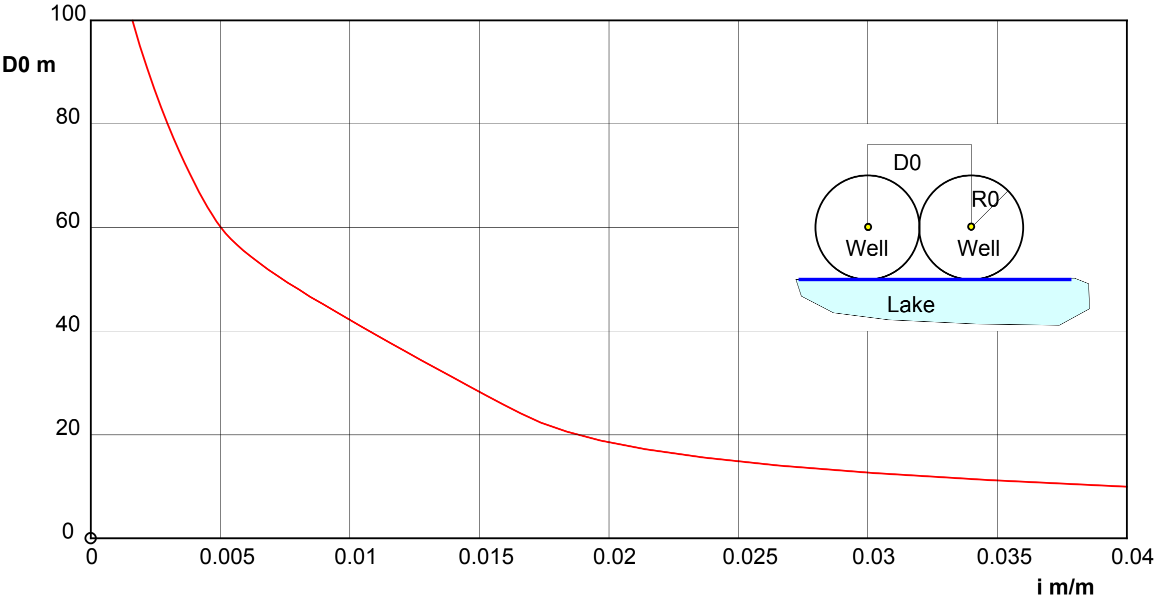

Figure 5-4: Minimum distance between two wells versus the slope of the aquifer.

An interesting feature of this result is that the distance between two wells remain high, even for high slopes higher than 1%. This means that this type of protection is very efficient and relatively cheap due to the limited number of wells.

Figure 5-5 gives the flow rate per well which is mainly a function of permeability. It is, in fact proportional to this parameter. It is given by :

$$q = \frac{k}{\mu} \rho \, g \, i \, h_0$$

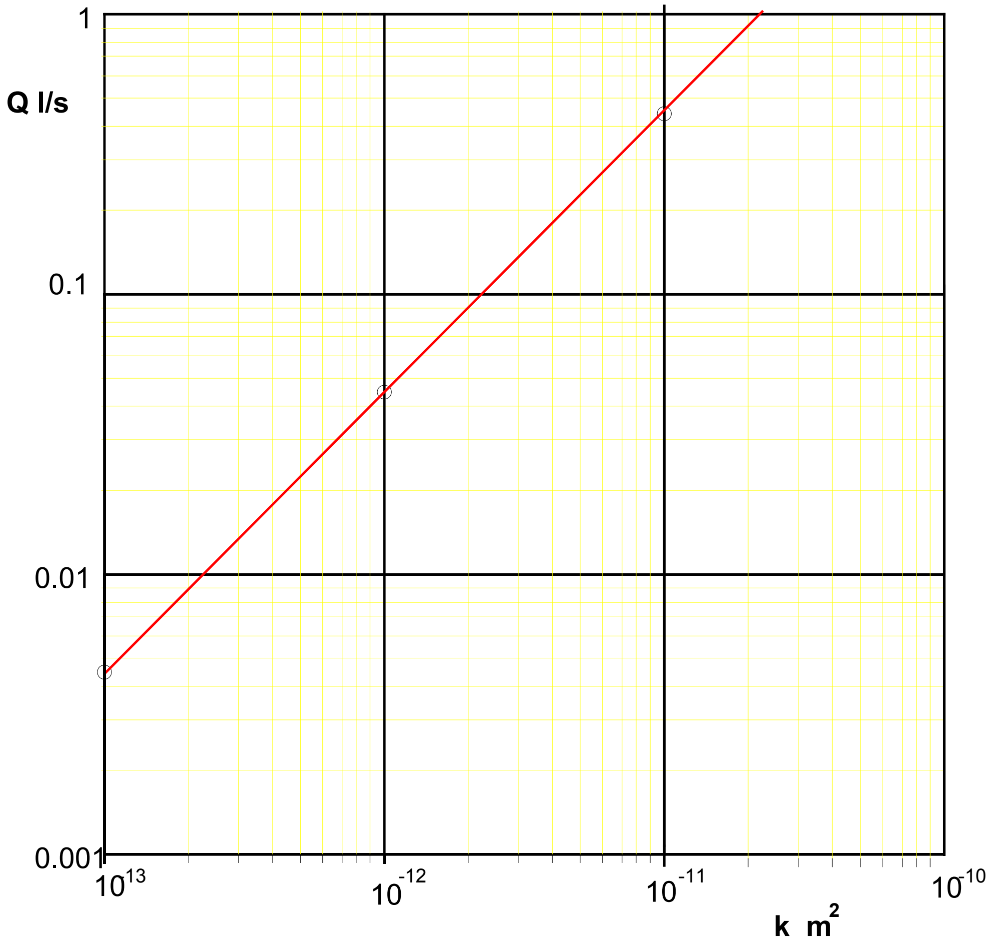

Figure 5-5: Typical flow rates versus permeability for D_0=30 m.

What is interesting is that the flow rate per well is low and that the installation of a single pump for each well would be very expensive.

5.3 THE TRIPLE PHASE EXTRACTION

The principle.

This concept has been developed by ATE Geoclean in France with the collaboration of Ylec Consultants. It is very well adapted to situations where the number of wells is high and their flow rate is relatively small. Moreover, the technique permits to take rid of floating pollution as well as of vapours contained in the so-called vapodose region situated above the water table.

The principle consists to equip the well with a tube whose tip is at a certain depth so that the drawdown is automatically fixed. By doing that, the flow rate in the well is also fixed.

The principle of the system is given on Figure 5-6.

All the wells are linked to a collecting pipe which is itself connected to a vacuum tank.

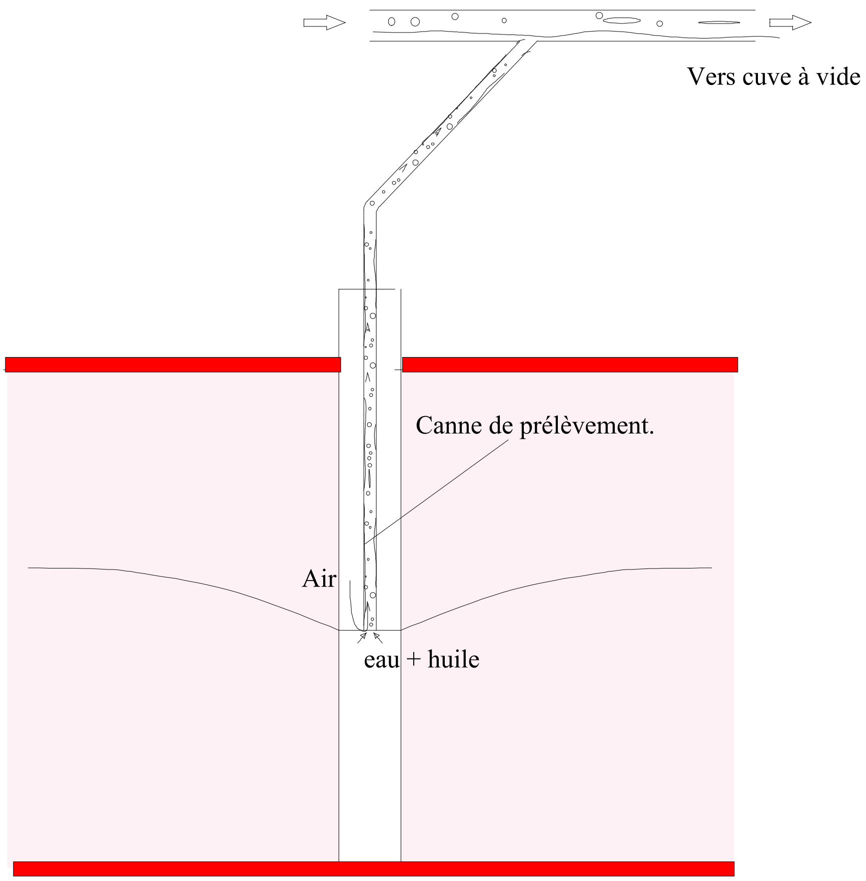

Figure 5-6: Principle of the triphase extraction (ATE Geoclean).

By adjusting the head losses at the exit of the wells, it is possible to regulate the water and gas flows.

Most installation of this type have problems in case of breakdown, because it may be difficult to put each well automatically in operation. Some special simple tricks have been proposed and patented by Ylec Consultants which make this sensitive part of the process very easy to initiate.

Special calculation procedures have been developed which permit a complete determination of the size of tubes, valves, static head losses, vacuum pumps and other equipment. It is completed by an automatic software to calculate the cost of the equipment and to optimise it.

Treatment.

The aim of this equipment is to treat the water before it returns to the lake. The type of treatment depends on the pollution concerned. These treatments can be either physical or chemical. Secondary treatment such as oxygenation or hyper oxygenation by dissolution of O_2 can be used to enhance the water quality before it is rejected to the lake.

The general principle of such an installation is given on Figure 5-7.

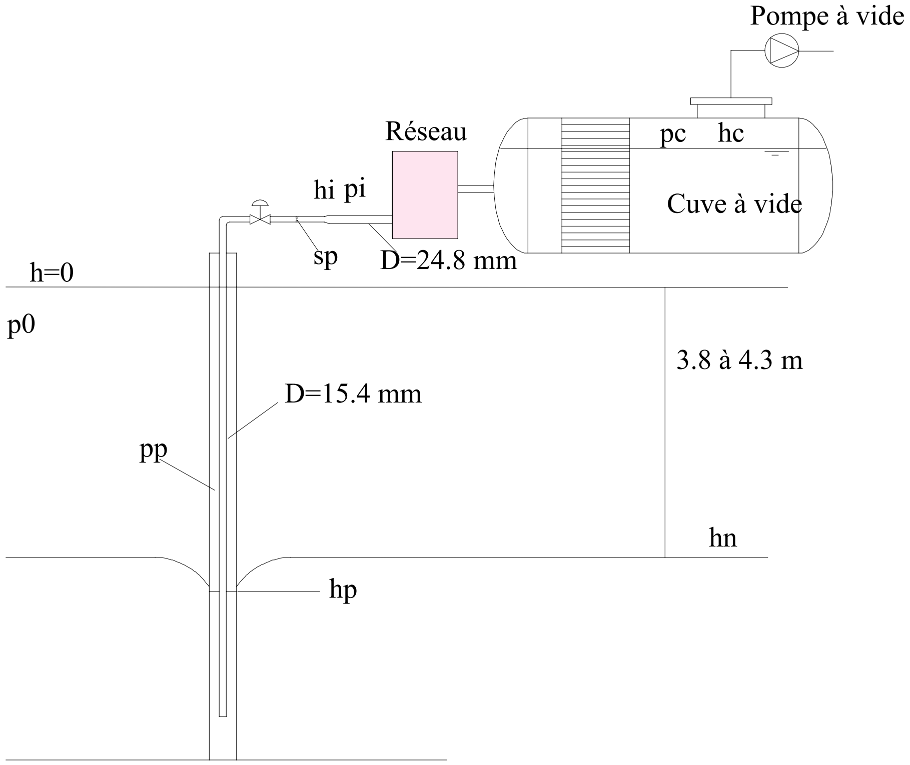

Figure 5-7: Pumping equipment for a triphase network.

Such equipment has been installed on a number of polluted sited in France and works very satisfactorily, preventing pollution of water bodies such as rivers.

6 NOTATIONS

b m	Width.
c	Chézy coefficient.
d_{10} µ	Diameter for which 10% of grain are smaller than d_{10}.
D m	Diameter.
F N	Force.
F_D N	Drag force.
F_x N	Projection of the force in the x direction.
F_y N	Projection of the force in the y direction.
g m/s^2	Gravity acceleration.
H m	Head.
k	Coefficient.
K m/s	Permeability.
k m^2	Intrinsic permeability.
h m	Height.
h_0 m	Height of the water table.
I	Slope
l m	Length.
L m	Length.
L m	Half length of the boat.
M kg	Mass.
MRT s	Mean residence time.
n	Rotation velocity.
n	Manning coefficient.
N rpm	Rotation velocity.
p Pa	Pressure.
Δp Pa	Pressure difference.
P kW	Power.
P N	Weight.
q m^3/s	Flow rate per unit width.

q_J N	Momentum of a jet.
Q	Coefficient.
r m	Radius.
r_0 m	Radius.
r_1 m	Radius OA.
r_p m	Radius of the well.
R_H m	Hydraulic radius.
s_t m	Drawdown at the well boundary.
s_p m	Drawdown in the well.
S m^2	Surface.
S m^2	Wetted surface.
S_w	Non dimensional wetted surface.
t s	Time.
T m	Draft.
U m/s	Velocity.
U_c m/s	Critical velocity.
U_T m/s	Tangential velocity.
V m^3	Volume of a lake.
x m	Abscissa.
y m	Vertical co-ordinate.
∇ m^3	Volume
α	Coefficient.
α °	Angle.
χ	Perimeter.
δ m	Thickness of the skimmed layer.
γ_c m/s_2	Centrifugal acceleration.
η	Efficiency.
λ	Scale ratio.
Λ	Pressure drop coefficient.
μPa.s	Dynamic viscosity.
ν m^2/s	Kinematic viscosity.

π_1	Rateau head coefficient.
π_2	Rateau flow rate coefficient.
ρ kg/m^3	Specific mass.
ω rad/s	Angular velocity.