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Working group on recharge wells

The clogging of recharge wells, main subjects

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PREFACE

This report is a shortened version of a much more detailed report (Olsthoorn*, 1982) and came about within the recharge-well-research projects, since 1973 part of the research programme of the VEWIN (Netherlands Waterworks Association). The research is carried out by five waterworks, an industry, two governmental institutions and KIWA, the testing and research institute of the Netherlands waterworks. The Author, employed by KIWA, led the recharge well research from 1974 to 1980. The research was convoyed by the KIWA-Recharge-Wells-Working Group of which the composition is given in appendix B.

Participants of the recharge-well research are:
- Netherlands' Waterworks Association (VEWIN, Rijswijk)
- Dune Waterworks of The Hague (The Hague)
- Municipal Waterworks of Amsterdam (Amsterdam)
- Provincial Waterworks of North Holland (Bloemendaal)
- Waterworks "Midden-Nederland" (Utrecht)
- Municipal Waterworks of Groningen (Groningen)
- Rijkswaterstaat, Directorate of Water Management and Hydraulic Research, District North (Lelystad)
- Governmental Institute on Drinking Water Supply (Leidschendam)
- ESTEL-Hoogovens B.V. (Steelworks, IJmuiden).

* T.N. Olsthoorn. Verstopping van Persputten (clogging of recharge wells), KIWA-mededeling 71 (KIWA-communication 71), KIWA, Rijswijk, 1982, 500 pp.

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SUMMARY

Artificial infiltration of (prepurified) surface water into the subsoil (artificial groundwater recharge) is an important process in drinking-water preparation in the Netherlands and elsewhere. The research by Dutch waterworks and other concerns was conducted and financed since 1973 under the auspices of the VEWIN* association and supervised by the KIWA** Committee on Artificial Recharge. Specialised areas such as the recharge-well research are being investigated in greater depth by various working groups. The participants (see PREFACE) are represented in the working groups and, more often than not, at the same time in the Committee. This report is produced within the terms of reference of the working group on Recharge Wells (see Appendix B).
The research under the guidence of other similarly functioning working groups reporting to the Recharge Committee (the "Working Group on the Hydrology of Recharge-Well Systems" and the "Working Group on Health Aspects allied to Soil Filtration"), will be reported separately.

* VEWIN = Netherlands' Water Works Association
** KIWA = Testing and Research Institute of the Netherlands' Waterworks

Artificial infiltration or recharge gives waterworks an opportunity to make reliable use of surface water, an intrinsically unreliable resource. If suitable soil strata are present but cannot be utilised or are not suited to creation of an open recharge basin, then recharge wells may be an alternative means to bring water into the soil. An important difference between groundwater recharge by wells and that via recharge ponds is the entry velocity, which with wells is some two orders of magnitude higher. As a result of this, the risk of clogging is very great with recharge wells. Therefore emphasis has long been layed on the study of the clogging phenomenae.
This report presents, in broad lines, the results of this research as well as a number of guidelines for the design of recharge-well systems. A much more detailed report is available as "KIWA-mededeling" (Olsthoorn, 1982, see footnote at page 3).

Since, thanks to modern knowhow, practical ways of preventing clogging of recharge wells exist, application of recharge wells is technically feasible in principle. One method is illustrated by the examples in chapters 5 and 6.

Clogging, however, is only part of a much more complex problem.
Hydrological factors such as divergent subterranean residence times and stock accumulation plus the problem associated with saline and brackish water can be decisive factors determining the sense or nonsense of a recharge-well system. The same applies to chemical and biochemical quality improvement during subsoil passage and the attendant degradation by bacteria and viruses. These factors

are still being subjected to intensive study within the VEWIN-research program.

The maximum permissible pressure in an injection well without causing soil fractures is a boundary condition, limiting application (see chapter 2). A permissible head of 2 m water column above ground level for every 10 m that the top of the gravel pack or the well screen is below ground level can be taken as a practical rule of thumb.

Chapter 3 deals with the cause and prevention of well clogging and mentions the hallmarks of this phenomenon. Apparently, obstruction by air bubbles and gas bubbles, by formation of precipitates, by reaction with soil material and by soil subsidence, can be prevented or, at least, restricted by simple technical expedients and good preliminary exploration. Removal of suspended matter is more difficult and more expensive.
However, although clogging can be greatly minimized even in respect of suspended matter, it may be more advantageous to weigh up the degree of prepurification against the number of wells and frequency of redevelopment (chapter 5).

In practically all cases, recharge wells can be redeveloped successfully (chapter 4).

On average, back pumping removes three quarters of any obstruction material accumulated since the previous redevelopment. Flushing rates and flushing times are almost irrelevant, unless they are extremely high or extremely long. More intractible residue, in the mean time, can only be eliminated by intensive methods.

In mechanical removal procedures repeated to and fro movement of water is essential. Intermittent pumping action, e.g. switching the pump on and off repeatedly is not conducive to redevelopment. A compressed-air system is the most flexible, least vulnerable and simplest means to clean injection wells, both mechanically and chemically.
It is at the same time a powerful flushing pump, a facility for propelling water back and forth in the formation and a means to submit the well to short, but intensive peakflows. Applying compressed air in this sense we call air-lift-juttering.
Only when a large number of wells has to be pumped clean very frequently, should preference be given to a system with fixed submersible pumps (no noise nuisance, for example). High-pressure jetting nozzles are suitable for flushing out wells which are clogged internally (within the well screen slots). This type of obstruction seldom occurs in recharge wells, but should be anticipated in the extraction wells of the system.
Where the action of mechanical methods of redevelopment does not come up to expectation, chemicals may be added, in which case the method is termed "chemical redevelopment". Chlorine-containing agents, acid and polyphosphates are the chemicals most commonly employed. The correct choice of agents will depend on the nature of the obstruction material and therefore on the cause of the clogging. The main clogging cause one faces in a particular situation, can be deduced only indirectly from various kinds of measurements and other indices while comparison with experiences acquired elsewhere (chapter 3) is also important.

Although it remains difficult to predict, a priori, the performance of a future injection-well system, with the data from adequate preliminary investigations it now is possible to make a good design that will yield lowest costs for the system as a whole. The general design procedure, based on such preliminary research, is discussed in chapter 5, while the planning of actual injection wells is discussed in chapter 6. Also at the level of the wells themselves, optimisation proofs feasible.

According to the insight given in this report clogging is no longer an insolvable problem and therefore recharge wells may be considered as a technical means to inject water into underground strata. Therefore the use of recharge wells becomese feasible for sites where fresh/salt-water interaction does not play. In such places, the soil can be used, in conjunction with an infiltration-recovery system, as a vast filter and mixing tank. It will thus convert prepurified surface water into a consistently good product which can be rendered fit for consumption, after recovery plus limited aftertreatment, possibly without disinfection. A system of this type is generally advisable for locations where conventional groundwater withdrawal might seriously harm other interests or where ingress of polluted groundwater is a potential hazard.

Should the supply of injection water be interrupted groundwater withdrawals may be continued for some time. The resulting lowering of groundwater levels may be acceptable on an occasional basis.

As a rule, injection wells are used primarily for creating an underground supply of fresh water which can be extracted whenever necessary.

In places where fresh water floats on saline groundwater with a somewhat higher specific gravity, a stock of fresh water might conceivably be created by horizontal and vertical displacement of salt groundwater. Movement of the fresh-water/salt water-interface, taking account of subsoil stratification, if necessary, is still difficult to calculate.
Description of the behaviour and formation of brackish water, both locally in the short term and more regionally in the longer term, is still problematical even now. Research into these aspects is under way.

Another important matter is the chemical and microbiological quality improvement obtained by the passage of the subsoil, after which research is also beeing done within the VEWIN research programme.

1 INTRODUCTION

In "artificial recharge" surface water - purified or unpurified - is brought into the ground, through which it flows towards the more remotely sited extraction facilities. The water extracted is of hygienically improved - and above all constant - quality. It is also important that extraction of the infiltrated water may be continued for a considerable time, whenever the supply of untreated water subsides. Looked at from this angle, artificial recharge represents a way of making reliable use of intrinsically unreliable waters. It has this in common with storage basins.

Recharge itself (also known as groundwater replenishment) is usually a simple process: water is allowed to percolate from purpose-built ponds or channels. Where this method is impracticable, whether because the uppermost strata are impermeable or because the space required for ponds is not available, the water can be introduced into the ground via wells (so-called injection wells, fig. 1).

Injection wells are being used on a large scale in other sectors, particularly in secondary oil recovery (Tazelaar, 1968; Case, 1970; Patton, 1974, etc.), secondly for deep-well waste injection (Donaldson, 1972; Anon., 1973) and in reinjection of groundwater (Brandes et al., 1978). Thirdly, they are being used in the water-distribution industry in Spain (Custodio-Gimea, 1970, 1980), Switzerland (Schmassmann, 1978), Germany (Dorn, 1974) and widely in California (Baffa, 1965; Bulten et al., 1974; Doshi, 1972; McIllwain, 1970) and in Israel (Anon., 1969; Harpaz, 1970).

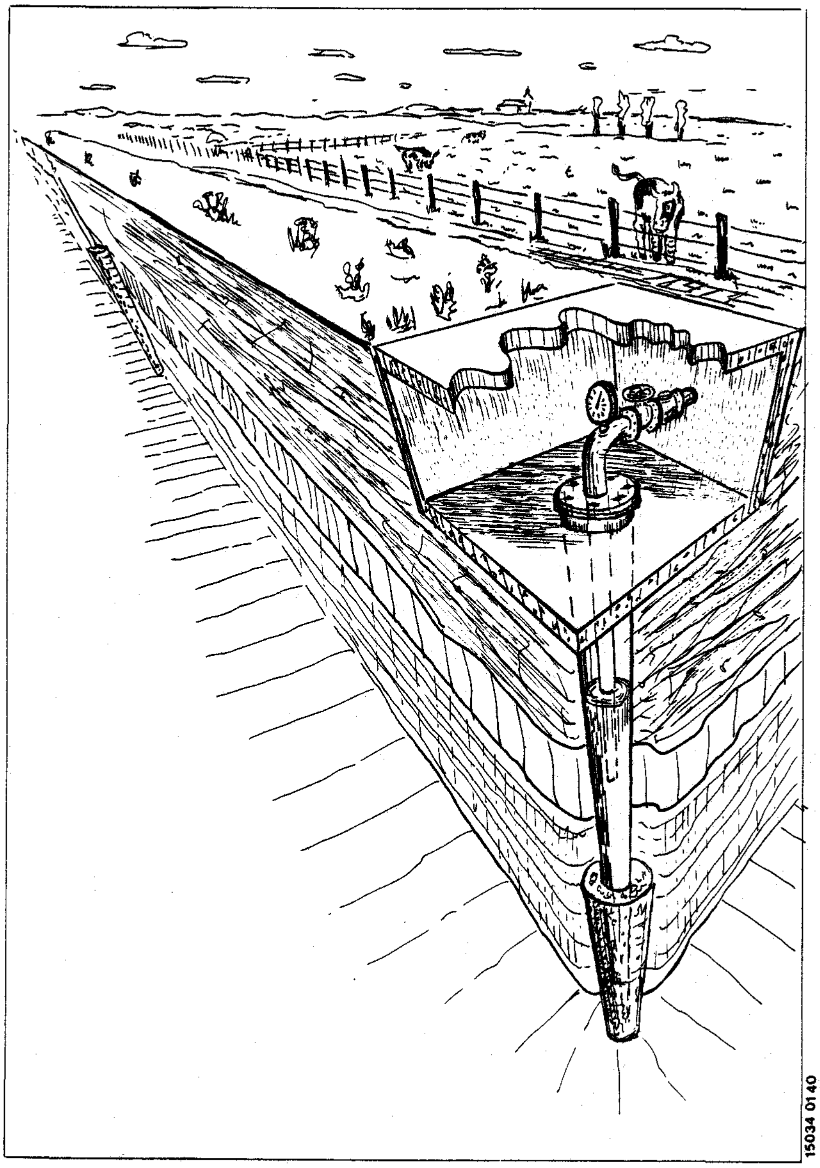

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Figure 1 - A simple injection well in unconsolidated sediment (sand)

In the Netherlands, application of the system is still at the experimental stage, although extensive research has been going on for the last decade and considerable knowhow and experience in this field has been acquired (Bulten, 1971, 1972; Olsthoorn et al., 1975; Olsthoorn, 1977, 1979; see also the data from wells investigated in Dutch injection well research, Appendix A).

This report is based on the latter research as far as clogging is concerned. However, the material included here is based on a more detailed report (Olsthoorn, 1982).

Structurally, there is hardly any difference between an injection well (figure 1) and an extraction well (Steinmetz, 1977, 1978; Sternau, 1967; Johnson et al., 1966; Monkhouse and Philips, 1978, etc.). The former tends to be of somewhat sturdier construction (Uil and Deelder, 1978) and includes an injection line plus accessories such as shut-off valves, possibly a water meter, a manometre and the like.

In contrast to deep-well drainage and injection wells in the oil industry, where working pressures often range from tens to hundreds of atmospheres (Doscher and Weber, 1957), in the water-distribution sector the word "pressure well" is hardly applicable. In view of the shallowness of wells sunk for water-distribution purposes (down to about 100 m), the pressure has to be limited strictly in order to prevent the ground around the well from fracturing (chapter 2). In many cases, injection is carried out without any excess pressure; there exists a free liquid level in the well while in operation.

Since the rate at which the water enters the formation via the borehole wall is a few orders of magnitude higher than that at which the water in usual recharge ponds penetrates into the soil, the risk of an injection-well clogging is a very real. Whereas an injection pond becomes clogged after only a few months or years, an injection well may clog in a matter of days to weeks. Clogging is therefore one of the most important technical aspects of injection-well application (chapter 3).

2 SOIL FRACTURING

The soil around an injection well fractures when the injection pressure is too high (Howard and Fast, 1970; Hubbert, 1972), where upon the well may be rendered unserviceable by an efflux of soil material (figure 2). The injection pressure must therefore stay below a maximum-permissible value determinable as follows.

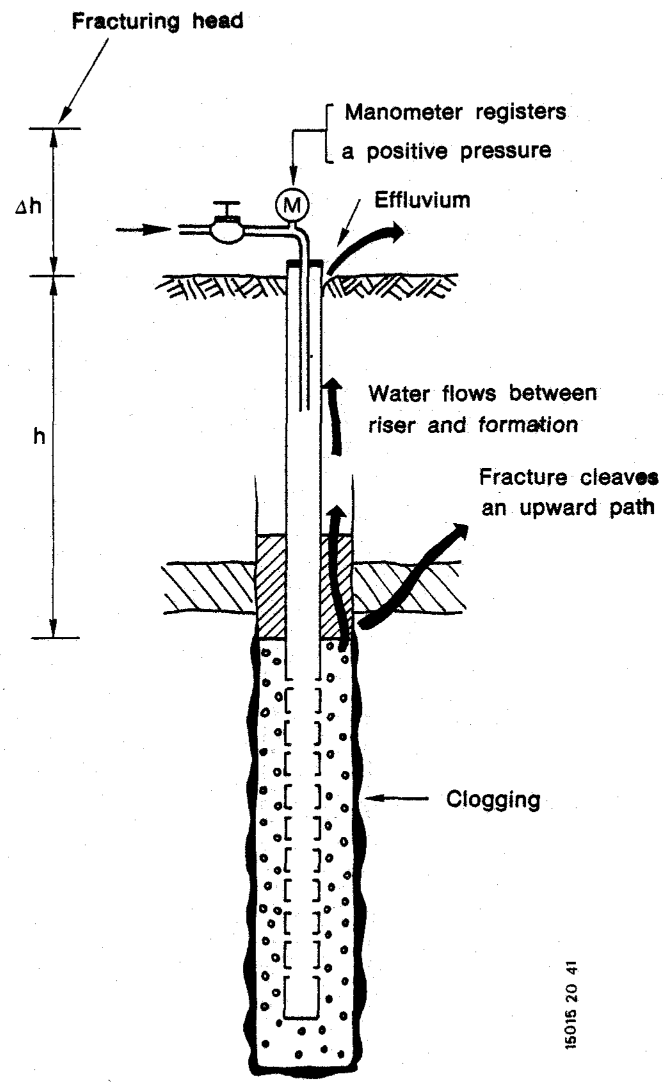


Figure 2 - Hydraulic fracturing of a recharge well

The minimal grain/grain stress σ_3 (N/m^2) at a point in the soil cannot be smaller than the maximal grain/grain stress σ_1 at that point, divided by the passive soil-pressure coefficient λ. For soils with negligible cohesion, such as sand, clay and peat, λ is dependent exclusively on the angle of internal friction of the soil ϕ (ϕ is a basic parameter. See standard soil mechanics references):

$$\frac{\sigma_3}{\sigma_1} > \frac{1}{\lambda} = \frac{1-\sin(\phi)}{1+\sin(\phi)} \tag{2.1}$$

In unconsolidated sediment with negligible tectonics (sand and clay), σ_1 can be equated to the total vertical pressure σ_g (soil pressure) less the in situ pore-water pressure, u. This water tension increases, during infiltration, by Δu. Since hydraulic fracturing of the soil is impossible so long as the minimum grain-to-grain pressure $\sigma_3 > o$ (Verruijt, 1967), the requirement:

$$\sigma_3 \geq (\sigma_g - u)/\lambda - \Delta u \tag{2.2}$$

is now valid.

Δu is a maximum against the borehole wall, whereas $\sigma_g - u$ is a minimum at the top of the gravel pack. This is therefore the critical place at which the above mentioned requirement must be applied. If the top of the gravel pack is located at a depth h below ground level it follows that given
$\sigma_g = \gamma_g h$, $u \approx \gamma_w h$, $\Delta u = \gamma_w \Delta h$,
where Δh is the head in the well above ground level, and γ_g and γ_w are the volumetric weights of wet soil and water respectively:

$$\Delta h = \frac{\gamma_g - \gamma_w}{\gamma_w} \; \frac{h}{\lambda} \qquad (2.3)$$

For a sandy soil $\gamma_g \approx 20000$ N/m^3, while, in general, $\gamma_w = 10000$ N/m^3. Since, normally ϕ 40°, a value of 5 can be retained as the upper limit for λ and hence the following rule of thumb is applicable:

$$\Delta h \leqslant 0.2 \text{ h}. \qquad (2.4)$$

Hence Δh is the maximum permissible positive head in the well above groundlevel and as such is equally valid for artesian groundwater. The rule of thumb must be adjusted according to equation (2.3) in cases where the average γ_g and/or γ_w deviate appreciably from the values taken here. Especially with thick layers of peat care should be taken.

The lower limit of 0.2 h also appears to hold good for the much deeper injection wells used in the oil industry in which "hydraulic fracturing" has been applied (see the experimental findings of Howard and Fast, 1970 and conversion to the quantities and dimensions discussed here in Olsthoorn, 1982).

3 CLOGGING

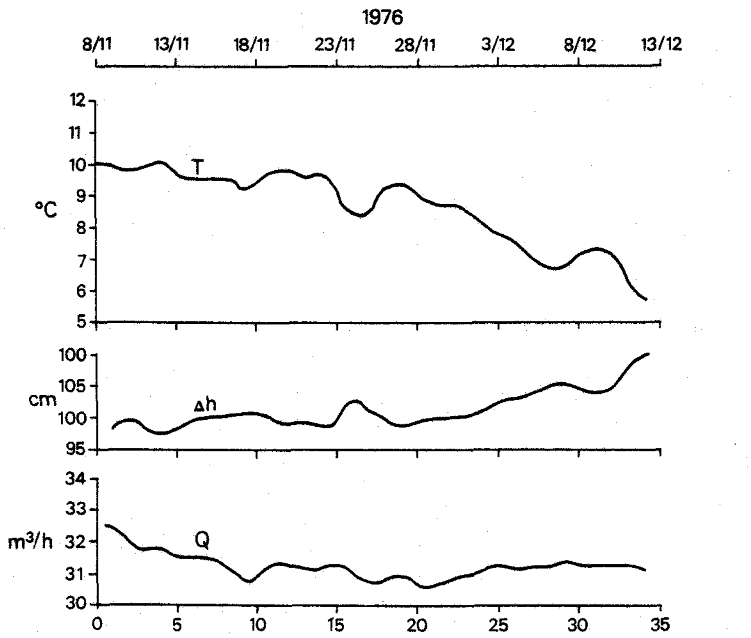
3.1 Causes of clogging

The following can be cited as causes of clogging:

1. suspended particles in the injection water;
2. gas bubbles in the water;
3. proliferation of bacteria in and around the well;
4. formation of chemical precipitates in the injection water and the well;
5. formation of chemical precipitates in the soil;
6. swelling and dispersion of clay;
7. erosion of soil structure and jamming of the aquifer.

Causes 2 to 7 inclusive may make the system unreliable and have very serious consequences. They therefore need to be eliminated beforehand and this can be achieved successfully by means of adequate research, properly designed installations and proper control.

Suspended particles and consequently clogging by suspended substances cannot always be avoided completely. The residual concentration of suspended constituents depends on the composition of the untreated water, combined with the method of prepurification. For a particular situation this concentration varies as a direct function of purification costs, which can be quite high if the objective is virtually complete removal. In practice, consideration can be given to the relative merits of further prepurification, a larger number of wells and more frequent cleaning (redevelopment) of wells (see chapter 5).



T = temperature in °C
Δh = difference in head in cm between the injection well and the observation well (5 m apart, 33 m deep)
Q = flow rate in m^3/h

Figure 3 - Reaction of head difference between the injection well and an observation well Δh to fluctuations in water temperature and injection rate (according to Steinmetz, 1977, Leiduin Municipal Waterworks' of Amsterdam well)

In contrast to groundwater which is always at almost the same temperature (T), the viscosity (μ) of the surface water used for injection must be taken into account (figure 3), for the resistance the water encounters is directly related to it. The difference between winter-injection water at 2 °C, for instance, and summer water at 28 °C is thus likely to produce a(n) (apparent) 100 % increase in resistance.

The viscosity of the water μ (Ns/m^2) is closely approximated using:

$$\mu = \{(\frac{510}{T + 43.1})^{1,502}\}(10^{-3}) \quad 0 < T\ (°C) \leq 100 \qquad (3.1)$$

The actual clogging of an injection well does not therefore appear until after temperature correction, which is normally carried out by reducing the measured value to what it would have been at 10 °C. The multiplication factor (μ_{10}/μ_T) required for this purpose is often approximated as follows:

$$\frac{\mu_{10}}{\mu_T} \simeq \frac{T + 20}{30} \ (T \text{ in } °C) \qquad (3.2)$$

The resistance (W^{10}) of an injection well is obtained, after correction for temperature, from the increase in the requisite pressure or rise of level (ϕ) per unit injection flowrate (Q).

The so-called clogging resistance (W_c^{10}) is the difference between the total resistance and the natural resistance of the well (beeing the resistance of the well when new) and is calculated as follows:

$$W_c^{10} = (\frac{\mu_{10}}{\mu_T}) \ (\frac{\phi_o - \phi_r}{Q}); \ (m/(m^3/h)) \qquad (3.3)$$

For ϕ_o, the water level or the pressure in the well is taken. ϕ_r is the water level in an adjacent observation well or is the original groundwater level (figure 4).

If a gauge pipe is available in the wells' gravel pack, then clogging of well-screen slots and the first few centimetres of gravel fill can be determined separately. Since, in the vast majority of cases, clogging tends to be concentrated at the borehole wall and penetrates at most a few centimetres into the surrounding formation sand, also the level in a gauge pipe in the same borehole, but now in fine sand above or below the gravel pack, may be used as the reference for the unclogged formation, as an alternative to using a separate observation well some distance away. However, for various reasons preference is usually given to one or more observation wells several metres or tens of metres away.

3.2 Suspended substances

When, as a result of good installation, clogging by air or gas bubbles, and, through chlorination, by bacteria can be excluded, while there are no indications that chemical reactions are taking place in the well, with the original groundwater or with the soil (see following paragraphs), then suspended substances are the most probable cause of clogging. Constituents suspended in the injection water may be of differing kind, shape and size, inorganic or organic in composition.

If the composition of the suspended matter is fairly constant, it will usually generate a considerable increase in resistance per kg of deposited material (figure 5, Vecchioli, 1980). If, in addition, the concentration is constant then the relationship between clogging resistance and total volume of water injected will also be linear (figures 6 and 8).

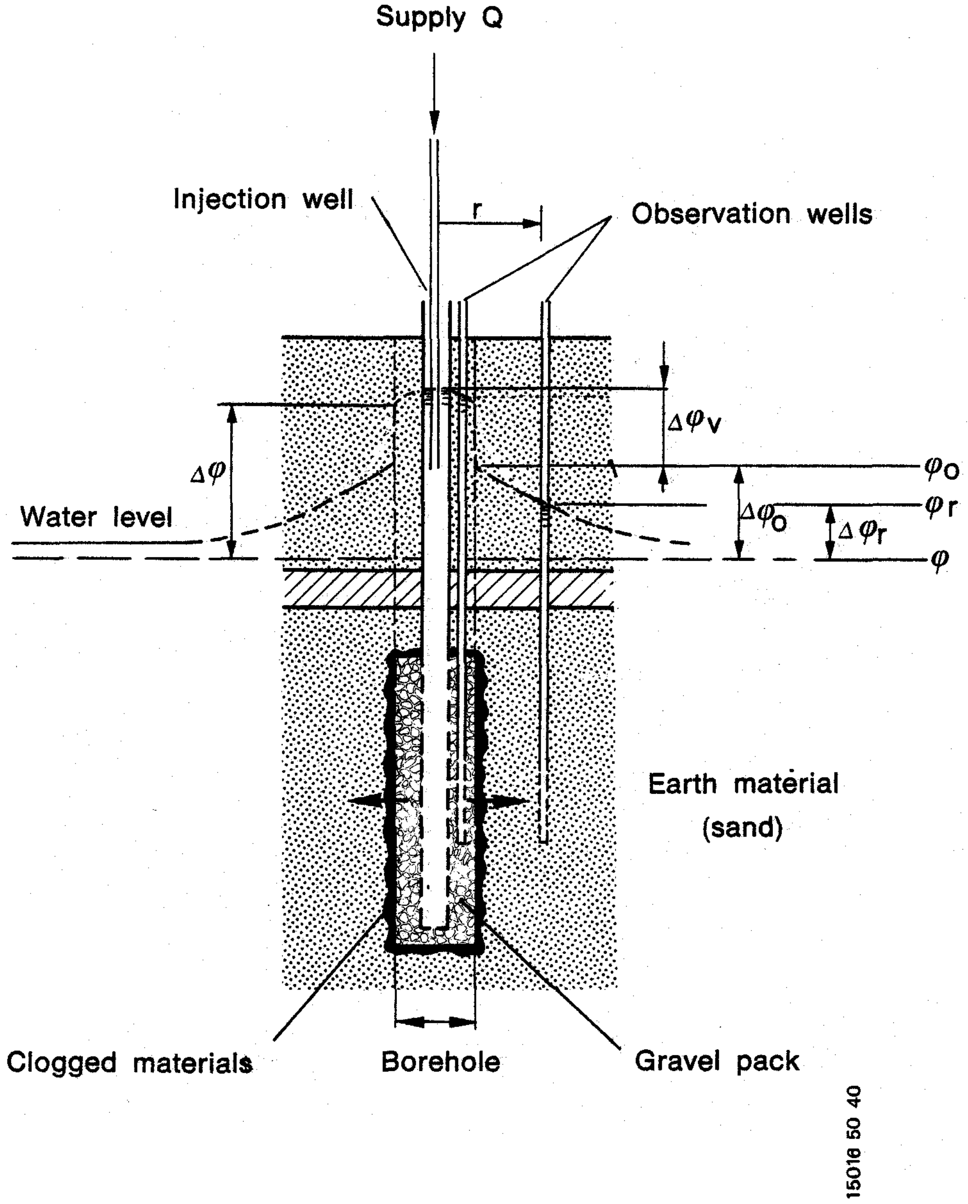


Figure 4 - Clogging of an injection well and relevant water levels (heads)

Since besides the normal relationship between pressure rise and flowrate (according to Darcy's law) also the supply of clogging material per unit time is proportional to the flowrate, there is a squared relationship between the rate at which the injection pressure or water level (ϕ) in the well rises and the injection flowrate (Q) (figure 7):

$$\frac{d\phi}{dt} \sim Q^2 \tag{3.4}$$

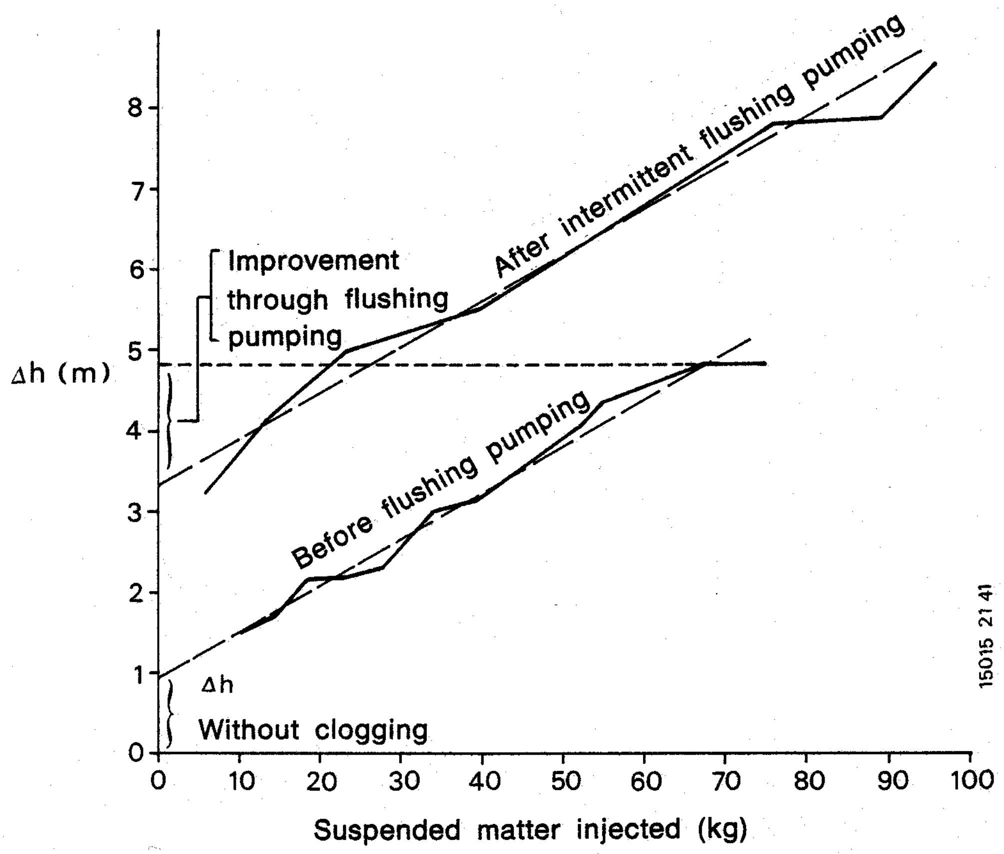


Figure 5 - Water level in the injection well minus that in the observation well 40 m away, as a function of the total quantity of suspended matter which has infiltrated. Hoogoven-well, period August 1970 to February 1971 (after Brandes, in Bulten, 1972, see Appendix A for further information).

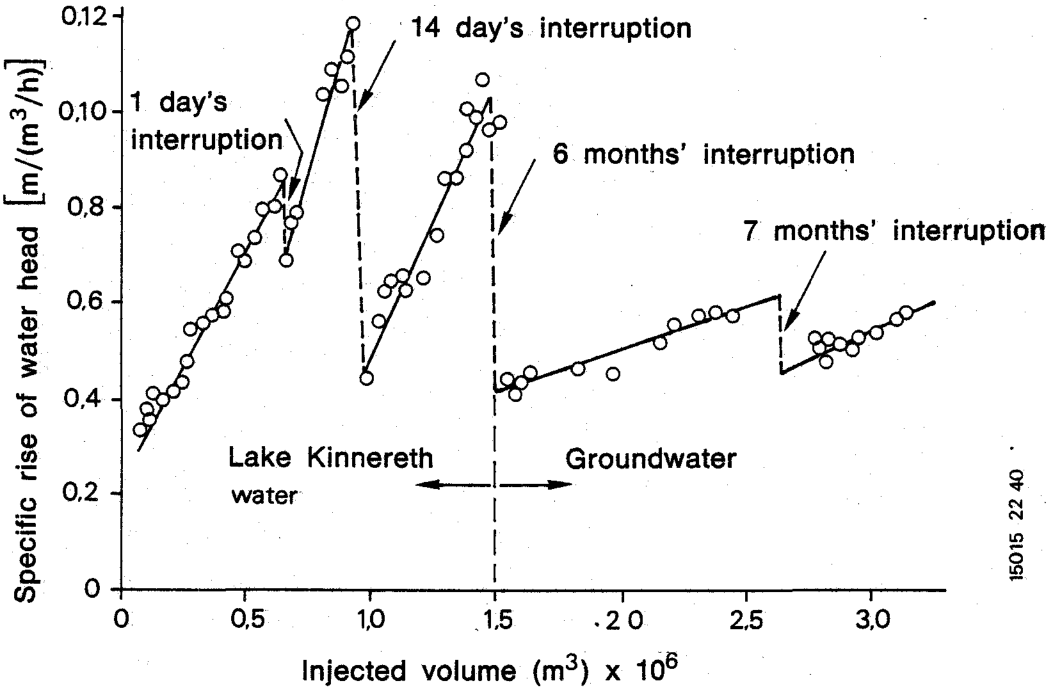


Figure 6 - Specific rise of water head as a function of the total injection volume (Gilgal-Well, Israel; Discussion, reported after paper Y. Harpaz, 1970).

In this equation (3.4) the measured pressure build-up can, despite variations in injection flowrate, be converted into a certain standard flowrate and be plotted against the total injected volume of water. As an illustration, this process is carried out here on the data from two tests of Sniegocki (1963). The relation thus obtained (see figure 8) is linear, despite a marked fluctuation in flowrate during the test.

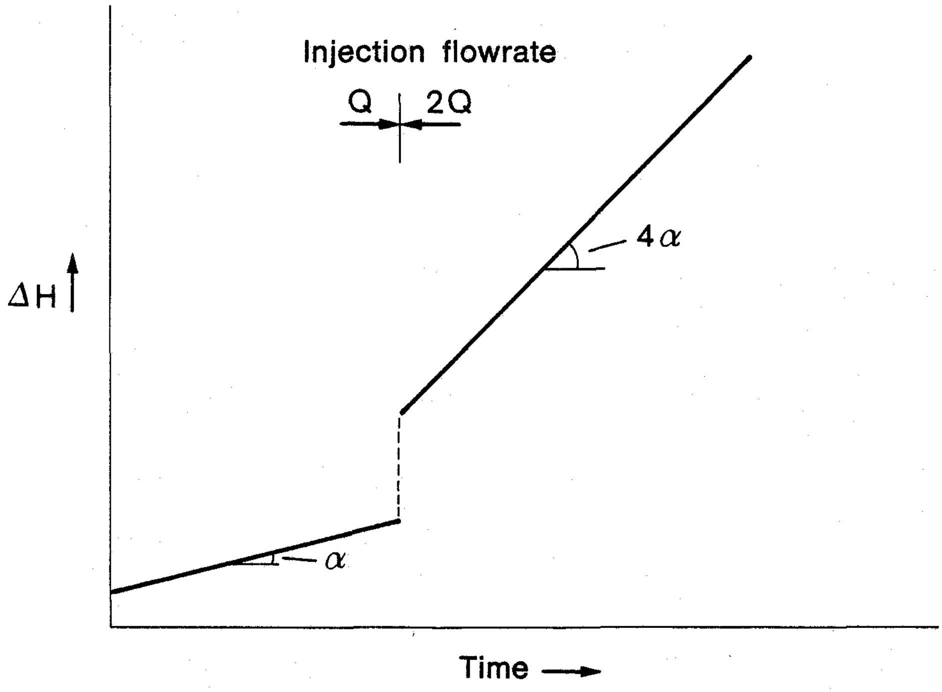


Figure 7 - The effect of doubling the flowrate on the requisite recharge pressure (clogging by suspended substances)

A similar linear relationship is also found for other wells.

Surface water harbours a very large variety of natural and man-made species of suspended matter which may behave variously depending upon the composition of the water and mutual interactions. Thus the clogging properties of an iron-hydroxide suspension (figure 9) appear to depend markedly on the pH rating of the water (Lerk, 1965).

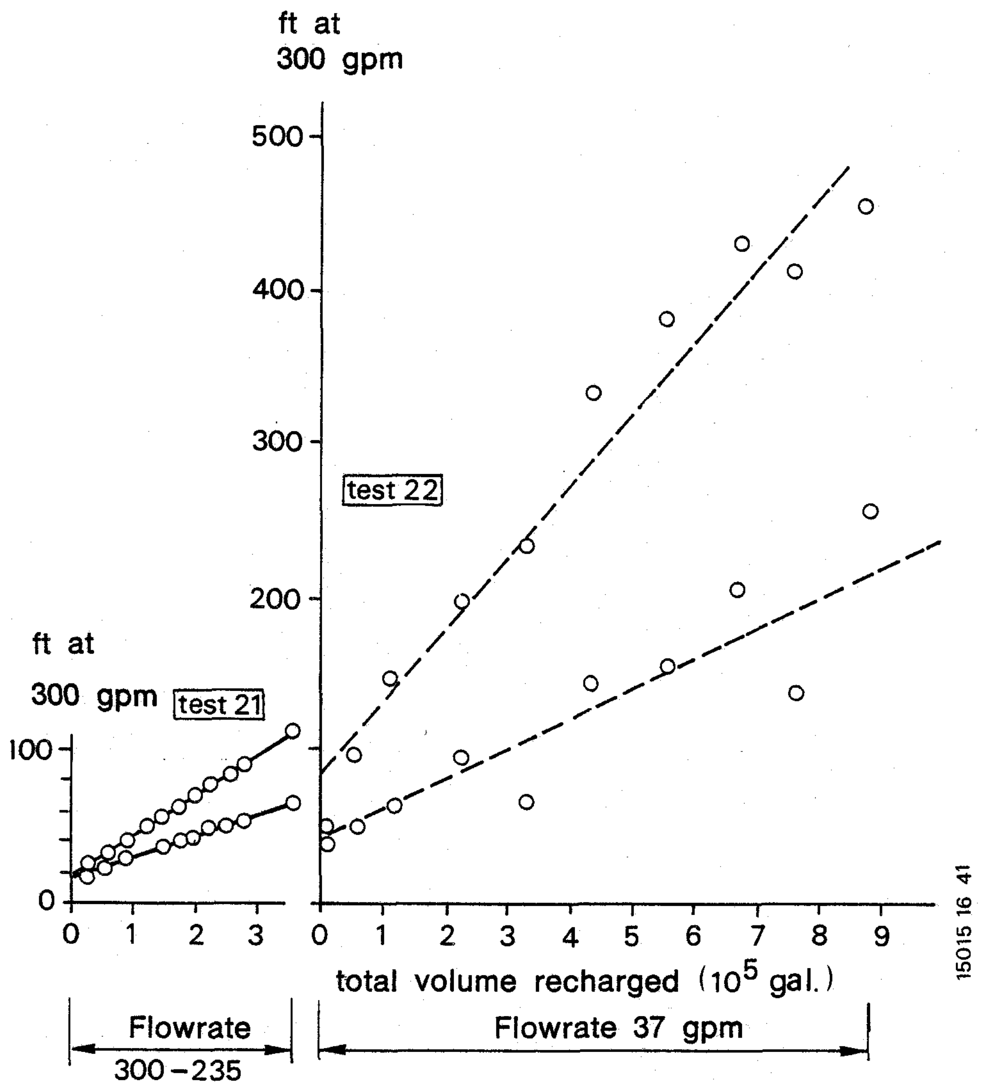


Figure 8 - The water head in the well or the sand pack, minus that in the observation well at 1.8 m distance, reduced to 10 °C and 300 gpm (68.2 m^3/h), as a function of total recharge volume. Sniegocki's tests 21 and 22, 1965 (gpm = gallons per minute, 1 gal = 3.79 l).

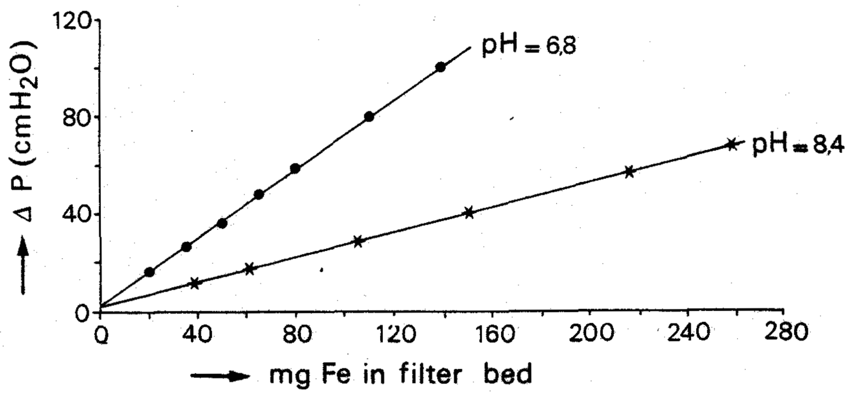


Figure 9 - Effect of pH on the pressure loss over a filter clogged by an iron suspension (After Lerk, 1965).

The clogging properties of clay suspensions of the same type of clay differ for clays originating from different sources (Signor, 1973).
Sometimes, the clogging rate does not lessen, despite a pronounced reduction in the concentration of suspended substances. This proved (Sniegocki, 1963) to be the case with interposition of a coagulation step in the prepurification process (see Olsthoorn, 1982), removing the negative charge on the suspended particles which are thus retained much more efficiently by the negatively-charged formation grains, now that particles and formation material no longer repel one another. In this way, particles become concentrated in a thinner clogging deposit with a relatively high build-up of resistance in consequence.

If filtration by the soil is highly efficient, just a few grammes of suspended matter per m^2 of recharge area suffice to cause a 1 m rise in water level in the well, for a recharge rate of around 1 m/h at the borehole wall (Marshall, 1968, Olsthoorn, 1979). At the admittedly high concentration of 1 mg/l, this situation may thus occur within a few hours. Where filtration by the soil is less efficient (fine negatively-charged suspended particles; coarse earth material, Rahman, 1969) the process can take much longer (figure 10 and Bichara, 1974).

The involvement of the gravel pack surrounding the well screen corresponds to that of the first coarse layer of a multi-bed filter. The pack traps the particles without appreciable increase in resistance, thereby relieving the formation wall where clogging concentrates. Generally, the pack provides a substantial reduction in clogging rate (Olsthoorn, 1982).

From the points mentioned, it seems that, if based solely on analyses of the injection water and the suspensoids which it contains, there are as yet no prospects of successfully predicting clogging of injection wells from behind the designers desk.

Investigation is further hampered by the fact that in water with a low concentration of suspensoids, e.g. drinking water, no relation exists between that concentration, the turbidity and the clogging of an injection well (Olsthoorn, 1979). Consequently, it is hardly possible to draw relevant conclusions concerning prediction and analysis of injection-well clogging, from data of this sort.

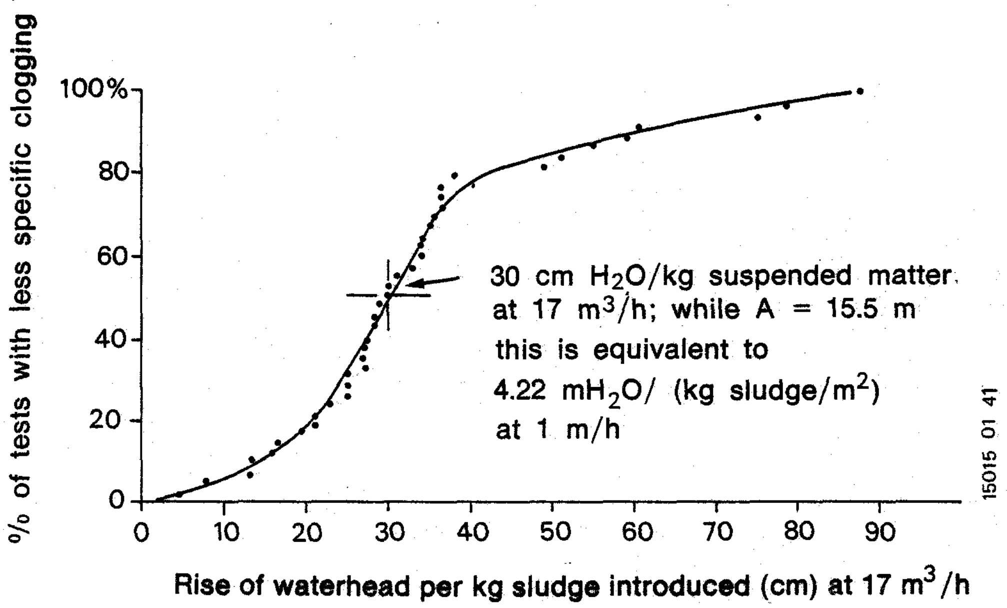


Figure 10 - Rise of waterhead (cm) in the Hoogoven-well per kg suspended matter at 17 m^3/h (see Appendix A for further details).

Better results are obtained from a membrane-filter test (figure 11, Felsenthal, 1956, Doscher & Weber, 1957, Stormont, 1958, Barkman & Davidson, 1972). The most modern membrane-filter test is that devised by Schippers & Verdouw (1980). Their test may be considered based on the work of Doscher & Weber (1957) and Barkman & Davidson (1972), but has now been specially designed and tested for the drinking water sector.

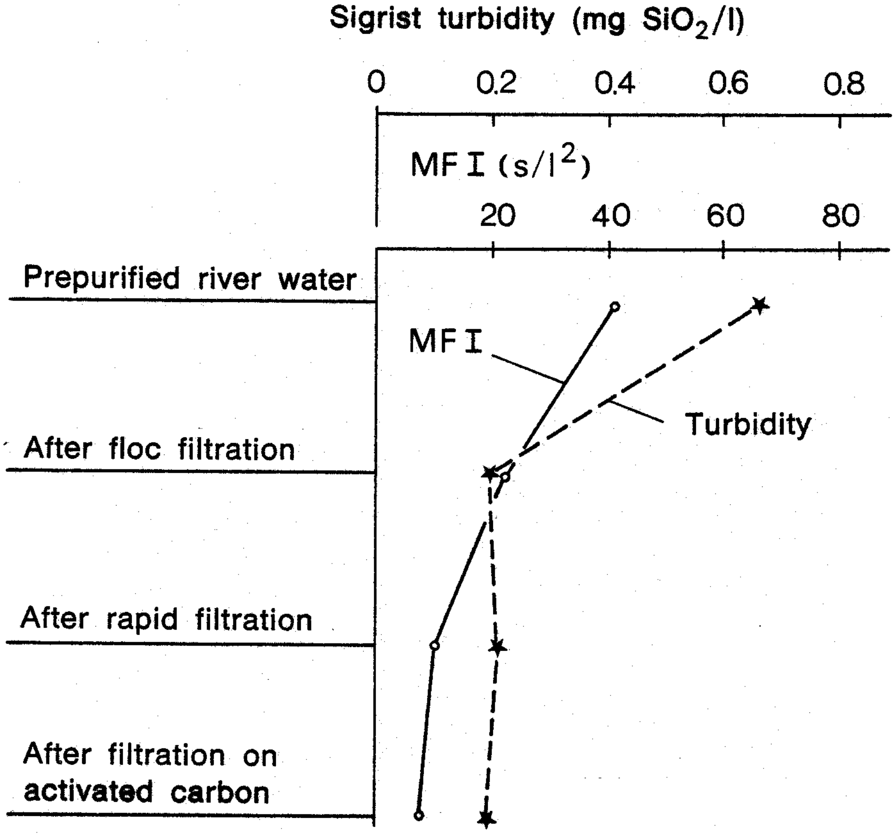


Figure 11 - Decrease of turbidity and membrane-filter index (MFI) in an experimental purification plan at Leiduin (Olsthoorn, 1979).

Schippers & Verdouw (1980) force the water being examined under constant pressure (2 bar) through a membrane filter of 47 mm diameter with 0.45 µm pores. The result is reduced to what is called a membrane-filter index (or "modified fouling index", shortened to MFI) expressed in s/l^2 (s = seconds, l = litres). For injection wells, MFI-values less than 3 are good and over 10 - 15 bad (figure 12).

To arrive at the final design, practical experiments will stay necessary. Interpretation of the data from experiments with test wells is presented in section 5 of this report.

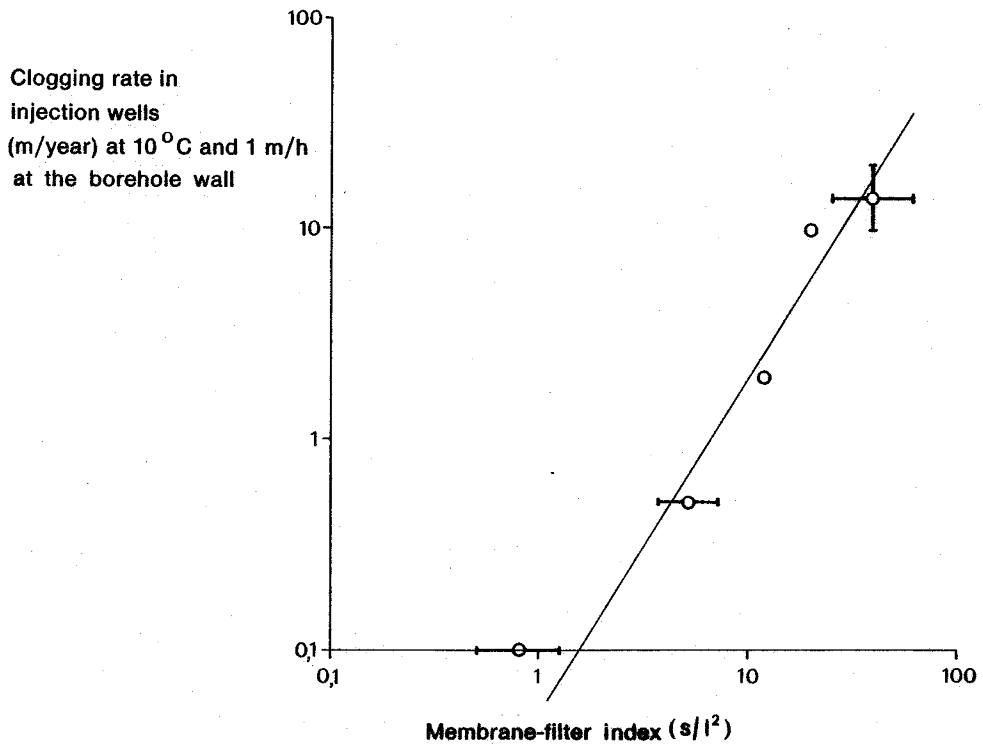


Figure 12 - Relation between injection-well clogging and membrane-filter index of Dutch injection wells.

In order to secure a good idea of this kind of clogging and interpretation of the test results obtained, a few basic relations will now be derived.

The rise of head, $\Delta\phi$, in an injection well above the static water level (or above the level in a neigbouring observation well) is now divided into a portion $\Delta\phi_o$ resulting from the soil itself and a portion $\Delta\phi_v$ caused by clogging (figure 4) which has reduced the permeability k between the distances r_o and r_v to k_v. In the case of well injection, the total required head rise in the well will be as follows:

$$\Delta\phi = \frac{Q}{2\pi k_v H}\ln\left(\frac{r_v}{r_o}\right)+\frac{Q}{2\pi kH}\ln\left(\frac{R}{r_v}\right)$$

$$= \frac{Q}{2\pi k_* H}\ln\left(\frac{r_v}{r_o}\right)+\frac{Q}{2\pi kH}\ln\left(\frac{r_v}{r_o}\right)+\frac{Q}{2\pi kH}\ln\left(\frac{R}{r_v}\right) \qquad (3.5)$$

where R is an integration constant (m).

Since the 2 last terms together give the natural soil resistance, namely:

$\Delta\phi_o = \frac{Q}{2\pi kh}\ln\left(\frac{R}{r_o}\right)$; it follows that for $\Delta\phi_v=\Delta\phi-\Delta\phi_o$:

$$\Delta\phi_v = \frac{Q}{2\pi k_* H}\ln\left(\frac{r_v}{r_o}\right) = \frac{Q}{2\pi h}\left\{\frac{1}{k_v} - \frac{1}{k}\right\}\ln\left(\frac{r_v}{r_o}\right) \qquad (3.6)$$

so that the summation in (3.5) is always admissible provided k_v be replaced by k_*:

$$k_* = kk_v/(k-k_v) \qquad (3.7)$$

Since normally $k \gg k_v$, equation (3.7) is reduced to $k_* \simeq k_v$.

These permeability coefficients allow for the viscosity of the water as follows:

$$k = \frac{\rho g}{\mu} k_i \qquad (3.8)$$

k = permeability coefficient (m/s), ρ = specific gravity of water (ρ = 1000 kg/m^3), g = strenght of gravity field (g = 9.81 N/kg), k is the intrinsic permeability coefficient (m^2) , a material property independent of the percolating liquid.

The pressure drop dp (N/m^2) resulting from flow through a clogging layer of thickness dl (m) and

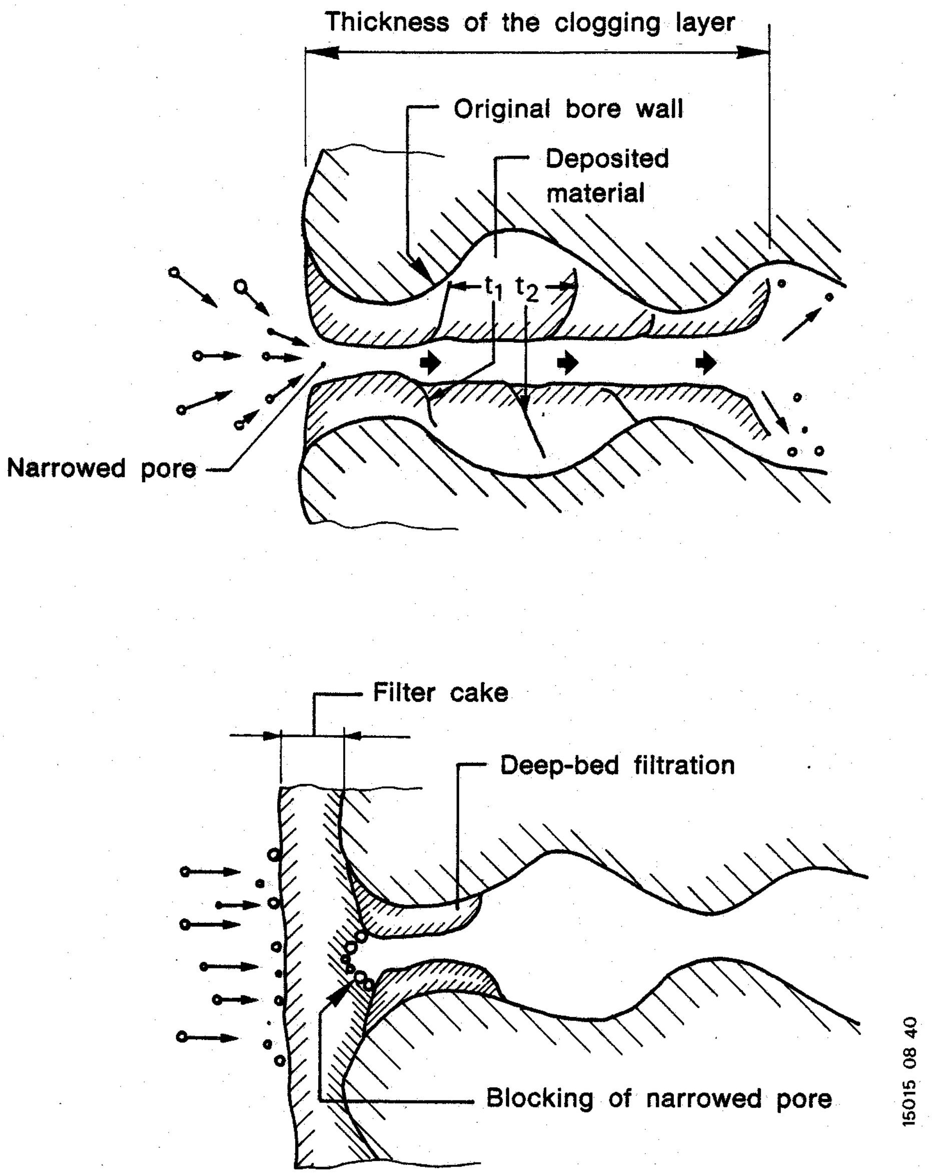


Figure 13 - Above: clogging in deep-bed filtration
Below: if the pores are excessively narrowed by deep-bed filtration, or the particles in the water are too large, then clogging will occur through cake filtration.

permeability k_i (m^2), will, at a flowrate Q (m^3/s) and filtering surface A (m^2), have the following value:

$$dp = (\frac{\mu}{k_i}) (\frac{Q}{A}) \, dl \qquad (3.9)$$

Since p is directly proportional to Q and μ, we can introduce the concept of resistance (W):

$$dW = d \, (\frac{p}{Q\mu}) = (\frac{1}{k_i A}) \, dl \qquad (3.10)$$

The volume of clogging layer brought in per m^3 water (dV) is directly related through A with dl: dV = A dl, while for a well: dV = $(2\pi r)(H)dr$, where r is the distance to the centre of the well from the place where the clogging occurs, and H the length of the gravel pack, so that:

$$dW = (\frac{dV}{k_i A^2}) = (\frac{1}{2\pi k_i H})(\frac{dr}{r}) \qquad (3.11)$$

where W = 0 if t = 0; this means that r = the bore radius, r_o:

$$W = \frac{1}{2\pi k_i H} \ln (\frac{r}{r_o}) \qquad (3.12)$$

In the case of cake filtration at the bore wall (figure 13 below) r steadily decreases further ($r < r_o$)

$$V = {}_o\!\int^t Qcdt = \pi \, H \, (r_o^2 - r^2) \qquad (3.13)$$

where c is the concentration of suspensoids (expressed in terms of m^3 clogged layer per m^3 water).

In the case of deep-bed filtration (figure 13 above) the reverse applies ($r \geq r_o$)

$$V = \int_o^t Qcdt = \pi H (r^2 - r_o^2) \qquad (3.14)$$

For $\phi = \frac{p}{\rho g}$, this will give:

$$\Delta \phi_V = \frac{(\mu/\rho g)Q}{2\pi k_i H} \ln (1 \pm \frac{V}{\pi r_o^2 H}) \qquad (3.15)$$

where the plus sign applies for deep-bed filtration and the minus sign for cake filtration (naturally another process will be attended by another permeability). For a short time, and thus a small V, the relation goes over to that for linear filtration:

$$\Delta \phi_V = (\frac{1}{\rho g})(\frac{\mu}{k_i})(\frac{Q}{A_o})(\frac{V}{A_o}), \text{ where } A_o=2\pi r_o H \qquad (3.16)$$

or:

$$\Delta \phi_V = (\frac{1}{\rho g})(\frac{\mu}{k_i}) v \bar{V} \qquad (3.17)$$

where v is the infiltration rate at the bore wall (m/h) and $\bar{V}$ the volume of suspended matter (m) introduced per m^2 borehole-wall area.

Since this relation is derived independetly of the instantaneous flowrate, it applies generally for this kind of clogging and can be used equally well for infiltration under constant pressure or under constant flowrate.

With constant concentration c (m^3 clogging layer per m^3 water), the following relation applies:

$$\Delta \phi_V = (\frac{1}{\rho g})(\frac{c\mu}{k_i}) v \bar{U} \qquad (3.18)$$

where $\bar{U}$ is the total volume of water (m) infiltrated per m^2 bore-wall area.
Should this flowrate also be constant, then we have:

$$\Delta\phi_v = (\frac{1}{\rho g})(\frac{c\mu}{k_i})v^2 t \qquad (3.19)$$

which is equivalent to the previously mentioned quadratic relation (3.4) between pressure increase and infiltration rate.

At constant pressure p or head ϕ and a total infiltrated volume $\bar{U}$ per m^2 bore wall, and with $v = v_o$ if $t = 0$, we have:

$$v = \frac{v_o}{a + 1}, \text{ where } a = (\frac{v_o}{p})(\frac{c\mu}{k_i})\bar{U} \qquad (3.20)$$

while elaboration with respect to time t instead of $\bar{U}$ gives:

$$v = \frac{v_o}{\sqrt{(bt + 1)}}, \text{ where } b = (\frac{2}{p})(\frac{c\mu}{k_i})v_o{}^2 \qquad (3.21)$$

Figure 14 shows that this relation covers the results of Bianchi & Nightingale (1978) better than the approximations that they actually used, which, moreover, lack any obvious physical background.

The formulae derived can be used to describe clogging by suspended matter. The actual water quality, which can be equated with the factor $\frac{c}{k_i}$, can only be determined experimentally and will vary in an unknown fashion in a natural water, while c and k_i depend also on the formation.

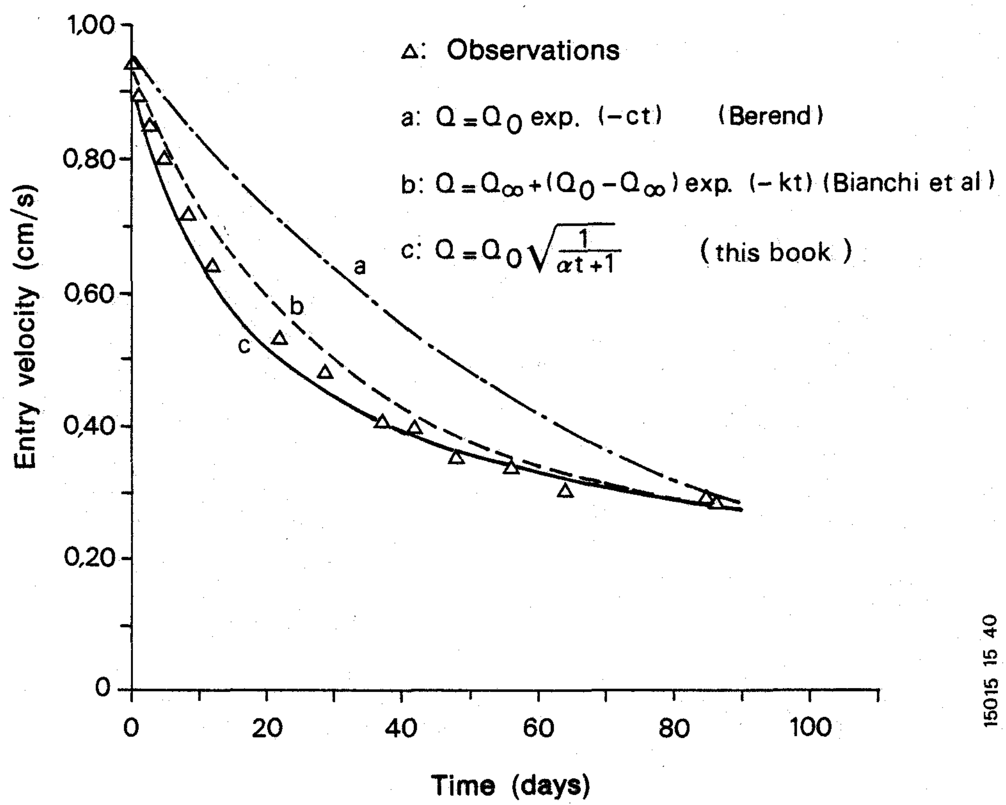


Figure 14 - Infiltration rate as a function of time; comparison with various formulae from Bianchi et al, 1978.

An important thing is that the formulae make it possible to compare the clogging experienced in different wells. To this end, the increase of pressure head is reduced to the number of metres of water-head rise that would have occured if the water had been infiltrated at a temperature of 10 °C for 1 year at a rate of 1 m/h at the bore wall.

In this connection it suffices to compare just one point at the start and one at the end of the period considered. At the start of the period considered the flowrate is Q_b, the water viscosity μ_b and the injection head with respect to the static head or that in a nearby gauge well ϕ_b. At the end of the period considered these quantities will be respectively Q_e, μ_e and ϕ_e. During the period of length t, a volume of water U was injected. The well has an infiltration surface A. The required standard-injection head $\Delta\phi_s$, which would occur at the standard infiltration rate of $v_o = Q_o/A = 1$ m/h, water viscosity μ_o (water temperature 10°) for the standard period of time t_o (8760 h) amounts to:

$$\Delta\phi_s = \{\phi_e(\frac{\mu_o}{\mu_e})(\frac{v_o}{Q_e/A}) - \phi_b(\frac{\mu_o}{\mu_b})(\frac{v_o}{Q_b/A})\}\{\frac{v_o t_o}{U/A}\} \quad (3.22)$$

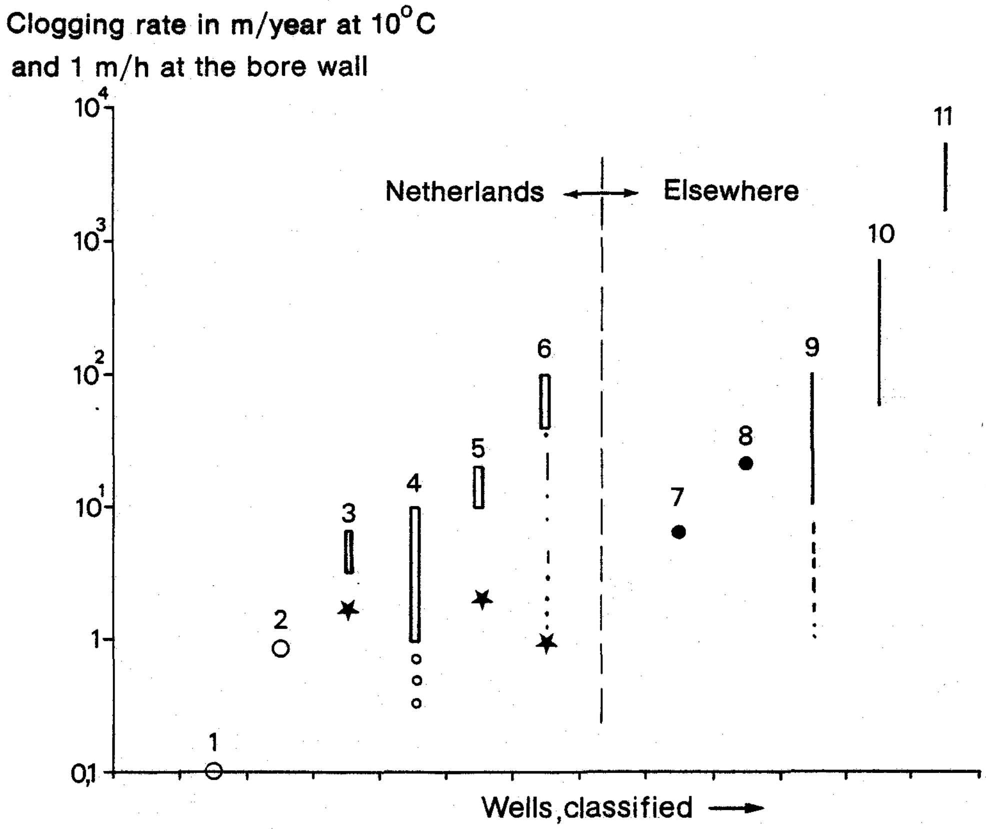
which, when $\mu_e = \mu_b = \mu_o$, $Q_e = Q_b = v.A$ and $U = vAt$, transforms to the known quadratic relation:

$$\Delta\phi_s = (\phi_e - \phi_b)\ (\frac{v_o}{v})^2 (\frac{t_o}{t}) \quad (3.23)$$

Data of injection wells from different origin and different places of the world, was reduced by means of these formulae, to a common denominator (see fig. 15). When this was done it appeared that installations thought to be successful sometimes contrast to some extent with others that were thought less successful.

3.3 Gas bubbles

Gas bubbles and air bubbles with diameters between 1 and 10 mm rise at a velocity of 0.3 to 0.4 m/s through water (Rautenberg, 1972). In most injection wells they can thus reach the well screen and penetrate through the screen slots and the gravel pack after which they block the formation pores in an extremely quick fashion (fig. 16, Sniegocki, 1963).



11. Brown & Silvey, Norfolk, Virginia (1973)
10. Vecchioli, Long Island N.Y. (1971)
9. Sniegocki, Texas (1963)
8. Los Angelos (Bulten et al. 1974)
7. Israel, Rebnun Schwarz (1968)
6. *PWN — Drinking water from Andijk
5. *HO — Filtered Rhine water
4. *DWL — Secondary purification in an experimental installation
3. *GW — River water after coagulation, etc.
2. *Leiduin model-injection well
1. *DWL — Drinking-water recharge

Figure 15 - Clogging rates measured in injection wells and model tests in fine to moderately coarse sand.

* See Appendix A

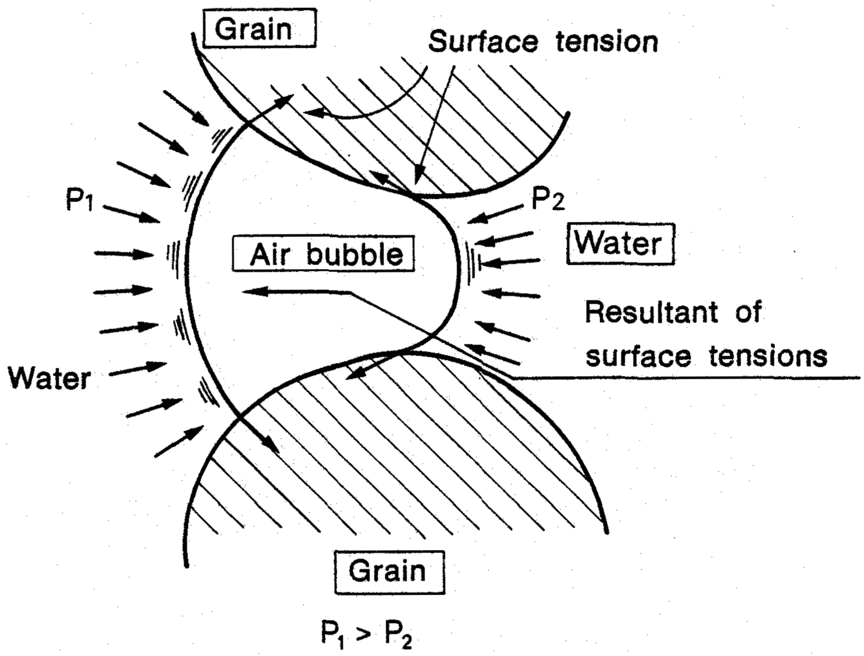


Figure 16 - Resistance of a jammed gas bubble, which, due to deformation, causes the surface tension to produce a resistant force.

When the injection line ends entirely at the bottom of the well, the bubbles, with their almost constant rising velocity, may clog the whole screen. But if this line ends above the screen, this cannot happen, since the vertical speed of the water decreases the deeper they go and they will therefore never be able to reach the lowermost part of the screen. Reducing the flowrate in a well of this kind while its clogging increases will stabilize the clogging at a certain level. There will then be equilibrium between supply and dissolution.
Clogging by gas bubbles is characterised by the

aforesaid equilibrium and the rapidity with which the well clogs after being brought into service.

Bubbles may form in the water as a result of:

a. free fall of water into the well (entrainment);
b. leaks in the conduits at points where there is a pression below atmospheric (valves, topside of injection line);
c. supersaturation with a certain gas (may occur through pressure fall or heating).

<u>to a</u>

Free fall of water and too low a pressure at the top of the injection pipe are prevented by furnishing the latter with a proper (possibly adjustable) restriction at its end or by fitting an injection pipe, narrow enough to ensure that its wall friction will provide the requisite counterpressure (Sniegocki & Reed, 1963).

The required friction f (resistance in m H_2O per m injection pipe) will be (figure 17):

$$f = h/L \tag{3.24}$$

where h is the distance from the highest point of the injection line to the lowest groundwater level (m) in the well and L the length of the vertical part of the injection pipe (m). Table 1 gives values of f as a function of the internal diameter d of the injection pipe and infiltration flowrate Q.

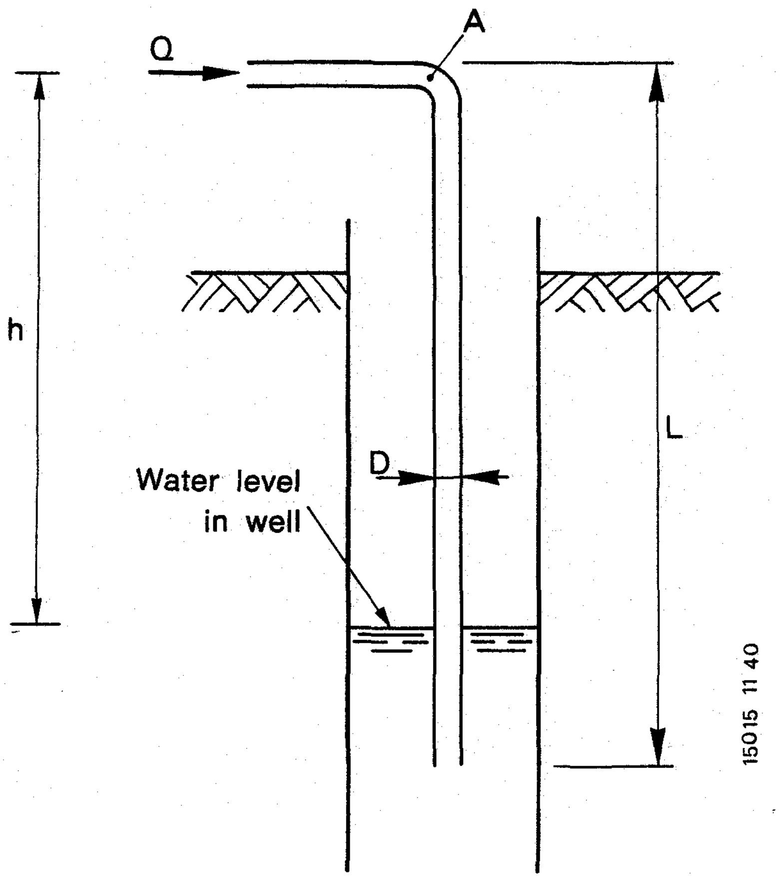


Figure 17 - Essential measurements of an injection pipe.

d (mm) Q (m^3/h)	25	30	40	50	60	70	80
5	0.33	0.15	0.04				
10	1.39	0.56	0.13	0.04			
25	8.10	3.19	0.75	0.24	0.10	0.05	
50		12.3	2.81	0.91	0.36	0.17	0.09
100			10.9	3.46	1.37	0.63	0.32
200				13.4	5.26	2.39	1.21

Table 1 - Friction f (m H_2O/m) of plastic pipes of internal diameter d (mm) at a flowrate Q (m^3/h), at 10 °C water temperature (calculated after Huisman, 1969).

to b

Just beyond the gate of a throttled valve a relatively low pressure exists (figure 18). A negative value (below atmospheric) may lead to the intrusion of air. This must be prevented by maintaining a sufficiently high pressure level down-stream of the valve. The throttle head h_2 (m) remains positive when the head downstream of the valve, h_3, satisfies the following relation:

$$h_3 > 2\ \Delta H \surd \left(\frac{v^2}{2g}\right) \qquad (3.25)$$

where ΔH is the piezometric loss across the valve (m) and v the average water-flow velocity further along the pipe (m/s) and g the strength of the gra-

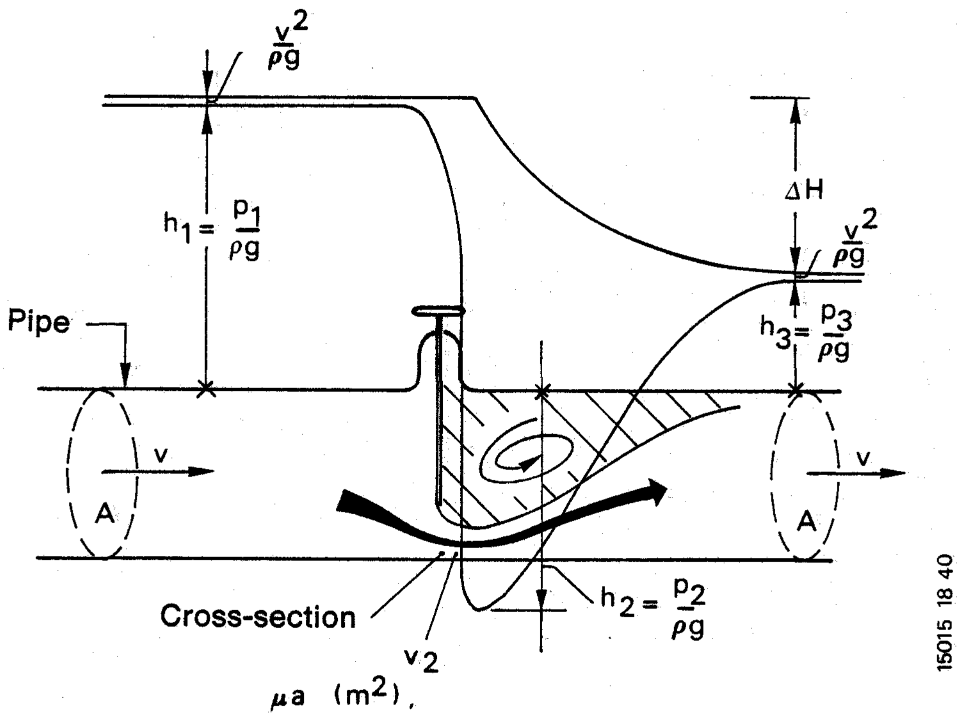


Figure 18 - Variation of energy- and pressure head across a valve.

vity field (N/kg). At a water-flow velocity of 1 m/s and an upstream head h_1 of 10 mH_2O, the head beyond the valve may thus not fall below 0.91 mH_2O, as can be readily checked from the foregoing equation.

<u>to c</u>

Heating the water lessens the solubility of gasses. The effect is however slight, some 2 % per degree centigrade.

Because the solubility rises in proportion to the increase of absolute pressure, the heating effect can be compesated easily by a slight rise of pressure. A pressure-increase equivalent to 1 mH_2O in a pipe carrying a head of 5 mH_2O will thus (assuming 10 mH_2O head as the atmospheric pressure equivalent) increase the solubility by 1/15 = 7 %. This slight increase of pressure thus suffices to compensate a temperature rise of about 3.5 °C. This being so, the water temperature offers no problem in infiltration via injection wells.

The solubility of gas is chiefly a problem when abstrated groundwater is to be reinjected, due to the fact that groundwater may sometimes contain a lot of methane (natural gas). Due to the marked fall in pressure caused by the raising of such water, the latter may become oversaturated, resulting in strong degassing with formation of bubbles. To be able to reinject such a water requires an adequate degassing installation or else maintenance of a high pressure in all pipes and conduits at the cost of a lot of energy and stronger piping (Brandes et al., 1978).

3.4 Development of bacteria in the well

The volume of a sand grain of only 1 mm diameter is the same as the combined volume of all the bacteria to be found in 1 cubic metre of water when this contains, what is for purified water the relatively high number of 1000 bacteria (globular, diameter 1 micron) per millilitre. Bacteria in the prepurified recharge water can therefore cause no significant clogging in practice. They only cause clogging when able to grow and multiply extensively in the well. Given a doubling time of say 8 hours, one single bacterium can, in theory, within 12 days produce 10^{16} progeny, enough for a 1 cm thick layer of slime over 1 m^2 borehole surface. Given a doubling time of 80 hours this situation would ensue after 120 days. Such a layer of slime and bacteria bodies on the borehole wall of the injection well would obviously clog it completely.

The extent to which extensive growth is possible, will, in the absence of disinfectants, be governed by the supply of organic matter decomposable by bacteria. Given a fair supply of such matter, clogging will set in at an accelerating rate after a propagation period of several days to weeks, possibly leading to almost complete sliming up of the well (Vecchioli, 1972). Then, after flushing the well, sufficient bacteria will still remain to restart the clogging process immediately afterwards, passing over any build-up period when injection is resumed. On the other hand, if the water carries little organic matter, equilibrium will be reached after some time, causing the clogging to stabilize. The dissimilation (food combustion) is then equal to the food supply per unit of time and assimilation (growth) is reduced to zero (fig. 19). Since the assimilatable organic matter in the injection water is the governing factor in this case, chemical standards describing the quantitiy of organic matter, such as COD and $KMnO_4$ are of little use. The BOD (Biochemical Oxigen Demand) is a better one, albeit an insensitive measure, suitable at best for recognising bad kinds of water, while the AOC (assimilatable organic carbon) (see Kooij V.d., 1978) is the most promising measure. However, no numerical values are yet available.

Logically bacteria are fed best at shallow depth, i.e. at the screen slots and in the first few centimetres of the gravel pack, instead of somewhere further on in the formation. In the case of strong bacterial growth, this leads to heavy clogging of the screen slots (fig. 20) and the gravel pack. Contrast to this suspended matter clogs the formation wall rather than the screen slots and gravel pack.

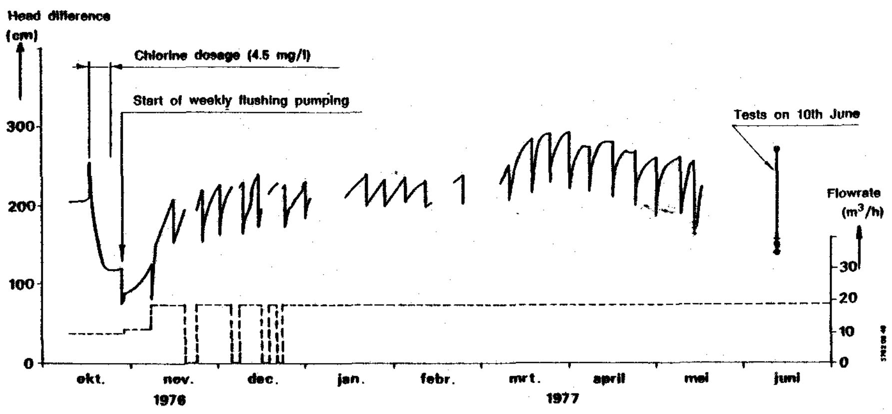


Figure 19 - Waterlevel in the gravel pack, minus that in the observation well 8 m away, as from the second injection test. PWN injection well in Castricum (see Appendix A).

Clogging by organic matter and bacteria leads to putrefaction when the infiltration is discontinued. Should the well cease operation for several weeks, the water will in such cases acquire a bad smell and taste and the resistance in the well will have decreased (Harpaz, 1970, Eren & Goldsmid, 1970), often quite considerably (figs. 21 and 6).

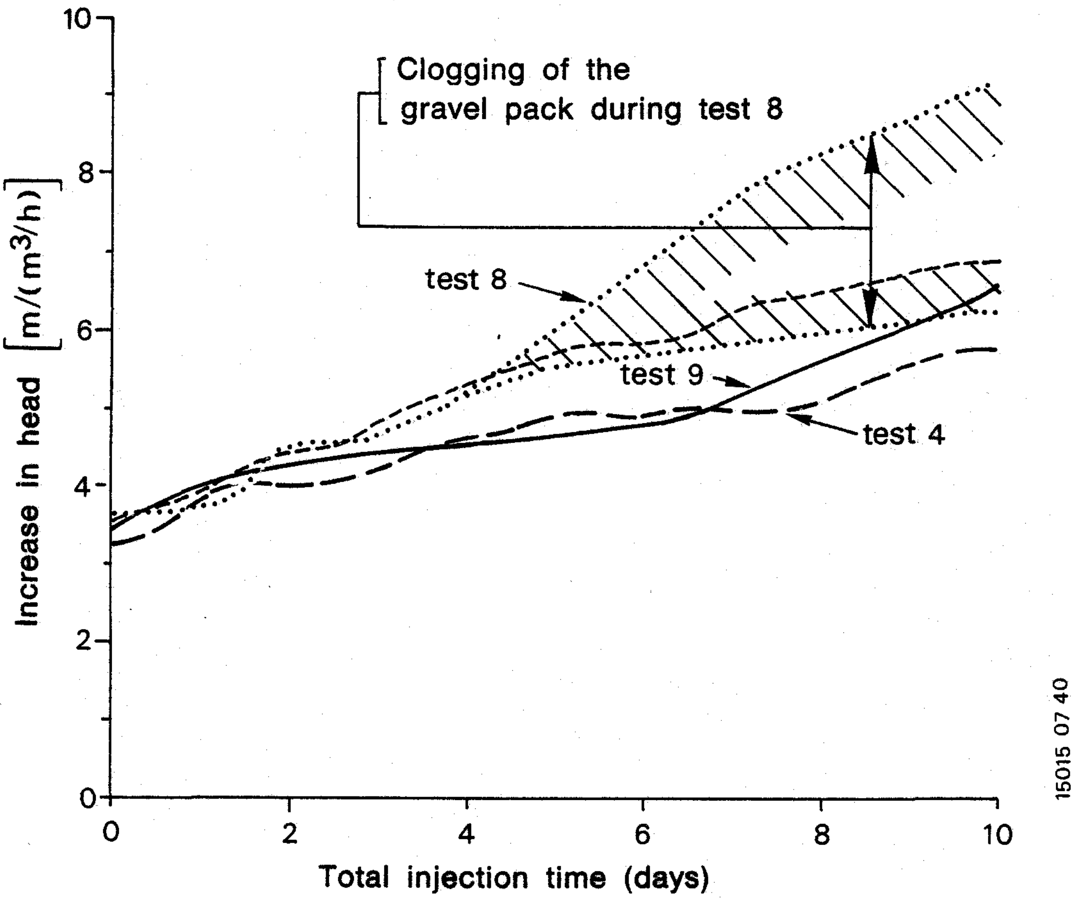


Figure 20 - Rise of water head in the well during 4 injection tests. Only in test 8 was the water not chlorinated and only in test 8 became the gravel pack clogged. After Ehrlich et al., 1973; see also table 3.9.

The bacteria and their eventual slime act as a "net" for inorganic particles, which, owing to the shallowness of the layer of clogging material, can hardly penetrate into the formation. A well clogged in this manner can therefore be cleaned fairly well by flush pumping. Endeavour is generally made to prevent clogging by injecting with 0.2 to 2 mg free chlorine per litre injection water. However, since chlorine is mostly consumed very quickly in the soil, bacterial development will still be possible at a certain depth. A variable chlorine dosage or a periodical shock dosage of chlorine in high concentration (100 mg chlorine per litre or over, Krone 1970) can suppress this and is therefore preferable to continuous chlorine dosage at a constant rate.

The shallow clogging, the initially low clogging rate that steadily increases after some days sometimes followed by attainment of an equilibrium state, the decrease in resistance that spontaneously occurs when the well is put out of action for several weeks, and the limitation of the intrusion depth for other clogging matter, all these factors, are characteristics of clogging by bacteria.

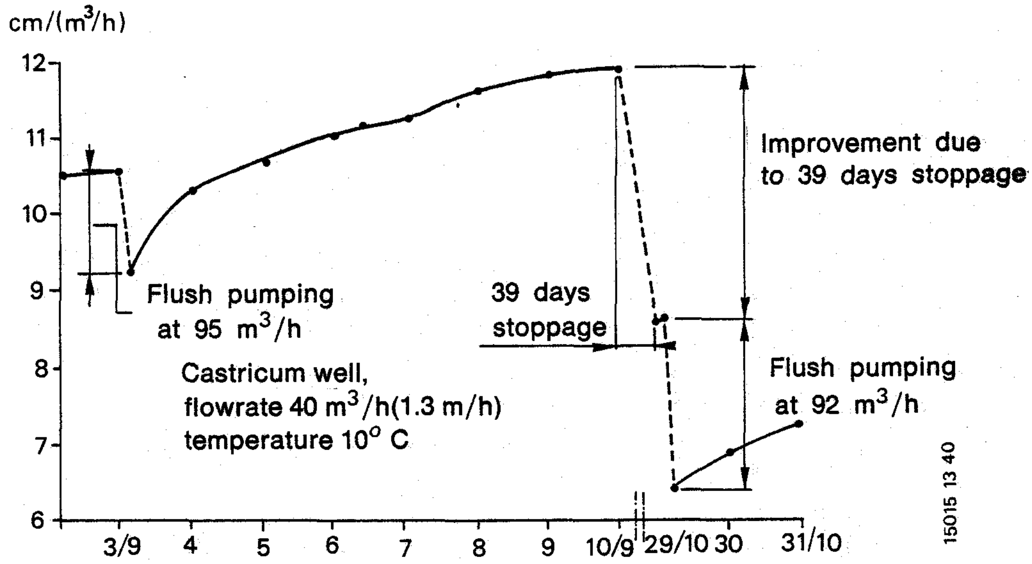


Figure 21 - Reduction of clogging resistance (injection head in well minus that in observation well 8 m away and divided by the injection flow) by stoppage for 39 days.
PWN injection well at Castricum, September-October 1979 (see Appendix A for further details)

3.5 Formation of chemical precipitates in the well

The "bacterial sludge" referred to in the previous paragraph could well be called a biological precipitate as a counterpart to a chemical precipitate. A chemical precipitate gives rise to suspended components in the water which can as such be recognised and traced. However the precipitate may escape from our view when formed just in the well. Still, there is no reason why reactions should occur in the well that do not occur in the supply line. But the situation changes when the water is "interfered with" immediately before the well, say by admixture of another kind of water (a known fault in injection wells of the oil industry (Farley & Redline, 1968; Case 1970; Patton 1974)) or when some addition, mostly chlorine, is made. The latter occured at a test well in The Hague, where the chlorine, added shortly before the well, caused precipitation of manganese dioxide (Olsthoorn 1979). This manganese was present as an impurity in the coagulant (iron chloride) but was not removed by the following filters. Its removal was not effected until chlorination was installed before the rapid sand filters, an action which solved the problem. In general, one must be prepared for reactions when adjustments are made to the composition of the recharge water immediately before an injection well. Preferably the last purification step should be filtration.

3.6 Reactions between injection water and groundwater

Mixture of groundwater (often devoid of oxygen and containing iron) with injection water (mostly rich in oxygen) can often cause precipitation (iron hydroxides, Warner 1966) although this does not occur to any great degree in practice; reactions can only proceed in the mixing zone between the expelled groundwater and the intrusive injection-water, this mixing zone remains relatively thin and moves steadily away from the well, while the reactivity of the mixing zone steadily decreases as a result of the reactions that have already taken place within it. Only in fissured formations where the flow is mainly through the cracks, but the bulk of the original groundwater still remains for a long time in the formation pores, is a prolonged and intensive commixtion of groundwater and injection water conceivable, with possibly unpleasant results.

For granular formations without fissures, Bernard (1955) showed in laboratory tests that neither parallel flow nor flow in series (expulsion) of two kinds of water forming reactive precipitates if mixed, will cause any significant clogging. Although the situation referred to in the first sentence of this paragraph is frequently encountered, experience with injection wells has produced no evidence of this kind of clogging.
However, mixing of different kinds of water does indeed occur in extraction wells (Case, 1970) as a logical consequence of the residence-time differences brought about by the differing flowlines attended with passage of water through the ground. Extraction wells in an injection-well system can therefore clog internally fairly soon. However, this clogging can be simply removed by cleaning the well with high-pressure-jetting nozzles directed horizontally, squirting the deposits from the screen slots.

3.7 Interaction between injection water and soil

The interaction between injection water and soil relates almost exclusively to the swelling and dispersion of clay minerals insofar as these are present around the grains of the aquifer (figure 22).

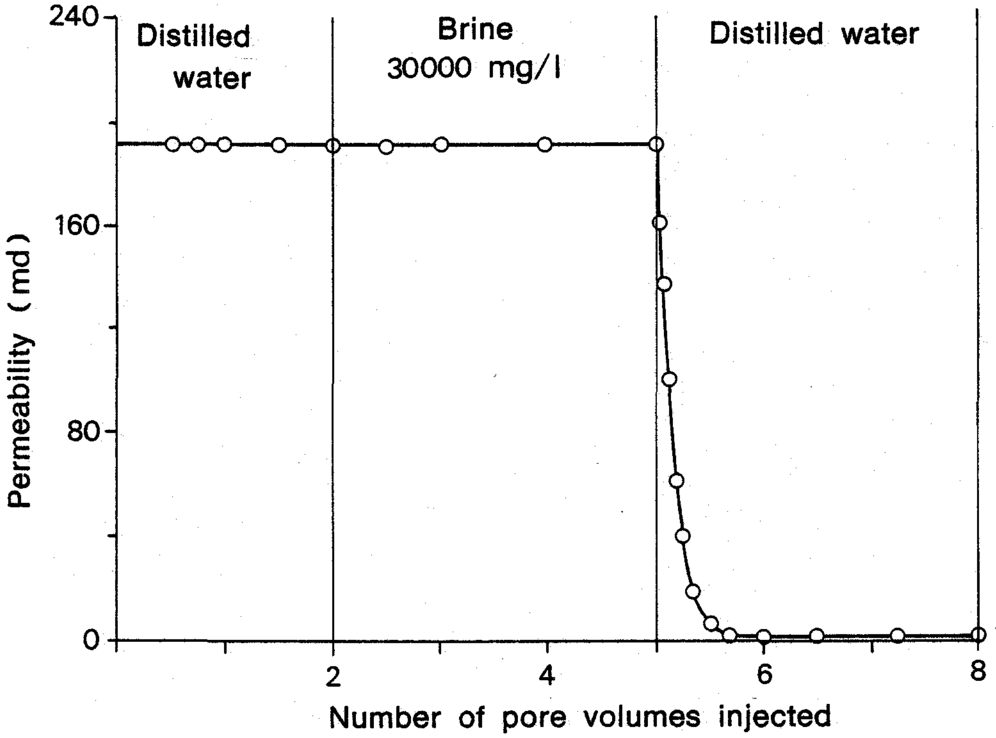


Figure 22 - Typical effect of salt content (NaCl) of the through-flowing water on the permeability of a core of Berea sandstone (Mungan 1965)

Clay minerals consist of negatively charged platelets, threads or flocs (Millot, 1979) held together by positive ions. The higher the valence of the positive ions in the water and the higher their concentration, the more densely and strongly will the minerals be packed together. Lowering of the ion strenght and replacement of high-valent ions (especially Ca^{2+}, Mg^{2+}) bij low-valent ions (Na^{+},

K^+) causes a looser packing (swelling) and often loss of the bond between the clay particles, which will then be entrained by the flowing water (dispersion). Swelling squeezes the pores and dispersion blocks them so that the entrained particles are trapped, after which hardly any water flow may still be possible. In Norfolk, Virginia, clay dispersion caused the uniquelly rapid injection-well clogging of a good 3500 mH_2O/year (0.4 mH_2O/h) at an infiltration rate of 1 m/h at the bore wall (Brown & Silvey, 1973, see also figure 15).

The SAR (sodium-adsorption ratio) of the water is often taken as a measure for the risk of swelling and dispersion of argillaceous minerals:

$$*SAR = \frac{[Na^+]+[K^+]}{\sqrt{\{[Ca^{2+}]+[Mg^{2+}]\}}} \quad \text{(conc. in mol/m}^3\text{)} \quad (3.26)$$

The higher the SAR the greater the risk. However, the influence of the ionic strenght is only partly incorperated in the SAR so that clay minerals in salt groundwater, despite its high SAR, are nevertheless strongly aggregated. Fresh water generally has an acceptably low SAR, but this is completely altered during the underground expulsion of brackish or salt water by fresh water (figures 22 and 23).

* The normal SAR-definition in the literature does not contain potassium, while the PAR (potassium-adsorption ratio does not contain sodium). Equation (3.26) should therefore probably be called PSAR, as it is the sum of the SAR and PAR.

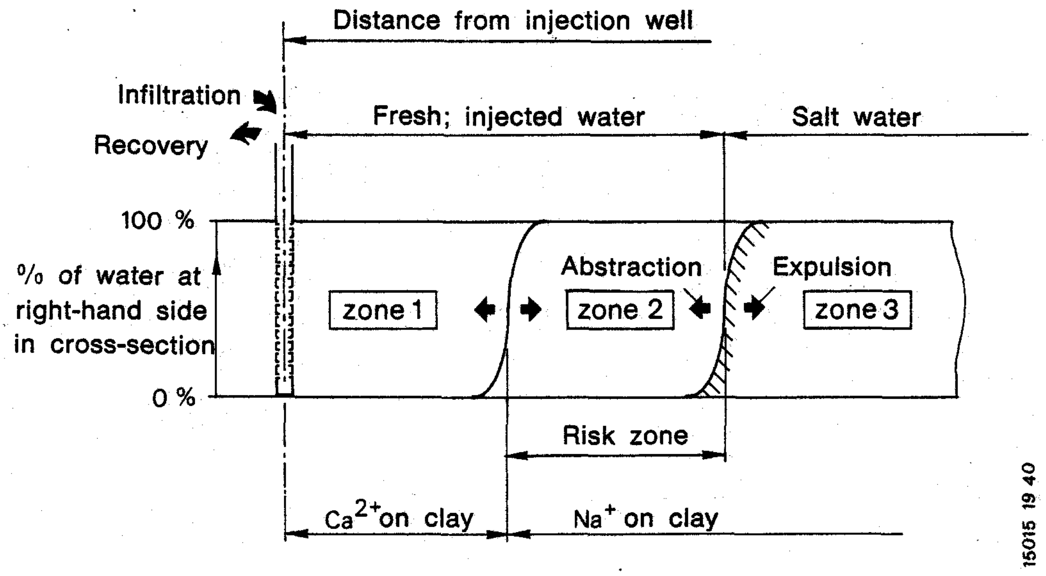


Figure 23 - Expulsion of salt groundwater (3) by fresh injection water (1) causes cation exchange and a sodium water at low ionic strenght in the first infiltrated water (2). This zone (2) offers a risk of swelling and dispersion of clay minerals.

By cation exchange (figure 24) between the first injection water (zone 2 in fig. 23) and the soil initially in equilibrium with the salt water (zone 3), this injection water acquires the same SAR as the expelled groundwater, but at the low ionic strenght of the injection water. This danger zone can, if the formation contains some sensitive clay minerals, induce swelling and dispersion of these clay minerals. Clogging then occurs immediately, after which hardly any flow may still be possible. When injecting into a well in salt groundwater, this danger must therefore always be borne in mind.

The risk arises when the SAR of the groundwater is too high (SAR > 3 to 5, Krone 1970). The SAR of the injection water does not matter.

Laboratory investigation of clay minerals of the injection layer and tests with undisturbed soil samples, can show whether the danger is real in cases where the SAR may seem too high (Brown & Silvey 1973). Should it be, then clogging can be reduced by 80 to 90 % by a preliminary injection of several cubic metres of water with a high $CaCl_2$-concentration, sufficient to delay the reaction as far as 1 to 2 m outside the well (Brown & Silvey 1973). The more efficient high-valent-metal ions with which the oil industry overcomes this problem (zirconium, hafnium, titanium and suchlike, see Veley 1969) are unsuitable for the water-supply sector.

In contrast to dispersion, swelling is a reversible process, if flow is still possible. Redevelopment of a soil clogged by swelling and/or dispersion around the well is possible only partly. To redevelope a formation clogged by clay dispersion, it is essential to move the water to and fro continuously, for instance by air-lift juttering (chapter 4).

3.8 Change in the granular structure of the soil

As a result of repeated injection and extraction, and acid leaching of the soil, the latter may tend to settle and so cause a reduction in permeability around the well (Johnson, 1966). But the results of this are limited as the reduction is only 1- to 3-fold, can extend to a depth of a few metres at most and then, after settlement, no further reduction

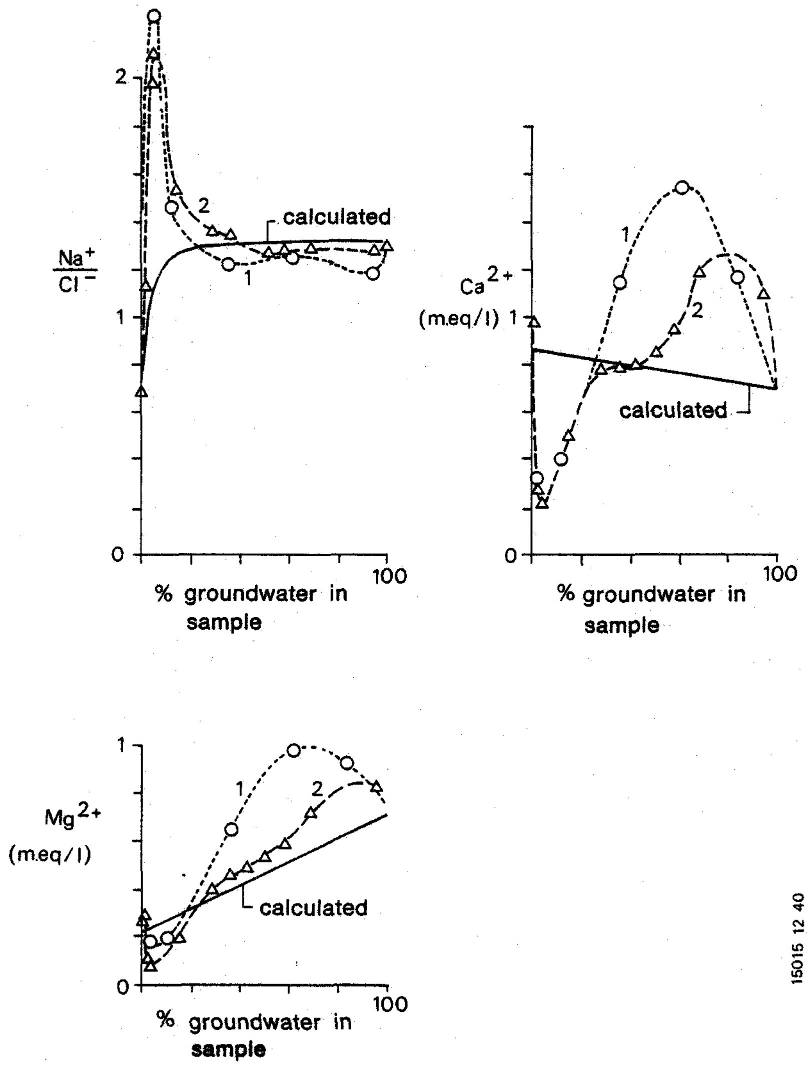


Figure 24 - Calculated (continuous line) and measured (pecked line) concentration of Mg^{2+}, Ca^{2+} and (Na^{+}/Cl^{-})-ratio as a function of the calculated (from chloride) mixing-ratio between groundwater and injection water in the water pumped up from the well. The deviations from the continuous lines show the cation exchange that has occured (calculated from data after Brown & Silvey 1973).

will occur. When the permeability coefficient, k, has decreased to k_v the resulting rise of pressure head can be calculated directly from a formula previously established (formula 3.7), i.e.:

$$k^* = kk_v/(k-k_v) \tag{3.7}$$

and

$$\Delta\phi_v = \frac{Q}{2\pi k^* H} \ln \left(\frac{r_v}{r_o}\right) \tag{3.27}$$

When, e.g., $v_o = Q/(2\pi r_o H) = 1$ m/h (24 m/d), k = 10 m/d, $k_v = 0.5$ k so that $k^* = k$; $r_v = 3$ m and $r_o = 0.25$ m it follows by way of example that:

$$\Delta\phi_v = \frac{(24)}{(10)} (0.25) \ln \left(\frac{3}{0.25}\right) = 1.49 \text{ m}$$

which means a limited reduction.

4 REDEVELOPMENT OF INJECTION WELLS

4.1 Generalities

Provided good methods are used, an injection well can be successfully redeveloped (cleaned) in almost every case. Indeed, few, if any, injection wells are known that could not be redeveloped. Those that could not usually were defective, silted up (Olsthoorn, 1977) or collapsed through excessive pumping up of circumjacent formation sand (Bruington & Seares, 1965) so that the failure was attributable to wrong or weak design.

There are mechanical (hydraulic) and chemical methods or redevelopment. A chemical method merely means that chemicals are introduced to reinforce redevelopment by a mechanical method. Chemical methods and the more labour-intensive mechanical methods are normally only used when simple mechanical methods give inadequate results or take too long to secure the desired results. This situation will apply when a well has been cleaned a (large) number of times in succession by a simple method (e.g. by flush pumping), which, although successful, will always leave behind a certain (small) proportion of the clogging resistance built up since the last redevelopment (figure 36). A sort of major "spring clean" is then desirable from time to time.

When rather less prepurification is employed, the resistance generally rises rather quickly. The wells will then be flush pumped - as a rule automatically - at frequent intervals of mostly once a day to once a month (figure 19).

If, through prepurification, resistance rises only slowly, the interval between two successive developments will be greater (usually 6 months to 5 years, Bruington & Seares 1965). An automatic flushing-pump system will then not be worthwile and it is best to give every well a more intensive treatment as soon as its turn comes round (Cooper 1971). This intensive redevelopment treatment involves about half a day to two day's work usually, by a team of two men.

A slight deterioration in quality of the injection water, as may sometimes arise through a fault in the prepurification system, can lead to rapid clogging of injection wells and may put the whole well system out of action. In such cases it should therefore be possible to flush pump injection wells quickly and easily, wether they are redeveloped with the aforesaid regularity or only when they are just about due for it.

4.2 Mechanical methods of redevelopment

4.2.1 Generalities

There is a large number of redevelopment methods, but by no means all of equal importance. In order of importance for the water-supply sector, these methods may be listed as follows:

a. flushing pumping;
b. juttering with compressed air;
c. sectional flushing pumping;
d. flushing spraying at high pressure (jetting);
e. surging and bailing;
f. brushing;
g. high-frequency vibration;

h. use of explosives in the well, and
i. hydraulic fracturing.

Methods g. to i. inclusive are used exclusively in deep injection wells in solid rock (in deep disposal wells and in the oil industry (Tazelaar 1968)) and are therefore outside the scope of this report. The remaining methods will now be discussed in greater or lesser detail according to their relative importance.

4.2.2 Flushing pumping

Flushing pumping of an injection well is undoubtedly the commonest redevelopment technique. With regard to required pumping capacity, pumping duration and pumping technique, we can now correct the misunderstandings encountered in practice by the factual material collected.

The flushing pumping of injection wells in fine-grained, unconsolidated formations removes on an average a good three quarters of the clogging resistance, built up since the well was brought into service or last redevelopment (see table 2 and figure 36).

Well or model test	v_i 5) (m/h)	Q_p/Q_i 6) (-)	percentage removed
Rebhun & Schwarz (coarse sand)	13	1)	40
Well GAT 24 (Final Report Israel)	13	2)	61
Hoogoven well	11	0)	72
The Hague	1.2	0)	73
Well GAT 21	37	2)	75
Hoogoven well	1.1	0)	76
Castricum	0.6	0)	76
Rebhun & Schwarz (fine sand)	13	1)	82
Well GAT 9	11	2)	83
Castricum	0.6	0)	85
Bichara (constant pressure drop, decreasing flowrate)	7.1	3)	89
Bichara	7.1	3)	92
Well GAT 6	8	2)	95
Veccioli test 7	0.6	4)	96
Mean			80

0) Calculated from original data

1) Calculated from observations in the paper by Rebhun & Schwarz (1968)

2) Calculated using data from the Final Report Israel, (1969). The borehole diameter was here estimated at 0.6 m

3) Calculated from data in thesis by Bichara (1974)

4) Test lasted 33 days. The infiltration rate lowered after 19 days from 1.1 to 0.6 m/h (Vecchioli 1972)

5) v_i is the infiltration rate, calculated at the bore wall

6) It is of interest to note the often small pump delivery in relation to the infiltration flowrate (Q_p/Q_i)

Tabel 2 - Review of percentage removal of clogging resistance by flushing pumping for different wells, arranged in increasing order of merit.

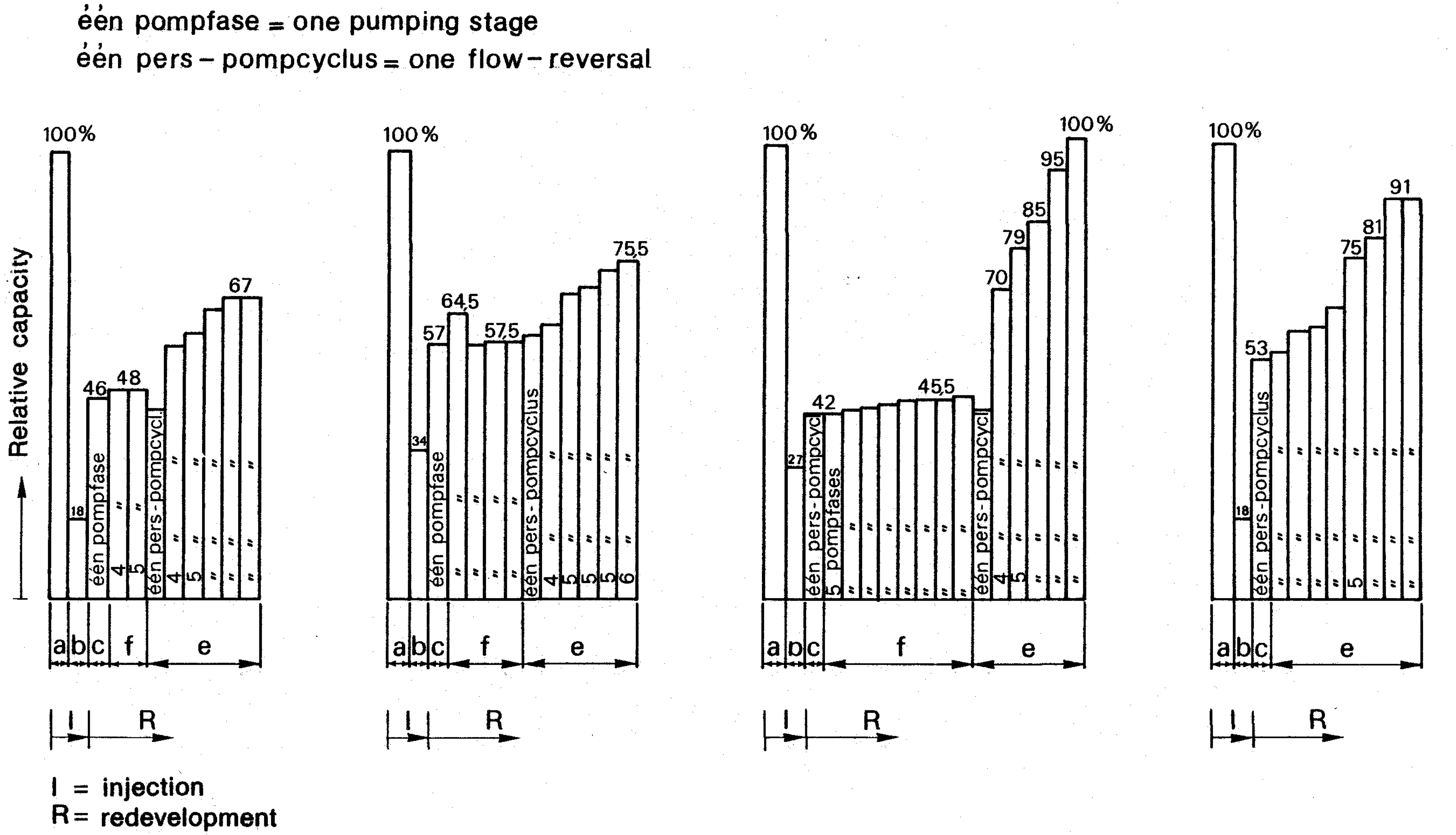


Figure 25 - Effect of intermittent pumping and flow reversal (flow reversals) on a clogged well (model test by Bichara 1974).

the actual work is done by the reversal of flow direction brought about by the pumping (figure 25). Whether pumping continues for two minutes or a whole day will make hardly any difference (table 3).

t(min) / Δh (mH_2O)	2	30	720	Average
2.5	90	88	89	89
5	94	89	92	92
Average	92	88	91	%

Table 3 - Reduction (%) of clogging resistance by flushing pumping as a function of pumping duration (t) and constant "pressure drop" (Δh) over the well model (Thesis by Bichara 1974).

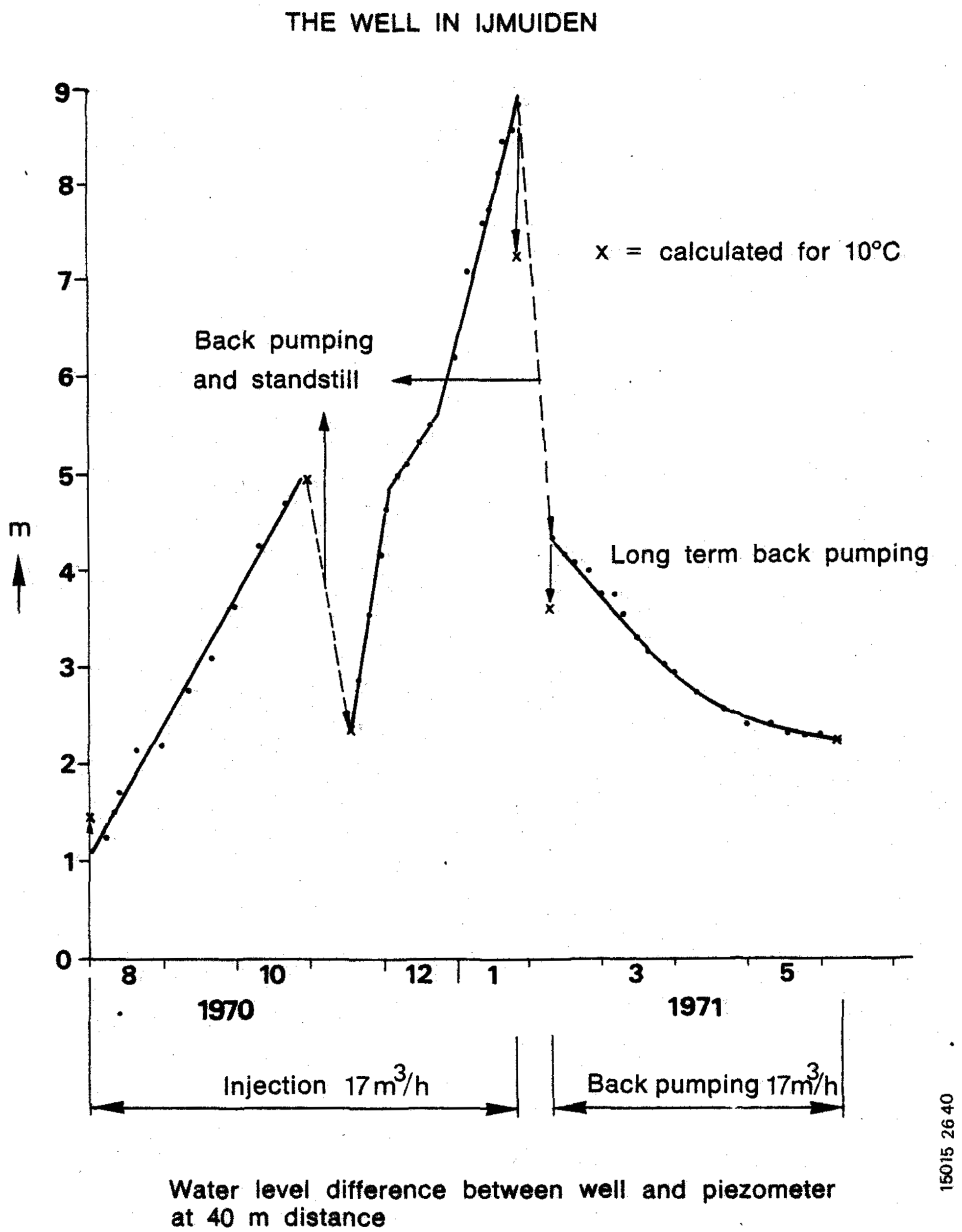


Figure 26 - The water level in the injection well belonging to ESTEL-Hoogovens B.V. at IJmuiden, minus that in the observation well at 40 m distance (during injection) and vice versa (during flushing pumping) demonstrates the immediate effect of flushing pumping and the very slow improvement from prolonged pumping.

Sraight pumping for more then a couple of minutes is only effective when carried out for weeks or months at a stretch (figure 26), a situation that normally exists only in dual-purpose wells (much used in Israel, table 4).

Well	Q_i	Q_p	V_i	V_p	$\frac{\phi_0}{Q_i}$	$\frac{\phi_1}{Q_i}$	$\frac{\phi_2}{Q_p}$	$\frac{\phi_3}{Q_p}$	W_∞	W_1	W_2	W_3
	(m^3/h)		(10^3 m^3)		(m/(100 m^3/h))				(%)			
Gat 6	220	170	0.21	0.75	2.7	6.7	2.9	2.1	0	100	5	-15*
Gat 9	425	160	0.90	0.37	0.9	4.5	2.5	2.0	0	100	17	3
GAT 21	780	500	1.7	1.95	0.28	1.9	0.67	0.33	0	100	25	3
GAT 24	520	440	1.0	0.39	0.76	3.1	1.7	1.1	0	100	39	15
Averages									0	100	22	2

* 15 % better than at the start of the recharge season.

Table 4 - Reduction of a resistance built-up during an injection season from $W_1 = (\phi_1-\phi_0)/Q_i$ = 100 % to $W_2 = \phi_1/Q_p-\phi_o/Q_i$ immediately after starting the pump or to $W_3 = \phi_3/Q_p-\phi_o/Q_i$ at the end of the pumping season for 4 dual-purpose wells in Israel. (Anon. 1969: Underground Water Storage Study Israel, Final Report, FAO, Rome 1969). Q_i = injection flow, Q_p = abstraction flow, ϕ = head in well relative to the value that would have been measured if clogging were absent (new well), ϕ_∞ = new well, 0 is start of recharge season, 1 = end of recharge season, 2 = some minutes after start of abstraction season, 3 = end of abstraction season. V_i, V_p are the total water volume injected and the volume pumped up again in the respective seasons.

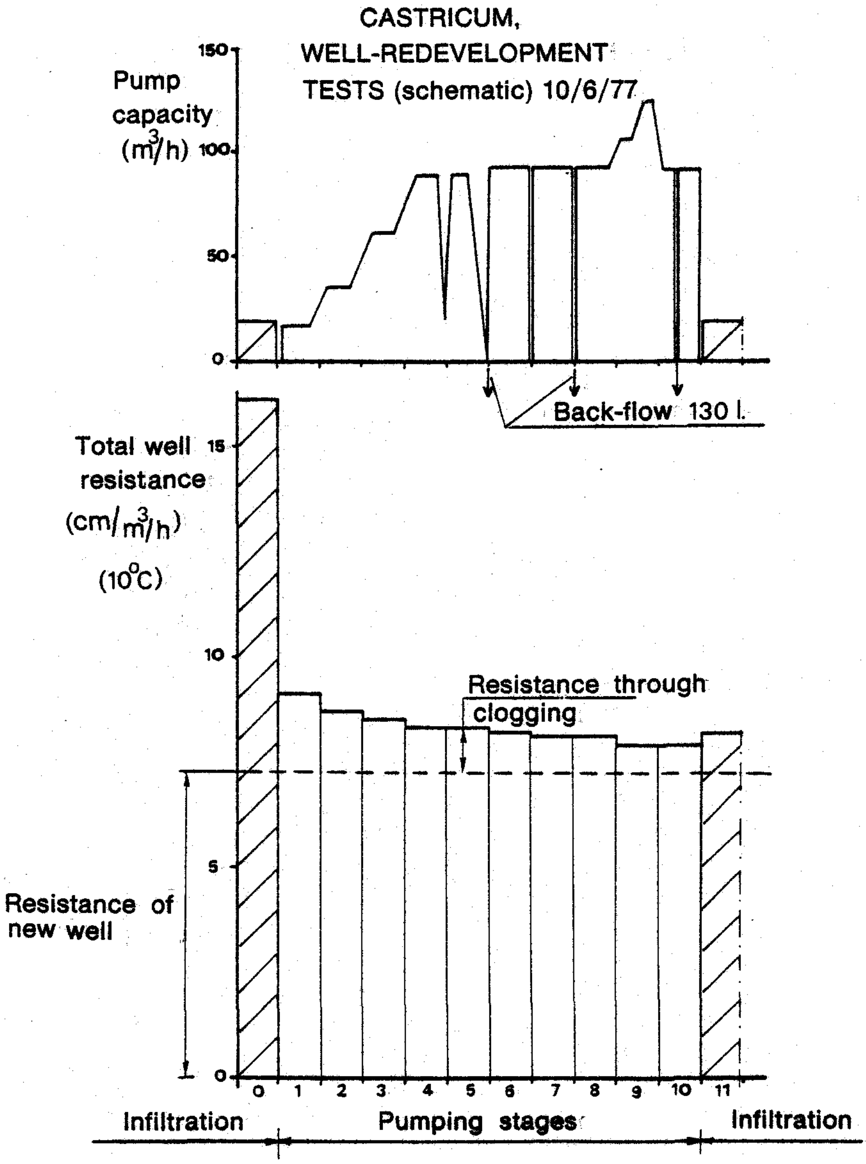


Figure 27 - The variation of resistance in the injection well at Castricum (10/6/1977, see also appendix A) shows that the bulk of improvement is brought about by the first time the pump is switched on, even when the pump has a low capacity. Even a considerable increase of pump capacity brings little further improvement.

The flushing-pump delivery as such has little influence inasmuch as a fivefold increase of capacity may perhaps increase the removal of the clogging resistance from 75 % to 85 % (figure 27). In all cases, even with high pump capacities, the flowrate of the water at the bore wall is limited to a few metres per hour, insufficient for thorough cleaning.

Intermittent pumping, that is with repeated switching on and off of the pump, appears to have almost no result in practice (figure 25, figure 28). The supposed pulsing action does not occur because every time the pump starts, its delivery then approaches its end value asymptotically. Good results are obtained with juttering, i.e. repeated reversal of flow direction by repeated injection and pumping. Juttering appears to be an admirable method of mechanical cleaning, capable of bringing down the resistance, in small steps, to very low values (figures 25 and 28).

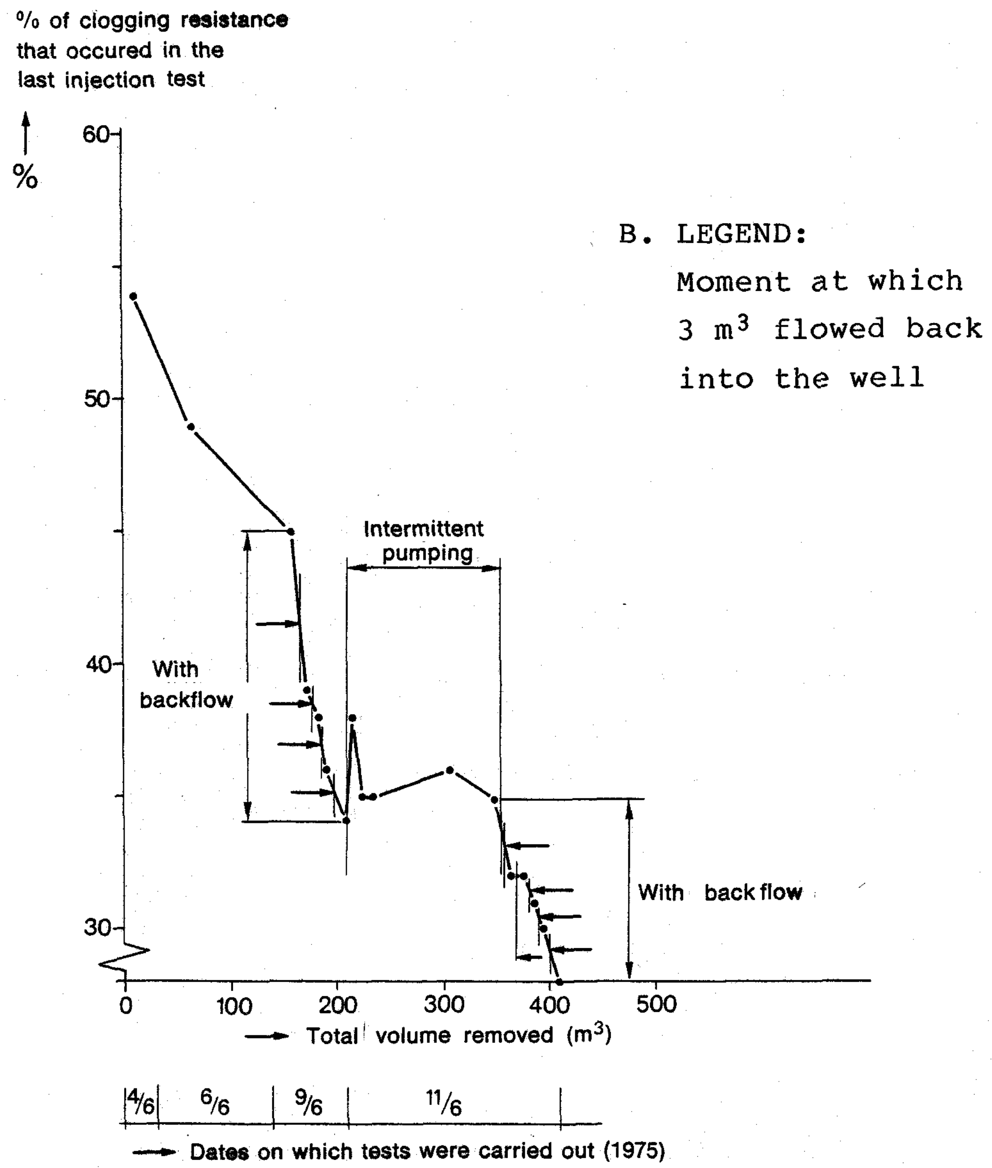


Figure 28 - Effect of flow reversal and purely intermittent pumping on clogging of the second injection well belonging to the Dune Waterworks of The Hague (see appendix A)

4.2.3 Juttering with compressed air

Compressed air lends itself admirably for the redevelopment of injection wells. The blowing in of compressed air at a certain depth creates a mammoth pump (also referred to as "airlift") which, given good design, can have a high capacity (Rautenberg 1972). If the well head is closed (figure 29) the compressed air will force the water level in the well downwards and so cause infiltration for a short time. By then opening the quick-acting valve to discharge, a sudden drop of pressure will be caused, with a rapid rise of the liquid level in the well as a result, attended by an extraction flow for several seconds, for which an extremely large submersible pump would normally be required. When this injection of air is continued, the loosened material is immediately removed. This repeated process, that is pushing down the water level in the well with compressed air, then letting it shoot upwards, followed by continued air-lift pumping we call "compressed-air juttering". The water in the formation can also be moved to and fro with the compressed air, which is desirable when using chemicals.

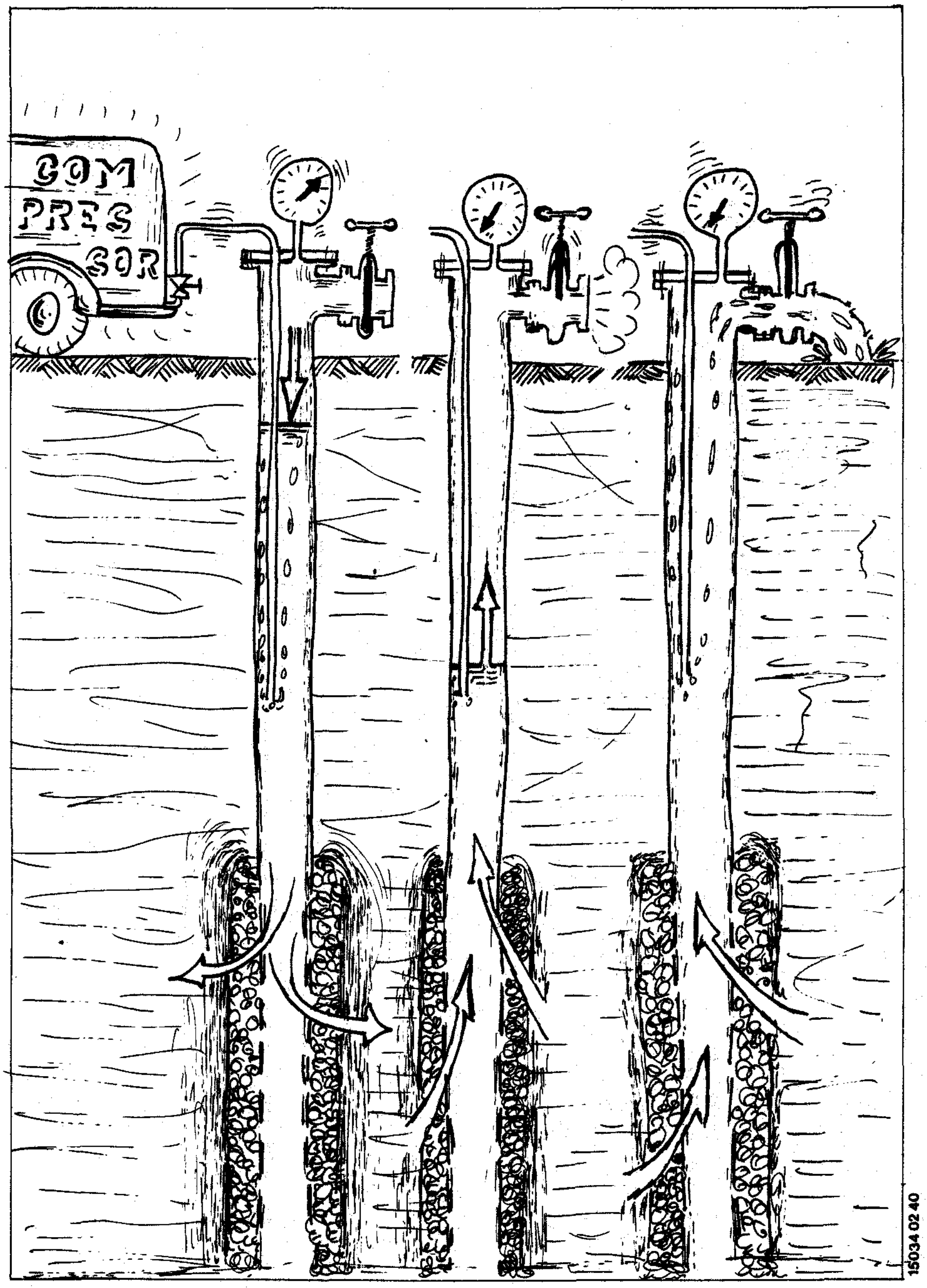


Figure 29 - Juttering with compressed air.

Simplicity, reliability and low capital costs more than make up for the rather low energetic efficiency of the mammoth pump (30 to 45 %). It is thus worthwhile when designing injection wells to allow for juttering with compressed air (chapter 6). Mammoth-pump design has been exhaustively treated elsewhere (Rautenberg 1972, Olsthoorn 1978).

The brief delivery brought about by the rapidly rising water level after opening of the quick-acting valve at the well head, can be approximated as follows (Kruseman & De Ridder 1970, Olsthoorn 1982):

$$Q = \Delta\phi/[W_c + \frac{1}{4\pi kH} \ln (\frac{2.25\ kHt}{Sr_o^2})] \qquad (4.1)$$

where Q is the flowrate (m^3/s), k the soil permeability (m/s), H the length of the gravel pack (m), S the storage coefficient (dimensionless), r_o the radius of the borehole (m) and $\Delta\phi$ the maximum-dowward expulsion of the liquid level in the well (m). W_c is the clogging resistance of the well ($m/(m^3/s)$). Given the values found valid in an experiment with a test well of the Municipel Water Works of Amsterdam: $\Delta\phi$ = 20 m, W_c = 0.5 $m/(30\ m^3/h)$ or 60 $m/(m^3/s)$, k = 20 m/d (0.00023 m/s), H = 15 m and r_o = 0.3 m, we obtain with (equation 4.1):

t (s)	0.1	0.2	0.3	0.5	1
Q (m^3/h)	660	570	530	490	440

Since the forces of inertia predominate for the first 0.2 seconds and after about 0.5 seconds the

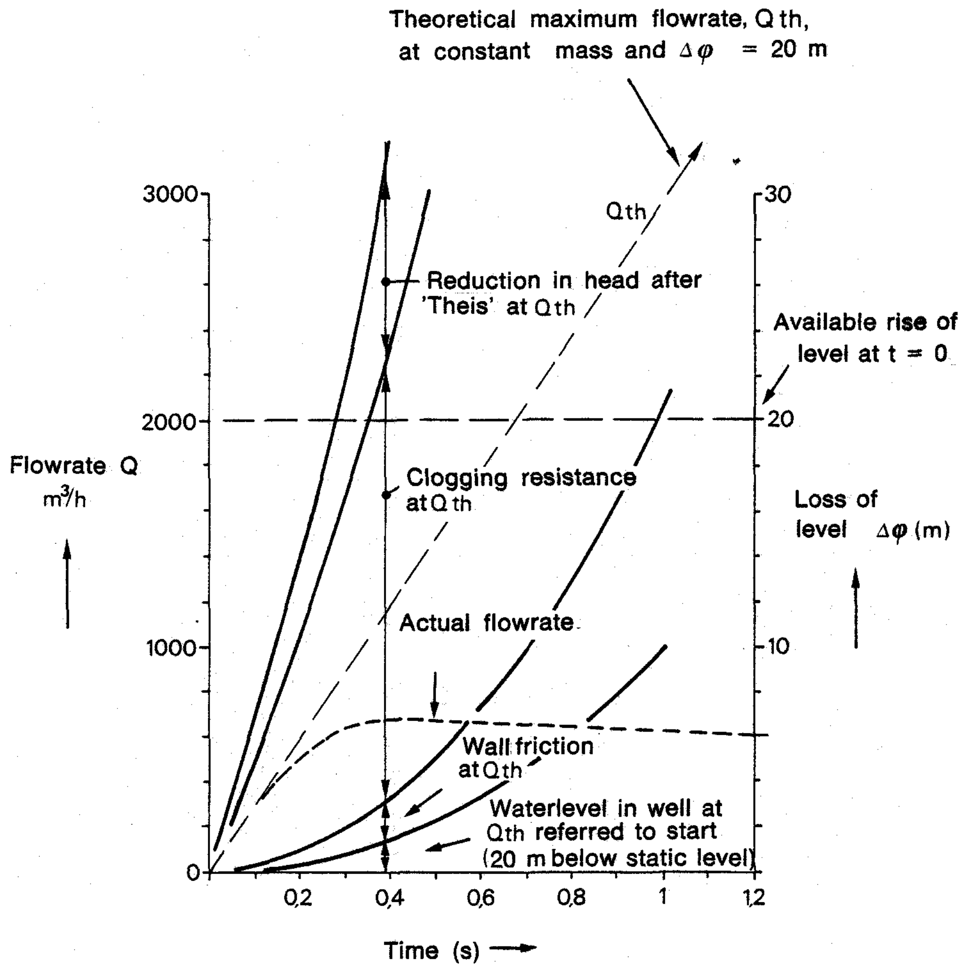


Figure 30 - Flowrate and water-level distribution as a function of time after opening of quick-acting valve after the water column has previously been forced 20 metres downwards by compressed air. Amsterdam Municipal Water Works well, see appendix A.

rise of liquid level has attained an appreciable magnitude (figure 30) the formula holds good altogether for 0.3 to 0.5 seconds after opening of the quick-acting valve. In this example the maximum flowrate amounts to about 500 m^3/h.

The magnitude of the maximum flowrate depends closely on the value of W_c. In a seriously clogged well the maximum flowrate may thus not be as high as expected at first but will then increase as redevelopment proceeds (decreasing W_c).
It is of course advisable not to delay redevelopment until the well has become badly clogged.

4.2.4 Sectional pumping

A sectional apparatus (figure 31) is a means to secure a high water-flow velocity locally, that is in the gravel pack, at a limited power impact.
However, due to the strong short-circuiting via the pack, hardly any of the desired high velocity remains at and behind the bore wall (Ellenberger & Aseltine 1973). This means that sectional pumping is only of value when the pack is substantially clogged. Such kind of clogging is to be expected with excessive propagation of bacteria in the well, but seldom occurs with other forms of clogging. In the case of chemical redevelopment a sectional apparatus can be used for accurate dosage of the chemicals.

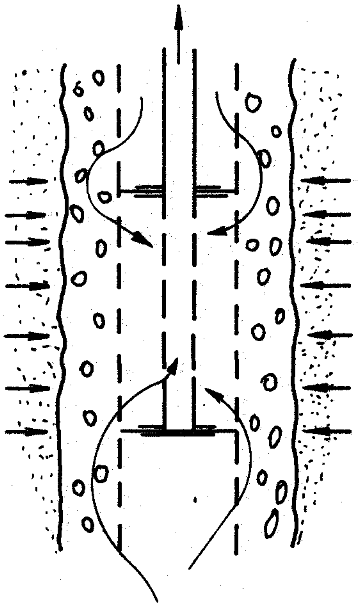


Figure 31 - Short-circuiting during sectional flushing pumping

Sectional pumping seldom pays, firstly because of its laboriousness and secondly due to the aforementioned short-circuit flow through the gravel pack.

4.2.5 High-pressure jetting

Jetting means removing dirt with a powerful water jet, as illustrated by way of example in figure 32.

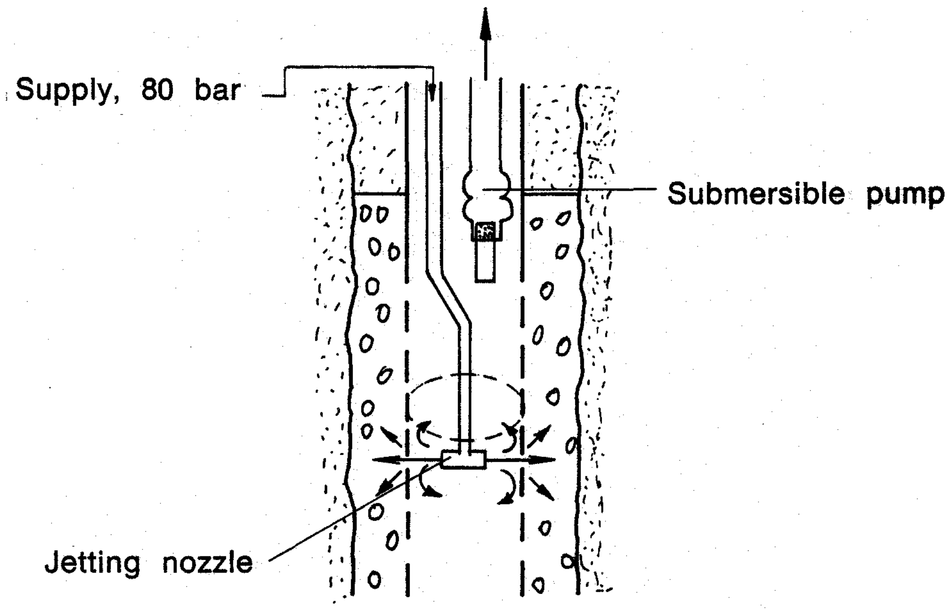


Figure 32 - Jetting (schematic)

The material removed is discharged immediately from the well by simultaneous pumping. Despite the high pressure applied - some 80 bars - the jet does not penetrate deeper than a few centimetres into the gravel pack. For deeper fouling, as mostly encountered in injection wells, the method has not much to offer, but it does lend itself well to the cleaning of wells which are fouled internally, that is where screen slots have been clogged.
This form of clogging is liable to occur in extraction wells with an injection system. The method has also been successfully used for accurate forced injection of chemicals.

4.2.6 Surging and bailing

Surging is the to and fro movement of the water in the formation, induced by the reciprocating motion of a piston in the riser or the filter screen.

If the piston be replaced by a bailer with clack valve, the reciprocating movement will, at the same time, have a pumping effect, whereby material, loosened during bailing, will be removed.
It goes without saying that bailing is better than surging, but it has no advantages over the compressed-air method, which can be carried out without any special apparatus or dismantling of the well head.

4.2.7 Brushing

Brushing is suitable only for cleaning the interior of risers and well screens. Whether it is of any use for injection wells is highly doubtful.

4.3 Chemical methods of redevelopment

4.3.1 Generalities

In some cases, part of the clogging material is so firmly adherent that mechanical means seem incapable to remove it. Mechanical methods can then be reinforced by chemicals, introduced into the well before the start of the mechanical redevelopment.

The following chemicals are important in this connection:

a. chlorine (as gas) and chlorine-containing agents such as chlorine-bleaching liquor (liquid) and calcium hypochlorite (grains);
b. acid, especially hydrochloric acid (obtainable as a liquid in carboys) and sulphamic acid (HSO_3-NH_2, grains);
c. polyphosphates (grains).

4.3.2 Chlorine

Chlorine and chlorine-containing agents are used to burn off and break up organic deposits (slime) and also for the purpose of killing bacteria. The concentrations applied amount to several hundreds or sometimes thousands of milligrammes free chlorine per litre.
The amount of chlorine water must be sufficient for the gravel pack plus 2 to 5 decimetres beyond it. If the clogging is also caused by inorganic components, it will be advisable to give the well a double treatment, namely first with chlorine and then with acid (Crowe 1968, figure 33). In this way the encapsulating organic matter is removed first, leaving the inorganic particles more easily accessible to the acid treatment.

Chlorine is a particulary dangerous gas (lung oedema) and the appropriate safety regulations must therefore be observed. It is preferably introduced by injecting it into the flow of recharge water. Chlorine bleaching liquor and dissolved calcium hypochlorite can be introduced in the same way, but this is often done through a thin pipe to the bottom of the well. This pipe is subsequently pulled up bit by bit till the screen has been supplied over its whole height. The chemicals are then driven into the gravel pack and formation with a certain amount of water. Sometimes the chemicals are simply poured into the well. In such case it is uncertain exactly how far they reach and the method is therefore not recommended. However, it is not clear wether the redevelopment is appreciably reduced by the simple pouring method.

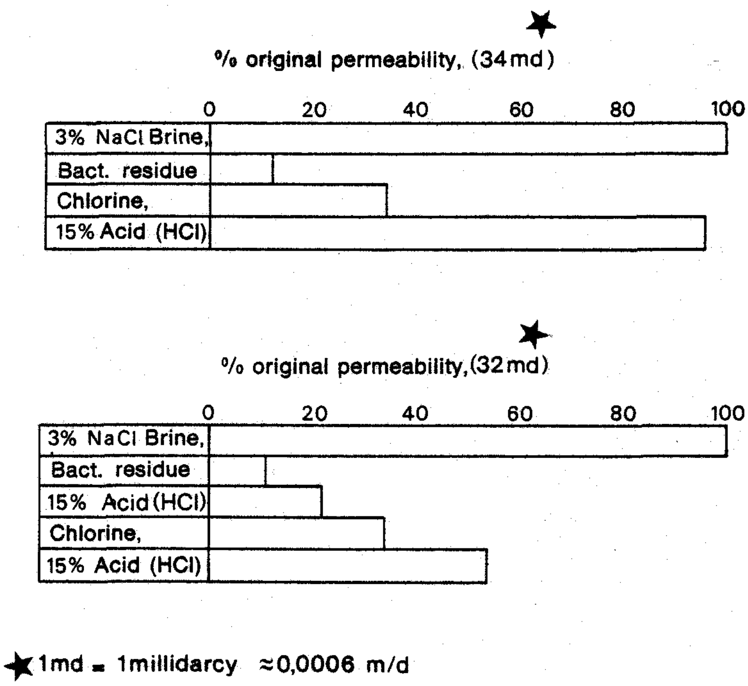


Figure 33 - Redevelopment of injection wells. The difference between "first chlorine, then acid" and "first acid, then chlorine" (after Crowe, 1968)

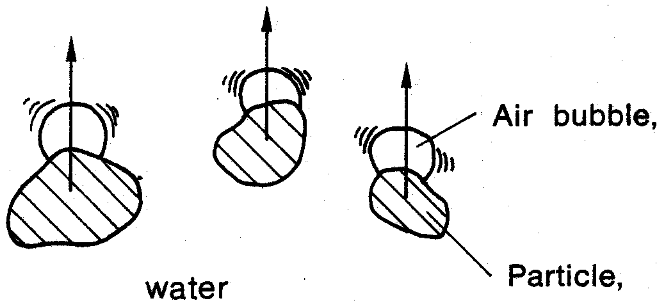
4.3.3 Acid

Acid is used chiefly for removing deposits of iron and aluminium hydroxides, originating from the coagulant employed, which may have passed through the filters to some extent. Of course, other soluble clogging materials can also be removed with acid. Sulphamic acid is obtainable in granular form and is therefore safer to transport and use than the hydrochloric acid supplied in carboys. But sulphamic acid is weaker and more expensive than hydrochloric acid (Schafer 1974).

The acid concentration will preferably be high enough to ensure that the pH of the acid water introduced is equal to or less than zero (Olsthoorn 1977). Introduction can be effected in the same way as for chlorine-containing agents and here too the quantity must be sufficient for a depth of 20 to 50 cm outside the borehole wall. This limited redevelopment depth is sufficient in fine sandy formations and for infiltration rates up to several metres per hour at the bore wall, appreciably limiting the total consumption of chemicals.

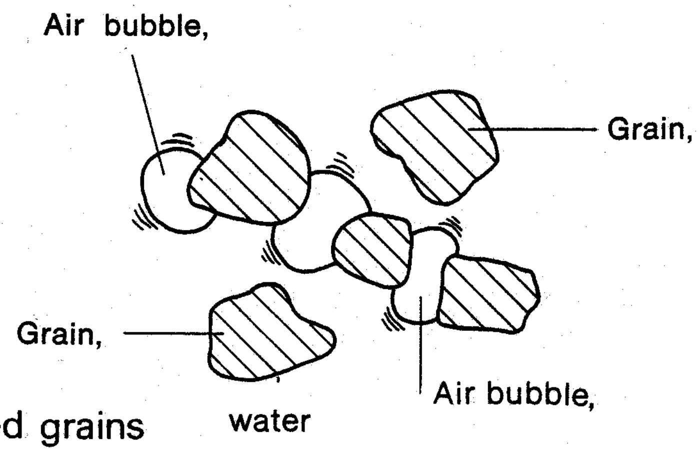
In formations containing lime, redevelopment with concentrated acid generates an enormous amount of carbon-dioxide bubbles making the pumped-up water bubbling as if boiling. Presumably because of the high surface energy (surface tension) of the formation grains pickled clean by the acid, the gas bubbles cannot adhere to them (complete grain wetting, figure 34) and they proved to be readily removed in practice without any residual clogging (Olsthoorn 1982); which is in contradiction to air bubbles (without acid) that got into the soil (Sniegocki 1963).
It is just as important to observe safety measures when working with acid as with chorine and chlorine-containing agents. Chlorine-containing agents must never be mixed with acid as this would generate chlorine gas!

Fig. 34a



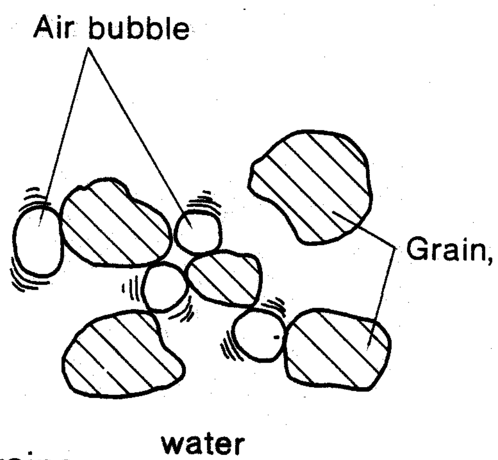
Flotation

Fig. 34b



Poorly wetted grains

Fig. 34c



Completely wetted grains

Figure 34 - Grain wetting with air or gas in the formation

The heavy, odourless carbon dioxide liberated by acid treatment can, for instance, collect in a well shaft with, obviously, lethal consequences for anyone who gets in (and for their rescuers). In formations containing, for instance, sulphur (pyrite), acid treatment liberates hydrogen sulphide which is very poisonous.

Should the well, the supply lines or the screen include metals be liable to be attacked by the chemicals employed, it is recommended to use inhibitors, substances that protect the metal by forming a skin.

Sometimes the action of the chemical can be improved by adding a wetting agent (Gawalek 1962, Osipow 1962, Schafer 1974). However, in practice this seems superfluous.

4.3.4 Polyphosphates

Polyphosphates are introduced for removing clay particles, sludge, silt and air bubbles (Kleber 1959, Sniegocki & Brown 1970). Polyphosphates are also known as "glassy phosphates", sodium hexametaphosphates and under the tradenames of for instance "Polyphos" and "Calgon", the latter derived from the phrase "calcium gone". Polyphosphates work as follows (Lyons 1973, Toy 1976):

a. Softening by combining high-valent metal ions (such as Mg^{2+} and Ca^{2+}) in dissolved complexes (sequestering);
b. Dispersion of clay, sludge, iron hydroxide and manganese-hydroxide particles by strong adsorption of dissolved polyphosphates;
c. Preventing formation of precipitates by interfering with forming crystals (threshold);
d. Improves the action of soap and detergents by

lowering the critical micelle concentration, and, finally;

e. Breaks down and emulsifies fats and alkanes through the high pH resulting from the lye addes in the production process (hydrolysis).

Just as in washing with soap, polyphosphates only act when the water is sufficiently agitated. This can be achieved quite simply with the compressed-air technique. The treatment must often be repeated several times to get the maximum effect (Schafer 1974, Sniegocki 1963).

Presumably due to the high surface energy, acquired by the soil grains through adsorption of the polyphosphates, complete grain wetting results (figure 34), so that any air bubbles can be easily removed (this is also the case after treatment with concentrated acid). This explains the succes of the use of polyphosphates to remove clogging air bubbles from the formation - as was reported by Sniegocki, 1963 - as well as the easy removal of carbon-dioxide bubbles after redevelopment with strong hydrochloric acid in formations containing lime - as reported by Olsthoorn, 1982. To realise complete wetting, agitation is required, for instance by compressed air juttering (chapter 4).

The optimum poly-phosphates concentration is not known. In general endeavour is made to dissolve as much polyphosphates as possible. This dissolution proceeds with difficulty, owing to the limited solubility and the glassy skin that forms on the grains on coming into contact with water. In some cases the polyphosphates are dissolved separately in hot water with continuous stirring. Then again,

use is often made of a perforated container in which the polyphosphates are moved continuously up and down in the riser.
Preference should be given to separate dissolution and introduction as for the chemicals previously discussed.

Since clay and sludge particles can penetrate further into the formation thanks to their negative charge, the treatment depth should be greater than with chlorine and acid. Polyphosphates are particularly indicated for removing such particles. A depth of 50 to 100 cm will generally suffice. Only when the dispersion of clay minerals plays a part, must the treatment depth perhaps be increased somewhat, say to 150 cm ouside the wells, assuming beforehand that flow is still possible after such a clogging.

In almost every case, calcium hypochlorite or chlorine bleaching liquor is added to the polyphosphates in order to remove any organic matter that may be present simultaneously. Because of this combined use of polyphosphates and chlorine, it is impossible to judge from the information yet available to what extent a given redevelopment result can be attributed to polyphosphates alone.

Agitation, essential in polyphosphate treatment, is always desirable when using chemicals. Besides the compressed-air method a sectional apparatus or high-pressure jetting nozzle can be used for this purpose and will incidentally also allow accurate dosage of the chemical.

4.3.5 Which chemicals?

Which chemicals to add is decided by the nature of the clogging material. Should this, as is so often the case, be a mixture of organic and inorganic components, several different treatments may be needed, e.g. polyphosphates plus chlorine, followed by a seperate treatment with acid. Should excessive bacterial growth be the main trouble, then, of course, chlorine treatment will suffice.
If the clogging is chiefly a result of coagulant penetration, then acid treatment should suffice. Air bubbles can be removed with acid or polyphosphates, provided sufficient agitation is applied. Dispersion of clay minerals and clogging by clay and sludge particles may require a polyphosphate treatment.

The question as to conclude what is the cause of clogging in any given situation, can only be resolved indirectly from the various indications such as the course of clogging build-up (figure 35), origin and composition of the injection water, prepurification method employed, suspensoids in the injection water and in the water pumped up when flush pumping, from clay-dispersion tests and throughflow tests with undisturbed soil samples, from analyses of soil samples, throughflow tests with other materials and other suchlike measures (see also chapter 3).

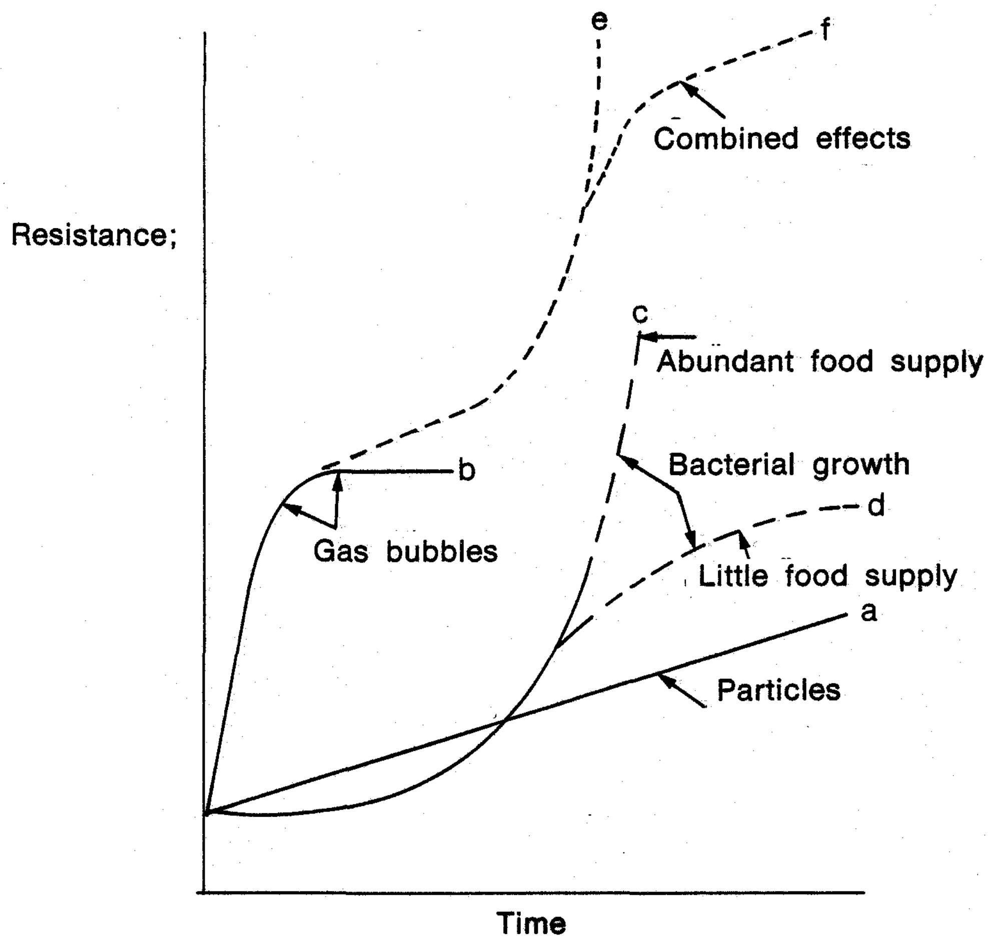


Figure 35 - Typical clogging history for suspended matter alone (a), gas or air bubbles (b), bacterial growth with large food supply (c) and with a limited food supply (d). In practice various forms can occur simultaneously (e) and (f).

5 MASTER DESIGN

By systematically considering the costs of prepurification, sinking more wells, quicker replacement of those wells and costs of redeveloping the wells, it is possible to minimize costs.

The one form of clogging that cannot in practice always be averted is that caused by suspended matter. Since in the case of suspended matter the rapidity of pressure rise in the well becomes four times less when the infiltration rate is halved (figure 7), optimization of the abovementioned parameters is an attractive way of minimizing the total cost per m^3 injected water.

Many injection-well systems have a flushing-pumping system permanently installed. The capital and running costs of this installation are then accounted under the heading of well costs and maintenance and need not be reckoned seperately as redevelopment costs. Redevelopment costs are thus limited to the costs incurred by more intensive cleaning of the well; that is a well-cleaning action for which a specially equipped redevelopment team comes into action (figure 36). Where a fixed flushing-pump installation is installed, clogging then means only that part of the resistance which remains after use of the said fixed pumping installation ($v_2 \Delta t$ in figure 36).

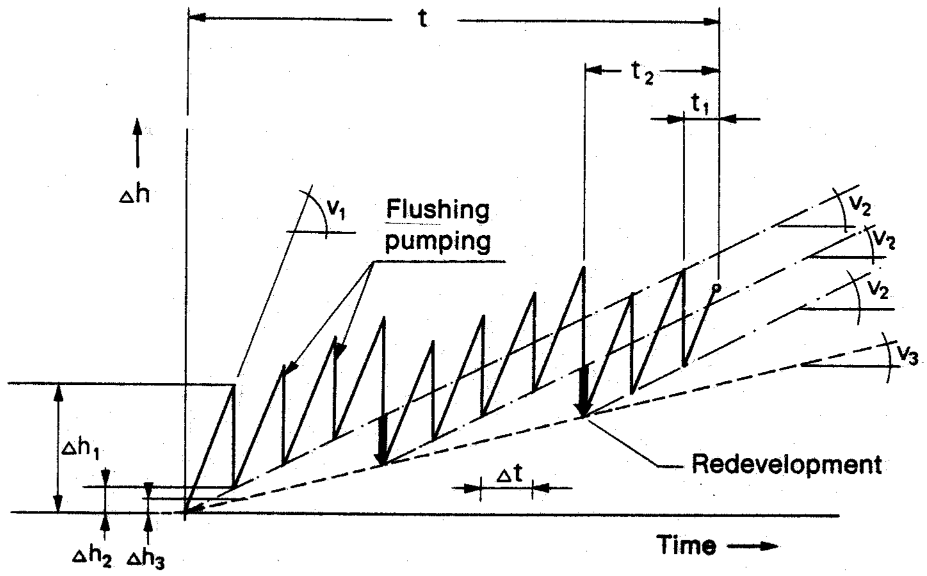


Figure 36 - Schematic varation of waterhead in an injection well as a result of clogging, flushing pumping and redevelopment. The time scale can be varied at option by choice of injection rate and water quality.

The basis of optimization is a choice of a redevelopment criterion, i.e. the signal for the redevelopment team to come into action. Better than arrival at the maximum admissible injection pressure (see chapter 2) is it to take a certain increase of resistance since the previous redevelopment as the criterion. The speed with which this criterion is reached will decide the redevelopment frequency.

To obtain the data required for design, preliminary investigations are necessary. This involves well tests within the selected formation to ascertain how fast the chosen redevelopment criterion is reached with the types of water envisaged.

This will give the redevelopment frequency, rf_o ($year^{-1}$), for the type of water tested, the wells used (size (A_o (m^2)), the given formation and given flowrate (Q_o (m^3/h)). This development frequency is thus a direct function of the purification costs. The result of an extensive investigation may look as follows (table 5).

Test prepurification		Redevelopment frequency rf_o ($year^{-1}$)	Purification costs (ref. Anon. 1978) (fl/m^3)*
1	Chlorination	52	0.02
2	BCL+SF	12	0.05
3	BCL+C+SF	4	0.11
4	BCL+C+SF+SC+SF	0.5	0.25
5	BCL+C+SF+AK+HF	0.1	0.50
* fl = Dutch guilders ≈ 0.4 American dollars.			

Table 5 - A possible result of investigations, using test wells with infiltration surface A_o and fixed flowrate Q_o (BCL + break-point chlorination, SF = rapid sand filtration, C = coagulation, SC = secondary coagulation, AK = activated carbon filtrarion, HF = hyperfiltration).

If the subsequent operational wells have another flowrate Q and size A, equation 3.19 will enable the respective redevelopment frequency rf to be obtained directly from the value of rf_o given by the tests:

$$rf = \left(\frac{v}{v_o}\right)^2 rf_o, \text{ where } v = Q/A \text{ and } \tag{5.1}$$

$$v_o = Q_o/A_o,$$

are the respective infiltration rates at the borehole wall. $A = 2\pi rH$ and $A_o = 2\pi r_o H_o$, where $2r, 2r_o$ are the borehole diameters and H, H_o the heights of the gravel packs of the operational and test wells respectively.

For the injection-well system as a whole, the purification costs are K_z (\$/year) for a total flow Q_t(m^3/a) and a price per m^3 depending on rf_o.

The well costs on an annual basis, including buried pipelines and possible fixed flushing-pump installation and suchlike, amount for a total of m wells at k_p (\$/well) to:

$$K_p = n\ k_p\ r^* \tag{5.2}$$

where r^*, the annuity, depends on the rate of interest re (fraction/year) and the repayment term T (years):

$$r^* = \frac{re\ \exp(re\ T)}{(\exp(re\ T)-1)} \tag{5.3}$$

T is taken as equal to the life time of the wells. Given a total number of possible redevelopments of N per well and a redevelopment frequency rf ($year^{-1}$), the wells will last for N/rf years.

Finally, the redevelopment costs K_r (\$/year) amount at a rate of k_r per well and per redevelopment (\$/redevelopment) to:

$$K_r = n\ rf\ k_r \tag{5.4}$$

The total cost $K_t = K_z + K_p + K_r$ must be minimized. Where:

$$n = Q_t/Q \text{ and } Q = Q_o(\frac{A}{A_o})\ \sqrt{(\frac{rf}{rf_o})} \tag{5.5}$$

Dividing by Q_t now gives us the following formula for the total cost, k_T, per m^3 injected water:

$$k_t = k_z + \frac{1}{A}\left[(\frac{A_o}{Q_o})\ \sqrt{(\frac{rf_o}{rf})}\ k_p \left\{\frac{re\ \exp(re\ N/rf)}{\exp(re\ N/rf)-1}\right\} + \right.$$

$$\left. + \frac{1}{A}\ (\frac{A_o}{Q_o})\ \sqrt{(rf\ rf_o)}\ k_r\ \right] \tag{5.6}$$

where A and rf are the independent variables.

Optimization with respect to well size A means choosing a drilling system that will give the lowest cost per m^2 of infiltration surface (see also chapter 6), i.e. so that k_p/A is a minimum, while at the same time taking care to ensure that the construction method so chosen does not produce wells that are more difficult to redevelope, leading to a considerable rise of k_r in consequence.

Unless wells of special type (e.g. Ranney wells) or of exceptionally large size (several metres diameter) are used, there is no need to fear a sharply rising k_r (see also chapter 6).

It now remains to determine the optimum redevelopment frequency rf. Zeroing the partial derivative of k_T with respect to rf gives, after the necessary transformations, the following general relation between the dimensionless terms $k_r N/k_p$ and $u = re\ T = re\ N/rf$:

$$\frac{k_r N}{k_p} = \left\{\frac{u\ \exp(u)}{\exp(u)-1}\right\} * \left\{1 + \frac{2u}{1-\exp(u)}\right\} \tag{5.7}$$

This relation (see figure 37) can be approximated by:

$k_r N/k_p < 1$:

$$\rightarrow rf \approx r_o N \left\{0.78 \exp\left(-0.53\ k_r N/k_p\right)\right\} \tag{5.8}$$

$k_r N/k_p > 1$:

$$\rightarrow rf \approx r_o N \left\{0.56 \exp\left(-0.23\ k_r N/k_p\right)\right\} \tag{5.9}$$

The optimum frequency, rf, thus rises as the rate of interest, re, increases and the construction costs of a well, k_p, rise in relation to the redevelopment costs, k_r. Since the N before the exponent is in all practical cases (k_r/k_p of the order of 10^{-2}) much more important than the N in the argument, the optimum rf will at the same time increase with the redevelopability of the wells.
That together with all this, the repayment term T remains within reasonable limits is illustrated hereunder for $k_r/k_p = 0.02$ and re = 0.1:

N (number)	1	2	5	10	20	50	100	200
rf(year^{-1})	0.077	0.152	0.37	0.70	1.26	2.2	3.5	4.5
T (year)	13.0	13.2	13.5	14.3	15.8	22.7	28.5	44.4

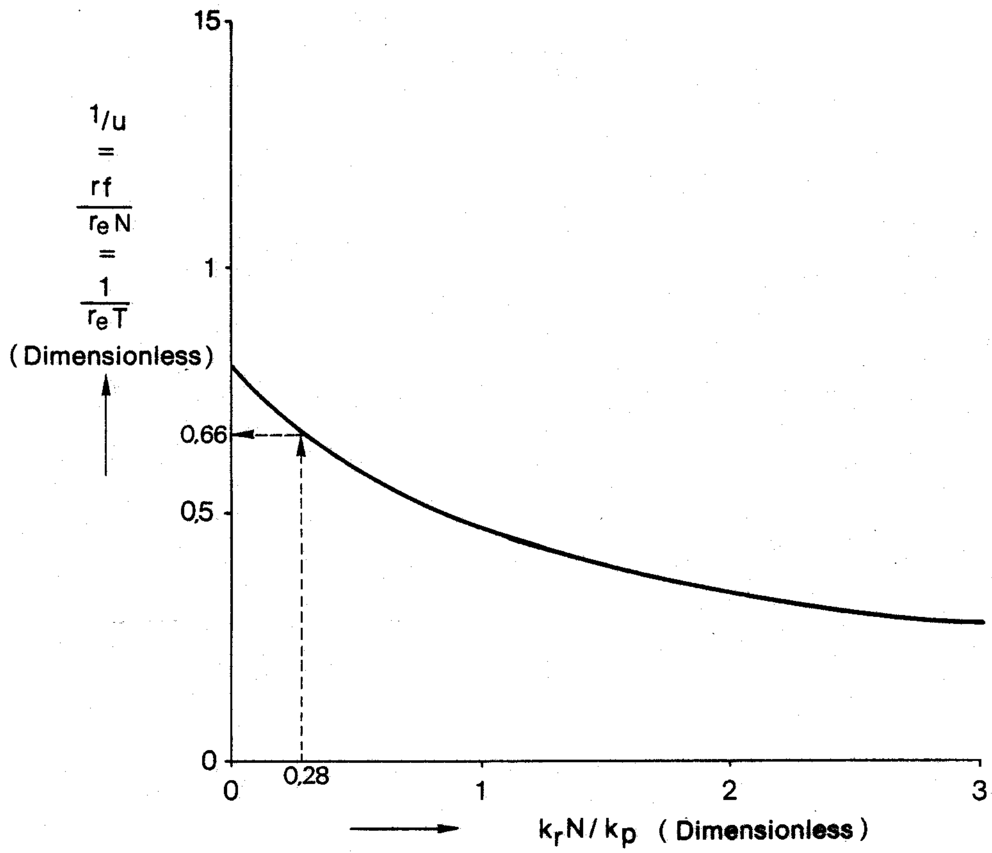


Figure 37 - Relation between the dimensionless terms rf/reN and $k_r N/k_p$; rf = optimum redevelopment frequency (year^{-1}), re = interest rate (% per year/100), N = maximum number of redevelopments that the well can sustain, k_r = cost of a single redevelopment, k_p is the capital cost of 1 injection well, including finishing and enclosing and terrain conduits belonging to it.

To illustrate the method described we shall now discuss the design of an installation with a capacity of 30 million m^3 per year, assuming that the investigations yielded the results in table 5 for

test wells with Q_o = 50 m^3/h and that these test wells had already been optimised with respect to the ratio k_p/A (see chapter 6) so that the operating wells to be built will have the same infiltration surface A_o and the test wells will be incorporated into the subsequent system.
Including onsite pipelines and permanently installed pumping equipment, the cost of a well in this example will be f 80,000 ($ 32000). Each redevelopment costs f 1,500 ($ 600) and the rate of interest re amounts to 10 %. The wells can be redeveloped at least 15 times in the course of their existence (N = 15).

The optimum redevelopment frequency rf can now be found from figure 37, namely:

$$\frac{k_r N}{k_p} = \frac{(1500)\ (15)}{(80\ 000)} = 0.28 \rightarrow \frac{rf}{re\ N} = 0.66 \qquad (5.10)$$

so that

$rf = (0.66)(0.1)(15) = 1.0/\text{year}$, and

$T = (15)/(1.0) = 15$ years.

After fillng the figures into the formula for total coast per m^3, the result will be:

$$k_T = k_z + (0.030)\ \surd\ (rf_o) \quad (fl/m^3) \qquad (5.11)$$
$$= k_z + (0.012)\ \surd\ (rf_o) \quad (\$/m^3)$$

k_z is found from table 5 as a function of rf_o so that k_T can be represented as a function solely of the quality of injection water (rf_o).
This has been done in figure 38. The cheapest solution is here obtained for rf_o = 9/year, while the

well-tried purification system (chlorination plus rapid filtration) gives much the same result with rf_o = 12/year.

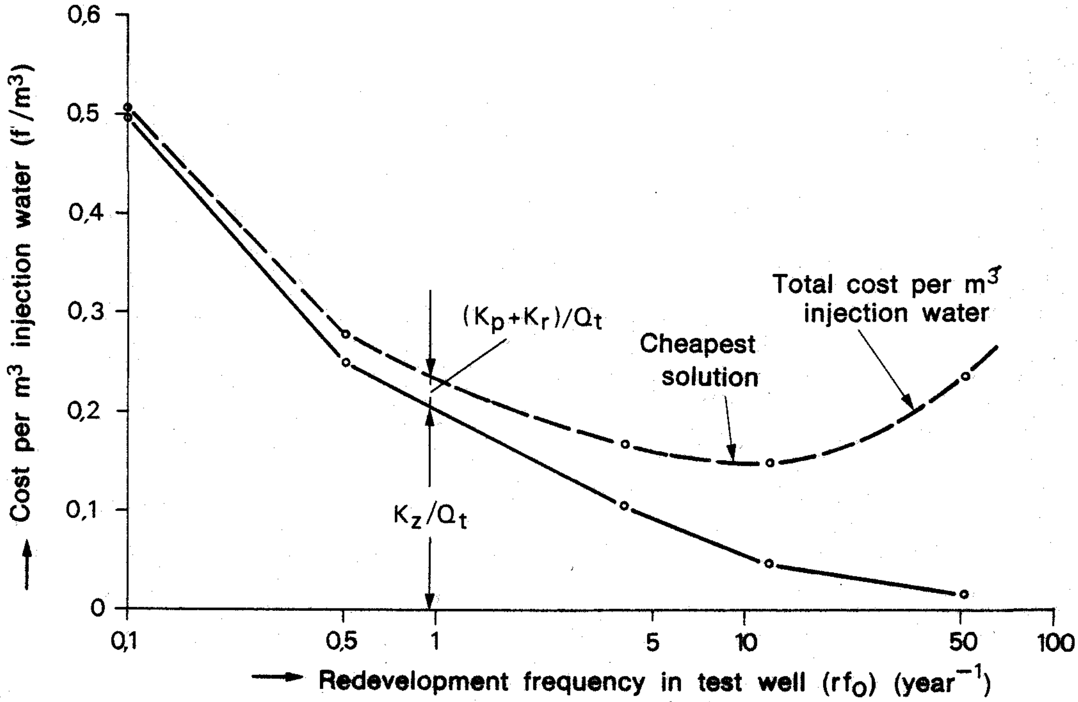


Figure 38 - Cost of prepurification per m³ (k_T) and total minimum injection cost per m³ $(k_z + k_p + k_r)/Q_T$ as a function of quality of injection water (rf_o).

However, values of N less than about 15 have a significant effect on the cost. 15 redevelopments per well is in practice achievable in most cases and dozens of redevelopments per well are often possible (see figure 39).

A redevelopment method must therefore generally guarantee N > 25 in order to assure minimum costs. Finally, figure 40 shows the effect of well cost k_p and redevelopment cost k_r on the optimum and the total costs.

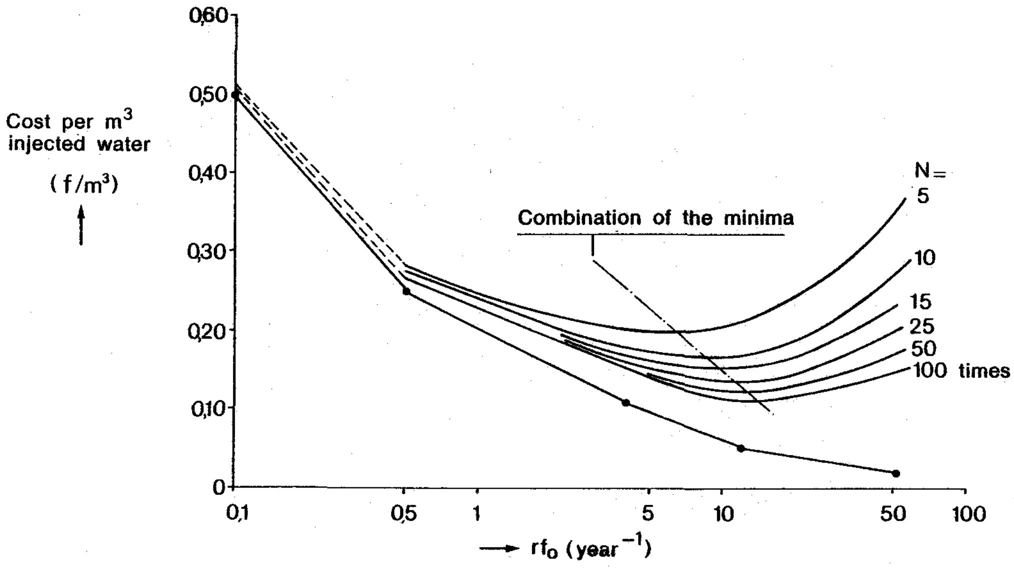


Figure 39 - The total minimum cost per m³ injected water for different values of the total possible number of redevelopments, N, that a well can sustain, as a function of water quality, rf_o, see figure 38.

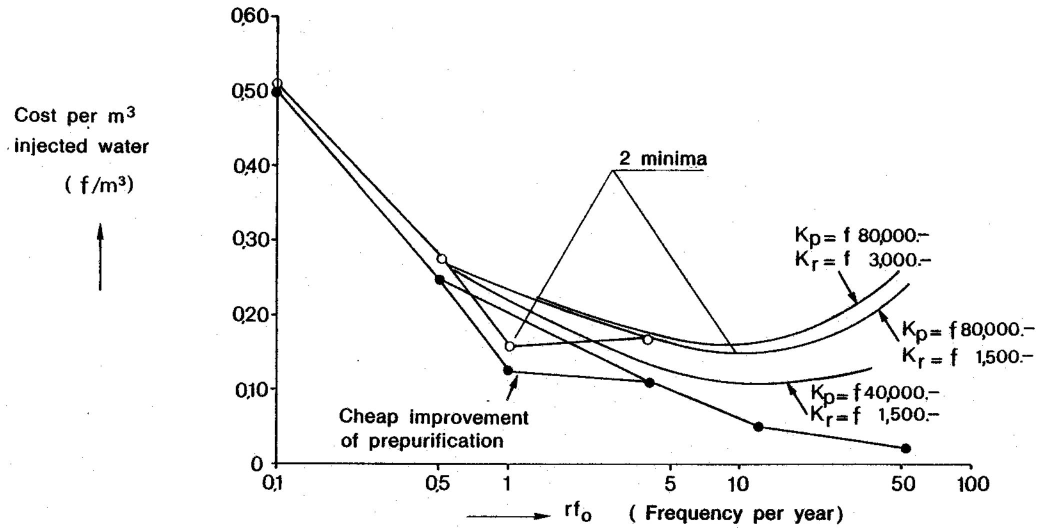


Figure 40 - The effect of a cheap improvement of prepurification, of halving the well cost and doubling the redevelopment cost on the cost per m^3 injected water, all in relation to the situation in figure 38.

This figure also shows the effect of a cheap improvement of prepurification. Compared to the solution already obtained, this appears here to offer just as expensive a solution, (i.e. more purification but fewer wells and redevelopment).

It is for the responsible authorities to decide on what shall be the final choice and in so doing they will, of course, take all other relevant factors into account. From the standpoint of protecting the soil, a more extensive prepurification will naturally be preferred.

6 WELL DESIGN

6.1 Economically

Using the methods discussed in the preceding chapter, a number of overall variables have been established prior to the design of the wells themselves and the variables concerning the individual wells must now be considered more fully.
The total required infiltration surface is one of the variables that has already been so fixed. For n wells of borehole diameter 2r and gravel pack length H, this surface area (A) will be:

$$A = n(2\pi r)H \tag{6.1}$$

In general, the construction costs of the requisite n wells can be described by the following formula:

$$K_p = n\ [K_o + \{K_1 + aH\}\{1 + b(\frac{r-r_o}{r_o})\}] \tag{6.2}$$

K_o represents the costs of supply and removal of the drilling equipment and suchlike, calculated per well, K_1 represents the drilling plus finishing costs up to the top of the gravel pack, a represents the drilling plus finishing cost per metre gravel pack length, H is the length of the gravel pack and b is a factor that allows for drilling to a diameter, 2r, other than the standard diameter, $2r_o$.

To minimise the drilling costs is a question of optimization where the increase in number of wells must be weighted against drilling deeper and drilling with another diameter. Both these alternatives are limited by the available thickness of the for

mation and feasibility of the various drilling techniques.
For instance, if A be 10,000 m^2 and choice be made to a first approximation of H = 15 m and $r=r_o=0.25$ m, then the number of injection wells required will be 424. In this example we also choose K_o = fl 10,000.- (\$ 4,000.-), K_1 = fl 10,000.- (\$ 4,000.-), a = fl 300.- (\$ 120.-) per metre and b = 0.5, so that drilling a hole twice as wide will cost fl 450.- (\$ 180.-) per metre.

With these figures every well will have an infiltration surface of 23.6 m^2 and costs fl 24,500.- (\$ 9,800.-). If the wells are sunk just so much deeper (dH) that requirements can be met with one well less, then:

$$2\pi r_o H = (n-1)2\pi r_o dH \tag{6.3}$$

so that

$$dH = H/(n-1), \tag{6.4}$$

and this will cost:

$$dK_p = \left(\frac{\partial K_p}{\partial H}\right) dH = a\frac{H}{n-1} = \text{fl } 4500,\text{- } (\$\ 1{,}800.\text{-}) \tag{6.5}$$

against which we have a saving of 1 well, i.e. fl 24,500.- (\$ 9,800.-).
If on the other hand the wells are sunk with a greater diameter, and likewise so that requirements can be met with 1 well less, we shall have:

$$2\pi r_o H = (n-1)2\pi(r-r_o)H, \text{ so that} \tag{6.6}$$

$$r - r_o = r_o/(n-1), \text{ and this will cost:} \tag{6.7}$$

$$dK_p = \left(\frac{\partial K_p}{\partial r}\right)_{n-1} dr = b\,(K_1 + aH) = \text{fl } 7{,}250.- \quad (\$\ 2{,}900.-) \tag{6.8}$$

In this example therefore to sink wells deeper is the best solution, to sink wells of larger diameter the next best, while increasing the number of wells should only be a last resort. In this way the necessary infiltration surface can be secured at minimum cost. Should the realisable filter length, H, be say 20 m and the maximum diameter of the drilling system employed 1 m, then, instead of the previously calculated 424 wells, only 160 will have to be sunk and the capital cost will be only fl 5,440,000.- ($ 2,176,000.-) instead of fl 10,388,000.- ($ 4,155,200.-).

6.2 Hydrologically

Since the admissible injection pressure is proportional to the depth of the top of the gravel pack (chapter 2) a reasonable distance below ground level will be maintained, say 10 m. The admissible water head in the well will then be 2 m above groundlevel so that, allowing a head build-up of 2 m for possible clogging, the design-head ceiling in the well will be at ground level. In this connection, the low viscosity of winter-surface water must also be allowed for if necessary. More wells (but then perhaps of smaller diameter) and/or at greater distances between them may perhaps be needed to satisfy this boundary condition.

If the injection wells are evenly distributed over a length, L, of say 5000 m and are flanked on either side at a distance of 200 m by a series of abstraction wells, there should normally be no effect on the groundwater levels outside these rows of abstraction wells. The maximum rise of water head in the (n) clean wells, spaced b = L/n apart at a total infiltration flowrate, Q, of say 25 million m^3/year, a formation transmissivity, k times H, of e.g. of 1000 m^2/d at 10 °C water temperature, will then be (Huisman, 1972):

$$\Delta\phi = \left(\frac{\mu_o}{\mu_{10}}\right) \left\{\frac{1}{2}\left(\frac{Q}{L}\right)\left(\frac{1}{kH}\right) + \frac{(Q/n)}{2\pi kH} \ln\left(\frac{b}{2\pi r_o}\right)\right\} \qquad (6.9)$$

Since:

$$\frac{b}{2\pi r_o} = \frac{LH}{n2\pi r_o H} = \frac{LH}{A} = \text{constant and} \qquad (6.10)$$

amounts to: $\frac{(5000)(20)}{(10000)} = 10$, so that in this example with $\frac{\mu_o}{\mu_{10}} = 1.37$:

$\Delta\phi = (1.37) \{1.37) + (25.1)/n\} = 1.9 + 34/n$

Although n can be calculated accurately from this relation, it is immediately clear that with a large number of wells, say n > 100, only the constant term, here 1.9 m, still plays a part.

In order to prevent waterlevels rising above groundlevel when operating with clean wells, the natural groundwater level in winter must therefore not be higher than about 2 m below ground level (in wells with the top of the gravel pack at a deeper level a higher injection head may be applied).

6.3 Hydraulically

Should it be decided to redevelope with compressed air, the well must be adapted for this, chiefly by choice of diameter of riser and screen pipe. Figure 41 illustrates this based on the relation between the flow of water pumped up and the air flow which this requires, both for several values of well resistance $W = \frac{\Delta\phi}{Q}$ [m/(m³/h)] which, with R = 1000 m, in the present example amounts for clean wells to about 0.03:

$$\frac{\Delta\phi}{Q} \simeq \frac{1}{2\pi kH}\ln\left(\frac{R}{r_o}\right) = \frac{(24)}{(2)\pi(1000)}\ln\left(\frac{1000}{0.5}\right) = 0.029 \qquad (6.11)$$

the dimension of which is $(mH_2O/(m^3/h))$.

The magnitude of the transmissivity, kH, and the clogging, W_c, have a very decisive effect on the water flow obtained, as also has the air-injection depth (which should be as large as possible) and the efflux loss at the well head (which must be kept as small as possible).

In this example the large riser diameter brings no advantage and a diameter of 0.3 m appears to be the practical optimum. With this, a powerful compressor (more than 4 m^3 (STP) air per minute) will allow a flow of 100 to 200 m^3/h depending on the clogging. With air juttering (see chapter 4) a high flow rate can be reached for a short time with a much smaller compressor (chapter 4):

$$Q = \phi/\left\{W_c + \frac{1}{4\pi kH}\ln\left(\frac{2.25\ kHt}{r^2 S}\right)\right\} \qquad (4.1)$$

The maximum flowrate occurs at t ≈ 0.3 s. At a clogging resistance, W_c, of 0.05 $m/(m^3/h)$, i.e. 180 $m/(m^3/s)$, a storage coefficient, S, of 0.001 and $\Delta\phi$ equal to 10 m, it follows that:

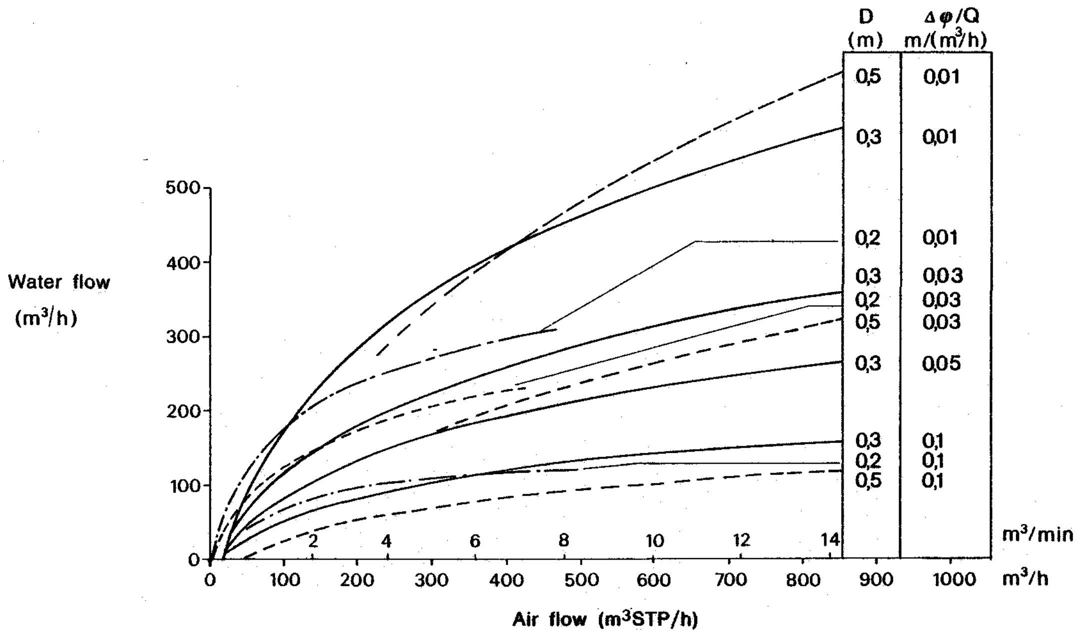


Figure 41 - Relation between water flow and required air flow (Olsthoorn, 1978) for 3 riser diameters and several values of specific lowering $\Delta\phi/Q$ $(m/(m^3/h))$. The wall friction coefficient λ equals 0.025, the efflux-resistance coefficient ξ of air pipe is equal to 1, the static water level is 3 m below ground-level, the compressed-air-injection point is at 30 m depth.

$$Q = 10/\{180 + 23.7\} = 0.049 \text{ m}^3/\text{s} = 177 \text{ m}^3/\text{h} \quad (6.12)$$

As the redevelopment proceeds and the clogging resistance falls, so does Q rise.

When the clogging resistance has dropped to 0.02 $m/(m^3/h)$, i.e. 72 $m/(m^3/h)$, Q will have risen to 380 m^3/h! Bearing in mind the effect of clogging on the juttering flow, it is recommended to redevelope in good time.

6.4 Technically

Now that the optimum riser and the screen diameter have been fixed, in this example, at 0.3 m, the annular space, left after positioning the necessary gauge pipes, is filled to 1 m above and below the screen with gravel. Except for formations consisting of very fine and uniform sand, 2 to 3 mm gravel is suitable for this purpose (Kobus, 1976).

Poorly permeable layers are sealed in order to prevent leakage and short-circuiting. It is good practice to place a metre of sand above the gravel pack and then one or more metres of clay or clay-cement. Thanks to its lower angle of internal friction (chapter 2), after a certain settling period, clay is better resistant to high water pressures than sand. Wells with a clay filling that has fractured due to too high an injection pressure can even be made usable again after a rest of several weeks or months (Brandes, et al., 1978).

Whether or not a given drilling system is advantageous, cannot be said with certainty. There is no practical information as yet on whether bailed boreholes are less advantageous than jetted or suctioned boreholes or vice versa, nor is there quantitative information about thought disadvantages of applying during construction drilling muds and chemicals; an injection well can operate satisfactorily with any good drilling system.

With an eye to possible use of chemicals, it is best to employ plastic for the riser and well screen. A high rigidity class is very adviseable in order to prevent collapse during redevelopment (Uil & Deelder, 1978). It is not known whether the large open surface of some stainless steel well screens (up to 30 %) offer significant advantages over plastic screens with their much smaller open surface area (about 7 %). The much higer cost of such sophisticated metal screens will generally outweigh their supposed advantages.

The injection line must always be under pressure. The design data required for this can be found in paragraph 3.2. In the present case a narrow pipe with adequate wall friction can be chosen for this purpose and run down to the bottom of the well. When redeveloping, the compressed air can then be blown in through this same pipe (figure 42).

At 18 m^3/h infiltration flow, (in this example namely 25.10^6 m^3/year with 160 wells) a minimum groundwater-level of 3 m below groundlevel and the bottom of the injection pipe at 30 m below groundlevel, the requisite friction will be about 3/30 = 0.1 m/m. A pipe with an internal diameter of 50 mm (friction: 0.12 m/m at 18 m^3/h, see table 1 at page 43) is suitable for this purpose.

The well head can be fitted by a quick-acting valve to be used during redevelopments. The redevelopment water is preferably discharged through a separate foul-water pipe (after neutralisation of any chemicals employed).

A gauge/sampling pipe, manometer, water meter, shut-off valve, compressed-air connection, discharge line and if possible a length of transparant pipe in the injection line for visual inspection (because of danger of bursting preferably do not use perspex but transparant PVC), complete the well accessoires (figure 42).

To prevent the pressure in the wells from rising too high, it is recommended to provide a control point, known as a "Christmas tree" at the end of the central supply line (Brandes et al., 1978). This also allows for flushing the pipe system.

To follow the course of clogging, gauge pipes are necessary. A practical arrangement is to have one gauge pipe in the well, one or two in the gravel pack and one or two in fine sand above or below the gravel pack. The latter give the head in the unclogged formation and comparison of this value with that in the gravel pack and the well itself indicates the course and location of the clogging (see also figure 4).

How an injection-well system would appear overall in places where the fresh/salt problem does not arise, is illustrated by the following example.

6.5 Illustration

Let us consider that an injection-well system has to be designed to protect a surface-water plant which applies direct purification of the raw-water up to drinking water.
The aims are then to provide a quality buffer and a reserve to cover interruption of the raw-water sup-

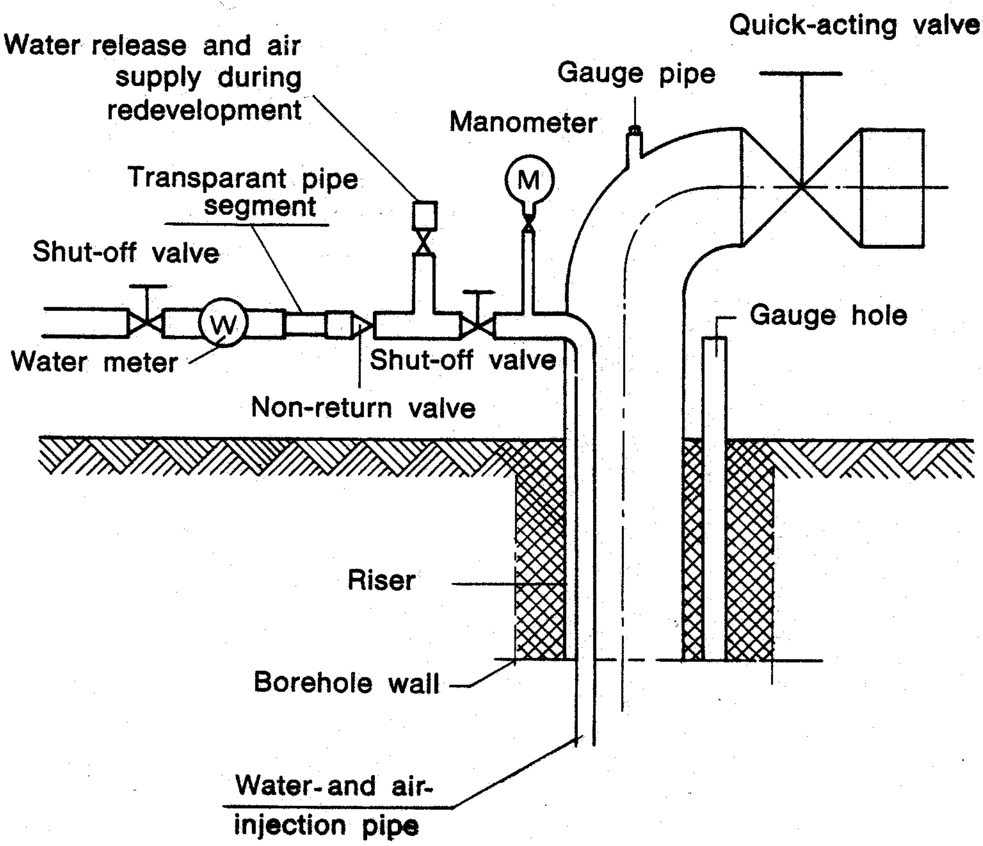


Figure 42 - Example of injection well accessoires (schematic).

ply. The required capacity is say 15 million m^3/year, while the available aquifer has a thickness of 30 m with a transmissivity of 1000 m^2/day, a porosity of 35 %, a storage coefficient of 0.001 and is covered by a clayey aquitard with a resistance, c, of 1000 days. The designed injection wells, with 30 m long screens in 0.6 m diameter boreholes, each have a capacity of 56 m^3/h at a regular infiltration rate of 1 m/h at the bore wall.
Consequently, 30 wells are needed for the system as a whole.

To avoid effects on the surrounding groundwater-tables as far as possible, the injection wells are placed at a circle completely enclosing the circle of abstraction wells.

30 abstraction wells, likewise of a capacity of 56 m^3/h, are placed 25 m apart in a circle of 120 m radius. To secure two months underground-residence time, the injection wells are placed at a circle of 300 m radius. The water-level at the ring of abstraction wells is now 6.7 m lower than the natural water-table. The lowering in the strata above the semi-pervious clay layer is much less and can in principle be countered by also recharging above the aquitard, possibly via ditches.

Should the supply of raw water be interrupted for some reason, abstraction will continue. Consequentely the groundwater levels will then fall. In this example, however, a practically steady-state situation will be reached within 10 days. The additional water lowerings occurring as a result of the interruption are tabulated below.

Distance: (m)	300	600	1000	2000	5000
Fall : (m)	9.0	5.1	2.8	0.8	0.0

At and within the ring of abstraction wells the total lowering is a maximum and will now amount to 9.0 + 6.7 = 15.7 m relative to the natural water-table. The water-levels in an aquifer above will also fall. With a resistance, c, of 1000 days the downflow through the clay layer, expressed in mm/d, is equal to the calculated drawdown. For $r < 300$ m, this downward flow of (9 mm/d) times (60 d) = 540 mm is to be considered an absolute maximum which might occur 60 days after stoppage of the raw-water supply. Given a storage coefficient for the overlaying, phreatic aquifer of say 25 %, this amounts to a fall in water level of roughly 2 m as a maximum.

During such an interruption of the recharge process, a particle of water starting at some 400 m from the centre of the installation, would, on an average, just reach up the ring of injection wells. Consequently, when normal operation resumes, there will be a chance of this particle being pumped up sooner or later through the abstraction wells.
Droplets originating on an average from more than somewhat over 410 m distance are not abstracted. In other words, the danger of attracting fouled groundwater over large distances, as is possible in normal groundwater recovery, will be viturally nonexistant in an injection-well system of this kind.
From this example we may conclude that an injection-well system of this capacity would be possible in the circumstances described, provided that the calculated incidental groundwater lowerings are accepted and the area of a radius of 300 to 500 m is protected and under the control of the waterworks. It is apparent that in this example an interruption of raw-water supply can be bridged without the need for the creation of some physical impoundage or surface-storage reservoir.

7 SYMBOLS USED

Dimensions are given in the ISO-units most commonly employed. Provided your system is consistent, choice of units in the formulae is unimportant. In other words, there is no need as a rule to use hours and seconds or m^3 and litres in any given formula.

a [$/m]	= drilling and finishing costs of an injection-well per metre well screen
A [m^2]	= well size or infiltration area of an injection well, calculated as borehole circumference multiplied by gravel-pack length: $A = 2\pi r_o H$
A_o [m^2]	= ditto, for test wells: $A_o = 2\pi r_o H_o$
b[*]	= price-factor for drilling and finishing wells of diameter (2r) instead of standard diameter ($2r_o$)
c[*]	= suspensoids concentration, expressed as m^3 clogging layer per m^3 injected water
f[*]	= wall friction of injection pipe, expressed as mH_2O loss per m pipe length
g[N/kg]	= strenght of gravity field
h[m]	= distance from ground level to top of gravel pack
Δh[m]	= head or rise of level in the gravel pack above groundlevel
h_1[m]	= head in a conduit before a valve
h_2[m]	= minimum head in a conduit, immediately beyond a valve

* dimensionless

h_3(m)	= head in a conduit at about 10 pipe diameters beyond a valve
ΔH(m)	= head loss across a valve, in the case of a non-horizontal pipe: loss of energy level or piezometric fall
H(m)	= gravel pack of operational wells
H_o(m)	= ditto for test wells
k(m/d)	= permeability coefficient of the unclogged ground
k_v(m/d)	= ditto for clogged ground
k_*(m/d)	= $kk_v/(k-k_v)$
k_i(m^2)	= intrinsic permeability coefficient, a soil property independent of temperature or liquid employed: $k_i = (\mu/\rho g)(k \text{ or } k_r)$
k_z(\$/m^3)	= purification cost per m^3 injection water
k_p(\$)	= capital cost per well, including associated onsite pipelines and pumps (if any)
k_r(\$)	= cost of one redevelopment
K_z(\$/a)	= total purification cost per year
K_p(\$/a)	= total depreciation cost of wells per year
K_r(\$/a)	= total redevelopment cost per year
K_T(\$/a)	= total cost per year: $K_T=K_z+K_p+K_r$
K_o(\$)	= fixed amount in the construction costs of a well
K_1(\$)	= construction and finishing costs of a well, excluding K_o, calculated from ground level tot top of gravel pack
l(m)	= thickness of clogged layer
MFI(s/l^2)	= membrane-filter index, a measure for the clogging rate of a membrane filter, expressed in seconds per

		litre squared
$Q(m^3/d$ or m^3/h or $m^3/s)$	=	injection-flow rate in the operational wells
$Q_o(m^3/d$ etc.)	=	ditto in the test wells
$Q_T(m^3/d$ etc.)	=	total injection-flow rate of the system
r(m)	=	borehole radius of operational wells or specified distance to centre of injection well
r_o(m)	=	borehole radius, sometimes specific radius of test wells
re (fraction per year)	=	rate of interest
rf(1/a)	=	redevelopment frequency of operation wells
rf_o(1/a)	=	ditto for test wells in fixed test conditions; is a direct function of the water quality
r_v(m)	=	distance from top of clogged layer to centre of well
R(m)	=	distance at which groundwater head is constant, geohydrological boundary condition
S(*)	=	storage coefficient, is a geohydrological constant
SAR[1])	=	sodium-adsorption ratio; SAR = $([Na^+] + [k^+])/([Mg^{2+}] + [Ca^{2+}])^{\frac{1}{2}}$, conc. in $[mol/m^3]$
t(s or h or d)	=	time
T (°C)	=	water temperature
T (years)	=	depreciation term or life time of injection wells (chapter 5)

* = dimensionless

1) = actually PSAR; the (sodium + potassium)-adsorption ratio = SAR + PAR

$u(N/m^2)$	= pore-water pressure around a well in the ground: Δu is an increment of u through infiltration (chapter 5)
$u(*)$	= $k_r N/k_p$ = reN/rf = reT (chapter 2)
$U(m^3)$	= total volume injected per well
$\bar{U}(m)$	= total volume of water injected per square metre borehole wall
$v(m/s)$	= water-flow velocity in a pipeline
$v(m/h)$	= flow rate at the bore wall of operational wells: $v = Q/A$
$v_o(m/h)$	= ditto in the test wells: $v_o = Q_o/A_o$
$v_o(m/h)$	= flow rate at the bore wall at time zero
$v_2(m/d)$	= daily increment of water-level or pressure head in an injection well
$V(m^3)$	= volume of clogging layer $V = \int_o^t Qc\,dt$
$W(h)$	= resistance of clogged layer, expressed as mH_2O per m/h flow rate
$W_c(h/m^2)$	= clogging resistance of an injection well, expressed as mH_2O per m^3/h injection-flow rate

* = dimensionless

Greek letters

$\gamma_g (N/m^3)$	=	volumetric weight of wet soil
$\gamma_w (N/m^3)$	=	volumetric weight of water
ϕ(deg. or or rad.)	=	property of the soil, angle of internal friction (chapter 2)
ϕ(m)	=	rise of groundwater level relative to a horizontal reference plain (chapters 3 to 6 incl.)
ϕ_o(m)	=	water-level or rise of pressure head in the well or gravel pack
ϕ_r(m)	=	water-level or rise of pressure head at a certain distance from the well
$\Delta\phi$(m)	=	rise of water-level or pressure head; increase rise in the well
$\Delta\phi_s$(m/a)	=	standard clogging rate: $\Delta\phi$ after 1 year infiltration at v = 1 m/h at the bore wall at 10 °C water temperature
λ(*)	=	passive soil pressure coefficient (chapter 2)
$\rho (kg/m^3)$	=	density of water
$\sigma_1 (N/m^2)$	=	maximum grain-to-grain stress at a point in the ground
$\sigma_3 (m^2)$	=	minimum grain-to-grain stress at a point in the ground
$\sigma_g (N/m^2)$	=	total vertical normal stress (= grain-to-grain stress (effective stress) + pore-water stress (neutral stress)) at a point in the ground
$\mu_T (Ns/m^2)$	=	water viscosity at T °C water temperature

* dimensionless

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APPENDIX A

Netherlands' Injection Wells (up to 1980) studied in depth by the Working Group.

Owner of the recharge test-well	ESTEL Hoogovens B.V.	Dune Waterworks of the Hague		Provincial Waterw. of North Holland	Municipal Waterw. of Amsterdam	Governmentel Institute on Drinking Water Supply
Details	Filtered river water	Recharge of drinking water	Purification in test plant	Drinking water from Andijk	River water after coagulation, settling and filtration	Model well, extended purification in test plant
LOCATION	IJMUIDEN	THE HAGUE	THE HAGUE	CASTRICUM	VOGELENZANG	VOGELENZANG
Drilling method	percussion	percussion	percussion	rotary, flush	percussion	-
Drilling depth (-msl)(m)	40.2	45.6	42.6	93.0	38.0	-
Bore hole diameter (m)	0.40	0.45	0.45	0.80	0.57	0.40
Length of gravel pack (m)	12.1	21.6	21.2	11.4	16.6	3.0
Outer gravel pack surface (m^2)	15.5	30.8	30.3	29.7	29.2	4.0
Gravel in pack (mm)	1-2/2-4	1.5-2.5	1.5-2.5	1.2-1.7	1.2-1.7	1.5-2.5
Well screen material	copper	PVC	PVC	PVC	PVC	PVC
Screen length (m)	10	19.4	19.4	9.4	14.0	2.0
Screen diameter (mm)	125/121	200/190	200/190	200/180	250/230	200/190
Screen area (m^2)	3.97	12.2	12.0	5.9	10.7	0.63
Screen-slots width (mm)	1-10	1	1	1	1	1
Open surface of screen (%)	7	6.7	± 7	6	7	± 7
Thickness of aquifer (m)	28	30	60?	60	60	4.5
Permeability (m/s)	$0.15.10^{-3}$	$0.28.10^{-3}$	$0.26.10^{-3}$	$0.35.10^{-3}$	$0.16.10^{-3}$	$0.2.10^{-3}$
Recharge flow (m^3/h)	16.7	36 - 60	36	10 - 30	30	3.5
Entry-velocity at bore hole wall (m/h)	1.1	1.2 à 2.0	1.2	0.34 - 1.0	1.0	0.9
Year of construction	1948	1973	1974	1974	1976	1978
In use as injection well	Aug. 1970 till Febr. 1974	Since March 1973, till 1980	Since July 1974, till 1980	Since Sept. 1975	Since Nov. 1976, till 1980	Aug. 1978 till Jan. 1979
Volume injected till may 1979 (m^3)	$0.13.10^6$	$2.5.10^6$	$0.7.10^6$	$0.7.10^6$	$0.3.10^6$	48000

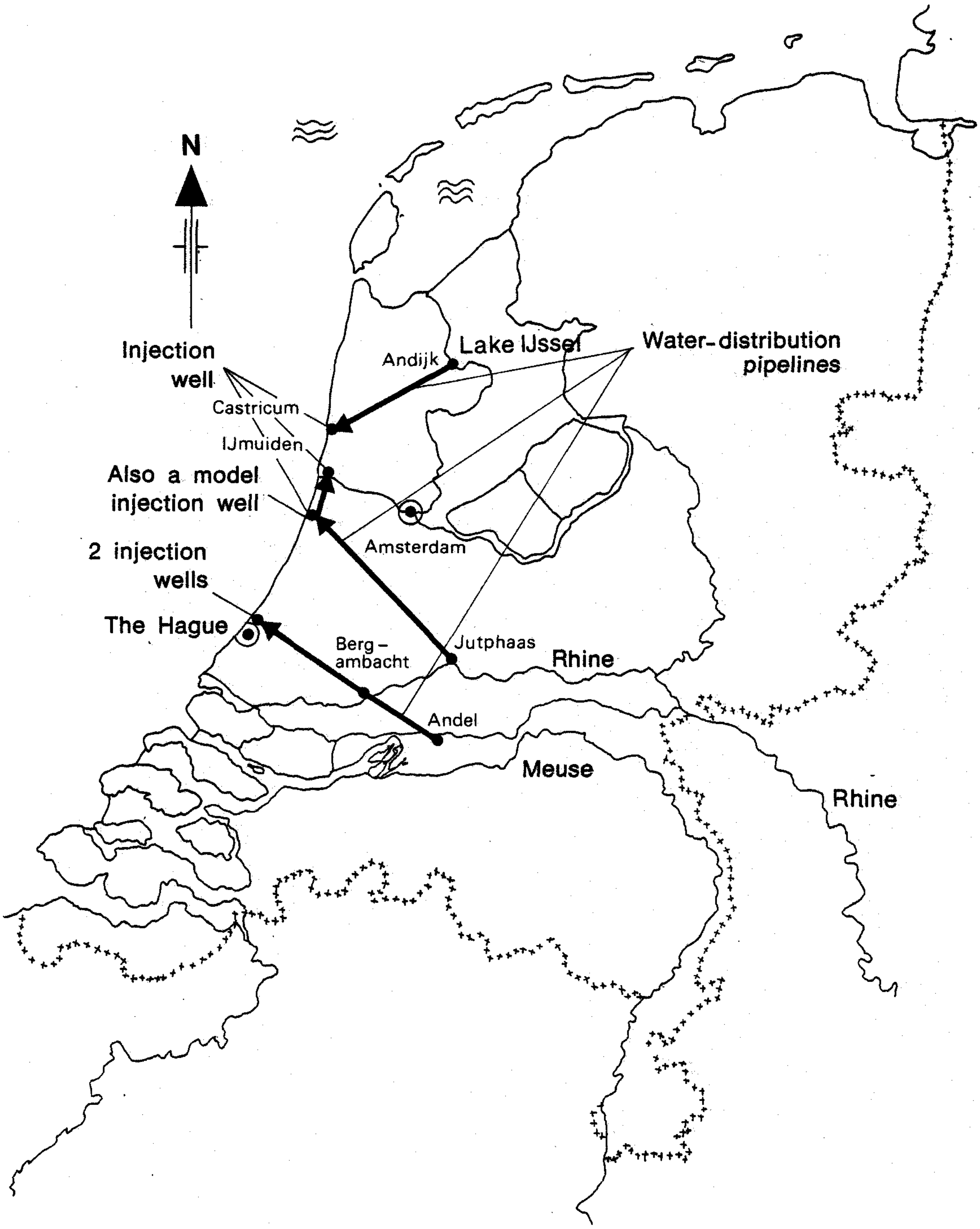

Figure 43 - Location of test-injection wells concerned in the joint Netherlands' injection-well investigation.

APPENDIX B

Composition of the Recharge Wells Working Group

Akker, C. van den, Gemeentewaterleidingen Amsterdam (Municipal Waterworks of Amsterdam, since 1980 with the Governmental Inst. on Drinking Water Supply). Member since 1975.

Beek, C.G.E.M. van, KIWA N.V. (Netherlands Waterworks Testing and Research Institute), Rijswijk. Member since 1974.

Bos, S.G., Duinwaterleiding van 's-Gravenhage (Dune Waterworks of The Hague). Member 1971-1973.

Boorsma, H.J., Rijksinstituut voor Drinkwatervoorziening, Voorburg (Governmental Institute on Drinking Water Supply). Member from 1970-1972.

Bulten, B., Chairman since the group was formed in 1970.
Till 1st June, 1975 employed by the Provinciaal Waterleidingbedrijf van Noord-Holland (North Holland Provincial Water Supply Works) at Bloemendaal, and since then by VEWIN (Netherlands Water Works Association) in Rijswijk.

Brandes, M.C., Rijksinstituut voor Drinkwatervoorziening (Governmental Institute on Drinking Water Supply), Voorburg. Member since 1972 (†, 1981).

Claessen, F.A.M., Till 1st June, 1977 employed by the Dienst der Zuiderzeewerken (Rijkswaterstaat Zuyderzee Project Authority) at The Hague, Since then by the Directie Waterhuishouding en Waterbeweging, District Noord: Rijkswaterstaat, Directorate of Water Management & Hydraulic Research, Northern District, Lelystad. Member since 1973.

Duin, H.J.E.M. van, Estel Hoogovens B.V., IJmuiden, (Steelworks IJmuiden). Member from 1970 till 1974.

Duyve, J., Gemeentewaterleidingen Amsterdam (Municipal Water Works of Amsterdam). Member since group was formed in 1970.

Felius, G.P., Dienst Zuiderzeewerken (Rijkswaterstaat Zuyderzee Project Authority), The Hague. Member from 1970-1973.

Haaren, F.J. van, Gemeentewaterleidingen Amsterdam (Municipal Waterworks of Amsterdams). Member from 1970 till 1972 (†).

Kieft, J.W. Gemeentelijk Waterbedrijf Groningen (Municipal Waterworks of Groningen). Member since November 1979.

Klaren, W., Hoogovens-Estel B.V. (Steelworks IJmuiden). Member from 1971 till 1972.

Dr. Kobus, E.J.M., KIWA N.V. (Netherlands Waterworks Testing and Research Institute), Rijswijk. Member from 1974 till 1975.

Krabbendam, J.P.J., Hoogovens-Estel B.V. (Steelworks IJmuiden). Member since 1974.

Kuipêri, J.C.H., Hoogovens-Estel B.V. (Steelworks IJmuiden). Member from 1970 till 1972.

Laan, J. van der, Waterleidingbedrijf Midden-Nederland, (Waterworks Midden-Nederland), Utrecht. Member since 1974.

Olsthoorn, T.N., KIWA N.V., Rijswijk (Netherlands Waterworks Testing and Research Institute). Member and project leader from 1974 till 1980, secretary from 1975 till 1979. (From Oct. 1982 with Governmental Inst. on Drinking Water Supply.)

Peters, J.H., KIWA N.V., (Netherlands Waterworks Testing and Research Institute), project leader since 1979.

Puffelen, J. van, Duinwaterleiding van 's-Gravenhage (Dune Waterworks of The Hague). Member since 1970, secretary till March 1975.

Romeijn, E., Rijksinstituut voor Drinkwatervoorziening, Voorburg (Governmental Institute on Drinking Water Supply). Member 1979 till 1972.

Tuinzaad, H., Duinwaterleiding van 's-Gravenhage (Dune Waterworks of The Hague). Member since group was formed in 1970.

Visser, Hoogovens-Estel B.V. (Steelworks IJmuiden). Member from 1970 till 1972.

Vlasblom, W.J., Provinciaal Waterleidingbedrijf van Noord-Holland, Bloemendaal (Provincial Waterworks of North Holland). Member till 1974.

Wildschut, R.J., Provinciaal Waterleidingbedrijf van Noord-Holland, Bloemendaal (Provincial Waterworks of North Holland). Member since 1975.

Winsen, P.J. van, KIWA N.V., Rijswijk (Netherlands Waterworks Testing and Research Institute). Member and secretary 1979 till 1981.