rijkswaterstaat

communications

groundwater infiltration with bored wells

by authors from

- national institute for water supply
- testing and research institute of the netherlands (waterworks undertakings)
- delft university of technology
- amsterdam public works department
- rotterdam municipal public works department
- pipelayers and drilling contractors federation
- department of public works

no. 39/1986

rijkswaterstaat communications

groundwater infiltration with bored wells

by authors from

- national institute for water supply
- testing and research institute of the netherlands (waterworks undertakings)
- delft university of technology
- amsterdam public works department
- rotterdam municipal public works department
- pipelayers and drilling contractors federation
- department of public works

the hague, 1985

all correspondence and applications should be addressed to

rijkswaterstaat

dienst getijdewateren

hooftskade 1

p.o.b. 20907

2500 EX the hague — the netherlands

the views in this article are the authors' own.

recommended catalogue entry:

groundwater
groundwater infiltration with bored wells / national institute for water supply . . . [et al.].
— the hague : rijkswaterstaat, 1985. — 204 p. : ann., ill. ; 24 cm. — (rijkswaterstaat communications ; no. 39)
with references.

Contents

Annexes

Preface

This report is the outcome of a study initiated by the Tunnel Technology Department of the Royal Institute of Engineers in The Netherlands and carried out by the study group on Groundwater Infiltration with Bored Wells.

Composition of the study group

M.C. Brandes†	Chief Engineer, Nationaal Institute for Water Supply; Chairman
H. Abbenhuis	Engineer, Department of Public Works, Locks and Weirs Division; secretary
J. Bardet	Chief Engineer, Water Works Division, Amsterdam Public Works Department
D. Cense	BV Grondboorbedrijf J. Mos, Rhoon; Pipelayers and Drilling Contractors Federation representative
H. Haitjema	Lecturer, Geotechnology Department, Delft University of Technology
T.N. Olsthoorn	Geohydrologist, Testing and Research Institute of the Netherlands (KIWA), Rijswijk
J.H. van Zutphen	Adviser, Geotechnology Engineering Bureau, Rotterdam Municipal Public Works Department

Mr. E. Horvat, an original member of the group, was replaced by Mr. J.H. van Zutphen (from the same organization). Upon taking up an appointment abroad before the study group had completed its work, Mr. H. Haitjema was replaced by Mr. H.J. Luger, also from the Geotechnology Department.

The report appeared in Dutch in November 1978 under the title 'Retourbemaling', and is obtainable from the Tunnel Technology Division of the Royal Institute of Engineers. The present publication in the series 'Rijkswaterstaat Communications' has been revised on the basis of data obtained from the De Bilt groundwater recharge test project.

1 Summary and conclusions

The installation of groundwater control systems when large building structures are being constructed can have undesirable consequences in the immediate vicinity (e.g. damage to buildings and vegetation). In some areas of the Netherlands restrictions exist on the amount of abstracted groundwater (if any) which may be discharged into surface waters. In such cases one solution consists of returning the water to the aquifer by bored wells (i.e. groundwater recharge). A number of groundwater recharge schemes in the Netherlands ran into problems, for which reason the Tunnel Technology Department appointed a study group on 'Groundwater Infiltration with Bored Wells' with the following terms of reference.

a. compilation of a survey of past groundwater recharge schemes in the Netherlands;
b. examination of the circumstances in which groundwater recharge is feasible in practice;
c. formulation of the requirements which a groundwater recharge system should satisfy.

For the first of these a survey was carried out which produced detailed particulars on all major recharge schemes in the Netherlands up to the end of 1977. This amounted to a total of 18 projects, the operating results of which are shown in Table 1. The projects to which the numbers used throughout this report refer are listed on p. 6, together with a general description of each project. The location of these projects in the Netherlands is shown in Figure 1.

Generally speaking the groundwater recharge projects in the Netherlands have been successful. The table below indicates that even large recharge projects can yield good results:

Project	no. of recharge wells	actual discharge (m^3/h)	length of pipeline system (m)
9	102	850	2000
14	427	320	750
16	83	800	480
17	17	530	1200
18	468	3000	3600

Clogging of the infiltration wells caused serious problems in only four of the 18 projects.

In three of the four cases the cause of the problem was the release of methane gas from the pumped-up water. Experience has shown that this form of obstruction can be prevented by the installation of effective degasification equipment or by maintaining the water under pressure.
In one case the clogging was caused by the flocculation of ferrous hydroxides resulting from the mixture of two different types of water. This can only occur in phreatic conditions, i.e. in the absence of a covering clay or peat layer. These circumstances were encountered only in project 8.
Clogging up by iron deposits can also occur as a result of the aeration of drainage water in the pumping wells or pipeline system. Although this is a fairly common phenomenon it does not generally result in major problems since the effects can be dealt with by careful groundwater infiltration procedures.
Since the survey was carried out further experience has been accumulated in the field of phreatic conditions in which clogging up by iron deposits did not occur, in this case because of the absence of iron-bearing groundwater in the layer in question.

To sum up the experience in the Netherlands, it may be said that groundwater infiltration can always be successfully carried out given the presence of covering clay or peat layers (with anaerobic groundwater, i.e. iron-bearing but free of oxygen).
This is also true in phreatic conditions if the water-bearing layer contains only aerobic groundwater (i.e. groundwater which is oxygen-bearing and hence free of iron). Reservations apply only in the case of an unconfined aquifer containing both aerobic and anaerobic water. The study group came to the conclusion, however, that recharge should also be possible in these circumstances provided that a number of supplementary measures were taken. A pilot groundwater infiltration project near the unsuccessful project no. 8 provided confirmation for this view.
The study group was unable to provide any general answer as to the economic feasibility of groundwater infiltration by bored wells. In order to conduct a cost comparison with other possible techniques (e.g. steel sheet-piling with underwater concreting, chemical injection or pneumatic caissons) one would have to have a general indication of the cost of installing and running a groundwater dewatering and infiltration system. No such general indication can be provided, however, since costs depend so heavily on the highly variable circumstances in which the work is to be carried out. Costs must therefore be worked out on an ad hoc basis. Chapter 8 contains a survey of hydrological calculation methods while Chapter 9 examines the factors that affect costs. It would, however, appear from the comparisons of the various techniques that groundwater recharge is often the cheapest.
As regards the execution of groundwater recharge there are two main systems:
a. the classical system with 'large' wells with a borehole diameter of approx. 0.5 m, constructed by means of reverse rotary drilling;

b. the system favoured in the last few years of a larger number of 'small' wells with a borehole diaeter of approx. 0.25 m. and constructed by jetting.

In geohydrological respects the two systems come to the same thing. The system of small wells affords somewhat greater operational reliability and can in certain circumstances be cheaper to instal and maintain.

It is important for proper research to be conducted before a groundwater recharge system is commenced. The study group concluded that guidelines needed to be laid down in this respect. Simple gas and filter tests can be conducted to establish whether clogging by gas bubbles or iron flocculation is likely. A pilot or test recharging scheme on a practical scale is generally essential.

Installation and operational requirements are summarized in Chapter 7. These requirements are largely based on four simple precepts:

a. the pumped-up water should not be subject to aeration at any stage of the system;
b. no part of the system should be subject to underpressure;
c. the water may not contain gas bubbles;
d. the pressure on the infiltration wells should not exceed approx. 1.5 m H_2O above ground level.

Further research is required into the following subjects:

a. effective means of degasifying methane-bearing groundwater;
b. means of preventing the flocculation of ferrous hydroxides in unconfined aquifers (such research has now been carried out in the pilot test undertaken after the failure of project 8);
c. means of combatting flooding in infiltration wells;
d. refinement of the gas and filter tests.

2 Introduction

In most parts of the Netherlands the groundwater table is not far below ground level. The foundations of many building structures go below the water table, for which reason dewatering is required.
The lowering of the phreatic surface or of water tension at greater depths achieved in this manner is not confined to the excavation itself but has a perceptible effect in a wider area, the range depending on such factors as the actual reduction of groundwater in the excavation and soil properties.
One of the consequences of dewatering is an increase in the granular tension in the soil. This can in turn lead to soil settlement and damage to building or engineering works which have their foundations in or on these layers.
In the case of a high groundwater table, a lowering of the level will directly reduce the amount of water available for trees and other vegetation. This could for example lead to a drop in agricultural yields.
Apart from these possible consequences it is becoming increasingly difficult in practice to obtain a licence in the Netherlands to discharge groundwater into surface waters.
These three factors have given rise to the concept of returning the pumped-up water to the ground at the points where it is required, thus bringing the groundwater table back to the same or an acceptable level. This principle has been applied in the United States for some time; it is discussed for example in a journal article of 1956 (1) concerning groundwater recharge in a New York excavation. This project prompted the designers of the IJ tunnel near Amsterdam to try this technique in the Netherlands in the same year, the first example of a major recharge system in the Netherlands. Since that time over twenty small and large groundwater recharge systems have been installed in the Netherlands, including a number of pilot schemes.
Although a number of problems have been encountered the results have in general been satisfactory. Partly in response to the difficulties that were encountered, the Tunnel Technology Department of the Royal Institute of Engineers decided to set up a study group into 'Groundwater Infiltration with Bored Wells'. The group comprised representatives from the National Institute for Water Supply, the Testing and Research Institute of the Netherlands, the Delft University of Technology, the Public Works Department of Amsterdam, the Rotterdam Municipal Public Works Department, the Public Works Department and welldrilling contractors in the Netherlands. The study group's terms of reference were:
a. to compile a survey of past groundwater infiltration schemes in the Netherlands;

b. to examine the circumstances in which groundwater infiltration was feasible in practice;
c. to formulate the requirements which a groundwater recharge system should satisfy.

The survey was compiled on the basis of a questionnaire sent to both the commissioning body and the contractor of each of the completed projects.

The replies received to the questionnaire also enabled the second and third aspects of the group's terms of reference to be carried out. As will be evident from the list of projects covered, the group confined itself not only to Dutch construction projects but also to those projects in which groundwater was reinjected near the spot where it had been obtained. The injection of cooling water or supplementation of groundwater for drinking water purposes therefore fell outside the scope of the study.

The study group expresses its gratitude to all those who cooperated in completing the detailed questionnaire.

3 Analysis of survey results

3.1 Introduction

As noted in the introduction to this report, a survey has been carried out in an effort to assemble all the available data in the Netherlands on the subject of groundwater infiltration schemes. This was done by means of a questionnaire drawn up by the study group which was designed to elicit information to provide a comprehensive picture of the installation and operation of such schemes. The questionnaire was sent to both the commissioning party and the contractor of all the known recharge projects at that time. In all 21 questionnaires were sent out, including two to projects which turned out not to have been implemented. Of the remaining 19, 18 questionnaires were returned almost entirely completed. Given the highly detailed nature of the questionnaire, this was a very high response rate.
Regrettably the nature of the questions sometimes meant that the replies did not contain all the data required.
Questionnaires were returned in respect of the following projects:

1. Tunnel at Zwijndrecht
2. Hospital at Leiden
3. IJ tunnel near Amsterdam, excavation C
4. IJ tunnel near Amsterdam, excavation E
5. IJ tunnel near Amsterdam, excavation F
6. IJ tunnel near Amsterdam, building dock
7. Utrecht power station
8. Underpass at De Bilt
9. Station building at Schiphol Airport near Amsterdam
10. 'Westraven' office block, Utrecht
11. Pilot groundwater recharge scheme for tunnel at Vlake
12. Pilot groundwater recharge scheme for Rotterdam metropolitan railway
13. Margriet tunnel at Uitwellingerga
14. Intersection at Maarssenbroek
15. Willem Dreeslaan office block, Utrecht
16. Metropolitan railway, Chris Bennekerslaan, Rotterdam
17. Sewerage treatment plant, Utrecht
18. Botlek tunnel, Rotterdam

The locations are shown in Fig. 1.
The information obtained is shown in tabular form at the end of this chapter in Table 3. Answers to questions that were not clear have been omitted.

Figure 1 Location of projects.

The results are shown in summary form in Table 1, which may be said to include *all major groundwater recharge projects to have been carried out in the Netherlands.* Serious difficulties were encountered in only a few of the projects. These were:

	no. of recharge wells	design discharge (m^3/h)	length of transmission pipelines (m)
1	3	120	350
3	33	320	1200
8	52	1900	c.500
13	6	90	none

It should however be noted that, despite the problems encountered at excavation C, the groundwater infiltration systems used in the construction of the IJ tunnel (projects 3-6) eventually functioned as planned. In the case of project 13 the problems were finally resolved by the introduction of certain technical modifications.
An analysis of the results obtained from the survey is provided below. Certain major features of the survey are first of all discussed, followed by a more detailed examination of the four problem cases referred to above.

3.2 General particulars

Leaving aside the two pilot schemes, a prior pumping trial to ascertain the required number of wells was carried out in ten of the 16 projects (i.e. 62%). A groundwater recharge trial was carried out in only nine cases (56%), viz. projects 3, 4, 5, 7, 9, 10, 13,16 and 17. It may be noted that the difficult projects (1 and 8) were among those in which no recharge trial was carried out.
In 11 of the projects (69%) the object of groundwater recharge was the protection of buildings, parks and the like. In three instances (19%) a groundwater recharge system was adopted on account of prohibitions or restrictions on the discharge of the water obtained as a result of dewatering. In once case (13) a trial was conducted while dewatering was already in progress. The projects generally achieved their object: 10 were successful (62%), while the remaining 6 (or 38%) were partly effective. No instances were reported of total failure by a project to achieve its object.

3.3 Data on the groundwater control system

Data on the dewatering system are of relevance in that the cause of clogging in recharging wells can usually be traced to earlier stages of the groundwater control

Table 1 Results of groundwater recharge schemes in the Netherlands

Project No	Commencement date	Duration (months)	Clogging	Cause	Recharge wells (type)	Pumping regularity 4 x per	Regeneration frequency	Regeneration method	Regeneration effect	Operation of recharge schemes	Aims achieved?
1	1955	?	yes	gas?	large	–	?	pumping	mod.	poor/mod	partly
2	1960	>12	little	unkn.	large	–	2 m.	pumping	?	good,later less so	partly
3	1963	23	yes	gas?/Fe	large	–	? (4)	acid	poor	mod./poor	yes
4	1962	26	no	•	large (2)	–	–	–	–	good	yes
5	1963	c.14	no	•	large (2)	–	–	–	–	good	partly
6	1962	30	yes	unkn.	large	–	?	acid	poor	?	partly
7	1967	c. 1	little	iron	large	–	limited	pumping	?	good	yes
8	1968	24	yes	iron	large	–	1 m.	acid	bad	poor	partly
9	1972	20	little	iron	large	month	1 yr.	syringe	good	good	yes
10	1974	7	yes	iron	large	3 wks	–	–	–	good	yes
11	1974	3½	no (1)	•	large	–	–	–	–	good (1)	n.a.
12	1974	c. 1	no	•	small	–	–	–	–	good	n.a.
13	1974	21	yes	gas	lg. (2)(3)	–	3 m.	misc.	bad	bad,later good (5)	partly
14	1975	24	little	debris	small	–	8 m.	airlift	good	good	yes
15	1975	5	yes	iron	large	2 wks	–	–	–	good	yes
16	1975	12	little	debris	small	–	limited	airlift	good	good	yes
17	1975	12	little	iron	large	–	limited	pumping	bad	good	yes
18	1976	18	little	•	small	–	limited	airlift	good	good	yes

(1) clogging in the beginning caused by fault in pumping well
(2) water from a deeper stratum
(3) combined pumping and injection wells
(4) acid treatment to begin with; later stopped
(5) after construction modifications (well R1)

system rather than to the recharging wells themselves. The key question is whether the water pumped up in the course of dewatering comes into contact with air or oxygen, thereby causing iron precipitation. As may be seen from Table 1 iron deposits are the most common cause of clogging in recharge systems.
Water can become aerated in the pumping wells in the dewatering process in two ways:

a. the water level in the well drops to (or near to) the intake opening of the submersible pump, in which case the pump may suck in air.
b. the water level drops below the top of the well filter; this produces a seepage zone in which water is subject to free fall in the filter and is intensively aerated.

It is not possible to determine from the results of the survey in which cases aeration occurred in the pumping well, but it is not uncommon for submersible pumps to suck in air, and this will no doubt have been a factor in some of the cases.
The data on the dewatering systems are summarized in Table 2. A number of points should be made on these data.
In a number of cases the pump was suspended in the filter (i.e. projects 3, 14, 15, 16 and 17). This reduces the risk of air being sucked in but also creates the risk of setting up a seepage zone.
Project 8 forms an exception in that it was the only project in which there was a lack of a covering, relatively impermeable layer (e.g. clay or peat). This was probably a major reason for the difficulties encountered in this project.
No direct conclusions can be drawn from the drilling technique used or from the method of well construction. In most cases reverse rotary drills or flushing augers were used, the latter sometimes with the addition of clay. This may have been a factor in the clogging of the recharging wells. This does not, however, apply in the two cases where bailers were used (projects 1 and 3). The fact that problems should have been encountered with these two projects is probably due to other causes, in that these were carried out in the early stages when very little experience had been accumulated in the field of groundwater infiltration with bored wells.
Similarly there are no indications that the materials used in well construction affected the results of the recharge systems. The use of steel in corrosive groundwater (projects 6 and 8) could lead to corrosion and clogging of the recharging wells, but this applies more to pipelines (section 3.4) than to wells. At the present time wells are almost always installed by suction drilling, synthetic materials being used to finish them off and for the pipelines, with good results.
Finally Table 2 shows pump delivery per metre of filter length (Q/1). This value generally lies between 2 and 12 $m^3/h.m$. The survey results did not provide any evidence to suggest that the actual rate of discharge from pumping wells was related to the effectiveness of the recharge system.

Table 2 Data on the groundwater control systems.

Proj. no.	Comm. date	Duration (mnths)	Covering layer	Drilling techn.	Screen			Pump		Q/1 (1)	Operation of rech. system	Aims achieved?
					mat.	diam. mm.	depth m.	depth m.	m³/h			
1	1955	?	yes	bailing	steel	150	13-24	10	35-115	3-10	poor/med.	partly
2	1960	>12	yes	rev.rot.	wood	220	17-25	18	60	7.5	good,later less so	partly
3	1963	23	yes	bailing	wood	150	24-34	35	25-30	2.5-3	mod/poor	yes
4	1962	26	yes	?	?	?	?	?	?	?	good	yes
5	1963	c.14	yes	flushing	wood	150	19-28	?	20-50	2-4.5	good	partly
6	1962	30	yes	?	?	?	?	?	?	?	?	partly
7	1967	c. 1	yes	rev.rot.	pvc	300	15-25	?	120	12	good	yes
8	1968	24	no	flushing	steel	300	12.5-22.5	12	45-110	4.5-11	poor	partly
9	1972	20	yes	rev.rot.	pvc	315	15.5-31.5	13.5	100	6	good	yes
10	1974	7	yes	rev.rot.	pvc	300	15-25	?	120	12	good	yes
11	1974	3½	yes	?	pvc	250	25-35	?	40	4	good (1)	n.a.
12	1974	c. 1	yes	rev.rot.	pvc	240	18-27	18	60	6.5	good	n.a.
13	1974	21	yes	rev.rot.	pvc	300	35-45	38	15	1.5	poor,later good (5)	partly
14	1975	24	yes	rev.rot.	pvc	300	15-25	18	50-100	5-10	good	yes
15	1975	5	yes	jetting	pvc	250	8-15	13	60	8.5	good	yes
16	1975	12	yes	rev.rot.	pvc	200	14-24	19	20-90	2-9	good	yes
17	1975	12	yes	rev.rot.	pvc	300	15-30	28	150	10	good	yes
18	1976	18	yes	rev.rot.	pvc	212	24-34	24	70	7	good	yes

(1) Q/1 = discharge per m filter length (m^3/h.m.)

All the recharge projects were carried out with water derived from pumping wells fitted with a submersible pump. In the case of project 18 the excavations had to be dewatered by means of vacuum pumping. The water pumped up in this manner was not used for recharging on account of the risk of air leakages in the suction pipes and pumps.

3.4 Data on the pipeline systems

Long pipelines can be used perfectly well provided they are kept under pressure at all points. Air managed to get in in a few cases (projects 1, 7 and 10), presumably because of defects. Generally speaking pressure in the pipeline system must be restricted to 3-3.5 mwk in order to prevent excessive pressure on the recharging wells, which might give rise to boiling. This occurs when the injection pressure is too high and the injected water finds its way directly to the surface and boils up next to the well shaft. This is discussed in more detail in Annex 3D.

Pressure in the pipeline system is usually regulated by means of a control manifold known as a 'Christmas tree' (see photograph on p. 18). This system was used in the case of the IJ tunnel. In the older recharge projects (nos. 1 and 2) and the experimental projects (11 and 12), as well as in a number of more recent cases (projects 13, 14, 16 and 17), presure was regulated solely by means of discharge valves.

There are however two reasons against using discharge valves:

a. The drop of pressure in such valves depends on the rate of discharge. As the recharging wells become clogged the rate of discharge and hence the pressure drop will decline, with a consequent increase in pressure in the pipeline system. If the rate of discharge should fall to very low levels, virtually the entire pumping pressure will be taken by the pipeline and the valve will no longer be able to perform its function as a pressure regulator.
b. In the case of a throttle valve, under-pressure can in certain circumstances occur behind the valve gate, so that air can leak in past the spindle. This phenomenon is discussed further in Annex 3A. It can mean that in a pipeline which is otherwise under pressure, iron borne in the water will nevertheless precipitate.

The former instance is presumably what occurred in project 1. Clogging by gas reduced the intake capacity, thereby leading to a build-up in pressure in the recharging wells. This happened so quickly that the pressure could not be adjusted in time and a number of the wells became subject to boiling within the space of a few days (see also 3.5 and 3.7.1).

In a number of recent cases (12, 14 and 15) pipeline pressures of up to a maximum of 5 to 10.5 mH_2O were permitted.

In the case of project 12 a number of return pits were deliberately overloaded to the point of boiling.

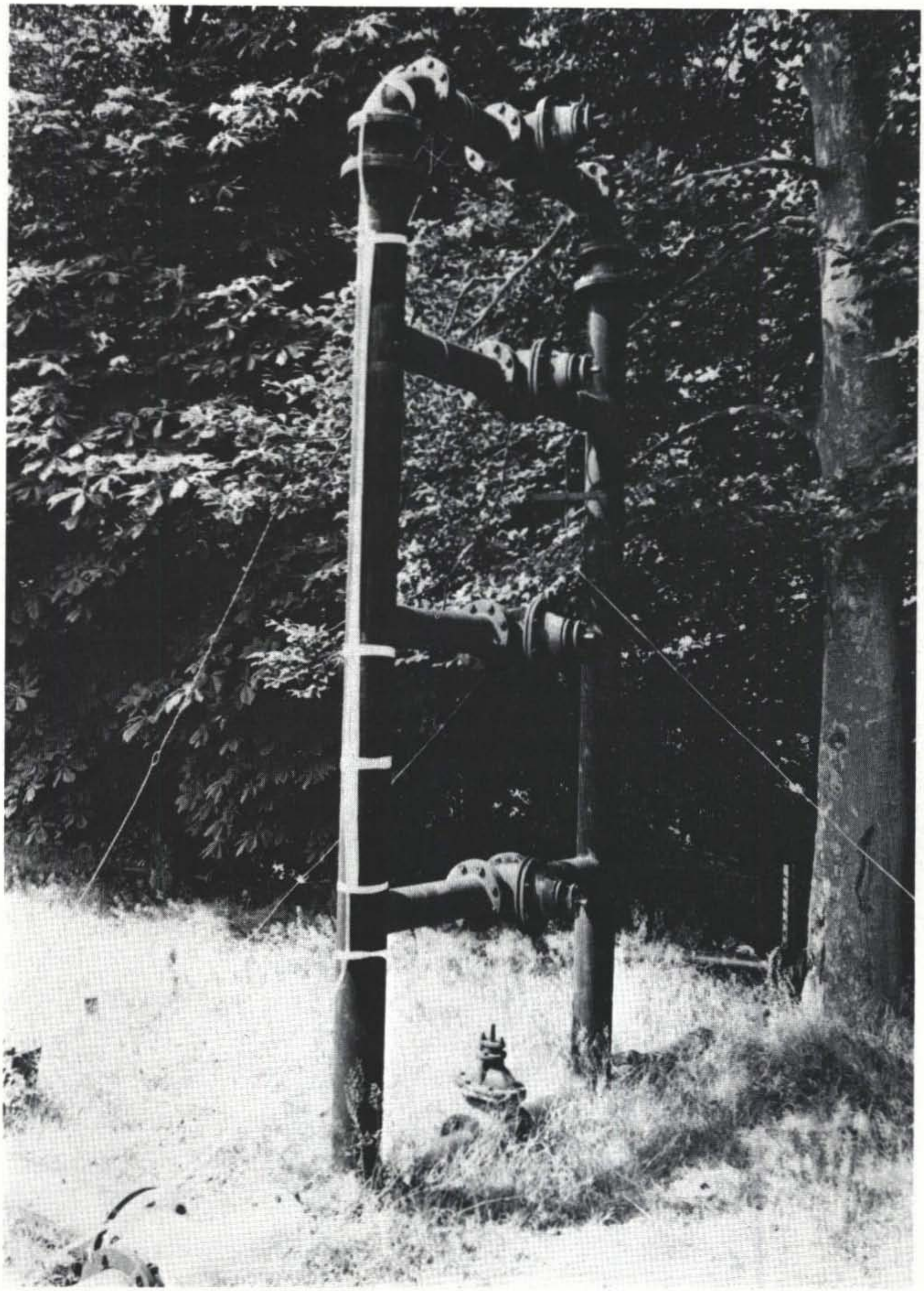


Photo 1 'Christmas tree'.

In three of the projects the infiltration system was fed by water drawn from a deeper sand layer. This avoided the need for long pipeline networks or expensive culverts.

3.5 Data on the recharging systems

As in the case of the pumping systems, the effectiveness of the recharging systems did not appear to be related to the particular drilling technique or well construction method used. In nearly all cases the wells were constructed by suction drilling or jetting; bailing was used only in project 1 and in the pilot projects (11 and 12). A wooden filter screen was used in the Zwijndrecht project, but this was the only such instance; the later projects all made use of pvc screens with graded filter material placed between the screen and the outer borehole.
As regards the diameter of the recharging wells, *two systems* were employed (see Table 1).

1. The old system with large wells (filter ø 150-300 mm)
2. Since 1974 narrower wells have also been used (filter ø 50 mm), especially in projects 12, 14, 16 and 18).

Small wells are closely spaced to form a 'curtain', and a large number is consequently required.
Large wells are always fitted with a special falling main designed to prevent the water from falling freely into the recharging well. These pipes used to reach to the bottom of the filter screen, but are now of limited length. They are also often used in the smaller wells, e.g. in project 17.
In general the recharging wells were under less load than the pumping wells. Taking all the projects together the *average* discharge (or intake) per filter metre (Q/1) was 7 m^3/h.m. in the case of pumping wells and 2.5 m^3/h.m. in the case of recharging wells. The averages are of course lower for the small recharging wells (c. 1 and 3 m^3/h.m. respectively).

The best index of the load on a recharging well is provided by the outflow velocity of the water (V_u) against the wall of the borehole. The bulk of any fines in the water will be deposited at the point where the relatively coarse graded filter material and the much finer sand formation meet.
For a given content of fines, the outflow velocity against the side of the borehole determines the speed with which a recharging well will become clogged.

$$\text{Now } V_u = \frac{Q/1}{\pi r_o} \ (m/h)$$

where $2r_o$ = the diameter of the borehole (m)

Unfortunately the survey did not provide data on r_0, so that outflow velocity can only be estimated approximately.
Generally speaking $2r_0 = 0.50$ m for large wells and 0.25 m for small wells. Leaving aside the atypical projects (nos. 7, 8 and 10), the following average figures apply to large wells:

$$Q/l \simeq 2 \text{ m}^3/\text{h.m}$$
$$\text{and } V_u \simeq 1.3 \text{ m/h}$$

In the case of the small wells used for the Metropolitan Railway and the Botlek Tunnel (16 and 17) the following apply:

$$Q/l \simeq 1 \text{ m}^3/\text{h.m}$$
$$V_u \simeq 1.3 \text{ m/h}$$

With a few exceptions the outflow velocity in large and small wells would appear hm be of the same order of magnitude.
Needless to say higher outflow velocities will not lead to more rapid clogging if the water does not contain suspended particles. In the case of project 10, for example, $Vu \simeq 5$ m/h without any problems being encountered. By contrast very rapid clogging occurred in project 8 where $V_u \simeq 2.5$ m/h but the water contained iron particles.

Project	Recharging wells Number	Q(m³/h)	l(m)	Q/l
with large wells				
1	3	20	9	2.2
2	4	40	15	2.7
3	33	9-12	12	0.75-1
5	4	12-15	8	1.5- 2
7	7	70	10	7
8	52	40	10	4
9	102	35	16	2.2
10	5	120	15	8
11	1	26	15	1.7
13	6	15	10	1.5
15	10	30	20	1.5
17	17	20-60	20	1-3
with small wells				
12	15	20	9	2.2
14	427	0.75	4	0.2
16	83	10	10	1
18	468	6.5	6	c.1

3.6 Practical operation and results

No conclusions can be drawn from the survey with regard to pressure regulation in recharging wells. In each case a valve was installed between the pipelines and the recharging well (or group of wells) with which the well could be regulated at the required rate of discharge or overpressure. As discussed in 3.4, it is possible for

pressure regulation to be affected by the rate of discharge and vice versa.
Despite precautions boiling occurred in over half the projects. It would appear that there is a tendency to take excessive risks, or that pressure regulation could be improved.
Once a recharging well has become subject to boiling it is virtually unusable and beyond redemption. It is therefore relevant to examine the means by which this condition can be avoided.
The survey question concerning the frequency of clearance pumping had been designed to determine in which projects systematic and frequent clearance pumping was treated as a routine maintenance step to avoid clogging. Only three projects appear to have treated this operation in that spirit:

Project	Frequency
9	every 4 weeks
10	every 3 weeks
15	every fortnight

From the replies to the questionnaire it is evident that 'regular clearance pumping' was sometimes confused with 'regeneration by means of pumping out'. The latter is done only when the condition of the recharge wells so demands rather than as a routine maintenance step. An example is project 8, where monthly pumping out had to be resorted to, even though it had little actual effect. In the case of project 18 the frequency of pumping out was 'variable' and thus a matter of regeneration rather than routine maintenance.
Operating results are shown in Table 1 and were summarized in section 1. In compiling Table 1 the assessments were sometimes modified in line with the considerations just discussed.
Apart from pilot project no. 12, which did not run for long, regeneration of more or less clogged recharging wells proved necessary in all cases. In most instances this was simply done by clearance pumping or by airlift. In project 9 cleaning was done once a year by syringe injection and section-by-section clearance pumping. Hydrochloric acid was used in a few cases: 3, 8 and 13.
In general poor results were obtained from regeneration; in many cases it was left till too late. Unless the cause of clogging can be eliminated regeneration is likely to produce only a short-lived improvement.
Where the routine maintenance of recharging wells not subject to rapid clogging is concerned good results appear to be obtained by airlift. This was done in the following projects:

Project	*Type of recharging well*
10	large
14	small
15	large
16	small
18	small

The introduction of air in the wells does not appear to be a disadvantage since the pumping action is greatly stepped up at the same time.

3.7 Special cases

3.7.1 *Project 1*

The following emerged from an examination of data that had been kept on the project after its conclusion.
The recharging wells had a required intake capacity of 30 to 40 m^3/h and clogged almost immediately. On the very first day intake capacity fell to below 3 m^3/h at an overpressure on the pipeline system of 1 m. The clogging made it difficult to regulate the pressure on the recharging wells (see section 3.4). In response to the build-up in pressure the wells became subject to boiling in a few days. The same occurred with another recharging well constructed at a later stage.
Submersible pumps with a capacity of 60 and 125 m/h were used for dewatering, but these had to be largely shut off. (One of the heavy pumps which was sucking in air had to be replaced by a lighter one.)
Despite the fact that the pipeline system was under pressure, gas or air managed to enter the system. Degasifying valves had to be fitted to the recharging wells since it was thought that the pressure drop might result in the release of methane (marsh gas).
The recharging wells were fitted with wooden screens surrounded by gauze-mesh. The extremely fine mesh used will have contributed to the clogging by solid particles.
One of the clogged wells was inspected by clearance pumping of individual sections. Analysis of the water and sediment in each section gave rise to the following conclusions concerning causes of clogging:

a. deposit of organic substances;
b. deposit of insoluble iron compounds resulting from the presence of traces of oxygen;
c. growth of sulphate-reducing bacteria and the deposit of ferric sulphides;
d. release and incomplete re-solution in groundwater of dissolved methane.

Rook refers to this research in his article (2), exmphasizing the chemical and biological processes: oxydation of iron, sulphate reduction and accumulation of bacteria. None of these processes, however, can cause serious clogging within a few hours. The main cause would therefore appear to have been the release of methane as a result of the drop in water pressure from approx. 15 mwk in the water-bearing layer to something like 3 mwk in the pipeline system. Whether such a pressure drop leads to degasification depends on the quantity of gas dissolved in the groundwater; unfortunately no data exist on this aspect.

The research carried out at the time did not produce any evidence to suggest that the organic substances and ferric sulphides had been formed by bacterial activity in the recharging wells. Sulphate reduction is a process that occurs naturally in the soil in river areas and takes some time to get going. The sulphides and organic particles therefore presumably originated in the water-bearing formation and were borne along in the pumped-up groundwater. If this is the case they can readily be removed by cleaning out the filter by pumping.
While it is possible that the traces of oxygen entered the system through the regulating valves, it is more likely that they entered the water through the suction pump of the appliance used for sampling the clogged filter section by section.
To sum up it may be said that the most likely cause of clogging was the release of methane. Further details on gases in groundwater and the formation of gas-bubbles in water are provided in Annexes 3B and 3C.

3.7.2 *Project 3*

Serious clogging arose in a prior recharge trial (1958) within two weeks. After three months both the trial wells were wellnigh fully clogged. The deposits in the clogged wells consisted primarily of FeS, so that sulphate reduction was once again identified as the cause. One of the wells was regenerated by acid treatment, which led to a build-up of H_2S gas (which is highly toxic, with an odour of rotten eggs).
For the same reasons as in project 1, it is unlikely that sulphate reduction was the cause of the problems encountered in the well recharge system. From the very outset the intake capacity of the recharging wells was much smaller than had been expected. Large quantities of combustiblegas were detected in the de-aeration process, and this was then identified as the main cause.
So as to be absolutely sure two sets of measures were taken:

a. Each of the header mains was fitted with a large degassing tank measuring about 3 m. high and 1 m. in diameter. Tall degassing pipes (up to Amsterdam Ordnance Datum + 5 m) or degassing valves were also fitted to a number of the recharging wells.
b. A number of the recharging wells were regenerated with hydrochloric acid in order to combat possible sulphate reduction. The remaining wells were simply pumped to waste in order to remove any accumulated gas (once the degassing had been improved).

It is no longer possible (15 years later) to isolate the effects of these measures. Acid treatment was, however, discontinued and the recharge system operated satisfactorily after 9 new recharging wells had been added. The water in the system proved moreover to contain only traces of sulphate, so that the sulphate reduction hypothesis was abandoned.
Once again the conclusion in this case must be that the initial clogging of the

recharging wells was primarily caused by the release of methane in the pumped up water. The problems were resolved by the installation of effective degassing devices and by reducing the capacity per well from 15 m^3/h to 10 m^3/h by increasing the number of recharging wells. The precise arrangements of the degassing tanks are no longer known.

3.7.3 *Project 8*

The difficulties encountered in this recharge scheme assumed such proportions that the project must be regarded as a failure. Extensive research was carried out into the causes, which is discussed in reference (3); this ascribes the cause of clogging in the recharging wells to the geohydrological situation. Dewatering was conducted in an unconfined aquifer, as a result of which both 'shallow' oxygen-bearing water and 'deeper' iron-bearing water were drawn up into the pumping wells. The mixing of the two types of water in the wells caused the precipitation of iron deposits which seriously clogged the recharging wells within the space of a few weeks.
This phenomenon had not been encountered in previous projects because groundwater recharge with bored wells is generally only used in circumstances where there are weak covering layers in which settlement could occur as groundwater is withdrawn. Layers of this kind which are subject to settlement (e.g. clay or peat layers) are generally relatively impermeable and thus prevent the groundwater from coming into direct contact with oxygen-bearing rainwater. Project no. 8 at De Bilt was not, however, concerned with the risk of settlement but with the prospect of damage to parks and other facilities in the vicinity resulting from the lowering of the water-table. The research carried out into the problems that were encountered indicated that little could have been done to prevent clogging in these circumstances once it had assumed serious proportions.
It may be asked whether any factors other than the above process may have contributed significantly to the clogging. It is possible that the groundwater may have been aerated in the pumping wells, e.g. if some of the submersible pumps sucked in air. Alternatively aeration may have occurred as a result of leakage in the wooden rising mains (which were constructed out of staves) or through the formation of a seepage zone (see section 3.3).
The sucking in of air by the pumps is unlikely to have been a major factor. A number of pumps were lowered to a greater depth at an early stage, after which the water level in the dewatering wells was kept regulated.
Aeration in a seepage zone is, however, a distinct possibility. A seepage zone can for example be set up between the shaft of the borehole and the much coarser gravel pack. The latter may in practice be regarded as an extension of the filter itself. In the case of the De Bilt pumping wells the gravel pack around the wellscreen was continued up to ground level in order to assist the dewatering of sand layers near the

surface. It is therefore possible that aeration may have occurred in the gravel pack above the filter.

3.7.4 *Project 13*

This project was based on a system of combined pumping and injection wells. Each well contained two filter installations, one set into a deeper sand layer and the other into a sand layer at medium depth. Groundwater was extracted from the deeper layer by means of a submersible pump and injected at the higher level. The advantage of this system was that it avoided the need to install a pipeline network in the village, which might have disturbed the residents. The lay-out of these wells and the results obtained with them are described in detail in reference (4).
It is of note that this project was preceded by a 5-month recharge trial which operated successfully. Following the trial six combination wells were installed for the project proper, but these malfunctioned from the very first day. Methane escaped from the deep groundwater and clogged the injection section of the wells, so that the injection intake had to be reduced from 15 to around 5 m^3/h. It proved possible to maintain an intake of 4 to 5 m^3/h throughout the dewatering project (1¾ years), but even so the recharge system fell short of expectations.
A number of tests were conducted on one of the wells in order to improve the injection intake. A small degassing unit was fitted at the well-head, but without clear result. Two impellers were then removed from the submersible pump with a view to reducing its lift. This had been 32 mwk for an intake of 5 m^3/h, although less than 10 mwk would have sufficed. Once this was done the desired intake of 15 m^3/h could be maintained without difficulty, presumably because the reduced pressure in the rising main meant that the dissolved gas was able to escape before reaching the degassing unit, thus forming larger bubbles which could escape more easily in the unit than before. Following a subsequent improvement it proved possible to increase the rate of intake to as much as 20 m^3/h. This was done by drilling large holes in the submersible pump rising main at the same level as the injection filter (18-27 m below ground level). This meant that the water was subject to a much lower drop in pressure in that it no longer had to be drawn up all the way to surface level.
Once these modifications had been effected the injection intake could no longer be regulated (apart from by discharging waste water). No further regulation was, however, required during the remaining eight months during which the well was in operation; nor was there any further clogging.
It is no longer possible to determine in retrospect why the prior trial was problem-free. During the trial 25 m^3/h were pumped up, of which 15 m^3/h were injected. The rest was discharged. During the project itself, however, discharge of surplus water was difficult or impossible. One possibility is that the pressure in the rising main was lower during the trial and that the bulk of the gas released in the pipe system was discharged with the waste water.

The following features are characteristic of gas clogging:

a. Clogging occurs very rapidly, in a matter of hours or at the most a day.
b. A state of equilibrium is then reached which remains unaltered; there is no further build-up of clogging.

These characteristics enable this type of clogging to be readily identified.

For table 3 see page 27.

Table 3 Recharge systems survey – 1.

Project no.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Commencement date		28.9.55	1960	April '63	June '62	Sept. '63	July '62	8.5.67	9.11.68	3.1.72	7.4.74	31.5.74	6.7.71	Nov. '74	Feb. '75	Sept. '75	Oct. '75	10.2.75	Nov. '76
Completion date				Feb. '65	July '64	May '66	Dec. '65	12.6.67	15.4.71	27.8.73	31.10.74	18.9.74	30.7.74	Aug. '76	Feb. '77	Mar. '76	Jan. '77	5.2.76	June '78
Duration		?	>1 y.	23 mnths.	26 mnths.	not continuous 60 weeks	2½ yrs.	5 wks	>24 m.	20 m.	7 m.	3½ m.	c. 1 m	1½ yrs.	>24 m.	5 m.	>12 m.	1 year	18 m.
Prelim. research:																			
Pumping test	yes/no	no	no	yes	yes	yes	no	yes	yes	yes	yes	yes	–	yes	no	no	no	yes	yes
– carried out by		–	–	(1)	(1)	(1)	–	v.Dijk	(2)	Tjaden	–	(2)	–	(2)	–	–	–	(2)	Mos
Soil parameters- kD	(m^2/24 h.)	± 500	–	220-290	160	160	500-850	2000	1500	1475	2000	335	1000	410	1400	2000	1000	1500	1000
	(m)	± 1000	–	1000	700	200	1200	–	–	500	–	800-1600	1500	150	680	–	1500	300	2000
Est. water load																			
– without recharge	(m^3/h)	–	–	180	–	–	820	560	700	1000	520	–	–	1830	–	275	200-240	950	2320
– with recharge	(m^3/h)	270	–	330	–	230	820	720	1900	1200	715	–	–	1900	–	350	600-1050	1050	3700
Recharge capacity	(m^3/h)	120	–	320	–	180	80	310	2135	1200	280	26	–	90	–	375	350-830	max 550	2735
Recharge trial	yes/no	no	no	yes	yes	yes	no	yes	no	yes	yes	n.a.	n.a.	yes	no	no	yes	yes	no
– carried out by		–	–	–	–	–	–	v.Dijk	–	Tjaden	v.Dijk	–	–	(2)	–	–	(5)	(2)	–
– clogging	yes/little/no	–	yes	little	little	–	–	no	–	little	yes	no	–	yes	–	–	no	yes	–
Reports available on																			
– dewatering research	yes/no	yes	no	yes	yes	no	–	no	yes	yes	–	yes	–	yes	–	–	no	yes	yes
– recharge trial	yes/no	no	no	yes	yes	yes	–	no	no	yes	–	yes	yes	yes	–	–	no	yes	no
Reason for recharge																			
– protection of buildings		+	–	+	+	+	+	+	–	–	+	–	–	–	+	–	+	–	+
– discharge prohibition		–	+	–	–	–	–	–	–	+	–	–	–	–	–	–	–	–	–
– groundwater conservation		–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	+	–
– other objectives		–	–	–	–	–	–	–	+(3)	–	–	+ (4)	+ (4)	± (4)	–	+	–	–	–
Aims achieved?	yes/partly/no	partly	partly	yes, after enlargement	yes	partly	partly	yes	partly	yes	yes	yes	yes	partly					
Technical data	Received																		
– well location	+/–	+	–	+	–	–	–	–	+	+	–	+	+	+	+	+	+	+	+
– soil profile sketch	+/–	+	–	–	–	–	–	–	+	+	–	+	+	+	+	+	(6)	+	+
– drill description	+/–	+	+	–	–	–	–	–	+	–	–	+	+	+	+	+	(6)	–	+
– sketch of rech. well	+/–	+	–	–	–	–	–	–	–	–	–	+	–	+	+	+	(6)	–	+
– chemical analysis	+/–	+	–	–	–	–	–	+	–	+	–	+	–	+	–	–	(6)	+	+

(1) Provincial Public Works Dept. + National Institute for Water Supply
(2) National Institute for Water Supply
(3) protection of parks
(4) trial
(5) Rotterdam Municipal Public Works Department
(6) see project 12

Table 3 Recharge systems survey – 2

Project no.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Dewatering data														(2)					
System: – pressure pipe		+	+	+	+	+	+	+	+	+		+	+	n.a.	+	+	+	+	+
– vacuum pipe		–	–	–	–	–	–	–	–	–	+	–	–	–	–	–	–	–	+
Number of pumping wells		5	5	18	8	4	39	6	28	12	–	1	6	6	40	5	48	8	52(3)
Type of pump		subm.	subm.	subm.	–	–	–	subm.	subm.	subm.	6	subm.	subm.	subm.	subm.	subm.	subm.	subm.	subm.
Intake depth	(m-g.l.)	10	18	35	–	–	–	–	12	13.5	subm.	–	18	38	18	13	19	28	24
Capacity per well	(m^3/h)	35-115	60	25-30	18	20/50	25/45	120	45-110	100	–	40	60	15	50 & 100	60	20-50(4)	150	70
Well drawdown	(m-g.l.)			max. 14.50	–	–	?	4.45	10	2.80	120	–	190	1.07	2 & 4	–	2.5	4	3
Drilling technique		bailing	suction	bailing	–	flushing	–	suction	flushing	suction	5	–	suction	suction	suction	jetting	suction	suction	suction
Drilling depth	(m-g.l.)	± 25	30	c.36.50	c.28	c.28	–	25	28	32	suction	35	28	46	25	15	26	30	–
Rising main – material		steel	wood	steel	–	–	–	pvc	wood	pvc	25	pvc	pvc	pvc	pvc	steel	pvc	pvc	pvc
– diameter	(mm)	150	260/220	250	–	–	–	300	300	315	pvc	250	240	300	250 & 300	100	200	300	225
Filter screen – material		steel	wood	wood	–	wood	–	pvc	steel	pvc	300	pvc	pvc	pvc	pvc	pvc	pvc	pvc	pvc
– diameter	(mm)	150	260/220	150	–	150	–	300	300	315	pvc	250	240	300	250 & 300	250	200	300	225
– top	(m-g.l.)	13	17	c.24	–	c.19	–	15	12.50	15.5	200	25	18	c.35	15	8	24	15	24
– length	(m)	1	8	c.10	–	9		10	10	16	15	10	9	c.10	10	7	10	15	10
											10								
Pipeline data																			
Length	(m)	100-250	100	1200	(1)	(1)	~150	450	c.500	2000		30	24	d.n.a.	750	250	480	1250	3620
Pipeline pressure – max	(m wk)	2	–	c.25	–	nil	~ 10	little	2.50	3.5	270	–	10.5	–	5	6	3-3.50	2.8	3.5
– min	(m wk)	1	–	–	–	–	0	–	–	2.5	2	–	3	–	1	–	0.5-1	1.8	2.5
Regulated by:											–								
– valves		+	+	–	–	?	–	–	–	–		+	+	+	+	–	+	+	–
– control manifold		–	–	+	+	?	+	+	+	+	–	–	–		–	+	–	–	+
Could air get in	-yes/no	yes	no	–	–	–	–	to ltd.	no	no	+	no	no	no	no	no	no	no	no
(half-full pipeline)?								extent			yes								
Recharge data																			
Total capacity																			
– design	(m^3/h)	120	–	320	–	–	80	310	1900	1200		–	–	90	350	375	830	550	2730
– actual (average)	(m^3/h)	75	120	320	50-60	45-80	30-60	310	1900	850	280	26	–	35	320	250	800/700	530	3000
No. of recharge wells		3	4	24, later	4	3.lat.7	9	7	52	102	280	1	15	6	427	10	83	17	387(6)
Capacity per well:				33							5								
– as a recharge well (new)	(m^3/h)	3 later 20	40	9-12	12	12-15	~7	69	40	7		26	20	5-15	0.75	30	10	20-60	6.5
• on an increase of	(m)	± 1	–	–	–	–	–	2.56	4	0.20	120	2.40	8	3	1	0.50	–	–	4
• Q specific	(m^3/h perm)	–	–	–	–	–	–	27	10	35	4.50	11	2½	1.7	0.75	60	–	–	1.6
Drilling technique		bailing	suction	–	–	flushing	-		flushing	–	26.7	bailing	bailing	suction	jetting	–	suction	suction	straight
Drilling depth	(m-g.l.)	22	–	c.38	–	c.30	–	–	–	–	suction	35	27.50	n.a.	13		24	30	
Rising main – material		steel	wood	st/pvc	–	–	–	pvc	pvc	pvc	25	pvc	pvc	pvc	pvc	–	pvc	pvc	pvc
– diameter	(mm)	150	200/160	250	–	–	–	300	300	250	pvc	315	50	300	50	–	50	300	50
Filter screen – materiaal		wood	wood	pvc	–	pvc	–	pvc	pvc	pvc	300	pvc	pvc	pvc	pvc	pvc	pvc	pvc	pvc
– diameter	(mm)	200x160	200/160	160	–	152 •	–	300	300	250	pvc	315	50	300	50	250	50	300	50
– top	(m-g.l.)	12	26	21.50	–	18.50		15	12.50	15.5	300	19	19	c.18	9	10	14	10	23
– length	(m)	9	15	c.12	–	8	–	10	10	16	10	15	9	c. 9	4	20	10	20	6
Gravel pack	(mm)	2-3	2-3	–	–	–	–	1.5-2	–	1.25-1.75	15	1.5-2.5	1.2-2.5	1.5-2.5	0.9-1.25	2-5	1.2-1.7	1.5-2	1.2-1.7
Falling main – length	(m)	24	–	35	–	30	-	+	20	6	1.5-2	10	n.a.	6.6	n.a.		–	3	n.a.
– diameter	(mm)	75	75	–	–	–	–		125	100	6	75	n.a.	82	n.a.	–	–	100	n.a.
Did water come into	yes/no	no	no	no	–	prob.	–	to ltd.	no	no	100	no	no	no	no	no	no	yes	no
contact with air?						yes		extent			yes								

(1) Recharge system fed by separate well in 3rd sandlayer.
(2) Extraction section of combined pumping and injection wells.
(3) Injection section of combined pumping and injection wells.
(4) 6 wells at 60-90 m^3/h and 42 wells at 20 m^3/h.
(5) Also 886 vacuum pumping wells, water from which was not injected.
(6) Also 81 shallow wells in holocene layer.

Tabel 3 Recharge systems survey — 3.

Project no.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
Operation of recharge wells																			
Well regulation for																			
– capacity	(limits) m³/h	–	0-60	per group	+	–	–	40-60	–	–	40-80	20-30	12.5-24	–	–	0-30	10-20	20-60	–
– overpressure	(limits) m-g.l.	c.1.30+	–	1.5-2.5+	~2.5+	none	3-3.5+	–	~4	2.5-3.5+	1+ –2+	2-2.5+	3.2-10.5-	0.5-1	0.5-1.50	–	1+ – 3+	–	2– –3+
Regular pumping out	yes/no	no	no	no	?	no	no	no	yes	yes	yes	no	no	no	no	yes	no	no	yes
– frequency	approx.	–	–	–	–	–	–	–	1x/m	4 wkly	1x/3 w.	–	n.a.	–	–	1x/2 w	–	–	–
Clogging	yes/limited/no	yes	limited	yes	scarcely	no	yes	limited	yes	limited	yes	yes	no	yes	limited	yes	limited	limited	–
– data received	+/–	+	–	–	–	–	–	–	–	–	–	–	–	+	–	–	–	+	–
– likely cause		various	–	gas/iron	–	–	unkn.	iron	iron	iron	iron	(5)	–	gas/Fe	debris	iron	debris	iron	–
Max. permitted overpressure	(m+g.l.)	>3.5(1)	2	5	–	·	–	–	2.50	3.5	1	0.50	5	1	0.50-1.50	6	3-3.50	c.0.5	3
Wells 'surcharged'	many/few/none	many	one	several	–	·	several	none	several	several	several	none	several	several	none	none	none	none	none
Regeneration	yes/no	yes	yes	yes(2)	–	–	yes	yes	yes	yes	yes	yes	no	yes	yes	yes	yes	yes	yes
– frequency	approx.		2 m.	–	–	–	–	not oft	1x/m	± 1 y.									
– method		pumping	pumping	pumping	–		pumping	pumping	flushing	(3)	pumping	–	–	pumping	airlift	p'g/airlift	p'g/a't	pumping	airlift
– chemicals		none	–	HCl	–	–	HCl		HCl	none	–	–	–	yes	–	–	–	none	–
– results	good/mod./poor	mod.	–	poor	–	–	poor	–	poor	good	good	–	–	poor	good	good	good	bad	good
Operating results							–							(4)					
– initial	good/mod./poor	poor	good	mod./poor	good	good	mod./poor	good	good	moderate	good	poor	good	poor	moderate	good	good	good	good
after 1 months	..	mod.	good	moderate	good	good	..	good	poor	moderate	good	good	good	poor	good	good	good	good	good
– after 2 months	..	good	good	moderate	good	good	..	–	poor	–	good	good	–	poor	good	good	good	good	good
– after 6 months	..	poor	mod.	good	good	good	..	–	poor	good	–	–	–	poor	good	–	moderate	good	good
– after 12 months	..	?	poor	good	good	good	..	–	poor	good	–	–	–	poor	good	–	good	good	good
– after 24 months	..	–	–	good	good	–	..	–	poor	good	–	–	–		good		–	–	–

(1) Result of overpressure.
(2) Initially, but later unnecessary.
(3) Jetting technique and section-by-section pumping out.
(4) Improvements were made to well R1 in Oct. 1975, after which it ran satisfactorily for 10 months.
(5) Defect in pumping well.

4 Circumstances in which a recharge system is practicable

The effectiveness of a groundwater infiltration system with bored wells depends critically on the properties of the water being recharged. A recharging well acts as a filter: suspended particles in the water are screened out, especially at the point where the relatively coarse gravel pack meets the much finer natural sand formation in which the well-screen has been installed. The filtration process results in the clogging of the injection well. Glogging can also be caused by gas bubbles, which get caught in the pores of the gravel pack and the surrounding formation, thereby reducing the available space for the water to flow through.
Chemical reactions between the injected water and the groundwater initially surrounding the recharging well are not a cause of serious clogging. This is because the original groundwater is completely displaced by the injection water, so that the recharging well is soon completely surrounded by injection water (5).
Instances are cited in the literature where the infiltration of fresh water in a clay-bearing sand formation containing salt or brackish water can produce clogging when the minerals present in the clay swell up (6).
We are not aware of any other instances of clogging caused by reaction with particles in the water-bearing stratum. In most recharge systems the abstracted water is returned to the aquifer from which it was withdrawn, so that interaction with the soil is unlikely. The injection of groundwater from a different (i.e. deeper) water-bearing stratum is inadvisable if the deeper layer contains fresh water and the higher one brackish or salt water. This is, however, seldom the case in the Netherlands, since groundwater generally gets saltier as one goes deeper.
As noted in the previous chapter, only four of the groundwater recharge schemes in the Netherlands ran into serious problems. In two of these cases the problems were resolved after a certain interval and the projects then ran as had been hoped. In both cases the problems had been caused by the release of methane in the abstracted groundwater. This was overcome in project 3 by the installation of degassing equipment and in project 13 by keeping the injected water under pressure.
Project 1 was the first groundwater recharge scheme to be carried out in the Netherlands. At that time the origin of the difficulties was still a matter for conjecture. With the experience acquired in the meantime it is now evident that the release of methane must also have been the major cause of the problem in this project. The installation of effective degassing equipment would have rectified the difficulties.
The fourth case, project 8, must be regarded as an exception. It was the only recharge system to be carried out in a water-bearing formation not covered by relatively impermeable layers, i.e. an unconfined aquifer. As it happened the

pumping wells extracted two different types of groundwater, namely shallow oxygen-bearing water and deep iron-bearing water. This led to the formation of iron deposits in the abstracted water and to serious clogging in the recharging wells. The conclusion should not however be drawn from this experience that a recharge scheme carried out in phreatic groundwater will invariably result in rapid clogging by iron deposits. No iron clogging occurred in a recent groundwater recharge project at Zeist (which could not be included in the survey) because in this instance the deep groundwater also proved to be aerobic (i.e. oxygen-bearing). Further particulars are provided in Chapter 11.

The effect of geohydrological conditions on the effectiveness of groundwater recharge schemes may be summarized as follows:

1. The formation from which groundwater is extracted is covered by semi-permeable layers (i.e. a semi-confined aquifer). The groundwater is anaerobic (i.e. contains no oxygen). No problems likely unless the groundwater contains significant quantities of methane. In this case the problems can be resolved by the installation of effective degassing equipment or by keeping the water under sufficient pressure. The study group is convinced that degassing can be effected simply yet adequately and has set out a number of recommendations in Chapter 10 for further research in this field.
2. The formation from which groundwater is being extracted is not covered by impermeable layers (i.e. an unconfined aquifer). In principle the groundwater will be aerobic (i.e. oxygen-bearing), although it may become anaerobic lower down. No problems arise if the formation contains aerobic water alone, but as shown in project 8 serious problems arise if both kinds of water are present. The study group considers that a recharge scheme should still be able to function satisfactorily in these circumstances provided that a number of supplementary measures are taken.

   The pilot recharge scheme discussed in Chapter 11, which was implemented in the immediate vicinity of project 8, provides confirmation for this view.

   The economic merits of recharge must be determined from case to case and depend on the available alternatives (e.g. steel sheet-piling with underwater concreting, chemical injection and pneumatic caissons). In order to facilitate a comparison between the various alternatives, Chapters 7, 8 and 9 examine the requirements which a recharge system must satisfy; the hydrological calculations required in the case of a dewatering/recharge system; and the factors which affect costs. It would, however, appear from comparisons of the various groundwater control techniques that a recharge system is often the cheapest.

5 Injection techniques

As noted in Chapter 3 recharge systems may either make use of large wells (i.e. filter diameter of 150-300 mm) or of small wells or wellpoints (filter diameter of 50 mm). These are the diameters of the inner casings; if the thickness of the gravel pack (15 and 10 cm respectively) is added, this makes for a borehole diameter of approx. 50 cm and 25 cm respectively. These construction differences affect the relative installation and maintenance costs, but in terms of the basic objectives of a recharge system the two systems come to the same thing. These objectives generally consist of the full or partial restoration of groundwater levels (and hence of the pore-water pressure in the granular structure) in places where this is required in order to avoid damage to buildings, vegetation etc. In order to achieve this objective a certain quantity of water per unit of length of the well curtain must be injected into the ground. This quantity (q; i.e. $m^3/h.m$) depends on local conditions and is determined on the basis of hydrological calculations, of which a number of examples are given in Chapter 8. Once the discharge per m of length (q) has been established it makes little odds whether it is returned to the ground by means of a few large or many small wells. This point is examined further in Annex 5A. The individual impact of recharging wells does not extend far beyond the well curtain itself, namely 0.5 times the well spacing. In practice the well spacing varies from 2.5 to 10 m for small wells to 10 to 25 m for large ones.

In certain circumstances it may be essential for the well curtain to be installed right next to the object it is designed to protect. This was the case for example with project 16, where the recharging wells were set into the pavement as close as 2.5 m from the fronts of the houses. In such cases well spacing must be kept small in order to ensure that the foundations are evenly protected. An effective design would therefore consist of a great many small wells each with a small discharge.

Provided there is sufficient space between the well screen and the object to be protected, well spacing and the construction of the recharging wells are a matter of free choice in a geohydrological sense. The actual selection will depend on practical considerations and installation and maintenance costs. One factor might be that a great many wellpoints tend to provide greater operational reliability than a small number of large wells.

A cost comparison in Annex 5B indicates that a system with small recharging wells is cheaper to install and run than a system with large wells. This is on the assumption that the wellpoints can be installed by means of the rotary straight-flush method with a jetting lance which can achieve the required drilling depth in one go. This effectively limits the depth to 35 m. Secondly it is assumed that the large wells have twice the capacity of the wellpoints.

As discussed in section 3.5, the latter assumption holds good if the outflow velocity (V_u) is the same for both types of wells. This is a reasonable basis for a cost comparison, but in practice the difference in capacity between large and small recharging wells can be greater than 2:1. In a particularly coarse water-bearing formation, therefore, or in a coarse and very thick stratum, a system with large wells can therefore be cheaper than one with wellpoints.

Hitherto wellpoints have been fitted with a filter and header pipe 50 mm in diameter. The friction in these narrow tubes sets definite limits to the intake capacity of the wellpoints, and also to the capacity with which they can be pumped out by air-lifting.

A calculation method for the air-lift system is provided in Annex 5C. This reveals that the pumping out capacity can be considerably augmented if a larger header pipe diameter is used. The optimal solution probably consists of the largest diameter filter and header pipe that can be achieved with jetting.

6 Preliminary research requirements

6.1 Introduction

In theory a wide variety of physical, chemical and biochemical processes can cause clogging in recharging wells; a good survey of these factors is provided by Rook (2). It is not possible to tell in advance which of these processes will apply in the case of a particular project. In the past this led to the view that the only means of determining whether a particular project was likely to be successful or not was to conduct a practical test. This view has undoubtedly been largely responsible for the present state of uncertainty concerning the general applicability of recharging systems.

The survey discussed in Chapter 3 provides for the first time an overview of the problems encountered in practice. It is evident that in practice clogging is the result of only two factors: degassing and the formation of iron deposits. In most cases it should now be possible to install a recharge system in such a way as to prevent clogging, or at least to keep it within manageable bounds.

At the same time the study group continues to regard a preliminary test as essential if failures are to be avoided, and has decided to specify certain guidelines for such trials. The accent of such trials is, however, rather different from what it used to be. As a first step a number of simple, qualitative tests are carried out to check whether degassing or the formation of iron deposits could occur, with the design of the recharge system being adjusted accordingly. The selected design is then tested on a practical scale. If clogging should still occur a test is carried out to see whether the wells can be regenerated by clearance pumping, and how frequently this would have to be done.

A satisfactory design for a recharge system can only be drawn up if the soil contents have been accurately established. This may be done by means of a pumping test (or pilot dewatering). The preliminary research thus consists of three phases which can be combined with one another.

With regard to the research design it should be borne in mind that the causes of clogging are related to the properties of the water being returned to the aquifer and not to the properties of the soil or the groundwater at the site of the recharging well. It is therefore important that the tests be carried out with groundwater pumped up at the site of the future excavation. If the recharge system is to be fed with groundwater from a deeper water-bearing formation, tests should be conducted with the same water as that which would be used in the actual project.

6.2 Pumping test

No general guidelines can be laid down for carrying out pumping tests; the design will depend on the geohydrological conditions and the required degree of accuracy of the results. In most cases one pumping well will be required at the site of the future excavation, with at least two and often more than 10 observation wells. The latter can be sited so as to account for any objects in the vicinity which might be liable to damage. If two different water-bearing formations are to be dewatered a separate pumping test will have to be carried out in each.
The length of the pumping test may vary from a few hours to as much as ten days. In many cases the pumping test will be followed by a stop test in which the restoration of the groundwater level is measured. In the event of two water-bearing strata the whole programme may take one to two months, or even longer in tidal areas.
The pumping well will generally provide the water for the remaining research as well. For this reason the depth, filter length and other construction aspects should correspond as closely as possible with those of the final pumping wells.

6.3 Gas test and filter test

These two tests are designed to provide a rapid and simple indication of whether iron or gas clogging is likely to occur or not. The tests do not establish the actual speed with which clogging will occur but provide an indication as to whether any special anti-clogging measures are required in the recharge system. Such measures can then be incorporated in the recharge test.
This type of preliminary research is new but the study group considers it should be carried out for each new project so that the recharge system can be adjusted in line with local conditions. These tests can be carried out during the pumping test or (in the absence of such a test) during pumping trials on one of the dewatering wells. The sampled water must however be the same as that to be injected during the actual recharge process. If the injection water is to be taken from a deeper water-bearing layer a special pumping well will have to be installed for extracting the same water. This well will in any case be required for the third phase, i.e. the recharge test.
Detailed accounts of the gas and filter tests are contained in Annexes 6A and 6B.

6.4 Recharge test

A trial on a practical scale is generally essential for the success of a recharge scheme. Detailed data required for the design of the installation can be obtained while any special arrangements (e.g. degassing equipment) can be tested.
In most cases it will not be possible for normal operating conditions to be

reproduced completely in a trial. During the actual project the recharging wells will operate in conditions in which the dewatering of the excavation will have resulted in a substantial drop in the groundwater level. A similar drop cannot effectively be achieved by a single pumping well during a trial. This means that the recharging well or wells can only be tested in conditions of lower underpressure than will in fact obtain during the project proper, in that the piezometric level in the well must, in view of the risk of boiling, remain confined to approx. 1.5 m above ground level. The injection intake obtained in a trial will therefore generally be lower than that achieved in practice.

In an effort to overcome this limitation, several of the recharge tests in the survey were based on a system of three wells. The wells were placed at the corners of an equilateral triangle with sides 6 m in length. One of the wells was used as a pumping well and the other two as recharging wells. This arrangement was used in projects 3-6, but suffers from the disadvantage that before long the same water is pumped round and round, so that the gas content (and other factors) no longer correspond with the true situation. As an additional complication, an abnormally high velocity of flow is built up underground between the pumping well and the recharging wells, so that fines can be transported from the soil. This can in turn produce a kind of clogging in the recharging wells which would not occur at normal flow velocities.

A recharging trial should be primarily designed to ensure that the quality of the abstracted water is as close as possible to that in the project itself. This means that the spacing between the pumping well and the recharging well or wells must not be much less than that in the real situation.

The inability completely to reproduce operating conditions in a trial also makes it impossible to determine with any accuracy the speed with which clogging will occur. One can only obtain a general impression of the frequency with which the recharging wells have to be cleaned by pumping out. It is however also important to obtain an impression of the effect of pumping out and of the discharge required for that process. In doing so various methods of pumping out can be compared.

The test installation should comply with the requirements laid down in Chapter 7 for recharging systems as such. If so indicated by the gas and filter tests, arrangements may have to be made to prevent clogging by gas or iron deposits. As noted earlier, the pumping well should be located on the same site as that from which the water for the final recharge system will be withdrawn. The pumping and recharging wells should be constructed along the same lines as the final wells and the spacing between them should not be too small.

The duration of the trial must be geared to the actual recharge project. In the case of a lengthy project (e.g. two years), the trial should last for between two and three months if the results are to be extrapolated with any degree of reliability. In the case of a short project a trial of a few weeks will suffice.

Precise measurements will be required, if the results are to be extrapolated, in order to detect any tendency towards gradual clogging. The piezometric level must be known in the recharging well itself and at a distance of several metres away. The difference provides a measure of well resistance, which is independent of all sorts of disruptive external factors. Resistance co-varies with the injection intake, for which reason it must be measured accurately for each recharging well.

7 Installation and operational requirements

7.1 Introduction

A number of simple rules must be observed if a recharge system is to run effectively. The most important of these is that the pumped up water should not be aerated at any point in the system. Even the smallest air leaks will eventually result in clogging by iron deposits, which leads to the second rule, that underpressure must not be allowed to occur at any point of the system. As a third rule the water must not contain any gas bubbles since these cause an almost immediate reduction in capacity, while as a fourth rule the pressure in the recharging wells may not exceed 1.5 m above ground level on account of the risk of boiling.
Although the main rules are simple, a great many details must be taken account of in practice. A summary of these details is given below; for safety's sake it has been assumed that all the conceivable problems could occur simultaneously. This will not of course always be the case, so that in certain circumstances it will be possible for some of the requirements to be relaxed. This applies especially to the exceptional case in which the groundwater contains no dissolved iron whatever.

7.2 Requirements with respect to the dewatering wells

1. Little or no clay or bentonite should be used in the drilling fluid.
2. The rising main must be leak-proof (i.e. no wooden pipes).
3. The top of the filter screen must always be well below the level to which the water-table has been drawn down.
4. Any impermeable layers above the gravel pack must be carefully sealed off with clay.
5. In the case of phreatic water the gravel pack may not be continued up to ground level. The borehole should instead be filled with impermeable material from a height of 1 m above the filter.
6. The pump must be a submersible one.
7. The pump must be installed at sufficient depth to prevent it from taking in air.
8. An inner dipping tube is required for monitoring water levels.
9. The well must be properly pumped out until the water is free of sand.
10. In phreatic water it may be necessary to pump up oxygen-bearing water and deep iron-bearing water separately. The practical means of doing so is a subject for further research. Allowance may have to be made for the need to install large and small pumps in each well.

7.3 Requirements with respect to the pipeline and regulating systems

1. The diameter of the pipelines should be on the generous side; the flow velocity should preferably be less than 1 m/s.
2. The pipeline material must be corrosion-proof; steel pipes must be internally bitumenized. Plastic pipes should be used for preference.
3. Pipelines should be under pressure at every point of the system.
4. Where roadways have to be crossed the pipeline should not be built over the top but installed underneath.
5. It should be possible to check the pressure in the system easily by means of transparent standpipes and manometers etc.
6. Gate valves should not be used for discharge regulation.
7. Pressure on the rising main should be confined to approx. 3 m above ground level. As a safety precaution a control manifold (or 'Christmas tree') should be installed at the selected pressure height.
8. A device for allowing the released gas to escape should be installed at a point beyond the regulating valves that are used to reduce the pressure in the submersible pumps to approx. 3 m above ground level. The technical specifications of such devices are a subject for further research.
9. Pressure in the recharging wells must not be allowed to rise to more than approx. 1.5 m above ground level. This should be something that can be readily checked (see point 7.4.7).
10. Each recharging well or group of recharging wells should be fitted with a regulating valve and a Woltman water meter.
11. Regulation of the recharging wells should not however be allowed to result in a substantial drop in pressure, since this would give rise to degassing.
12. The entire system should be drained and de-aerated before being commissioned. This will require appropriate discharge valves and vent taps.

7.4 Requirements with respect to the recharging wells

1. Little or no clay or bentonite should be used in the drilling fluid.
2. The top of the filter should be at least 10 m below ground level, and preferably more.
3. The grain size of the gravel pack should be as coarse as possible consistent with the safe pumping out of the well for cleaning purposes.
4. The remainder of the borehole above the gravel pack should be filled in with impermeable material (e.g. a clay-sand or clay-cement mixture). Any clay layers should be sealed off with clay.
5. The length of the filter should be related to the discharge capacity in such a way that the outflow velocity of the injection water against the side of the

borehole is no more than 1.5 m/h. The lower the velocity the slower any clogging will take place.

6. The injection water should not be allowed to fall freely in the recharging well.

6.1. In large diameter wells a falling main should be installed terminating well below the lowest water level. If the water level in the well is below ground level, the pipe should be fitted at the foot with a restriction designed to prevent underpressure in the pipeline.

6.2. A fall pipe may also be inserted in small-diameter wells (i.e. wellpoint curtains). In order to prevent undesired frictional losses the diameter and length of the falling main should be geared to the well discharge capacity. This is explained in Annex 5A by means of an illustrative calculation

6.3. In wellpoints without falling mains, a sufficient volume of water must be added to ensure overpressure in the system. If the supply of water should be inadequate a number of wells will have to be shut off.

7. It must be possible for each recharging well to be deaerated. It should also be possible to conduct a simple check to ensure the injection pressure does not exceed approx. 1.5 m above ground level. One means consists of fitting a transparent standpipe or hose to the well-head to a height of about 2 m.

8. If the water level in the well is below the well-head an internal access tube will be required for checking purposes.

9. Before being commissioned, each recharging well should be thoroughly pumped to waste.

10. Once the recharge system is in operation it should remain possible for individual wells to be easily and if necessary frequently pumped clean. Facilities for this purpose should either be fitted in each well or be capable of being added later, e.g. suction pipes, an air-lift system or a submersible pump.

7.5 Operational requirements

Many of the installation requirements have little point unless certain operational requirements are observed.

1. The system should be drained down and de-aerated before being placed in commission.
2. The system should be brought into commission gradually; maximum overpressure on the recharging wells should preferably be achieved in stages.
3. Over-pressure (or the water level in the well) and the injection intake should be checked daily.
4. Recharging wells should be pumped out in good time and preferably regularly. The following guidelines could be applied:
   - all recharging wells to be pumped out once or twice after being brought into commission.

– pumping out to be conducted before the over-pressure (or water level in the well) has risen by 1 m.

5. A rise in over-pressure may only be regulated by reducing the injection intake as an emergency measure. The sole remedy is clearance pumping.
6. The water level in the pumping wells should be checked regularly (e.g. once a week).
7. If water is injected which is not obtained from the dewatering of the excavation but from a deeper layer (or elsewhere), some form of automatic safety mechanism will be required enabling the recharge system to cut out the moment the dewatering system should stop.

8 Hydrological design calculation

8.1 Introduction

Apart from the technical arrangement of the pumping and recharging wells and connecting pipeline system, the number, capacity and location of such wells must be determined when designing a groundwater control system. In most cases hydrological calculations will have to be made for this purpose.

In the case of a dewatering system without recharge it will often be sufficient for a rough estimate to be made of the water load, on which the number, capacity and siting of the wells can then be based. The required drop in the groundwater level is then checked at a number of 'critical' points in the excavation by means of aggregating the influence of the selected well configuration.
If the solution is not satisfactory the configuration can then be adjusted and a new calculation carried out. An experienced hydraulic engineer can generally arrive at a satisfactory design fairly quickly by means of this trial and error approach.

If, however, the circumstances are more complicated – e.g. if recharge is required – the positioning and capacity of the pumping and recharging wells will be less clear-cut. In this case the trial and error method can easily become cumbersome and fail to produce the optimal result. In a recharge system, withdrawal and return infiltration are closely interrelated, for which reason a method should be used in which both the positioning and capacity of pumping and recharging wells may be derived directly by calculation.

The working group made a special study of a number of these 'direct' solutions. The latter may be divided into the following categories:
1. Analytical calculations without computer.
2. Analogue models.
3. Computer models.

The various methods are briefly discussed and compared in this chapter. A more detailed description of the methods is provided in Annexes 8A to 8F. To facilitate comparison Annexes 8A to 8F are all based on the same imaginary recharging project.

8.2 The test problem

In order to enable a quantitative comparison to be drawn between the various methods, the latter have each been applied to the same problem (hereafter referred to as the test problem). This test problem consists of an arbitrarily selected situation containing the sorts of elements that often arise in practice and that make the application of a more straight-forward approach very difficult if not impossible. The test case thus consists of an elongated excavation which cannot readily be represented diagrammatically in circular form, together with an 'object' (or building, as it is frequently referred to in the Annexes) at a short distance from the excavation which requires careful protection (see Figure 2). Since groundwater infiltration with bored wells is generally the obvious solution in those conditions where there are thick clay or peat layers near the surface which might otherwise be subject to settlement upon dewatering, the test problem has similarly assumed that the water-bearing stratum directly beneath the covering clay layers forms a semi-confined aquifer. The methods can, however, equally as well be applied to other circumstances. With regard to soil parameters, values have been selected which are representative of those encountered in the west of the Netherlands, while the water-bearing layer has been assumed to extend infinitely (i.e. several km) in all directions.

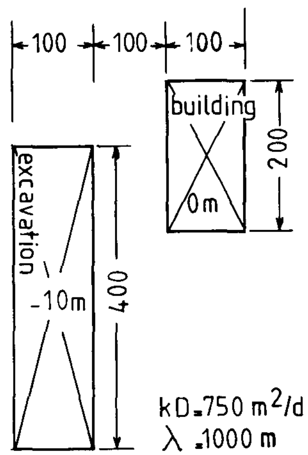


Figure 2 Ground plan of the test case.

8.3 Analytical calculations without computer

If the excavation and object to be protected are small in relation to the distance between them, the siting of the pumping and recharging wells is not critical. In these circumstances a calculation of the total withdrawal discharge and infiltration intake will suffice. This has been worked out for the test problem in Annex 8A, the calculation coming down to the solution of two equations with two unknowns, which can be done by hand. In most cases, however, the excavation and object will be closely spaced in relation to their size. In these circumstances it may well be possible to apply the trial and error technique, especially if an effective diagrammatic representation can be made of the object to be protected.

However, once a recharge problem has been reduced to a simplified diagrammatic form, it will often be possible for the siting and capacity of pumping and recharging wells to be solved 'by hand'. A number of mathematical examples are provided in Annex 8B in which well capacity has been calculated along 'impact boundaries'. These calculations are based on complex potential theory (i.e. computation using complex numbers) and the method of conformal representation. It is assumed for purposes of calculation that no increases or decreases in the water table are allowed to occur beyond a given line or lines as a result of the groundwater control system. The Annex explains how the calculated discharge profile may be regarded as a configuration of wellpoints with a given, constant capacity. The method of conformal representation similarly enables discharge profiles to be calculated for numerous other, complicated zones of influence. The calculations are valid for both a confined aquifer and phreatic water, and often provide a good approximation in the case of semi-confined aquifers.

8.4 Analog models

Another means of solving groundwater flow problems consists of using analog models. A well-known example is based on the use of carbonized paper. Carbonized paper is paper covered with a thin layer of conductive material with a constant specific resistance in all directions. This paper is conceived as a model for a water-bearing soil layer. In this model variations in the piezometric level of groundwater take the form of potential variations in the paper, which give rise to an electric current. Lines connecting points in the ground where there is an equal piezometric level are represented on the carbonized paper by lines of equal electrical potential. The fact that both the excavation and the object to be protected by recharging have a constant if differing potential (i.e. piezometric level) renders the use of carbonized paper particularly simple. After it has been ensured that the perimeters of the excavation and the recharge scheme outlines on the paper are freely conductive, a potential difference is introduced by means of a power source. This now enables equipotential lines to be traced with a voltmeter. The discharge and intake distribution around the excavation and the recharge scheme may now be determined by drawing a so-called quadrangular network. The test problem has been solved in this manner in Annex 8C. In this case, however, the flow lines were determined by

means of the so-called inverse problem, in which flow lines are converted into measurable equipotential lines, which can then be solved along the lines described above.
Both the anaytical calculations (8.3) and this analog model can be carried out with relatively simple mathematical and technical materials. The designer must however have a sound (theoretical) knowledge of groundwater mechanics. The analog method only provides an accurate representation in the case of confined aquifers and phreatic water. As shown in Annex 8C, however, excellent approximations for semi-confined aquifers can often be obtained with this method as well.

8.5 Computer models

Both general and more complicated problems can be solved by computer. Computers can however only be used effectively if the person designing the groundwater control scheme is able to assess and check the computer results.

8.5.1 *Finite element method*

A well-known numerical method for solving groundwater flow problems is the 'finite element' method. In this method the area to be analysed is divided into a large number of triangular elements (or subregions). For each corner of each element (i.e. at the nodal points of the element network) the piezometric level is calculated on the basis of the piezometric level and abstraction or infiltration of water at other nodal points. It is possible to formulate the problem in such a way that the required abstraction or infiltration levels at points where the piezometric level has been laid down (e.g. around the excavation and in the recharging wells) are taken as computer output. In this way the finite element method provides a direct calculation of the discharge profile around the excavation and the object to be protected.

The particular strength of the element method is that it enables even quite complex problems to be solved without difficulty, e.g. problems with complicated types of excavations and/or objects and complicated boundary conditions such as canals and rivers and decreasing or variable geotechnical parameters.

The element method can also be used to solve semi-confined aquifer problems, as done in Annex 8D for the test problem. One disadvantage of the finite element method is that it often requires a computer with a large memory and that even so the remaining memory capacity will set limits to the density of the element network, allowing more detailed analysis only in certain locations. For this reason it may be necessary for the element network to be wholly or partially fed into the computer point by point for the design calculation of each new dewatering scheme. This renders programming rather clumsy and time-consuming.

8.5.2 *Well method*

The well method (thus named by the authors themselves) is an analytical method in which the discharge levels for previously sited pumping and recharging wells are calculated on the basis of achieving the required piezometric level at a number of selected points. This method thus provides a direct calculation of the required discharge and intake levels in existing wells, rather than the discharge profile. It is therefore desirable for their positions to be chosen as carefully as possible beforehand. If the choice is a poor one the calculation will have to be repeated with fresh well positions. The well method requires a much smaller memory capacity than the finite element method (or the finite difference method). The programme can also be more readily adjusted for new problems.
Since the method is analytical, extremely detailed calculations can if necessary be made. On the other hand, constant geotechnical parameters have to be assumed. The method may also be applied to semi-confined aquifers by replacing the formulas for confined aquifers by those for semi-confined aquifers. In Annex 8E the test problem has been solved for both confined and semi-confined aquifers.

8.5.3 *Line sources method*

Generally speaking the design calculation of a dewatering and/or recharge system is concerned with abstraction or infiltration levels next to the excavation or object to be protected. Instead of discretizing this discharge profile over a series of points around the respective perimeters (i.e. the finite element and well methods) a superior approach may consist of the selection of line segments each with a constant specific discharge. In those places where the largest discharge variations are anticipated, a greater number of (i.e. smaller) line sources or sinks are selected around the perimeter than elsewhere. It is also possible to select linear specific discharges along the line segments if a more accurate discharge profile is required. Greater accuracy can however also be achieved by selecting a greater number of line segments with constant specific discharges.

The test problem has been analysed by this method in Annex 8F. The line sinks and line sources each have a constant specific discharge, the levels being calculated by computer. If the pumping and recharging wells are assumed to have a certain constant discharge, the (discretized) discharge profile arrived at may be regarded as a distribution of pumping and recharging wells.

Like the well method, the line sources method is an analytical one, so that the remarks at the end of the previous section also apply in its case. The line sources method has not so far been adapted for use in the case of semi-confined aquifers, although in the authors' view this should be possible. In comparison with the well method it is easier to incorporate boundaries; this can moreover be done in a more realistic manner. Individual wells can also be added in this method, which therefore lends itself to being combined with the well method.

8.6 Comparison of results obtained by various methods for the test problem

In the interests of quantitative comparison the various techniques were applied to a test problem (see Annexes 8A and 8C-8F, where these methods are set out in greater detail). By way of comparison the results have been listed in the table below, showing total water abstraction from the excavation and total injection in the vicinity of the object to be protected:

Table 4 Total discharges in m^3/d obtained by various methods

Method	Annex	Notes	Abstraction (m^3/d)	Infiltration (m^3/d)
1) general	8A	3)	39570	23330
2) carbonized paper	8C	2)	39300	22500
3) finite element	8D	1) 3)	41740	23670
4) well method	8E	2)	39000	22770
5) ,,	8E	3)	39610	23700
6) line sources	8F	2)	35950	23400

Notes: 1) Excavation width: 80 instead of 100 m.
2) Confined aquifer.
3) Semi-confined aquifer.

There is a close degree of correspondence between the results obtained by the various methods. The differences are attributable to variations in the way in which the problem was schematized. A certain amount of schematic variation is inherent in the use of the various techniques, but some of the variation is the result of the personal insights or preferences of those tackling the problem.
With respect to the schematization, only methods (1), (3) and (5) took the supply from the covering layer into account (i.e. a semi-confined aquifer). This was not done in method (6), although it should be possible.

Only method (2) is restricted in aplication to either confined or unconfined aquifers. Given the results obtained in the case of the test problem, it would appear that in certain cases (see Annexes 8C, 8E and 8F) the supply from the semi-permeable layer can be disregarded without adverse consequences.

In the case of method (2) irregularities in the carbonized paper can affect the results. It should also be noted that the results of all the methods discussed above (with the exception of (1)) provide the discharge profile along the perimeters of the excavation and the object to be protected, from which the optimal well distribution along those perimeters can then be directly derived. Method (1), however, does not provide information on this aspect, so that one of the more sophisticated methods has to be made use of in order to determine the optimal siting of the wells.

8.7 Conclusions

All the hydrological calculation methods described above are practical in nature and will if properly applied lead to a sound hydrological design for a recharge system.

Space does not permit an exhaustive examination of all the available and effective techniques.

The choice of technique will depend on the complexity of the problem and the aids at the designer's disposal. Three different computer-based techniques have been described in the report, of which the last (the line sources method) is possibly the most versatile.

A particularly effective approach consisted of first of all calculating the discharge profile required to obtain a given piezomatic level along the perimeters of the excavation and the recharge system. This then enabled the locations and discharge levels of individual pumping and recharging wells to be selected, if necessary, on other than purely hydrological grounds. The advantages of the finite element method over the more common analytical methods (namely the ability to deal with soil discontinuities and complex boundary conditions) are not fundamental in nature. As shown in the annexes, complex boundary conditions can also be introduced in most of the other methods. Soil discontinuities are in fact encountered only rarely in practice. Complex boundary conditions can moreover be introduced in the gallery abstraction method by means of 'dipole allocations'. The advantage of an analytical approach over a numerical method such as the finite element technique is that calculations need be carried out only for those points of the area in which one is interested rather than for all the points of the network. Nor therefore does accuracy depend on the fineness of the network and hence on the size of the computer (i.e. application of networks with variable mesh width). Analytical methods such as the gallery abstraction technique are therefore powerful aids, which can also be combined with the use of the increasingly widespread mini computer.

9 Cost factors in a recharge project

The capacity and geographical size, and hence the costs, of a recharge project depend on:

a) its objectives, e.g. maintenance of groundwater levels or discharge prohibitions; and
b) local circumstances.

Point b) covers the location of the dewatering scheme and the site at which the recharge water is to be injected, together with the geological and hydrological properties of the sand layer to be recharged.

The cost of the recharge system will depend on:

1 The local terrain and the existence of any external constraints with regard to the activities to be carried out, such as:
1.1 The accessibility of the project site.
1.2 The location of boreholes, i.e. proximity to buildings, and whether located in open country, roads or pavements.
1.3 The piezometric level in relation to the drilling method used.
1.4 The site at which working water can be obtained.
1.5 The geological soil profile. Soil composition determines the type of drilling technique to be used.
1.6 The need or lack of need to dig in feed lines and recharging wells.
1.7 Condition in which the site must be left upon completion of the project.
2 The size and scale of the project in relation to the length of the pipeline system.
3 The quantity of recharge water to be injected. This determines:
3.1 The number of rechargingwells required (once the type of well has been decided upon).
3.2 The diameters of the feed lines.
3.3 Dimensions of water meters and valves etc.
4 The required number of special extraction wells or additional pumping wells needed to supply the injection system.
5 Operational requirements such as:
5.1 The number of observation wells required for recording groundwater levels.
5.2 The method of discharge measurement.
5.3 Monitoring of groundwater levels and processing of data obtained in 5.1.
5.4 Monitoring of the actual operation of the scheme and processing of data obtained in 5.2.

5.5 Project regulation.
5.6 Safety aspects, such as the installation of back-up charging wells and use of feed lines with surplus capacity.
5.7 Maintenance, especially 'clearance' pumping of recharging wells.
6 Operational duration of the project.
This determines:
6.1 The size of the total rental charge for the installation.
6.2 The level of monitoring and regulation costs.
6.3 Maintenance costs.
6.4 The level of likely repair costs, e.g. for wear and tear, clogging and corrosion.
6.5 Energy requirements.

The cost-increasing factors will to some extent be offset by a reduction in discharge costs if the groundwater abstracted in the dewatering system is wholly or partly re-injected in recharging wells.

For a comparison of the cost of ordinary recharging wells with a wellpoint 'curtain' see Annex 5B.

10 Recommendations for further research

A number of problems emerged from the survey of the recharge projects conducted to date in the Netherlands which stand in need of further research. The study group has made a start on a number of these points. An indication is provided below of the areas already covered and of those where the study group considers further research to be required.

10.1 Research into gas clogging

The major cause of gas clogging is in all probability the formation of gas bubbles in the abstracted groundwater. As may be seen in Annex 3B, methane is commonly found in groundwater in the west and north of the Netherlands. The groundwater also contains other dissolved gases, small quantities of which are carried along by the released methane, but which do not readily escape from the water. The question as to whether other gases such as nitrogen and carbon dioxide are released from groundwater in sufficient quantity to cause clogging in the recharging wells is difficult to answer, but is of little practical import. The degree of clogging does not depend on the nature of the gas but on the quantity released in the water being pumped up. The latter may be determined in each individual case by means of the gas test referred to in section 6.3, and which is discussed in more detail in Annex 6A. For degassing to be carried out effectively the bubbles must have sufficient time to grow to the point at which they can escape easily. The rate of growth of a gas bubble in water is discussed in Annex 3C.

The degassing appliances used in past projects to control gas clogging have not been particularly effective. The study group believes that the development of an effective degasser would be a major contribution towards improving the reliability of recharge schemes. For this reason the group sought the advice of an expert from the Delft University of Technology, Mr. J.A. Wesselingh, whose initial assessment is contained in Annex 10A. Further research on a practical scale is required in order to arrive at a satisfactory design.

Other but less important aspects for further research include:
1. clogging mechanisms and means of controlling them;
2. the elimination of gas from clogged formations.

10.2 Research into iron clogging

In principle there are two possible solutions to iron clogging which is caused by the mixing of 'shallow' oxygen-bearing groundwater and 'deep' iron-bearing water as occurred in project 8:

1. separate abstraction of the two types of water;
2. very frequent clearance pumping of the recharging wells (e.g. once a day) to prevent clogging from assuming serious proportions.

In the long run, frequent clearance pumping is unlikely to be sufficient, for which reason the study group would give priority to further research into the first of the two solutions listed above. An experiment carried out in the vicinity of project 8 along these lines is described in detail in Chapter 11. The need for further research will depend on the results of this experiment, which will form the subject of a separate publication.

10.3 Research into boiling

The material used for back-filling the borehole of a recharging well has a bearing on the risk of boiling, as discussed in Annex 3D. In this respect clay is an unsuitable filler and sand or even concrete are to be preferred. It would be desirable to investigate whether higher injection pressures could be achieved than at present with other filler materials. Such research would be required to furnish recommendations concerning back-filling and guidelines on permissible injection pressures.

10.4 Execution of gas and filter tests

The study group attaches considerable importance to the identification of simple means for obtaining an early indication of the risk of clogging in recharging wells. Timely action can often avoid failures. Research in this field has led to the gas and filter tests referred to in section 6.3, and which are discussed in more detail in Annexes 6A and 6B. Both tests have produced satisfactory results.
Further research is, however, required into the best means of carrying out these tests, while it would also be useful to have a set of operating instructions. The tests also need (where possible) further application in practice.
The filter test was for example used in the later recharging experiment conducted in the vicinity of project 8.

11 Recharging experiment near project 8

The results of the various recharging projects carried out to date in the Netherlands were analysed in Chapter 3. It was seen that the recharging scheme in project 8 was the sole instance of an outright failure.

This also proved to be the sole project in which there was an absence of semi-pervious upper layers, i.e. an unconfined aquifer in which phreatic water was being pumped up. Despite the success of many other recharge projects, the poor results obtained in the case of this underpass have – in our view unfairly – adversely affected the reputation of recharge schemes as an effective aid in construction work below groundwater level.

Possible causes of the difficulties encountered at the time include the following:
1. oxygen-bearing upper water resulting from the direct contact between the oxygen-bearing rainwater and the groundwater;
2. the fact that the gravel packs in the pumping wells was continued through to ground level, thus providing a possible source of aeration;
3. direct aeration because the submersible pumps drew in air;
4. aeration through leaks in the joints of the wooden rising mains;
5. dissolved gas;
6. transported sand.

It may be noted that the two last reasons are unlikely to apply in this particular instance since the phenomena normally encountered in such cases (e.g. very rapid clogging by gas, as in project 1, and the presence of transported sand) were not in evidence. Since it should be possible to carry out a successful recharge scheme in an unconfined aquifer given our present knowledge and the current state of the art, a design for such a recharge facility was drawn up which took account of the possible causes of failure noted above.
Financial contributions from a number of the organizations participating in the study group made it possible for this installation to be tested in practical circumstances in the form of a recharge experiment lasting over a year in the immediate vicinity of project 8.

The installation consists of a pumping well with a tapping capacity of approximately 100 m^3/h, a pipeline system and ten injection wells approximately 2.5 m apart.

The pumping well consists of:

a. a borehole of approx. 600 mm in diameter constructed to a depth of 30 m below ground level by means of the reverse rotary method;
b. a p.v.c. rising main with glued joints and an inner diameter of 300 mm;
c. a filter, the top set at 15 m below ground level and the bottom at 30 m below ground level;
d. a 1.5-2.5 mm gravel pack exending to 13 m below ground level;
e. the remainder of the back-filling consists of and sand/clay/cement mixture extending to the surface;
f. the following are fitted in the rising main:
   1 submersible pump with a capacity of 40 m^3/h, with the intake at 15 m below ground level;
   1 submersible pump with a capacity of 100 m^3/h, with the intake at 20 m below ground level (the two pumps being independently regulable); 1 access tube with a screen from ground level to 30 m below ground level.

- an observation piezometer at a distance of approx. 1 m from the pumping well with a screen extending to a depth of 3 to 4 m below the water table;
- the well is pumped to waste until it is free from sand.

Two independently regulable pumps are used for eliminating the upper oxygen-bearing layer of groundwater and the lower iron-bearing layer of groundwater. Only the deep water is injected; the upper water is discharged in a nearby ditch.

The pipeline system consists of:

a. plastic pipes with an inner diameter of 200 mm leading to the injection wells;
b. a valve, manometer and water meter near the pumping well;
c. a discharge facility near the injection wells;
d. two valves, a water meter and manometer for each injection well;
e. a plastic pipeline system with an inner diameter of 100 mm for discharging the upper water;
f. a plastic pipeline system with an inner diameter of 150 mm for discharging dirty water during the air-lifting of wells;
g. a 'Christmas tree' fitted to the pipeline leading to the injection wells.

The injection wells consist of:

a. a jetted 300 mm diameter borehole constructed to a depth of 30 m below ground level;
b. a p.v.c. rising main with glued joints with an inner diameter of 70 mm (7 wells) and 150 mm (3 wells);

c. a screen running from 22 to 30 m below ground level with the same diameter as the rising main;
d. a p.v.c. falling main with an inner diameter of 1½", the bottom set at 29 m below ground level;
e. a 1.2-1.7 mm gravel pack;
f. 3 wells have been filled with 2 m of cement/sand above the filter; for the rest they have been back-filled with borehole material;
g. 4 wells have been completely back-filled with borehole material above the filter;
h. a 'tell-tale' tube has been fitted to the falling main for checking the water level in the well.

In addition a reference well was installed half-way between the pumping well and the infiltration curtain, consisting of:
a. a borehole approx. 500 mm in diameter constructed to a depth of 30 m below ground level by means of reverse rotary drilling;
b. a p.v.c. pumping filter with an inner diameter of 150 mm running from 2 to 30 m below ground level;
c. a p.v.c. observation filter with an inner diameter of 50 mm, running from 2 to 30 m below ground level;
d. the two filters set 15 cm apart, the gap being maintained by spacers;
e. a 1.5-2.5 mm gravel pack with a minimum thickness of 5 cm;
f. a water gauge indicator fitted to the well.

The borehole material was used to establish the soil profile, while the well itself was used for water level and oxygen and iron content measurements.
Since this project is to form the subject of a special report, a general account of the results must suffice at this stage.

The local situation is shown in Fig. 3 (p. 56). The preliminary research indicated that, contrary to expectations, the shallow groundwater did not contain any measurable quantities of oxygen, but this situation altered once the pumping well went into operation. The localized drawdown of the water table caused the infiltration of oxygen-bearing groundwater from a nearby brook, thereby reproducing in the test the situation of two different types of groundwater on top of one another. The oxygen-bearing brook water (B) is shown above the iron-bearing groundwater (A) on the left of Fig. 4 (p. 56).
The recharge system commenced operation on 7 August 1979 and, apart from a few teething problems, functioned satisfactorily until the end of December. The deep groundwater (A) was pumped up by means of the lower submersible pump and injected in the recharging wells.

The shallow groundwater (B) was dealt with by the upper pump and discharged some distance away in a ditch. The optimal operating conditions proved to be 70 m^3/h for the lower pump and 30 m^3/h for the upper pump. This system of separate abstraction fully measured up to expectations: the water returned to the recharging wells was free from oxygen and also contained virtually no suspended particles (as indicated for example by the filter test).

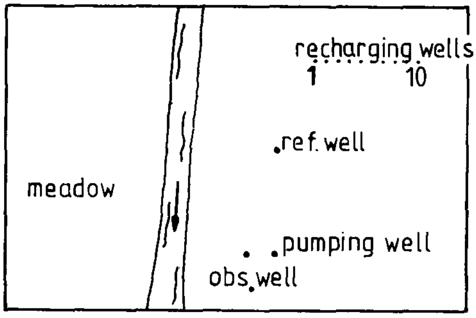
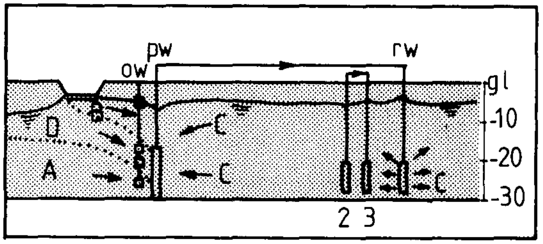


Figure 3 Local situation.



pw – pumping well
ow – observation well
rw – recharging well

Figure 4 Diagram of water types.

Despite the satisfactory operation of the system the recharging wells eventually began to get clogged. The wells were pumped out three times by means of air-lift in the period to the end of December, but it then proved necessary for this operation to be carried out more frequently and even for hydrochloric acid to be used in regeneration. Despite the precautions taken to prevent the entry of oxygen into the system the sludge brought up when the wells were pumped out turned out to contain ferric hydroxides. After several months of research the clogging was traced to the serious pollution of the groundwater with nitrate from a farm on the opposite side of the brook. The nitrate-bearing water (D in Fig. 4) was sucked in by the pumping well and, mixed with other types of water, pumped up by both pumps. In April 1980

the nitrate content of the infiltration water was over 2 mg NO_3-/1. In normal circumstances bivalent iron in groundwater is not oxydized by nitrate. This can only happen given the intervention of nitrate-reducing iron bacteria which convert nitrate into nitrogen. The nitrogen thus released is then used in the oxydation of iron. The research indicated that this double conversion was taking place in the immediate vicinity of the recharging wells: as close as 2.5 m to the wells the injected water was free from nitrate. The injection water flowed back towards the pumping well (C in Fig. 4) to form as it were an underground bubble of iron-bearing but nitrate-free water.
In order to demonstrate that nitrate was the cause of the clogging, a final experiment was conducted in which nitrate-free water was pumped up out of this bubble in recharging well 2 and directly injected into recharging well 3, while the remaining wells were supplied as usual with water from the pumping well. This test ran for five weeks, during which time well 3 showed no signs whatever of clogging, while the other wells continued to clog up as before.

This kind of clogging may be prevented in practice by siting the recharging wells in a ring around the excavation, thereby screening the pumping wells from outside pollution. The same water is then pumped round and round within the circle as occurred in the experiment with wells 2 and 3. It should also be noted that the nitrate pollution encountered at the test site was an exception. A value of 94 mg NO_3-/1 was recorded in the groundwater at a depth of 5 m in the meadows to the west of the brook. An extremely high level of this kind is only encountered in the vicinity of a major source of pollution.
Despite the clogging of the recharging wells it proved possible to keep the recharge system going by frequent clearance pumping by means of chasing and air-lifting. If the intake capacity of the wells should drop excessively they can if necessary be regenerated with hydrochloric acid. The recharge system remained in operation (with certain gaps) until 3 October 1980. In all some 360,000 m^3 of water were injected in the recharging wells.

12 Bibliography

(1) 'Solution to Wet-Foundation Problem', Engineering News Record, 12 April 1956.

(2) 'Chemische en biochemische aspecten van het in de bodem terugpompen van grondwater' (Chemical and Biochemical Aspects of Groundwater Recharge). Article by **J.J. Rook** in De Ingenieur, 20 Dec. 1968, Vol. 80, no. 51.

(3) 'Mededelingen 73-6', National Institute for Water Supply.

(4) 'Rapport inzake de resultaten van de proeven en de praktijkervaring met gecombineerde winnings- en injectie-putten te Uitwellingerga (Friesland)' (Report of the Results of the Tests and Practical Experience with Combined Abstraction and Injection Wells at Uitwellingerga, Friesland), issued by the National Institute for Water Supply.

(5) **Bernard, G.G.** 'Effect of Reactions between Interstitial and Injected Waters on Permeability of Rocks', Producers Monthly, December 1955, pp. 26-32.

(6) **Brown, D.L. and W.D. Silvey.** 'Underground Storage and Retrieval of Fresh Water from a Brackish Water Aquifer'. Second Int. Symposium on Underground Waste Management and Artificial Recharge, New Orleans, 1973.

(7) 'De tunnel voor autoverkeer onder het IJ' (The Road Tunnel under the IJ). Article by **B. Jansen,** De Ingenieur, 3 April 1964, vol. 76, no. 14, and 'Technische aspecten van het terugpompen van grondwater' (Technical Aspects of Groundwater Recharge). Article by **J. Bardet,** De Ingenieur, 20 December 1968, vol. 80, no. 51.

Annex 3A Under-pressure in valves

T.N. Olsthoorn and M.C. Brandes

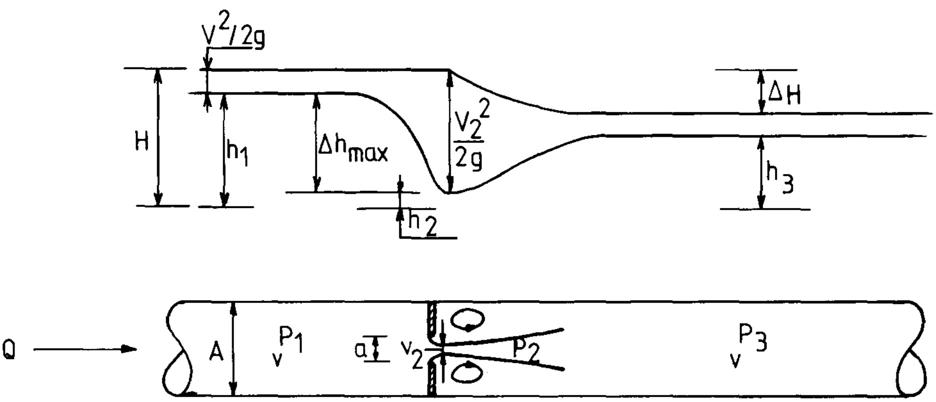
Symbols

A	= pipeline corss-section (m^2)
a	= cross-section of throttled valve (m^2)
μ	= contraction coefficient
v	= average velocity of flow in a cross-section (m/s)
Q	= discharge (m^3/s)
P	= pressure (N/m^2)
ρ	= water density (kg/m^3)
g	= acceleration of gravity (N/kg) or (m/s^2)
ΔH	= energy loss across the valve (m)
Δh	= maximum drop of the pressure head in relation to the pressure head before the valve (m)
K	= power (N)
m	= mass (kg)
t	= time (s)
H	= pressure head (p/g) (m)

Annex 3A Under-pressure in valves

Introduction

The lowest pressure in a partially throttled valve may be calculated in accordance with the principles of elementary hydraulic mechanics. In this respect the valve may be regarded as a caliber plate of cross-section a (m^2) placed in the pipeline.



1 Assumptions

The following laws may be applied:

a. Continuity:

$$Q = v \quad A = v_2 \mu a \tag{1}$$

b. The flow from (1) to (2) may be derived by Bernouilli's equation:

$$\frac{v^2}{2g} + \frac{P1}{\rho g} = \frac{v_2^{\,2}}{2g} + \frac{p2}{\rho g} \tag{2}$$

c. Velocity comparison for the flow from 2 to 3, with the pressure in the vertical cross-section at the point of maximum constriction being assumed to take a hydrostatic course.

$$Kdt = d(mv)\,(P_2\text{-}P_3)\,A = \rho Q\,(v\text{-}v_2) \tag{3}$$

2 Solution

Writing $\frac{P_i}{\rho g} = h_i$, it follows:

(2) and (1) give:

$$h_1 - h_2 = \left\{ \left(\frac{A}{\mu a}\right)^2 - 1 \right\} \frac{v^2}{2g} \tag{4}$$

(3) and (1) give:

$$h_2 - h_3 = 2 \left\{ 1 \quad \frac{A}{\mu a} \quad \frac{v^2}{2g} \right\} \tag{5}$$

(4) and (5):

$$h_1 - h_3 = \left(\frac{A}{\mu a} - 1\right)^2 \frac{v^2}{2g} \tag{6}$$

alternatively one may write:

$$h_1 - h_3 = \Delta H$$

so that

$$\frac{A}{\mu a} - 1 = \sqrt{\frac{\Delta H}{v^2/2g}} \tag{7}$$

(7) and (5) gives:

$$h_2 = h_3 - 2\sqrt{\Delta H . \frac{v^2}{2g}} \tag{8}$$

The occurrence of under-pressure in a valve can be readily calculated by means of equation (8).

3 Examples

1. Beyond a valve $h_3 = 1.5$ m in a pipeline at ground level.
   ΔH amounts to 0.5 m and $v = 1$ m/s:

Applying (8) it follows that:
$h_2 = 1.5 - 2\sqrt{0.5 \times 1/20} = 1.18$ m
The valve is thus not subject to under-pressure.

2. Pipeline at ground level; $v = 1$ m/s; $\Delta H = 28$ m;
h_1 (pressure head before the valve) $= 30$ m.
$h_2 = (30\text{-}28) - 2\sqrt{28 \times 1/20} = 2 - 2.37 = -0.37$ m
In this case there is a limited amount of underpressure. This may be prevented by limiting the pressure drop to 27.5 m, with a negligible effect on the discharge:
$h_2 = (30\text{-}27.5) - 2\sqrt{27.5 \times 1/20} = 2.5 - 2.35 = +0.15$ m.

Annex 3B Gases in groundwater

M.C. Brandes

Annex 3B Gases in groundwater

Groundwater is fed by rainwater and from surface waters such as ditches, polder catchwater basins, brooks and rivers which manage to infiltrate at relatively high points of the water table. Being in direct contact with the atmosphere the infiltrating water contains various dissolved gases found in the atmosphere. According to Henry's Law the quantity of dissolved gas is proportional to the partial pressure of the gas in the atmosphere and the absorption coefficient:

$$c = a.p. \tag{1}$$

where c = concentration of the dissolved gas (mg/l)
a = absorption coefficient (mg/l per atmosphere of partial pressure)
p = partial pressure of the gas (atm).

This formula enables the average concentration of dissolved gases in infiltrating water in normal circumstances to be calculated (disregarding any pollution).

Although infiltrating rainwater or surface water contains virtually no methane, this gas has been included in the above table since it can be a significant component of groundwater.

The concentrations of the dissolved gases alter during the infiltration process as a result of biochemical processes. These processes are primarily confined to upper soil layers, the most important being the oxydation of organic materials present in the soil. This generally uses up the oxygen present, thereby producing considerable quantities of new CO_2. In certain (i.e. anaerobic) conditions, significant quantities of methane can be formed through fermentation processes. There can also be an increase in nitrogen levels, but not generally to the same extent.
The solubility of gases in water varies in proportion to the pressure; this in turn varies directly with the depth of the groundwater. In the lower-lying areas of the Netherlands the water-bearing sand layers are generally covered by clay and peat layers of low permeability. In general the absolute water pressure in sand layers is higher than 2 atm, so that the groundwater in those layers is under-saturated in respect of the dissolved gases, unless significant quantities of new gas have been formed during the infiltration process. This applies particularly to CO_2 and CH_4 and, in certain circumstances, to N_2.

The saturation concentration (c_v) of a gas in water which is **not** in direct contact with the atmosphere (or with another gas phase) corresponds to equation (1):

$$c_v = a.h$$

where h represents the absolute pressure (atm). In the case of groundwater sealed off from the atmosphere, c_v may be obtained directly from Table 1.

Table 1 Principal gases in rainwater and infiltrating surface waters.

Gas	Partial pressure (atm)	Absorption coefficient (at 10°C)	Concentration (mg/l)
nitrogen (N_2)	0.7808	23.2	18.1
methane (CH_4)	–	29.9	–
oxygen (O_2)	0.2095	54.3	11.4
carbon dioxide (CO_2)	0.0003	2345	0.7

Only the CO_2 content is measured in normal groundwater analysis. This may vary from a few mg/l to several hundred mg/l, thus indicating that groundwater is always heavily under-saturated with CO_2. Oxygen analysis is also occasionally carried out; this has indicated that the groundwater in the low-lying areas of the Netherlands generally contains no O_2, while the higher, sandy areas contain no more than 10 mg/l. Groundwater is thus also always undersaturated in respect of oxygen.

Virtually no data exist on the content of nitrogen and methane as dissolved gases. The groundwater in the polder areas of the Netherlands is, however, known frequently to contain significant quantities of methane. As 'marsh gas', methane used to be tapped on a fairly large scale from groundwater for domestic use on farms.

Around 1941 there were some 5,000 gas wells operating in the deep polders of North and South Holland (1, 2 and 3); they may still be seen in many places today.

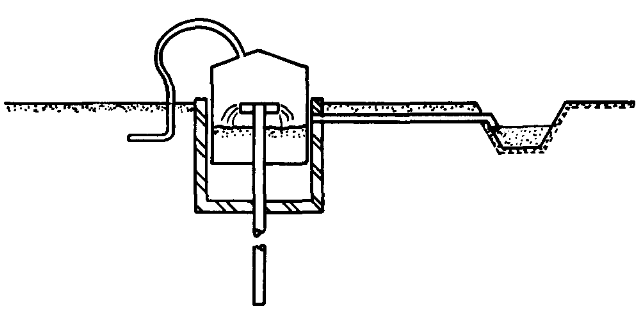


Figure 1 Diagram of a gas well.

A gas well is a well the screen of which is set into a coarse sandy layer beneath covering clay layers. The depth may vary from 10 to 40 m. Gas wells were often sited immediately next to the ditch surrounding a polder, the top of the main being set at 5 to 10 cm above polder level. If the piezometric level of the groundwater is higher than the top of the tube, the groundwater will well up, bringing gas up with it, which can then be caught in a tank or reservoir.

The gas yield is generally 1 m^3 for every 10-15 m^3 of water, while the average water yield is in the order of 4 m^3/h.

Gas wells are found only in deep polders because the groundwater is allowed to well up by itself. In less deep polders the groundwater would have to be pumped up, where upon the process would no longer pay. The presence of methane is not, therefore, restricted to deep polders. The groundwater must, however, be located in an area where the formation of methane is possible, i.e. in conditions where there is a total absence of oxygen and sufficient organic material in the ground in the infiltration area.
At a depth of 20 m the groundwater pressure is around 3 atm, giving a CH_4 saturation concentration of $3 \times 29.9 = 90$ mg/l. Once the water reaches the surface the pressure declines to 1 atm and the saturation concentration to approximately 30 mg/l (see Table 2). This means that 60 mg CH_4/l of water will be released from groundwater abstracted from a depth of 20 m; this corresponds with $60 \times 0.00145 = 0.09$ l gas/l of water or 1 l gas from 11.5 l water. These figures correspond closely with the average gas well yields referred to earlier, so that it may be assumed that the groundwater in deep waters often has a high methane saturation concentration.
It is even possible for the groundwater to be locally over-saturated with methane, thereby producing an underground gas phase. If a water layer of this kind should be drilled into an eruption of methane will follow (1).

Table 2 Saturation concentration of gases in groundwater (10°C and at 1 atm pressure).

Gas	c_v (mg/l)
nitrogen (N_2)	23.2
methane (CH_4)	29.9
oxygen (O_2)	54.3
carbon dioxide (CO_2)	2345

As soon as gas bubbles have formed in the water rising to the surface a gas phase is produced consisting initially of methane alone. This permits other gases to escape as well, until the partial pressure in the bubbles has been brought into equilibrium with the concentration in the water in accordance with equation 1. The process is known as 'scrubbing' and is often applied in the chemical industry in order to eliminate gases from liquids. Due to the entrainment of other gases the quantity of pure methane which can escape is somewhat reduced. The proportions of the various gases released in groundwater may be seen from the average composition of marsh gas abstracted from polders in the Netherlands (1).

Table 3 Average composition of marsh gas.

Gas	%
methane (CH_4)	79-86
carbon dioxide (CO_2)	9-12
nitrogen (N_2)	4-10

When it leaves the gas tank as shown in Fig. 1, the water still contains some 30 mg/CH_4/1. Once the water is discharged into a ditch it comes into contact with the open air and the remaining gas will escape until the concentration is brought into equilibrium with the partial methane pressure in the atmosphere, which is virtually nil. This method of extracting gas is not therefore particularly efficient since approximately a third of the gas in the groundwater is lost.

In a recharge scheme the abstracted groundwater is under a pressure of approximately 1.2 atm in the pipeline system (i.e. 2 m H_2O over-pressure in relation to the atmosphere) and is sealed off from the open air. It may therefore be expected that the quantity and composition of the gas released in a recharge scheme in the right circumstances will broadly correspond with the quantity and composition of marsh gas.

It is conceivable that groundwater may contain sufficient dissolved nitrogen in certain localities for this gas to escape in the same way as it is pumped up, especially since its saturation concentration is somewhat lower than that of methane (Table 2). On the other hand, the quantities of new nitrogen formed in the ground are unlikely to be great (5). The study group did not look into this possibility because it was primarily concerned with the fact that degassing can be a significant factor in recharging schemes, rather than with the actual gases released in that process.

Bibliography

(1) **Voogd, J.G. de. Brongas** (Marsh Gas), Tijdschrift Het Gas (1941), nos. 16 and 17, pp. 213-218, pp. 225-230.

(2) Water, Bodem, Lucht (Water, Soil, Air). Gasbronnen en Gas (Gas Wells and Gas) (1941), No. 5, p. 87 and 93.

(3) **Gruijter, P. de, and E.L. Molt.** Rijnlands boezem, deel III: De hoedanigheid van het boezemwater (Rhineland Catchment Basin, Part III: The Properties of Catchment Water). Hoogheemraadschap Rijnland, Leiden (1943).

(4) **Institute for Land and Water Management Research.** Regional Studies 9: Hydrologie en waterkwaliteit van Midden West-Nederland (Hydrology and Water Quality in Central West Netherlands). Wageningen, 1976.

(5) **Rook, J.J.** Chemische en biochemische aspecten van het in de bodem terugpompen van grondwater (Chemical and Biochemical Aspects of Groundwater Recharge). De Ingenieur (80) 1968, pp. G 88-G 92.

Annex 3C Rate of growth of gas bubbles in water

T.N. Olsthoorn

Symbols

D	= diameter of air bubble	(m)
k	= Botlzmann's constant	(J/mol.k)
γ	= Henry's constant	(mol/J)
T	= absolute temperature	(K)
p_o	= hydrostatic pressure at which liquid is exactly saturated with the gas	(N/m^2)
p_g	= gas pressure in bubble (hydrostatic pressure)	(N/m^2)
β	= gas/liquid diffusion coefficient = 2.10^{-9} at 20°C	(m^2/s)
V	= bubble velocity in relation to liquid	(m/s)
t	= time	
ν	= kinematic viscosity of liquid	(m^2/s)
g	= strength of gravitational field	(N/kg)
$k\gamma T$	= 0.017 at 20°C	(dimensionless)

Annex 3C Rate of growth of gas bubbles in water

Introduction

Clogging of the recharging wells occurred in four of the twenty or so recharge projects examined in the Netherlands. In three of these four cases the cause was methane gas bubbles (see also Chapter 3).
The presence of methane in the groundwater to be abstracted can therefore be of considerable significance for a recharge scheme.

The presence of gases in groundwater and their physical behaviour are described in Annex 3B. This Annex examines the consequences of the physical behaviour of gas for the design of a recharge scheme.

Mechanics of clogging by released gas

The amount of dissolved gas that water can contain varies directly with pressure. At a depth of several dozen metres below the water table, groundwater is subject to a hydrostatic pressure of several atmospheres. As borne out in practice, this means that deep groundwater is able to contain more gas than shallow groundwater.

As groundwater is brought to the surface the hydrostatic pressure drops, so that the water can become over-saturated with gas. If this happens the gas will then separate from the liquid in the form of bubbles.

If these bubbles are carried along by injection water into the recharging well this will produce an accumulation of gas in the gravel pack. This in turn leads to clogging because gas bubbles can only be re-dissolved with great difficulty once they have formed, for the following reasons:

1. As a bubbles grows its surface area (through which gas transmission takes place) declines in relation to its content.
2. This effect is accentuated in that small bubbles join to form larger ones.
3. A considerable amount of time is available for the gas bubbles to be released and to grow, viz. in the pumping well and in the pipeline system (which is generally subject to only low pressure in a recharge system) until it reaches the recharging well.
4. Part of the gas will accumulate in the upper reaches of the gravel pack sur-

rounding the recharging well, where the pressure is relatively low and the rate of solution consequently very slow.

5. The difference in the velocity of the water and the gas is a significant factor for the rate of growth of bubbles. In the pumping well and the pipeline system the velocity of the bubbles in relation to the velocity of flow of the water is the decisive factor; in the gravel pack surrounding the recharging well, where the gas is trapped, the velocity of flow through the filter material is decisive.
   At approx. 0.001 m/s the latter is considerably smaller than the former, which measures 0.01 to 1 m/s.

From this summary it may be seen that the conditions for the release of dissolved gas in a recharge scheme are much more favourable than those for the re-absorption of any bubbles once formed.

Rate of growth of a single gas bubble

The key to tackling the problem of gass clogging lies in an understanding of the rate of growth of gas bubbles. A model for this process has been worked out by Kranenburg in ref. (1). Kranenburg has calculated the quantity of gas that will diffuse out of an ideal, non-viscous liquid into a spherical bubble while the bubble moves through this liquid at a constant velocity. Kranenburg arrives at the following equation:

$$D = 2.3\left[k\gamma T \quad \left(\frac{p_{\cdot}-p_g}{p_g}\right)\sqrt{\beta^V t}\right]^{2/3}$$

For an explanation of the symbols see the list at the end of the Annex.
The velocity v of the bubble in relation to the liquid is a determining factor. In the case of bubbles of between 1 and 10 mm the velocity of flow is between 0.3 and 0.4 m/s. In the case of very small bubbles the following applies:

$$v = \frac{g}{18\nu} D^2 \; (vD \ll \nu)$$

It should be noted that, as the formula indicates, time is required for the bubbles to reach a certain dimension. The drop of pressure in the throat of a throttled valve thus has little effect on the escape of gases. Moreover, the pressure in the throat of a valve is not a great deal lower than that immediately after the valve (see Annex 3A). Of much greater importance are the drop in hydrostatic pressure and the comparatively long period which the water spends under low pressure in the pipeline system.

Example

Groundwater is pumped up and returned to an infiltration well 300 m away. The pressure head in the pipeline system, which is installed on the surface, is 5 m above ground level. The groundwater becomes saturated with methane once the pressure head drops to 10 m H_2O. Once saturation point has been reached the water remains in the rising main of the abstraction well for a further 10 seconds. The average pressure during this 10-second period is 7.5 m H_2O. The velocity of flow in the pipeline is 1 m/s.
If the average velocity of flow of the gas bubbles this formed in relation to the water is put at 0.01 m/s, the size of a single growing bubble may now be calculated.

At the top of the rising main of the pumping well D = 0.1 mm, while at the end of the pipeline before entry into the recharging well D = 1.8 mm.

From this it will be evident that gas and liquid separation does not occur to a marked degree until the water reaches the pipeline system.

Consequences

In practice gas will collect in the upper part of the pipeline, so that a gas layer will develop on the top of the water. It should therefore be a straightforward matter to tap the gas from the pipeline. The pipeline must however be horizontal, preferably rising slightly before the recharging wells so that it can be tapped more easily.
If the water passes through the pipeline system too quickly for the gas to escape properly, a special degassing unit will be needed. Once again this should preferably be positioned as close as possible to the recharging wells and at a higher point. Finer points of such installations are taken up in Annex 10A.

Bibliography

(1) **Kranenburg, C.** Transient cavitation in pipelines. Dissertation, Delft 1974.

Annex 3D Boiling in recharging wells

T.N. Olsthoorn

Annex 3D Boiling in recharging wells

1 Introduction

Boiling (sometimes called piping) means that the infiltration water forces its way directly up to the surface. This occurs if the injection pressure is too high, i.e. if the well is surcharged.

If the injection pressure is too high the ground will fissure or the material with which the borehole has been backfilled will be pressed aside. The water thus acts as a wedge which opens up the fissure as it goes along.

In loose formations (i.e. clay, peat, sand and gravel) this phenomenon will occur wherever the water pressure exceeds soil pressure (although a certain degree of over-pressure may be required in order to open up the particular formation).

Fissuring will occur when grain pressure drops to zero as a result of excessive injection pressure. Grain pressure in the soil is not generally uniform in all directions. The critical point is where grain pressure is smallest, where the soil will fracture (Fig. 1) at right angles to the direction of weakness (Fig. 1).

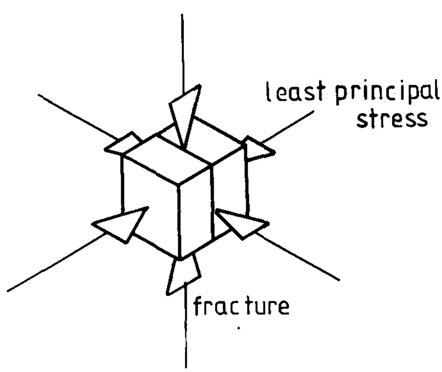


Figure 1 Stress element and preferred plane of fracture.

Since the horizontal grain pressure is generally the lowest in practice, this gives rise to vertical fissures. Grain pressure gets smaller as one goes up, so that fracturing will set in immediately above the gravel pack and seek to force its way through to the surface. Only a small amount of over-pressure is required to produce long fissures

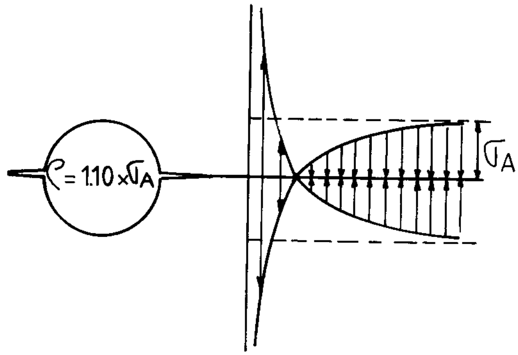


Figure 2 Stresses in vicinity of a crack in a stressed elastic material when pressure acting on walls of crack is slightly greater than stress within the material.

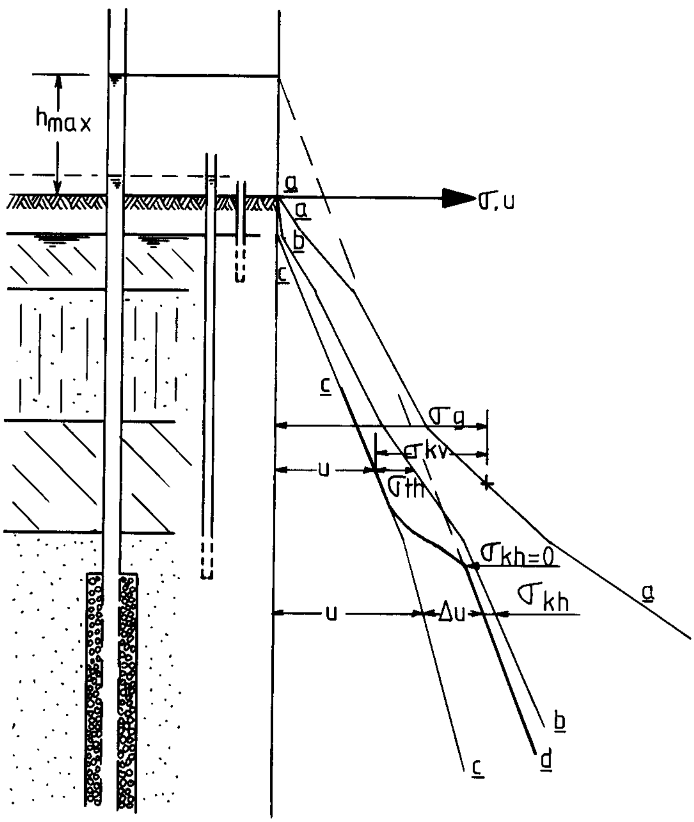


Figure 3.

because the leading edge of the fracture gives rise to peak stresses (see Fig. 2). Figs. 1 and 2 have been taken from ref. (5).

Figure 3 reproduces the situation encountered in many recharging wells. Line a-a indicates the path of vertical soil pressure. This is equal to the weight of the soil and water above a small horizontal plane at an arbitrary spot. The line b-b indicates the path of minimum horizontal soil pressure (= horizontal grain pressure + water pressure) in and around the borehole. Line c-c shows the path of water pressure in the natural undisturbed state. The difference between b-b and c-c is the minimum grain pressure available.
The pattern of water pressure is altered as a result of groundwater injection. The thick line d-d in the diagram represents the situation in which the horizontal grain pressure above the gravel pack has dropped to zero. Any further increase in the injection pressure will result in fissure formation.

From Fig. 1 it is evident that h_{max} is the maximum permissible water level in the well.

Practical experience has shown that water in a surcharged well can reach the surface in various ways. In one well it will rise up beside the header main while in another the water will well up as far as several meters away. Horvat and Boutsma (1967) have described a situation in Rotterdam in which long fissures developed when piling was being jetted into sand, the fissures connecting up the piles in a regular pattern.

2 Impact of various factors on boiling

After the screen and rising main have been installed, a borehole is back-filled with parent material. The compactness of such back-filling varies, as does the extent to which it adheres to the rising main and the wall of the borehole. In all probability the stress within the finished well will differ from that in the original situation. Fresh clay plugs are a tricky matter, particularly if they are directly above the gravel pack and of any length. As the borehole is back-filled the ground pressure in the clay plug rises, but this is to a large extent absorbed by the hydraulic tension. This means that the grain pressure required to prevent boiling is very low in a clay plug, at least immediately after the well has been completed. This situation will improve during the first few months after back-filling.
The Rotterdam Municipal Public Works Department has confirmed the above experimentally by producing boiling upon subjecting new injection wells to a given pressure. The boiling ceased when the pressure was reduced. A few months later the same wells proved able to deal with an injection pressure higher than the initial injection pressure.

The permissible injection pressure is greater the larger the lowest grain pressure immediately above the gravel pack. The grain pressure will in turn be higher:
a. the more carefully the borehole has been back-filled;
b. the shorter the clay plugs;
c. the fewer the clay plugs used;
d. as the clay plugs are given time to consolidate;
e. as the back-filled material is able to settle;
f. the greater the depth below surface level of the gravel pack;
g. the lower the equilibrium piezometric level below the surface;
h. the heavier the back-filling material.

3 Permissible injection pressure

It will be seen from Fig. 3 that the permissible injection pressure (or head), i.e. h_{max} (mH_2O), is determined by the horizontal grain pressure immediately above the gravel pack. How large is that grain pressure?

The size of the vertical grain pressure in the initial situation may be deduced directly from the vertical equilibrium. The size of the horizontal grain pressure is unknown. It is, however, possible to estimate the minimum possible horizontal grain pressure σ_h at a given vertical pressure σ_v by means of Coulomb's Law and Mohr's circle of stress. According to these theories σ_h must not drop below the value at which the circle through σ_h and σ_v touches the containing line of rupture (or shear strength line).

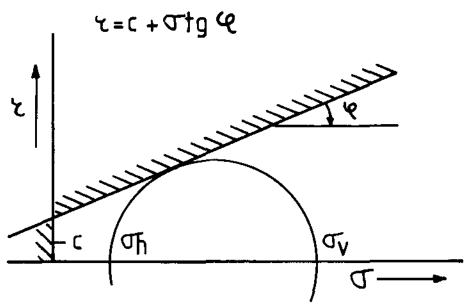


Figure 4.

In the case of sand: φ = 30-40° C = 0 – 0.01 .10^5 Pa
while for clay: φ = 20-30° C = 0 – 0.01 .10^5 Pa

In practice C may be disregarded. If this is done the maximum possible relationship between σ_v and σ_h becomes a constant dependent on the soil type.

$$\sigma_v/\sigma_h = \lambda \leq \frac{1 + \sin\varphi}{1 - \sin\varphi}$$

Given the above data it follows:

$\varphi = 20°\ \lambda = 2$
$\varphi = 30°\ \lambda = 3$
$\varphi = 40°\ \lambda = 4.6$

It follows from the above that the minimum horizontal grain pressure in clay is at least 33% of the vertical pressure while in sand it is at least 22%. By calculating the vertical grain pressure in the original situation at the depth of the top of the gravel pack and using the figures as derived above, the permissible injection pressure may now be calculated. Allowance must however be made for the lower weight by volume of clay and especially peat layers. Typical figures are:

Soil type	*wet density (kg/m³)*
sand	2000 (dry: 1600)
clay	1500-2000
peat	1000-1200

For safety's sake it is advisable to take the minimum values in any calculation.
General calculation:
Vertical soil pressure at depth z_o: $\sigma_g = \int z_o\ \gamma z\ d_z \rightarrow Tg = {}_o\int z_o\ \gamma_z\ d_z$

If the original pore-water pressure at depth $z_o = u$
The original vertical grain pressure at depth z_o: $\sigma_{kv} = \sigma_g - u$
The horizontal grain pressure at this depth is at least: $\sigma_{kh} = \sigma_{kv}/\lambda$

Permissible increase in pore-water pressure above the original value:
$\Delta u = \sigma_{kh} = \sigma_{kv}/\lambda$

Examples:

from 0-5 m clay γ wet $= 1500\ kg/m^3$
γ dry $= 1300\ kg/m^3$
(above groundwater level)

5-15 m peat	$\gamma = 1000\ kg/m^3$
15-20 m clay	$\gamma = 1500\ kg/m^3$
20-22 m sand	$\gamma = 2000\ kg/m^3$

Shallow groundwater level: – 2 m
original deep piezometric level + 2 m

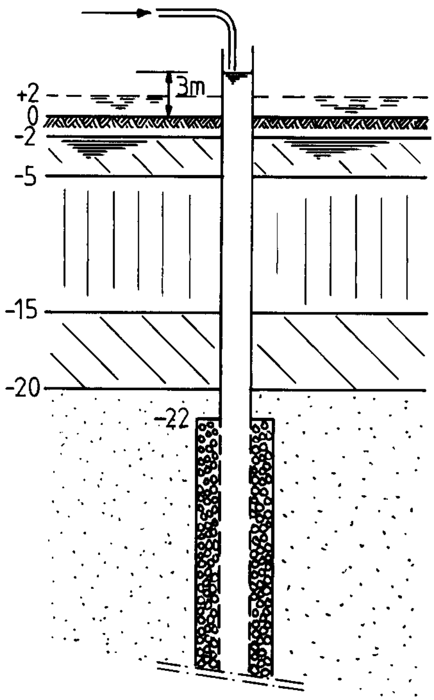


Figure 5. Situation 1

$\sigma_g \quad = g \, \Sigma \, p_i d_i$

$\sigma_g \quad = 10\,(1300.2 + 1500.3 + 1000.10 + 1500.5 + 2000.2) = 28.6\,.10^4$ Pa (= 28.6 m H_2O)

$u \quad = 10\,(24.1000) = 24.10^4$ Pa

$\sigma_{kv} \quad = (28.6 - 24)\,.10^4 = 4.6\,.10^4$ Pa

Soil above backfill is sand = $\sigma_{kh} \geq 0.22\ \delta_{kv}$

$\sigma_{kh} < 0.22\,(4.6\,.10^4) = 1.0\,.10^4$ Pa

In this case the permissible injection pressure is only 10^4 Pa (1 m H_2O) above the original value.

The permissible water level in the well is therefore 2 + 1 = 3 m above surface level.

Same situation except that there is only sand below the upper clay layer:

σ_g = 10 (1300.2 + 1500.3 + 2000.17
41.1 . 10^4 Pa
u = 24 . 10^4 Pa

σ_{kv} = 17.1 . 10^4 Pa
$\sigma_{kh} \geq 0.22\, \sigma_{kv}$ = 3.8 . 10^4 Pa

Permissible injection pressure above original situation:
3.8 . 10^4 Pa (= 3.8 m H_2O).

The permissible water level in the well above ground level is therefore 2 + 3.8 = 5.8 m.

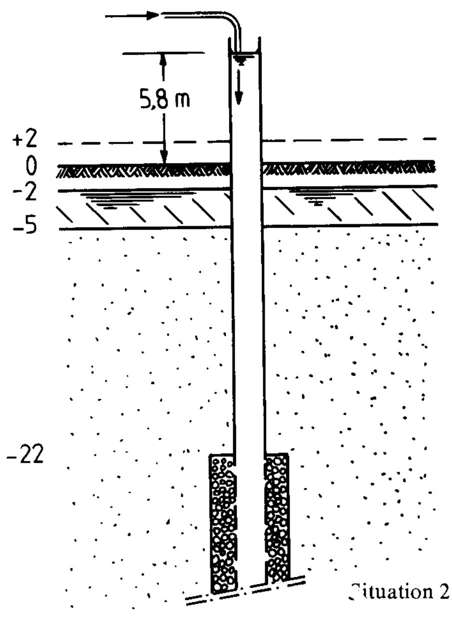


Figure 6.

4 Summary and conclusions

Boiling occurs when a recharging well is surcharged. The condition will be improved by reducing the injection pressure, unless substantial quantities of soil have already been washed away between the rising main and the surrounding soil formation. Despite the lack of certainty concerning the minimum grain pressure in the soil, the permissible injection pressure can still be determined by means of a simple calculation. The permissible injection pressure is very low and the water level in the well or the pipeline must remain confined to no more than a few m H_2O water above ground level. The calculated injection pressure will however only hold good if the borehole is carefully back-filled using as few clay plugs as possible. If clay plugs have been used the injection pressure must initially be kept even lower in order to give the clay plugs time to consolidate.

These consolidation problems can also be prevented by the use of clay-cement or concrete plugs instead of the usual clay plugs.

Bibliography

(1) **Anonymous:** Properties of Soil; Soil mechanics lecture course Part I, Delft University of Technology, c. 1969.

(2) **Nanninga, N.:** Foundation Engineering, part A-B-C; Soil mechanics lecture course, Delft University of Technology, c. 1970.

(3) **Boutsma, K. and E. Horvat:** The advantages and disadvantages of the application of sand drains and preloading at an excavation in Rotterdam. Proceedings of the geotechnical conference, Oslo 1967, Vol. I, pp. 9-11.

(4) **Verruijt, A.:** Commentary on article by Boutsma and Horvat, same proceedings, Vol. II, page 125.

(5) **Hubbert M.K. and D.G. Wills:** Mechanics of hydraulic fracturing. In: Fracture controlled production, AAGP Reprint Series No. 21 Tulsa, Oklahoma 1977, pp. 199-217.

Annex 5A Hydrological and hydraulic aspects of gallery design

T.N. Olsthoorn

Contents

Annex 5A Hydrological and hydraulic aspect of gallery design

1 Summary

Dewatering is often done by means of well galleries. The design of such galleries must take into account the well spacing required in order to achieve the objective of as uniform a drawdown as possible at previously selected points.

Since the impact of individual wells peters out at a distance of 0.5 × the distance of the well from the remainder of the gallery, much larger spacing is (on hydrological grounds) in fact permissible than is generally applied. In the case of galleries of a limited length fixed well spacing is all too often rigidly adhered to. This annex takes up both these points.

The narrow falling mains frequently used in recharging wells have a significant effect on intake capacity and its distribution along a gallery. It was considered desirable to demonstrate this effect, since it can if neglected have adverse consequences in a recharging system. On account of the energy losses involved, it is not advisable for intake capacity to be regulated by means of the falling mains; well siting should instead be optimalized with the aid of hydrological calculations.

2 Comparison of a gallery of infinite length with a trench of infinite length

On the basis of symmetry considerations we may confine our attention to the hatched area in the diagram below. It is also assumed that we are dealing either with a confined aquifer or a semi-confined aquifer in which $\lambda \gg b$, so that the problem may be treated as a confined aquifer problem, which it always is in practice.

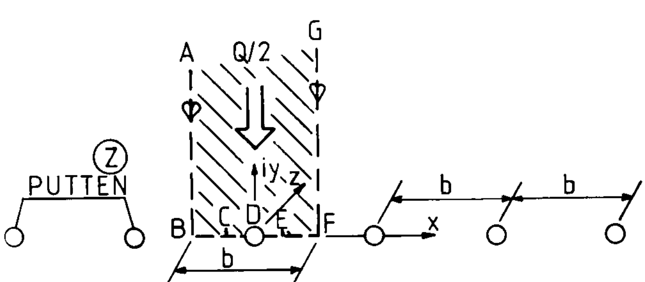


Figure 1.

The solution is obtained by means of complex transformation of the strip A, B, C, D, E, F, G, while if we shift the zero point (see the second term of equation (1)), the solution for larger values for y (the distance from the gallery) transposes into that for a perfect trench of infinite length (equation 2).

2.1 *General solution for the trajectory of the drawdown between A and G*

Solution for the gallery:

$$\Omega = s + i \frac{\Psi}{kD} = \frac{Q}{2\pi\, kD} \ln \sin \left(\frac{\pi (x + iy)}{b}\right) + \frac{Q}{2\pi\, kD} \ln 2 \tag{1}$$

in which s represents the drawdown in relation to the water level in the equivalent trench of infinite length, and the flow function.
Solution for the trench:

$$s = \frac{q}{2kD} y = \frac{Q/b}{2kD} y = \frac{Q}{2\pi\, kD} \left(\frac{\pi y}{b}\right) \tag{2}$$

2.2 *Drawdown trajectory near the gallery along characteristic flow lines*

a. *Path between the wells (trajectory BF)*
In this case y = o, so that (1) changes to:

$$s = \frac{Q}{2\pi\, kD} \ln \left(2 \sin \left(\frac{\pi x}{b}\right)\right) \tag{3}$$

with as characteristic points:
– at the edge of the well ($x = r_o\, b$)

$$s = \frac{Qo}{2\pi\, kD} \ln \left(\frac{2\pi ro}{b}\right) \tag{4}$$

– Half-way between two wells: (x = b/2)

$$s = \frac{Qo}{2\pi\, kD} . (0{,}693) \tag{5}$$

– For x = b/6 : s = o (6)

b. *Trajectory at right angles to gallery*

– line through a well (x = o, z = iy)

$$s = \frac{Q}{2\pi\, kD} \ln (\exp\ (\frac{\pi y}{b} - \exp\ (\frac{-\pi y}{b})) \qquad (7)$$

for y → o, (7) changes to (4)
– symmetry line between two wells; at right angles to well gallery:
trajectory BA and FG:
x = b/2, 2 = b/2 + iy)

$$s = \frac{Q}{2\pi\, kD} \ln (\exp\ (\frac{\pi y}{b}) + \exp\ (\frac{-\pi y}{b})) \qquad (8)$$

for y → o, (8) transposes to (5).

2.3 *Difference between the drawdown trajectory in a well gallery of infinite length and that in an equivalent trench*

Since both (1) and (2) hold in respect of the water level in the equivalent trench, (3) must be the required solution for the trajectory BF between the wells.
In the case of the two calculated trajectories at right angles to the gallery, equation (2) must be subtracted from (7) and (8) respectively in order to calculate the difference. For this purpose (7) and (8) must first be rewritten:

$$s = \frac{Q}{2\pi\, kD} \ln (\exp\ (\frac{\pi y}{b}) (1 - \exp\ (\frac{-2\pi y}{b})) =$$

$$\frac{Q}{2\pi\, kD}\ (\frac{\pi y}{b}) + \frac{Q}{2\pi\, kD} \ln (1 - \exp\ (\frac{-2\pi y}{b})) \qquad (7a)$$

(7a)–(2) yields, for the line at right angles to BF through D:

$$\Delta s = \frac{Q}{2\pi\, kD} \ln (1 - \exp\ (\frac{-2\pi y}{b})) \qquad (9)$$

equally (8a) – (2), for BA and FG:

$$\Delta s = \frac{Q}{2\pi\, kD} \ln (1 + \exp (\frac{-2\pi y}{b})) \qquad (10)$$

Δs very rapidly approaches zero as y increases. For example if $y = b/2$ (i.e. distribution of the impact of the individual wells at less than 45°), Δs varies only

between $\pm\ 0.04 \dfrac{Qo}{2\pi\ kD},$

which is of no consequence. (This may be compared with (5)).

The conclusion thus obtained is that the influence of individual wells may be regarded as fully elapsed at distances of more than half the well spacing from the gallery; the latter will give the same groundwater level as a trench of equal length which fully penetrates the water-bearing stratum and has a constant discharge per unit length.

3 Galleries of limited length

From the above it emerged that lengthy galleries may safely be regarded as trenches. The suggested loss of influence at an angle of less than 45° to the gallery obviously also applies in the case of galleries of more limited length, which may thus also be regarded as trenches. (For a quantitative analysis see (1), p. 57 ff.).

It may be shown that in the case of trenches of limited length in which there is a constant discharge per unit length, the drawdown gets lower towards the ends of the trench, while conversely an increasing discharge per unit length is required towards the ends in order to achieve a constant drawdown along the trench. If a constant drawdown is required this can in practice best be achieved by decreasing the well spacing towards the ends of the trench in accordance with the required discharge per unit length (1). If the original groundwater level was constant, each well would then obtain roughly the same discharge. If, however, the piezometric level is not constant when the wells are not being used a different distribution of the wells will be required in order to obtain a constant drawdown along the gallery together with the same discharge in each well. The optimal distribution of wells can often be calculated by hand as discussed in Annex 8A; alternatively more complicated techniques, such as those discussed in Annexes 8c-8f, may be used.

4 Influence on infiltration capacity of falling mains in recharging wells

It is often necessary for a recharging well to be fitted with a falling main in order to prevent the infiltration water from falling freely into the well, which could give rise to undesired degassing, the transportation of bubbles and aeration, and consequent clogging up of the recharging well.

It is therefore important that the correct dimensions be selected for a falling main. An excessively thin (or long) falling main will produce such frictional losses that the well's intake capacity will drop below the required and calculated level, while an excessively wide main will lead to the problems noted above.

It is sometimes claimed in practical circles that the relationship between infiltration capacity and elevation of the groundwater level is not a linear one. This is incorrect. The absence of a linear relationship between pressure in the infiltration system and groundwater pressure or the infiltration capacity in the wells must be ascribed to the above and other losses in pipelines and appendages (e.g. valves), and not to the incidence of turbulence (or deviations from Darcy's law) in the strata in which the water is being injected.

In the conditions encountered in the Netherlands, turbulence could only be expected to arise – and then only in a narrow band around the wells – at intake capacities some 10 to 100 times higher than those applied in practice.

Given a hydraulic energy head loss H in a falling main (m H_2O) of length L (m), diameter D (m) and a friction coefficient λ in which the discharge is Q (m^3/h), we obtain:

$$H = \frac{\lambda L}{D} \frac{V^2}{2g} = \frac{\lambda L}{2Dg} \frac{Q^2}{\left(\frac{\pi D^2}{4}\right)^2} \qquad ((2))$$

with $\lambda = 0.02$ as a reasonable average for PVC and the usual range for Reynolds number:

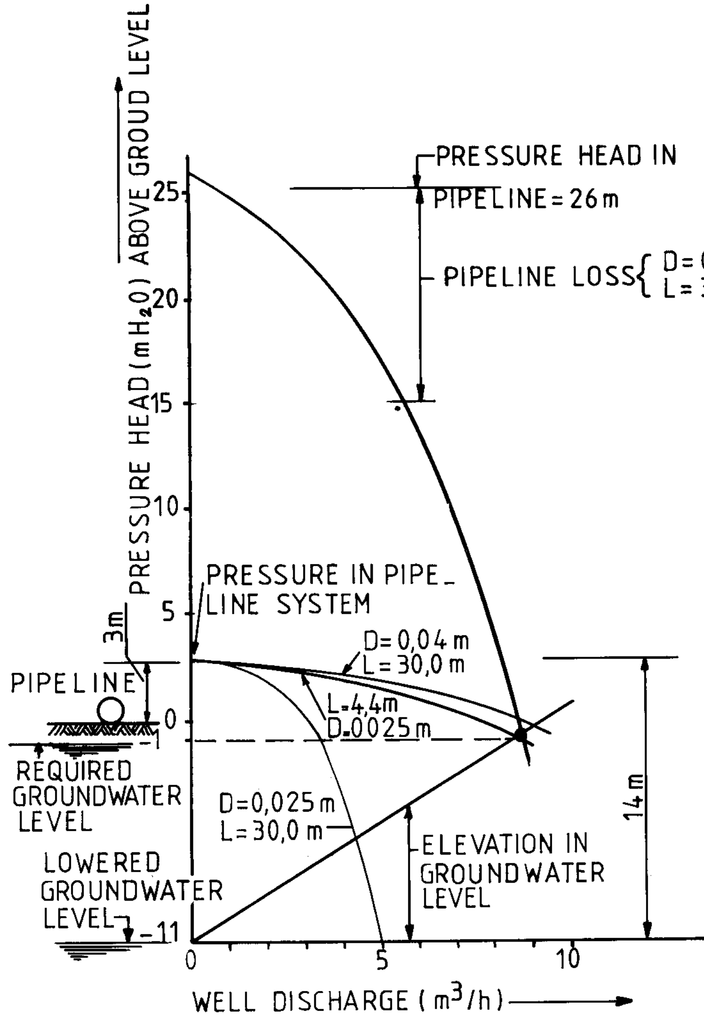
$$\frac{H}{L} = 1.28 \times 10^{-10} \times \frac{Q^2}{D^5} (m^3/h; m)$$

Table 1 shows various values for H/L for selected values of Q and D.

Table 1 Energy head loss per m of falling main (H/L) as a function of discharge Q (m^3/h) and pipeline diameter D (m) for PVC pipes.

D (m) Q (m^3/L)	0.025	0.030	0.040	0.050
5	0.33	0.13	0.03	0.01
10	1.31	0.53	0.12	0.04
25	8.17	3.28	0.78	0.26

The mathematical example below demonstrates the importance of selecting the correct falling main and the influence on the intake capacity of recharging wells.



∞ Long gallery
kD = 1000 m^2/d
λ = 500 m
b = 5 m
Well losses = 0
Individual
Well elevation = 0

Figure 2 Relationship between infiltration discharge, groundwater level, pipeline pressure and selected falling main.

4.1 *Mathematical example of falling mains in a recharging well gallery*

Assume recharging well gallery of infinite length. On account of dewatering the groundwater level has dropped from 1 m to 11 m below ground level. The available pressure head in the pipeline, which is installed at ground level is 3 m H_2O. The total available head (H) therefore amounts to 14 m.kD = 1000 m^2/day, λ = 500 m. The object of the gallery is to return the groundwater to its former level (i.e. an elevantion s_0 of 10 m).
The required discharge per lineal metre is:

$$q = \frac{2s_0\,kD}{\lambda} = 40\ m^3/day.m.^*$$

Spacing between the wells is 5 m.
For each well: Q = 5.40 = 200 m^3/day (8.3 m^3/h).
Assuming a falling main 30 m in length, the required internal diameter is 0.025 m.

Figure 2 shows the relationship between:

Q and the elevation in the groundwater level

Q and the pressure head loss in the falling main (i.e. of the pressure head in the pipeline system proceeding down the falling main) α (imperfections in the wells and own elevation of the wells being disregarded.

The point of intersection indicates the equilibrium situation. From this and further calculations it emerges (see Figure) that the maximum elevation in groundwater level that can be achieved with this falling main and preliminary head is only 5.6 m instead of 10 m.

In order to achieve the required 10 m elevation with these wells and falling mains the pressure head in the supply of pipelines must be increased from 3 m not to 6 m but to 26 m! It is therefore obvious that the relationship between the pressure head in the supply pipeline and infiltration capacity is not linear (although that between infiltration capacity and groundwater elevation is). The required elevation can be achieved at a head of 3 m in the supply pipeline provided that the falling mains have an internal diameter of approximately 0.04 m. Another alternative is to reduce the 0.025 m diameter falling mains in length to 4.44 m. This solution is, however, inadvisable, since the injection water will then fall freely into the well for some time during the starting-up phase of the recharge system (i.e. until the groundwater level rises to the foot of the falling main). Another disadvantage is that a short falling main of this kind cannot be used as an air pipe for clearance pumping by air-lift.

---

* In the case of a finite gallery, q must be multiplied by a factor n, n being a function of the gallery length L and of λ. See (1), pp. 57-60.

5 Falling mains in galleries of limited length

It will be evident that, on account of their hydraulic resistance, narrow falling mains will reduce any variations in the intake capacity of individual wells. The regulation of discharge capacity by this means is not, however, to be recommended, in view of the appreciable energy losses; the objective can instead be achieved by improving the well configuration in the gallery on the basis of hydrological calculations.

Bibliography

(1) **Huisman, L.** Groundwater Recovery, Macmillan Press Ltd., London and Basingstoke, 1972.

(2) **Huisman, L.** Stromingsweerstanden in buisleidingen (Flow resistance in pipelines). Testing and Research Institute of the Netherlands (KIWA), monograph no. 19, Rijswijk 1968.

Annex 5B Practical and financial aspects of a recharging system with a large number of small wells

D. Cense

Annex 5B Practical and financial aspects of a recharging system with a large number of small wells

1 Practical aspects

A system of this kind is based on the use of infiltration screens. In a screen the infiltration wells are located at frequent intervals, in practice generally between 4 and 8 metres apart.

The boreholes for small wells can be drilled by the rotary straight flush method. If this can be done with the aid of a jetting lance able to drill to the required depth in one go, the initial drilling costs will be low. The costs rise beyond a drilling depth of approximately 16 metres, while the method becomes totally impracticable at depths of over 35 metres.
For drilling depths of between 16 and 35 metres the choice of drilling technique will depend on the required number of boreholes.
In order to prevent the borehole from fouling up, clear water should if possible be used. If this is impractical a basin should be installed enabling the abstracted groundwater to be recycled. The basin should be sufficiently large to permit proper settlement of suspended soil particles brought up in drilling.
One practical solution is to excavate a ditch beforehand alongside the drilling points, the capacity of the ditch being at least two to three times as large as the volume of soil to be extracted.
Boreholes are constructed with a diameter of 20 to 30 cm; a constant diameter can be achieved by chasing the drill up and down several times while drilling.

When the drill is withdrawn from the borehole the latter must be carefully 'kept full' in the usual manner for a borehole without casing. The screen and rising main (with a minimum diameter of 50 mm) are then lowered into the borehole. Central positioning of the screen section can be ensured by means of centering brackets.
The screen section is then surrounded by a gravel pack graded according to the size of the screen perforations and the soil composition.
The following points should be taken into account in fitting the screens to the 'discharge' system:

a. stagnation in the infiltration process must not be allowed to lead to stagnation at the dewatering stage;
b. excessive infiltration pressure can lead to irreversible damage to the recharging systems if the wells become subject to boiling.

Both these aspects can be safeguarded by the installation of an overflow or control manifold designed to discharge into surface water or into a sufficiently large sewer. Care should, however, be taken to avoid siphoning or the sucking in of air.

The installation may be properly monitored in the following way:
a. Regular readings in observation wells.
b. Regular observations of the intake capacity per group of infiltration wells (e.g. with the aid of water meters).
c. Regular observations of the pressure at various points in the recharging pipeline system.

These measurements can if necessary be automated so that any manfunctioning will be automatically registered.

The following points are important for the successful operation and ease of maintenance of the system:
a. The system should be air-tight.
b. The screens should be properly de-aerated when first put into use and after each stagnation.
c. The supply pipeline should be of a generous size so as to ensure constant injection pressure.
d. Provision should exist for fitting devices to clean the screens, e.g. by means of air-lifting.
e. It should be possible to close off the wells.
f. With a view to possible frost damage, the system should not contain pipeline sections in which there is no flow.

2 Financial aspects

There follows a comparison of financial aspects in relation to systems in which 'large' wells are used. Those parts of a recharging scheme which are common to both a large and a small well (i.e. wellpoint) system, such as the feeder pipelines and measuring equipment, have been left out of consideration. This section instead examines the differences between the two methods, namely installation (including the different dimension materials used in the boreholes) and maintenance of the two types of recharging wells.

1 *Positioning of recharging wells*

1.1 The drill diameter required for drilling a large well is twice that required for a small well.

1.2 The most common drilling method for large wells is the reverse rotary, while for small wells or wellpoints the straight flush method is commonly used.

1.3 The diameter of the screen and header main installed in the borehole differ. In general the filter and header main diameter used in large wells is some four times as large as that in small wells. The volume of the gravel pack used in large wells is at least three times as large as that required in small wells.

1.4 Maintenance of the two types of wells.

Maintenance of large wells generally consists of cleaning the screen by means of clearance pumping, while in the case of small wells the screen is cleaned by means of air-lift. This means that a greater number of small than large wells can be cleaned per man-hour.

It should be noted that even if the drilling conditions are identical, the cost variations resulting from the above factors will not always be the same from case to case. The following example drawn from practice provides an illustration of the size in the variations. It should, however, be interpreted with a certain amount of caution.

a. The ratio of drilling, installation and connecting up costs for large as against small wells is 5:1.
b. The ratio of maintenance costs of large as against small wells is 10.1.
c. The capacity of large wells is twice that of small ones. This at least proved to be the case in a system subject to clogging up; twice as many wellpoints as large wells were required in order to inject an equal quantity of recharge water. The overall installation costs of large as against small wells was therefore in a 2.5:1 ratio.

   Overall maintenance costs in this case for large and small wells were in the ratio of 5:1.

If we assume that in most cases the installation of a system of large conventional wells will amount to 25% of total installation costs, and maintenance to another 15%, this will mean that the total costs of a recharge system using small wells will be 73% of the total costs required in the case of large, conventional wells.

Annex 5C Dimensions of recharging wells with a view to air-lift clearance pumping

T.N. Olsthoorn
Computer calculations by
A.N.G. de Vogel

Contents

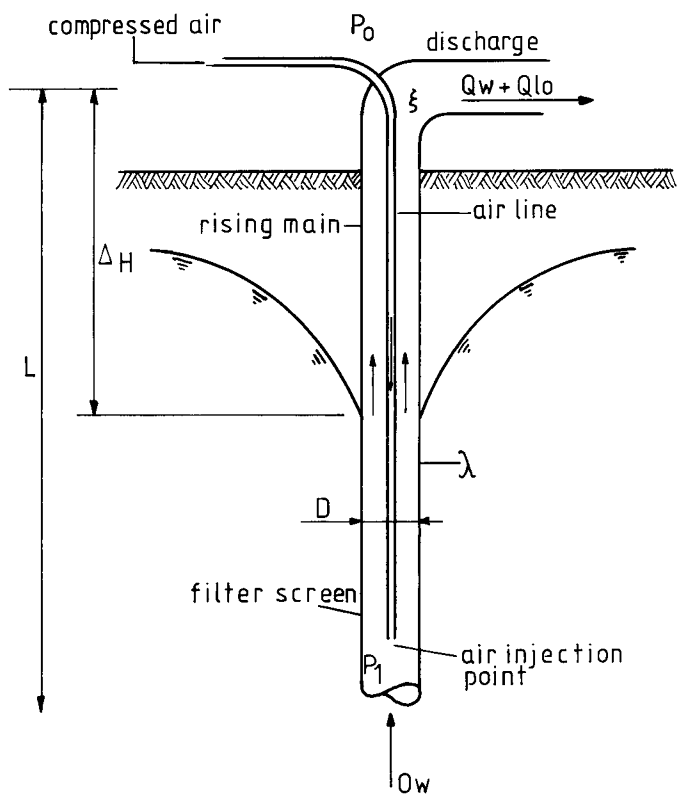


Figure 1 Symbols.

Symbols

p_1	= energy in the injected air	(W)
p_p	= gain in potential energy per second of water brought to the surface	(W)
p_s	= slippage resulting from friction between air bubbles and water	(W)
p_W	= losses due to friction between wall of pipe and air-water mixture	(W)
P_k	= kinetic energy of water leaving the well per second	(W)
p_b	= power losses in appendages and bends in the discharge pipe outlet	(W)
Q_w	= water discharge	(m^3/s)
Q_l	= air discharge	(m^3/s)
Q_{lo}	= air discharge at the outlet (atmospheric pressure)	(m^3/s)
p_0	= atmospheric pressure	(Pa)
p_l	= absolute pressure at point of injection during air-lifting ($p_l - po + \rho_w g (L - \Delta H)$	(Pa)
ρ_w	= density of water	(kg/m^3)
g	= gravitational force	(m/s^2 or N/kg)
L	= length of discharge pipe from injection point to top of pipe	(m)
ΔH	= distance between lowered piezometric level and top of discharge pipe	(m)
ε	= H/L (relative lift)	(–)
V_l	= air volume of all bubbles during air-lifting	(m^3)
F	= cross-section of discharge pipe minus air line	(m^2)
O	= inside circumference of discharge pipe plus outside circumference of air line	(m)
D	= hydraulic diameter $D = 4.F/O$	(m)
ξ	= bend friction coefficient before outlet	(–)
λ	= pipe wall friction coefficient	(–)
u	= rising velocity of bubbles in relation to surrounding water	(m/s)

Annex 5C Dimensions of recharging wells with a view to air-lift clearance pumping

1 Summary

Recharging wells are generally cleaned out by the injection of compressed air at a depth of 20-30 metres. The air bubbles present above the injection point reduce the weight of the water in the well to such an extent that it is forced out of the well by the deeper water, with a more or less constant air-water flow being built up. This technique of pumping is known as air-lift. In practice the simplicity of this type of 'pump' often more than compensates for its lower efficiency in relation to electrically driven pumps.

It may be noted at the outset that the force with which the air-water mixture blows out of the well is not an indication of the extent to which the well will be cleaned. The effectiveness of air-lift clearance pumping depends on the velocity with which the water strikes the side of the borehole and, in some cases, the jolting effect obtained as individual bubbles are dislodged at the point of injection. In terms of practical relevance we may confine ourselves to the velocity of the water against the side of the borehole.

The (filter) velocity against the side of the borehole is equal to the well intake capacity divided by the surface area of the outside of the gravel pack, i.e. it depends on the diameter of the discharge pipe and filter screen. On the other hand the discharge that can be achieved by means of air-lift is, because of such factors as friction inside the pipe, heavily dependent on the diameter of the discharge pipe (in the case of small diameters in proportion to that diameter to the 5th power!). This Annex accordingly concludes that with a view to air-lift clearance pumping, the diameter of the rising main (i.e. discharge pipe) in a recharging well should be as large as possible in relation to the borehole.

2 Calculation of an air-lift

Accurate calculation of an air-lift is based on an energy balance in which the force of the injected compressed air is balanced against the sum of the loss items and the increase in the kinetic and potential energy of the water.

A rigorous calculation in which Reynolds number and similar factors are taken into

account is complicated and can only be satisfactorily carried out by computer (see bibliography).
In practice, however, the following formula can effectively be used for determining whether a given compressor is large enough to pump the required quantity of water per unit of time from a given well.

The energy balance is as follows (see also the list of symbols at the beginning of this annex):

$$p_l > p_p + p_s + p_w + p_k + p_b$$

The various items in the balance are arrived at as follows in the literature:

Injected air

$$p_l + p_o Q_{lo} \ln \frac{p_l}{p_o}$$

Per second of water lifted to the surface

$$p_p = p_w g \, \Delta H \, Q_w$$

Slippage

$$p_s = \frac{V_l}{FL}(p_l - p_o)\, uF \approx \varepsilon\,(p_l - p_o)\, uF$$

The relationship between ε and u has been empirically determined (see Figure 2).

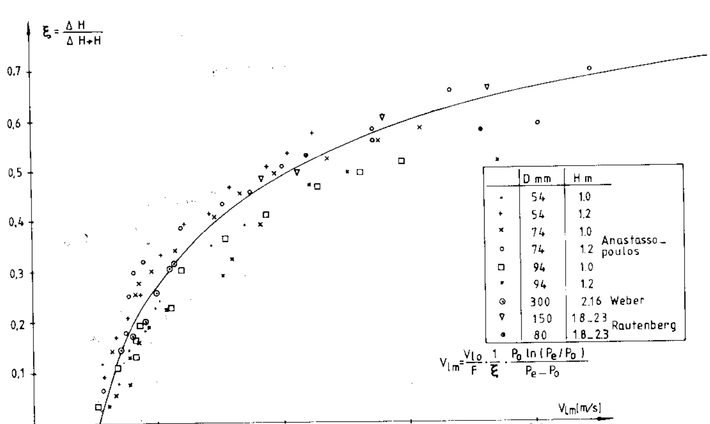


Figure 2 Relative air-bubble velocity as a function of relative lift = H/L (from (3) 1972).

Kinetic energy and discharge loss

The discharge loss is equal to a factor (depending on the form of the outlet) ξ times the kinetic energy of the water discharged. ξ may be looked up in textbooks on hydraulics (e.g. (1)).
Because it goes to waste, the kinetic energy is a loss item. Taken together:

$$p_b + p_k = (\xi + 1) \quad \frac{p_w Q_w}{2F^2} (Q_w + Q_{lo})^2$$

Pipe-wall friction

An approximation of pipe-wall friction may be obtained by taking a value for Q_l in relation to the total discharge $Q_w + Q_l$ lying between that at the air injection point

$$(Q_l = \frac{p_o}{p_l} Q_{lo})$$

and that at the point of discharge (where $Q_l = Q_{lo}$).
The precise equation is (2):

$$p_w = \frac{\lambda L}{D} \quad \frac{\rho_w Q_w}{2F^2} \quad (Q_w + Q_{lo} \sqrt{\frac{po}{pl}})^2$$

$\lambda = f\,(\text{Re and } \frac{D}{K})$ (1). In rough terms 30 Re_{water} can be taken for Re, with Re_{water} representing the Reynolds number $\frac{(vD)}{\upsilon}$, (ignoring any air present).

3 Mathematical example

It is required to examine whether a particular compressor is able to pump up a pre-determined amount of water from a given well.

Data

Compressor	:	capacity 2 Nm^3/min (normal cubic metres of air per minute) $Q_{lo} = 2/60 = 0.033\ m^3/s$ maximum pressure: 7 bars (= 7.10^5 Pa)
Well	:	PVC discharge pipe and screen: Ø 160 × 147 mm PVC air line: Ø 32 × 28 mm foot of air line: 25 m below top of discharge pipe pumping out of clogged well: 2 m / 25 (m^3/h)

Groundwater : groundwater state at rest: 4 m below top of discharge pipe
Required discharge: 100 m^3 water/h (0.028 m^3/s)
Atmospheric pressure taken as 1 bar (10^5 Pa).

Calculations:

1. Hydraulic diameter:
   This is the diameter of a pipe with the same relationship between cross-section and inner pipe circumference as the wet cross-section of the combination of the discharge pipe and air line:

   in the case of a pipe: $\frac{F}{O} = \frac{\pi D^2}{4 \pi D} + \frac{D}{4} \rightarrow D = 4 \frac{F}{O}$

   while for the air-lift:
   $F = \pi (0.147)^2 - \pi (0.032)^2 = 0.016 \text{ m}^2$
   $O = \pi (0.147) - \pi (0.032) = 0.562 \text{ m}$
   so that the hydraulic diameter is:
   $D = \frac{4.\ 0.016}{0.562} = 0.115 \text{ m}$

2. $\frac{\rho_w Q_w}{2F^2} = \frac{1000.\ 0.028}{2.\ (0.016)^2} = 54.3\ .10^3$

3. $\Delta H = 4 \text{ m} + \text{drawdown at } 100 \text{ m}^3/\text{h} = 4 + 8 = 12 \text{ m}$
4. $P_l = po + \rho_w g\,(L\text{-}\Delta H) = 10^5 + 10^4\,(25\text{-}12) = 2.3\ .10^5$ pa

ENERGY BALANCE

1. Energy injected by compressor:
   $pl = 10^5\ .\ 0.033 \ln\left(\frac{2.3}{1}\right) = 2750$ Watts
2. Required for lifting 100 m^3 water/h:
   $p_p = 1000.10.12.\ 0.028 = 3300$ Watts.

In other words $pl < p_p$ so that this compressor could certainly not achieve a pumping lift of 100 m^3/h.

Analysis of whether same compressor can pump up 50m³ water/h

1. P_l remains unchanged 2750 W
2. P_p becomes halved 1680 W

3. p_s: ΔH now becomes $4 + 8/2 = 8$ m
   so that $\varepsilon = 8/25 = 0.32$ from which it follows from Fig. 2: $u = 0.6$ m/s.
   $p_s = 0.32\,(2.3-1).\,10^5.\,0.016.\,0.6$ — 400 W
4. Bend loss and kinetic energy
   For an average bend it may be assumed that $\xi = 0.5$ so that $p_b + p_k = (0.5 + 1).$
   $\dfrac{54.3.10^3}{2}\,(0.014 + 0.033)^2 = 90$ W
   (for concrete examples see ref. (1)).
5. Wall friction
   In the case of PVC a representative value is $\lambda = 0.02$

$$P_w = \frac{0.02.25}{0.115} \quad \frac{54.3\,.10^{+3}}{2} \quad (0.014 + 0.033 \quad \frac{1}{2.3})^2 = 150 \text{ W}$$

Aggregated:	injected air:	2750 W
	Sum of losses and energy gains	2280 W

On the basis of these relatively straight forward calculations it may therefore be seen that 50 m^3 of water can be abstracted from the well with this compressor.

4 Impact of the discharge-pipe diameter on air-lift capacity

On the basis of the formulas given in (2) a computer program has been written which, after input of the necessary data, will calculate the relationship between the amount of water discharged and the amount of air injected. The drawdown in the well is kept constant in the program, as would be the case if the discharge pipe were standing in open water. The relationship between Q_w and Q_{10} can then be determined for a series of drawdowns.
The cluster of curves obtained in this way is therefore solely a function of the dimensions of and frictional losses in the air-lift pipes. The actual relationship between Q_w and Q_{10} for a given well will also be dependent on the soil properties and degree of clogging in the well. This relationship may be obtained by determining the discharge on each curve consistent with the drawdown for which the curve was drawn.
A number of curve clusters are shown in the following figures. The figures differ only with respect to the hydraulic diameter of the discharge/air line combination; both are made of PVC with a friction coefficient of 0.025.
The figures next to the individual curves indicate the groundwater level beneath the top of the discharge pipe during pumping. The foot of the air line is set at a depth of 29 m, in line with the fact that recharging wells are commonly constructed to a depth of 30 m.

The last figure shows the relationship between water discharge and air injection for wells of differing hydraulic diameter in a given formation. The curves have been derived from the other figures by assuming that a 1 m drawdown is achieved in the wells at a discharge of 10 m^3/h and that, in the absence of pumping, the groundwater level reaches the top of the discharge pipe.
These assumptions are not critical but have simply been taken for illustrative purposes. The user can instead apply the actual normal groundwater level/drawdown, provided, however, that the depth of the air line is reasonably close to 29 m as assumed here and that the coefficient of friction within the PVC tube is similarly in the order of 0.025.

5 Conclusion

The figures clearly reveal the major impact of the hydraulic diameter on the discharge that can be achieved. The main reason for these variations are the varying frictional losses. In terms of clearance pumping and the ability to obtain a high discharge level, therefore, wide-diameter pipes are indicated.

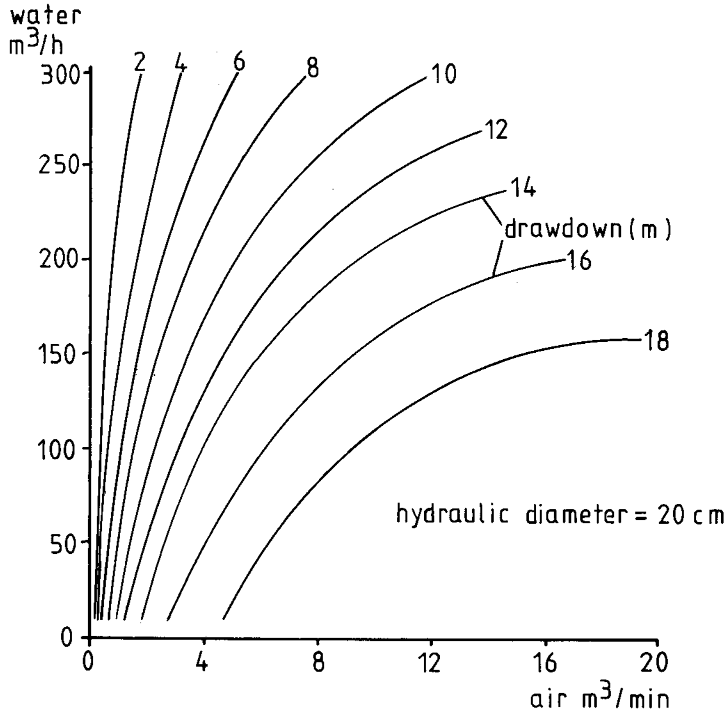


Figure 3 Relationship between water and air discharges during air-lift pumping. For various drawdowns in m, injection depth = 29 m, λ = 0.025.

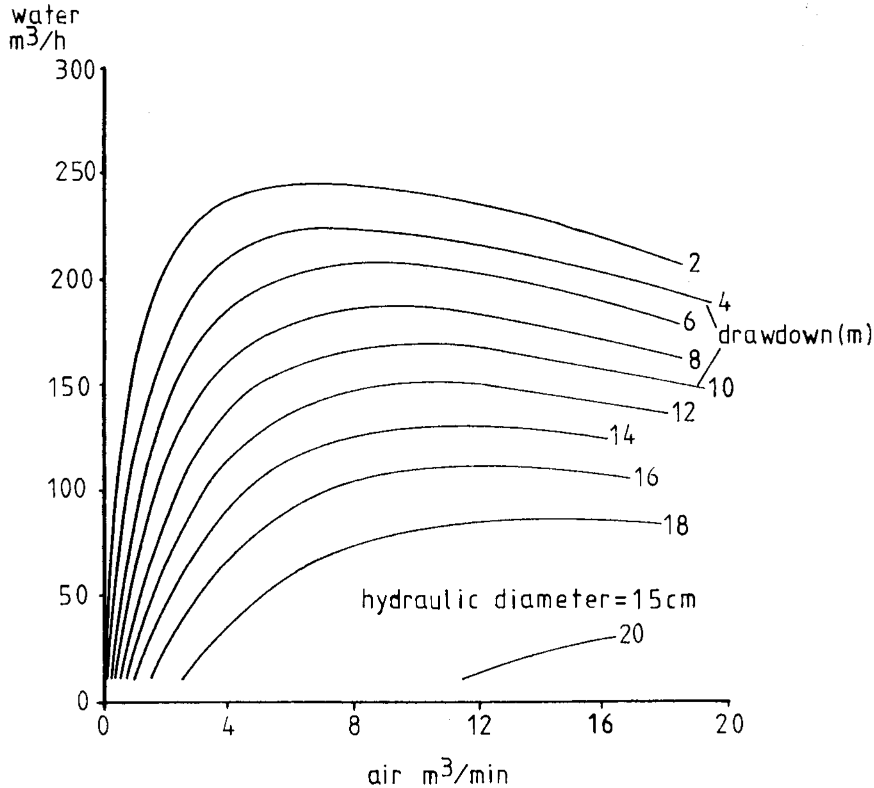


Figure 4 Relationship between water and air discharges during air-lift pumping. For various drawdowns in m, injection depth = 29 m, λ = 0.025.

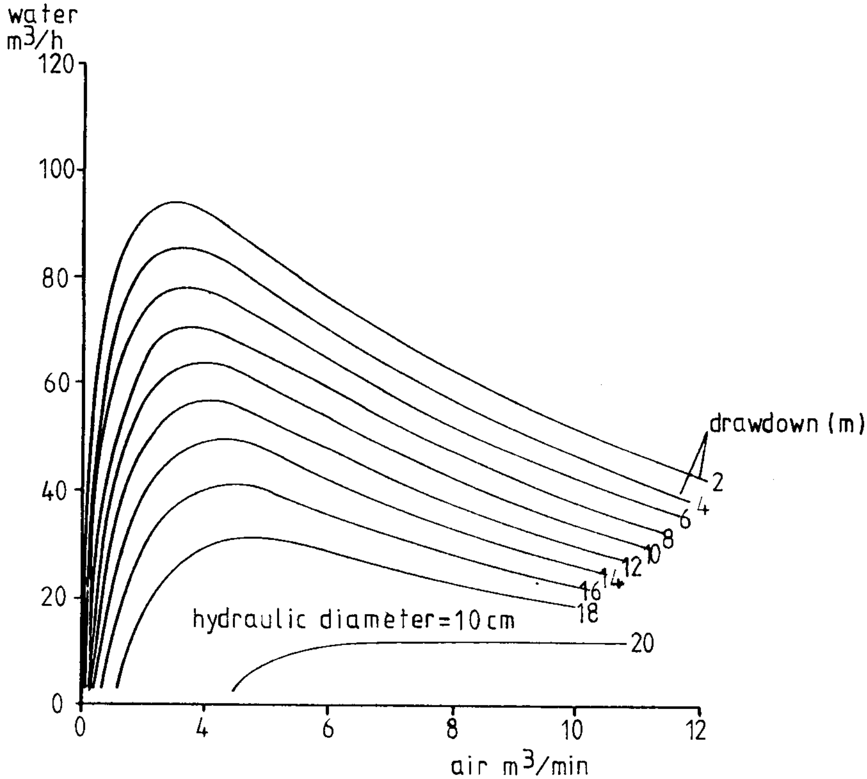


Figure 5 Relationship between water and air discharges during air-lift pumping. For various drawdowns in m, injection depth = 29 m, λ = 0.025.

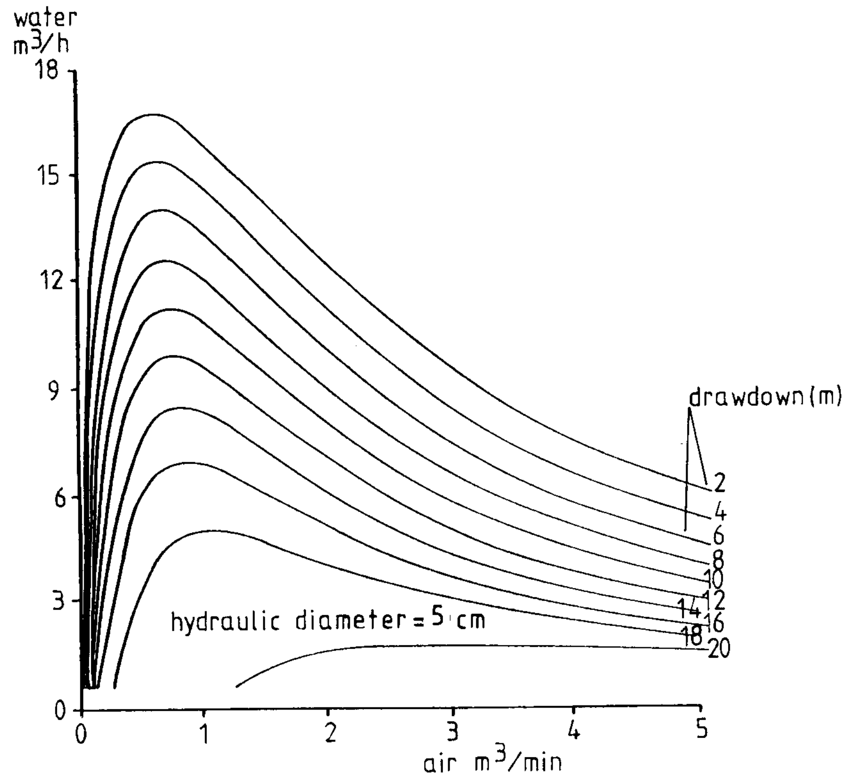


Figure 6 Relationship between water and air discharges during air-lift pumping. For various drawdowns in m, injection depth = 29 m, λ = 0.025.

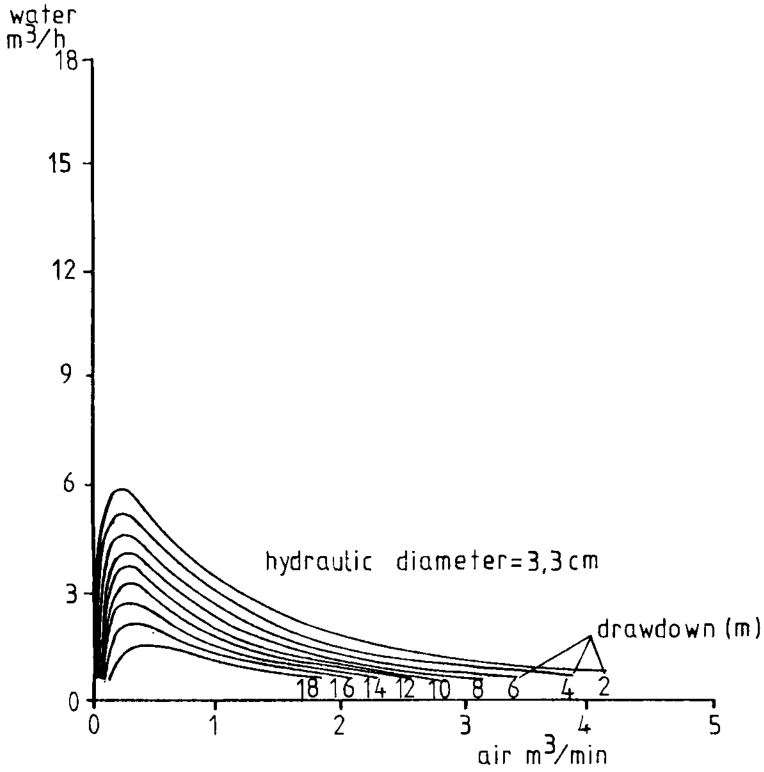


Figure 7 Relationship between water and air discharges during air-lift pumping. For given drawdowns in m, injection depth = 29 m, λ 0.025.

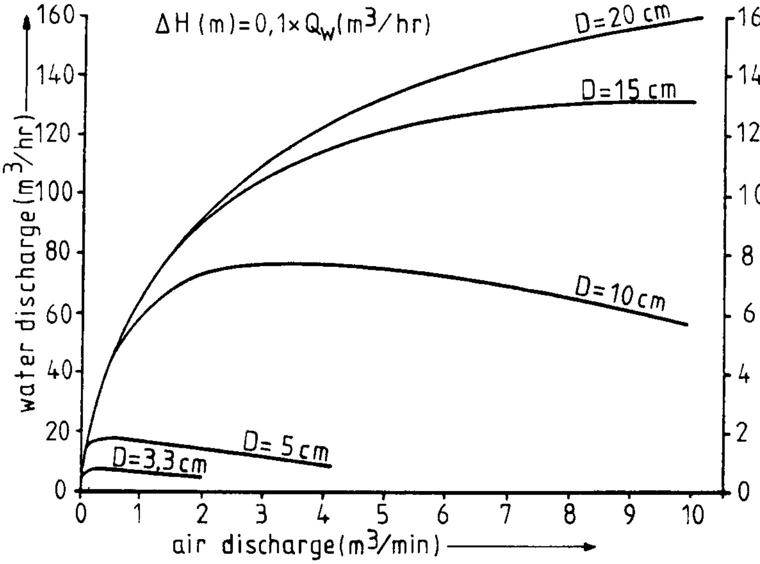


Figure 8 Relationship between water and air discharges for various hydraulic diameters of the discharge pipe, where $\Delta H(m) = 0.1 \times Q_w\,(m^3/hr)$

Bibliography

(1) **Huisman, L.** Stromingsweerstanden in buisleidingen (Flow Resistance in Pipelines). KIWA communication no. 14, Rijswijk, 1969.

(2) **Olsthoorn, T.N.** Het berekenen van een luchtlift (Calculating an Air-lift). KIWA report SWE-192, Rijswijk, 1978.

(3) **Rautenberg, J.** Theoretische und experimentelle Untersuchungen zur Wasserförderung nach dem Lufthebeverfahren (Theoretical and Experimental Investigations into Water Abstraction by Air-lift). Thesis, Karlsruhe, 1972.

(4) **Schmid, W.L.H.** Over de werking van de luchtlift voor water (Use of Air-lift Pumping for Water). Dissertation, Delft, 1930.

Annex 6A Gas test

M.C. Brandes

Annex 6A Gas test

Introduction

As a result of the drop in pressure, degassing may occur in groundwater as it is pumped up in a recharge scheme. Small bubbles will form in the water, which can then cause a certain degree of clogging in the recharging wells. The aim of the gas test is to enable an impression to be obtained quickly of the likely degree of degassing.

Theory

The theory of bubble formation during degassing is discussed in Annexes 3C and 10A. The following data are sufficient for the design of the test.

A certain amount of time is required for the formation of gas bubbles. This depends among other things on the drop in pressure. The pressure in the pipeline at surface level in a recharging scheme is approximately equal to atmospheric pressure. The time required for bubbles 1 mm in diameter to form is therefore:

$$t \sim \frac{40}{p} \text{ (seconds)}$$

where p = hydrostatic pressure in the aquifer (bars).
In the case of groundwater pumped up from a depth of 10 m, the time required is therefore $t = 40$ seconds. Because the time spent by the water in the pumping well and rising main is only brief, the testing appliance must have a flow-through time of at least 40 seconds.

Bubbles 1 mm in diameter have a rising velocity in relation to the surrounding water of at least 0.1 m/s. Section 3 describes a very simple test in which part of the pumped up water is injected downwards into a 1 litre flask. The velocity of the water must be regulated in such a manner that the gas bubbles can escape readily ($V \ll 0.1$ m/s).
In the case of a 1 l flask a retaining period of 40 seconds implies a maximum rate of through-flow of 1.5 l/minute.
Given a flask diameter of approx. 10 cm, the vertical velocity of the water will have to be around 0.003 m/s if the bubbles are in fact to escape readily.

The actual quantity of dissolved gas depends on various factors and the test outlined in this annex is only designed to determine whether degassing is likely or not; it does not enable any quantitative determination of the gas content of the water to be made.

3 **Implementation** (see Fig. 1)

3.1 *Equipment*

a. 1 litre glass flask with dual inlet stopper for two hard plastic (PE) tubes of ø 4 × 6 mm, the inlet tube just penetrating the stopper and the outlet tube reaching the bottom of the flask.
b. plastic 2 litre measuring flask.
c. connecting tubes and coupling.

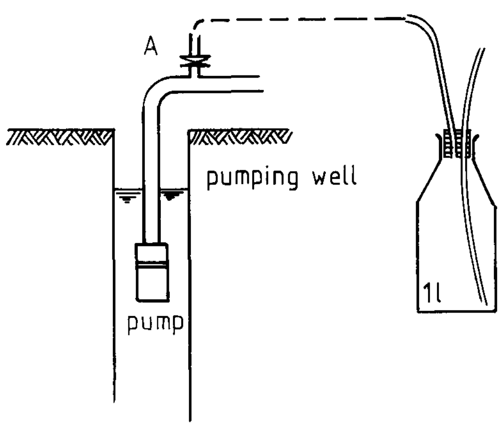


Figure 1 Gas test design (not to scale).

3.2 *Mode of operation*

a. switch on pump and set discharge from tap A at 1-1.5 l/minute
b. fill flask with water and seal with stopper
c. connect inlet tube to A and allow water to flow through flask for approx. 15 minutes.

Annex 6B Filter test

M.C. Brandes

Annex 6B Filter test

Introduction

In the oil industry membrane filter tests are commonly conducted in order to examine the suitability of a particular type of water for underground injection. The most common of these are Millipore membrane filters. These are manufactured out of a mixture of cellulose acetate and cellulose nitrate, with pores of 0.45 μm. The extreme fineness of the pores means that all solid particles (even bacteria) in the water are screened out. Membrane filters thus clog up much more readily than the natural soil formation into which the water is to be injected.
There is no quantitative relationship between the speed with which a membrane filter clogs up and that of an injection or recharging well, but experience indicates that membrane filter tests do provide a qualitative indication of the suitability of the water for injection purposes. Another advantage of the rapid clogging is that an accurate reading can be quickly obtained even in the case of extremely low particle concentrations.

The Testing and Research Institute (Waterworks Undertakings), KIWA, of Rijswijk in the Netherlands has conducted extensive research into the best means of conducting filter tests (1). This research has resulted in the definition of a membrane filter index (MFI) (2). This index has been applied in the case of the filter test discussed below so that the results may be directly compared with those of the KIWA surveys.

Theory

According to Millipore the flow through a membrane filter may be described as follows:

$$Q = \frac{dV}{dt} = \frac{P.A}{\eta} \left(\frac{1}{r_f + r_c}\right) \tag{1}$$

where:

Q = discharge ($m^3.s^{-1}$)
V = through-flow volume (m^3)

t = time (s)
ΔP = pressure drop across filter (Pa)
A = through-flow surface (m^2)
η = dynamic viscosity (Pa.s)
r_f = membrane filter resistance (m^{-1})
r_c = resistance of filterbed (m^{-1})

Equation (1) has been derived from Poiseville's Equation (cf.ref. (4)). During filtration the trapped particles form a filter bed on the membrane, the resistance (r_c) of which depends on the properties and concentration of the particles in the water. In the case of a given concentration of a given type of particle it may be assumed that the resistance rises in proportion to the through-flow volume:

$$r_c = V. \frac{S}{A} \tag{2}$$

in which S is a constant characteristic of the clogging properties of the water (labelled 'Silting Index' by Millipore).
Combining (2) with (1) produces:

$$\frac{dt}{dv} = \frac{\eta}{P.A} \quad (rf + \frac{V.S.}{A})$$

and after integration:

$$t = \frac{\eta}{\Delta P.A.} \; (rf.\, V + \frac{V^2 S}{2A})$$

$$\frac{t}{v} = \frac{\eta.rf}{P.A} + \frac{\eta.s}{2\Delta P.A^2}.V \tag{3}$$

Equation (3) is the formula for a straight line in a graph in which t/V is plotted against V, the slope being:

$$b = \frac{\eta}{2\, P.A^2}.S \tag{4}$$

which is solely determined by:
a. the clogging properties of the analysed water (the constant S) and
b. the conditions under which the test was carried out, viz.
  - the viscosity,, which varies with temperature;
  - the pressure drop across the filter;
  - the surface area of the filter.

The membrane filter index (MFI) is now defined as the slope (b) obtained if the experiment is conducted under standard conditions, i.e.

$$(MFI) = \frac{\eta o}{2\Delta\ P_o . A^2_o} .S \tag{5}$$

The standard conditions are: η_o = dynamic viscosity of water at 10°C = $1.31.10^{-3}$ Pa.S

$\Delta Po = 2.10^5$ Pa (c. 20 m H_2O)

$Ao = 13.8.10^{-4}\ m^2$

The standard filter surface area (Ao) is the through-flow surface area of a 47 mm diameter Millipore membrane filter used in a Millipore filter holder (Swinnex or Inline 47).

The slope of the straight line in the graph may be written as:

$$b = \frac{t_2/Vs - t1/V1}{V2 - V1} \tag{6}$$

in which V1 and V2 are two measurements of the through-flow volume at times t1 and t2 respectively. Combining (4), (5) and (6) yields the general formula for the membrane filter index:

$$(MFI) = \left(\frac{\eta_o}{\eta}\right) . \left(\frac{\Delta P}{\Delta P_o}\right) . \left(\frac{A}{A_o}\right)^2 . \left(\frac{t2/V2 - t1/V1}{V2 - V1}\right) \tag{7}$$

If the measurements are taken under standard conditions (i.e. $\eta = \eta_o$; $\Delta P = \Delta P_o$ and $A = Ao$), equation (7) becomes:

$$(MFI) = b = \frac{t2/V2 - t1/V1}{V2 - V1} \tag{8}$$

The standard Millipore filter in which $A_0 = 13.8.10^{-4}\ m^2$ may be used for recharge research. At a depth of 7 m below ground level the groundwater in the Netherlands is at a virtually constant temperature of approx. 10°C, so that it may be assumed that $\eta = \eta_0$. As may be seen from Table 1 variations of a few degrees from the average groundwater temperature have little effect on the index. The most important variation from test to test is the pressure under which it is conducted.

For the purposes of research into the suitability for recharge of a particular type of groundwater, equation (7) can therefore be reduced to:

$$(MFI) = \frac{\Delta Po}{\Delta P} . \left(\frac{t2/V2 - t1/V1}{V2 - V1}\right) \tag{9}$$

Table 1 Dynamic viscosity of water.

Temperature (0°C)	Viscosity x 10^3 (Pa.s)
5	1.52
6	1.47
7	1.43
8	1.39
9	1.35
10	1.31
11	1.27
12	1.23
13	1.20
14	1.17
15	1.14

To simplify the test t is measured in seconds and V in litres. This also produces handy values for the MFI, which may then be expressed as ($s.l^{-2}$). The pressure drop ΔP can be expressed in any desired unit, provided that $\Delta P_0 = 2.10^5$ Pa is expressed in the same units.

Research in the Netherlands has indicated that the filter test does not always entirely follow equation (3). At the beginning of the test the absence of a filter bed allows extremely fine particles to get caught in the filter pores. When this happens the filter resistance (r_f) rises until the point at which a filter bed is built up, after which the particles can no longer get through to the pores. It is from this point onwards that a straight line is obtained in accordance with equation (3), the slope of which determines the MFI. In certain tests the results also deviated from the straight line towards the end of the test when the filter bed became compressed, as indicated in Fig. 1. Deviation produced by clogging filtration occurs particularly in relatively clean water and takes longer if a test (with the same water) is conducted with a lower pressure drop.

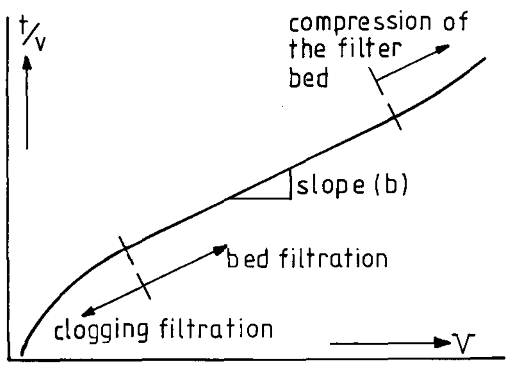


Figure 1 General relationship between t/V and V.

3 Practical experience

Little practical experience has so far been acquired with MFI determinations in respect of recharge systems. For this reason other experience with so-called 'pressure' wells must be drawn on. (The difference between recharging and pressure wells is that in the former, groundwater is pumped back, while in the latter surface water, either treated or untreated is used.)

Very clean water (e.g. tap water) has an MFI of 0.5-1 s/l^2. Highly treated surface water has an MFI of c. 8 s/l^2 and causes little clogging in a pressure well. Surface water which has been only partly treated has an MFI of 34 s/l^2 and results in rapid clogging.
A number of measurements made in **anaerobic** groundwater produced MFI values of approx. 0.5 s/l^2. In the case of one recharging test a value of 2.2 s/l^2 was initially recorded in a pumping well that had just been drilled in anaerobic conditions; one week later this value had fallen to 0.5 s/l^2. There was no measurable increase in the resistance of the recharging well during the 12 weeks over which the test was conducted.

In the case of a recharge scheme in **aerobic** groundwater with a very low iron content, MFI values of 2.7, 0.6, 2.5 and 8 s/l^2 were obtained in four pumping wells. The last, comparatively high, figure was presumably caused by the presence of slurry in the abstracted water (as shown by grey discoloration of the filter). This case does not provide a solid basis for assessing the filter test because air was able to enter the falling main in the recharging wells, with consequent gas clogging.
During the initial months of the recharging test conducted as part of project 8 (see Chapter 11), the following general results were obtained:
- deep groundwater (iron-bearing): 2.5—4.5 s/l^2
- shallow groundwater (mixed water): 15 —40 s/l^2

Finally it should be noted that the particular batch from which the membrane filters are taken and which way up they are placed in the holder can have some effect on MFI readings. These differences are, however, relatively small, and of little account in the general assessment of the suitability of groundwater for injection.
To sum up, it may be said that water with an MFI of 8 s/l^2 or less is suitable for injection purposes. If the MFI is significantly higher, clogging problems are likely to be encountered. It is not, however, possible to lay down a precise cut-off value. In the case of a well that has just been drilled the MFI will often be higher during the first few days than the value at which it ultimately settles.

It should be borne in mind when conducting a filter test that a low MFI does not necessarily provide a guarantee against clogging. In the first place the composition of the abstracted water can alter over time as a result of the dewatering of the surrounding area. Secondly the test can only provide indications of possible clogging by suspended particles in the water. Another form of clogging – admittedly a rare one – is described in the report on the recharge test in Chapter 11. Installation defects can also result in clogging, e.g. through the admission of air beyond the measuring point.

4 Implementation

4.1 *Equipment* (see Fig. 2)

a. Millipore membrane filters (HAWP 4700; 0.45 μm)
b. Millipore Swinnex filter holder (SX00 04700)
c. manometer 0.40 mH_2O
d. tap (open and shut)
e. 2 2-litre measuring flasks (plastic)
f. stopwatch
g. connection tubes and couplings
h. thermometer

(Further research will be needed to indicate whether a reducing valve is required in order to maintain constant pressure on the filter.)
Membrane filters and holders are obtainable from:

– Millipore Nederland
Ambachtstraat 6B
3732 CN De Bilt
tel. 0.30-764744

– Millipore Benelux
Heliotropenlaan 10
B-1030 Brussels
tel. 3222-158818

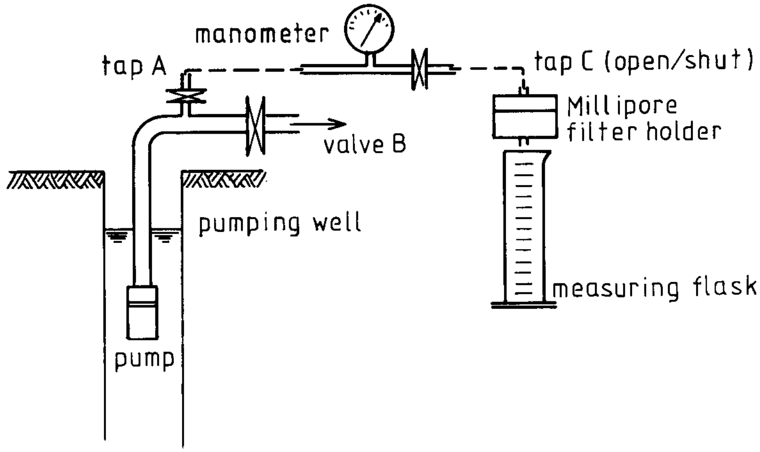


Figure 2 Circuit diagram of filter test (not to scale).

4.2 *Operating instructions*

1. Switch on pump and allow to pump to waste until water is completely clear. If necessary switch pump on and off several times. Thorough clearance pumping will be required for new wells.
2. Connect equipment to A.
3. Regulate pipeline pressure with valve B (tap A should be fully open and tap C closed). Ideally a pressure of around 20 mH_2O should be obtained, but the test can still be conducted with differing pressures.
4. Insert membrane filter in filter holder, taking particular care that the space above the filter is properly de-aerated (by inverting the inflow section of the holder).
5. Place filter holder over measuring flask.
6. Open tap C and start stopwatch (t = 0).
7. Note elapsed time for each litre of filtered water.
8. The measurement series must be continued until such time as t/V plotted against V has become a straight line. In the case of very clean water 20 litres or more will have to be filtered. If clogging properties are present in the water the discharge will tail off quickly and the series can be terminated after a few litres.
9. Plot the observations on a graph chart after the first series and check that they form a straight line. The duration of the test can be adjusted (either shorter or longer) as appropriate for the second set of measurements and if necessary the interval between measurements can be reduced (e.g. every 0.5 or 2 l).
10. Each test should produce two comparable sets of results. If necessary a third set of measurements should be taken.
11. Note the pressure and temperature of the water in each test.
12. Carefully inspect the membrane filter after each test. Any white patches on the filter indicates the presence of air bubbles, which reduce the flow-through area, in which case the test should be repeated.
13. Note colour and any special features of the material screened out by the filter.
14. Number filters and retain for further laboratory testing as required.

Bibliography

(1) **Schippers, J.C., et al;** Five years experience with a reverse osmosis plant for the treatment of River Rhine water. Proceedings of the 6th international symposium, Fresh Water from the Sea, Las Palmas 17-22 September 1978, Vol. 3, p. 370-372.

(2) **Olsthoorn, T.N.;** Use of Membrane Filter Tests in clogging Research in Pressure Wells). KIWA report, ROPP 78-21, June 1978.

(3) **Millipore Filter Corp.; Silting Index Apparatus,** Principles and operating instructions, p. 8.

(4) **Ives, K.J.;** Filtration through a porous septum. Proc. Inst. Civ. Engre., 17 (1970), 333-338.

Annex 8A General calculation of total abstraction and infiltration discharges

M.C. Brandes
T.N. Olsthoorn

Annex 8A General calculation of total abstraction and infiltration discharges

Introduction

In this section the total discharges from the dewatering and recharge systems (Q_1 and Q_2 respectively) in the test problem have been calculated by means of fundamental equations (which may be obtained from any textbook on groundwater mechanics). These equations do not, however, permit the optimal location of the wells to be calculated; this must be done with the aid of more sophisticated techniques (see following Annexes).
In order to apply these fundamental equations, the excavation and the building to be protected must first be schematized into circles (see Figure 1).
The radii of the circles in the diagram (r_1 and r_2) have been selected so that their circumference is equal to the perimeter of the two rectangles (see appendix to this Annex):

$$r_1 = \frac{a+b}{\pi} = \frac{500}{\pi} = 159 \text{ m } r_2 = \frac{c+d}{\pi} = \frac{300}{\pi} = 95 \text{ m}$$

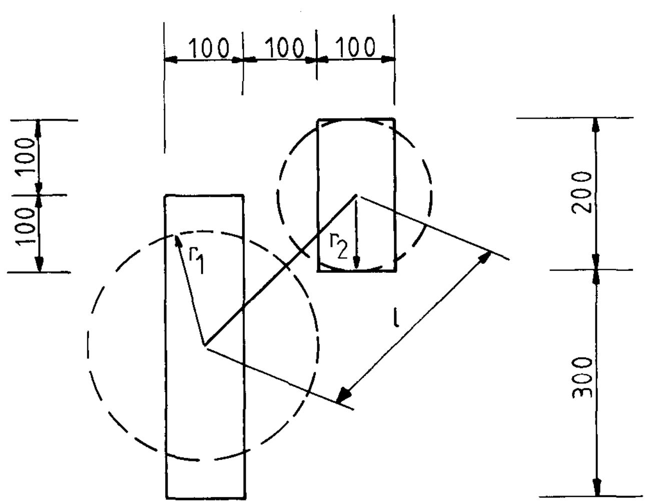


Figure 1.

The distance between the centres of the two circles (l) is $200\sqrt{2} = 283$ m.
The drawdown (s) in a water-bearing formation in a semi-confined aquifer at a distance r from the pumping well may be obtained as follows:

$$s = \frac{Q_0}{2\pi kD} K_0 \left(\frac{r}{\lambda}\right) \qquad (1)$$

(kD – permeability factor (m^2/day) and λ = characteristic length = $\sqrt{kDc}$ (m)). If the circles are regarded as wells, the condition for (1), viz. that the diameter of the borehol ($2r_0$) should be small in relation to λ, no longer holds. In these circumstances formula (2) must be used (see ref. 1, pp. 101-103):

$$s = \frac{Q_0}{2\pi kD} \frac{k_0 \left(\frac{r}{\lambda}\right)}{\left(\frac{r_0}{\lambda}\right) K_1 \left(\frac{r_0}{\lambda}\right)} \qquad (2)$$

Since the drawdowns s_1 and s_2 have been laid down for the two circles, the required total discharges can now be found from the following set of 2 equations with 2 unknowns.

$$s_1 = \frac{Q_1}{2\pi kD} \frac{K_0 \left(\frac{r_1}{\lambda}\right)}{\left(\frac{r_1}{\lambda}\right) K_1 \left(\frac{r_1}{\lambda}\right)} + \frac{Q_2}{2\pi\, kD} \frac{K_0 \left(\frac{l}{\lambda}\right)}{\left(\frac{r_2}{\lambda}\right) K_1 \left(\frac{r_2}{\lambda}\right)} \qquad (3)$$

$$s_1 = \frac{Q_1}{2\pi kD} \frac{K_0 \left(\frac{r_1}{\lambda}\right)}{\left(\frac{r_1}{\lambda}\right) K_1 \left(\frac{r_1}{\lambda}\right)} + \frac{Q_2}{2\pi\, kD} \frac{K_0 \left(\frac{r_2}{\lambda}\right)}{\left(\frac{r_2}{\lambda}\right) K_1 \left(\frac{r_2}{\lambda}\right)} \qquad (4)$$

The following data have been filled in:
r_1 = 159 m; r_2 = 95 m; l = 283 m; λ = 1000 m; kD = 750 m^2/d

x	$K_0(x)$	$K_1(x)$	$xk_1(x)$
r_1/λ = 0,159	1,98	6,09	0,97
r_2/λ = 0,095	2,48	10,42	0,99
l/λ = 0,283	1,43	3,26	–

See also (1), p. 147, where s_1 = 10 m and s_2 = 0 m.
With the aid of the above data Q_1 and Q_2 may be calculated from (3) and (4):

Q_1 (abstraction) = 39570 m^3/day
Q_2 (infiltration) = 23330 m^3/day

Substitution in the calculation of a circle for a rectangular perimeter with a constant drawdown

In order to apply equations adapted for axially symmetrical cases of groundwater flow, the rectangles with a constant drawdown must be replaced by circles.
In order to replace a square with a circle, a radius may be selected which is the mean of that calculated for the area:

$$\pi r^2 = ab \rightarrow r = \sqrt{\frac{ab}{\pi}}$$

and that calculated for the circumference:

$$2\pi r = 2(a + b) \rightarrow r = \frac{a + b}{\pi}$$

(see Figure 2).

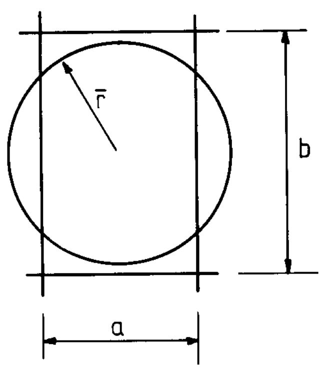


Figure 2.

In the case of a rectangular shape, however, best results are obtained with a radius calculated from the perimeter.

The above calculations have also been verified by simulation on carbonized paper (see also Annex 8C). The results are summarized in the tabel 1, although this substitution technique should be used with a certain amount of caution.

Table 1 Calculated and measured equivalents of radius $\bar{r}$ for the substitution of a circle for a rectangle with sides a and b for discharge calculation purposes. The length of side a has been taken as 1 unit.

a x b	$\frac{1}{2}\left\{\frac{ab}{\pi}+\frac{a+b}{\pi}\right\}$	$\frac{a+b}{\pi}$	$\bar{r}$
1 x 1	0.60	0.64	0.59
1 x 3	1.13	1.27	1.26
1 x 5	1.59	1.91	1.89
1 x 7	2.02	2.55	2.48
1 x 9	2.44	3.18	3.07

Bibliography

(1) **Huisman, L.** Groundwater Recovery. Macmillan Press Ltd., London and Basingstoke, 1972.

Annex 8B Specimen calculations for a recharge system (analytical calculations)

H.M. Haitjema

Contents

Annex 8B Specimen calculations for a recharge system (analytical calculations)

1 Introduction

It is assumed that, for certain reasons, the impact of a dewatering scheme must be confined within certain limits. This means that the dewatering system and associated recharge scheme must not be allowed to result in any variations in the piezometric level outside the 'zone of influence'. It must therefore be determined where the recharging wells should be sited, and what their intake discharges should be. A customary approach is for the number of recharging wells and their required discharge to be planned on the basis of the anticipated drawdown. The precise location and discharge of individual wells can then be optimalized by means of groundwater elevation calculations.

In the technique discussed in this Annex, however, the location and discharge of the recharging wells are solved directly from the boundary conditions to be satisfied. In line with the customary practice in groundwater mechanics, recharging wells have been termed *sources* and pumping wells *sinks*.

The method discussed below is only suitable in the case of water-bearing formations where there is no leakage. The method can, however, be used to provide an initial approximation for semi-confined aquifers. This approximation can then be checked for accuracy by calculating the drawdown at a number of critical points with the aid of well formulae for semi-confined aquifers (1).

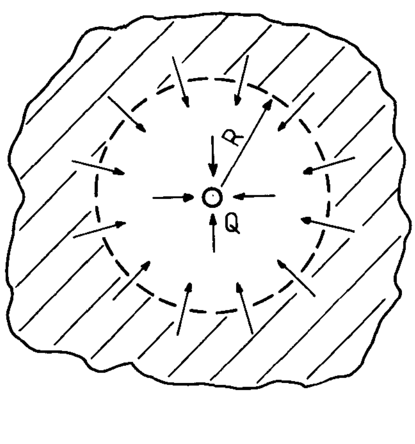


Figure 1.

2 Circular zone of influence around a sink

The shaded area in Fig. 1 is to be kept outside the zone of influence of the sink. The zone of influence boundary may therefore be taken as the potential line for the unshaded area. For the purpose of calculating the drawdown, this area may thus be regarded as a circular island of radius R with a sink at (or near) the centre. If a discharge Q is abstracted from the formation by the well, a quantity of water

$$q = \frac{Q}{2\pi R} \tag{1}$$

will flow out of the shaded area, per unit of length, towards the sink across the zone of influence boundary. If a specific discharge is infiltrated along the zone of influence boundary equal in volume to the discharge profile, the flow from the hatched area will be neutralized. This will maintain a constant groundwater elevation outside the zone of influence, with the total intake discharge being equal to the total abstraction discharge. If infiltration should be lower by a fraction ΔQ, the resultant effect on the hatched area can be calculated by assuming there to be a sink with a tapping capacity ΔQ in the absence of any recharge.
In practice the specific discharge will often be discretized into n sources. The impact of such discretization may, however, be disregarded at a distance with

$$\sigma = \frac{\pi R}{n}$$

3 Sink with a straight zone of influence boundary

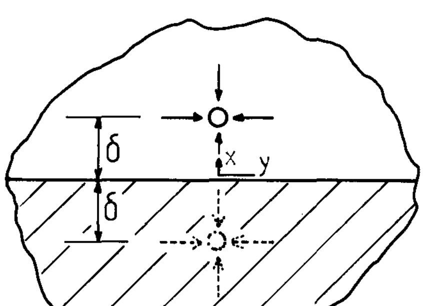


Figure 2.

In Fig. 2 the x-axis has been taken as the zone of influence boundary. The water-table elevation is required to be kept constant in the shaded area. The x-axis is therefore the potential line. The flow outside the shaded area may be solved by means of a mirror source. It may now be determined how much water flows across the x-axis from the shaded area. As in section 1, a specific discharge may be applied along the x-axis with the same discharge profile as that for the mirror source. The flow in the shaded area in relation to the sink will then be zero, so that the groundwater elevation in that area will be undisturbed.

Consider the complex potential function $\Omega = \varnothing + i\,\Psi$, in which the stream function Ψ may be derived as follows:

$$\frac{\partial\psi}{\partial x} = + q_y \tag{2}$$

$$\frac{\partial\psi}{\partial y} = - q_x$$

The complex potential function in the case of Fig. 2 may be expressed:

$$\Omega_{(z)} = \frac{Q}{2\pi}\{\ln(z + i\delta) - \ln(z - i\delta)\} \tag{3}$$

in which $z = x + iy$.

The stream function may then be expressed as

$$\psi = \text{imaginary}\,(\Omega) = \frac{Q}{2\pi}.\quad\{\text{arctg}\;(\frac{x}{y+\delta}) - \text{arctg}\;(\frac{x}{y-\delta})\}$$

Along the x-axis ($y = O$):

$$\psi_{(y=o)} = -\frac{Q}{2\pi}\,\text{arctg}\;\{\frac{2\delta x}{(x^2 - \delta^2)}\} \tag{4}$$

In order to obtain the discharge crossing the x-axis, q_y may be solved along the x-axis ($y = 0$):

$$q_y = \frac{d\psi}{dx} = -\frac{Q}{2\pi}.\quad\frac{d}{dx}\,[\text{arctg}\;\{\frac{2\delta x}{(x^2 - \delta^2)}\}]$$

This reduces to:

$$\frac{d\psi}{dx} = \frac{Q}{\pi}\,\frac{\delta}{(x^2 + \delta^2)} \tag{5}$$

Equation (5) gives the discharge profile required along the zone of influence boundary (x-axis) in order to neutralize the impact of the sink in the shaded area in Fig. 2. This discharge profile has been shown graphically in diagram 1 with positive values for x. The dimensionless variables x/δ and $\frac{\pi.\delta}{o} \cdot \frac{d\psi}{dx}$ or, following equation (2), $\pi.\delta \cdot q_y$, have been plotted along the coordinate axes.
In diagram 2 the infiltration discharge Q* between $-x/\delta$ and $+x/\delta$ has been plotted against the absolute value of X/δ.

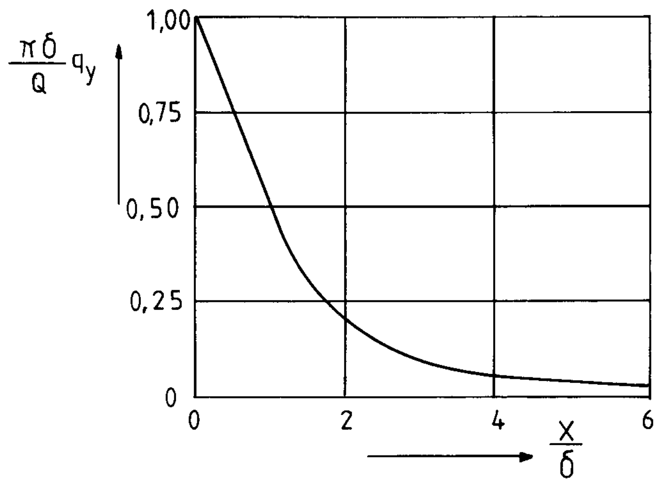


Diagram 1

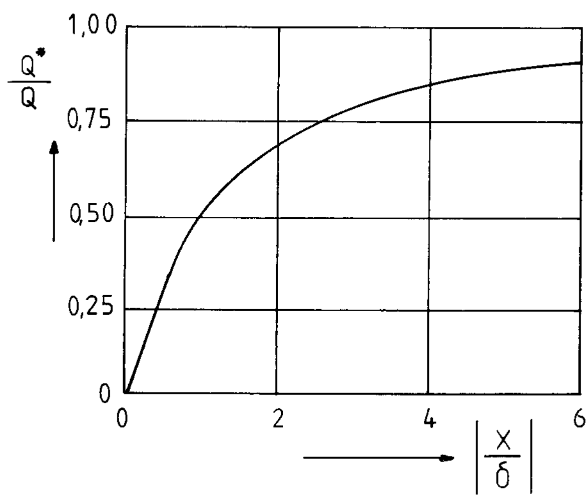


Diagram 2

On the basis of diagrams 1 and 2 it may be assumed that in practice it will generally suffice to take a specific discharge between $x = -6\delta$ to $x = +6\delta$. The total infiltration discharge will then be approximately 90% of the abstraction discharge. If this discharge should be any lower the impact on the groundwater elevation in the hatched area can be calculated as indicated in section 2.

4 Sink with two parallel straight zone of influence boundaries

No direct calculations with the aid of mirror sources can be made for the zone of influence boundaries (potential lines) in Fig. 3. With the aid of the Schwarz-Christoffel formula, the area z in Fig. 3 may be mapped onto a half plane in which the two potential lines (DB) are presented as a single line (see Fig. 4).

The transformation formula is:

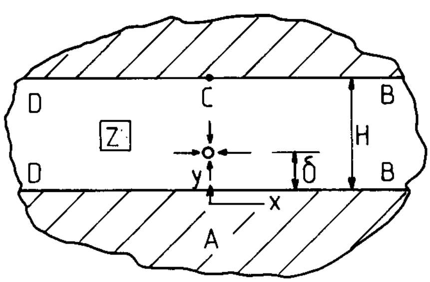


Figure 3.

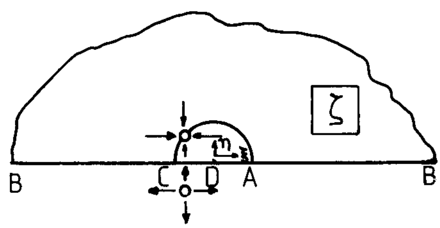


Figure 4.

The complex potential in the ζ area is:

$$\Omega(\zeta) = \frac{Q}{2\pi} . \ln \left(\frac{\zeta - \zeta_b}{\zeta - \zeta_b}\right) \tag{7}$$

The stream function ψ gives with $\xi = \zeta + i\eta$

$$\psi(\zeta) = \frac{Q}{2\pi} \operatorname{arctg} \left\{ \frac{-2n_b . (\zeta - \zeta_b)}{(\zeta - \zeta_b)^2 + n^2 - n_b^2} \right\} \tag{8}$$

The associated function in the area z does not provide any simple expressions for x and y.
The variation of ψ along the zone of influence boundary in the ζ area ($\eta = 0$) gives:

$$\frac{d\psi}{d\zeta}(n = 0) = \frac{Q}{\pi} \left\{\frac{n_b}{(\zeta - \zeta_b)^2 + n^2_b}\right\} \tag{9}$$

In the z-plane it follows with:

$$\frac{\partial \xi}{\partial x} = \frac{\pi}{H} . \cos \left(\frac{\pi\, y}{H}\right) e^{\frac{\pi\, x}{H}}$$

$$\frac{\partial \psi}{\partial x} = \frac{d\psi}{d\zeta} . \frac{\partial \xi}{\partial x} = \frac{Q}{2\pi} \left\{\frac{(\pi/H)\cos(\pi\,.\,y/H)\sin(\pi.\delta/H)}{\cosh(\pi.x/H) - \cos(\pi.y/H)\,.\,\cos(\pi.\delta/H)}\right\} \tag{10}$$

Along the x-axis ($y = 0$), equation (10) becomes:

$$\frac{d\psi}{dx} = \frac{Q}{2H} \left\{\frac{\sin(\pi.\delta/H)}{\cosh(\pi.x/H) - \cos(\pi.\delta/H)}\right\} \tag{11}$$

The discharge profile has been shown diagrammatically along the x-axis in Diagram 3 for a number of values of δ/H.

In Diagram 4 the infiltration discharge Q^* between $-x/H$ and $+x/H$ has been plotted against the absolute value of X/H for various values of δ/H.

5 Practical application

Although discharge measurements are often carried out for recharge projects, it would certainly be impractical to specify and maintain the individual discharges for each of the wells in a series. In the mathematical examples dealt with in sections 2-4, no such individual discharge regulation was required. The wells were instead sited along a potential line, so that, given constant pressure in the supply pipeline and a correctly selected total infiltration discharge, the required discharge profile will be self-regulating. This does, however, assume a constant infiltration resistance along the entire well gallery. This in turn means that where the discharge per unit of length is high, a greater number of wells per unit of length will be required. If an equal nominal discharge is assumed in the case of wells of identical construction, the required number of wells per unit of length may be derived from Diagrams 1 and 3. The total discharge required to be infiltrated in a well gallery of a given length may be derived from Diagrams 2 and 4.

The mathematical examples were confined to a single well, but if the dimensions of the well system are small in relation to the distance from the zone of influence boundary, the entire system may safely be regarded as a single well.

In other cases a more accurate discharge profile along the zone of influence boundary may be obtained by means of superposition.

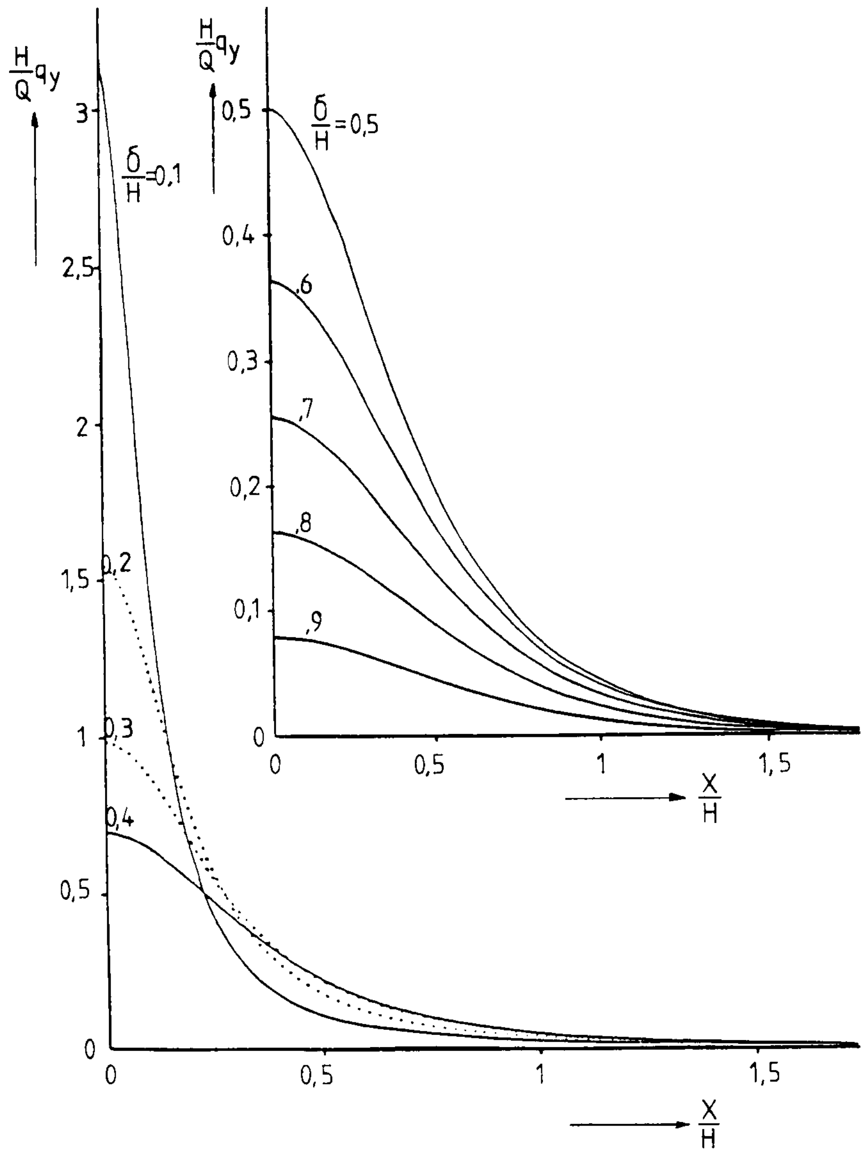


Diagram 3

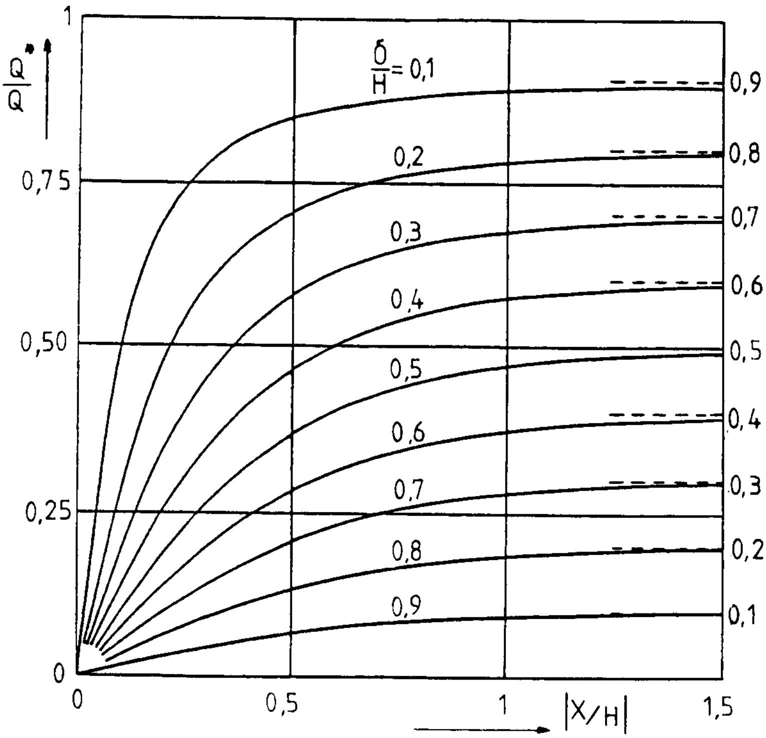


Diagram 4

6 Bibliography

(1) **Verruyt, A.** Theory of Groundwater Flow. Macmillan 1970.

Annex 8C Calculations with the aid of an analog model (Teledeltos paper)

T.N. Olsthoorn
A.N.G. de Vogel

Contents

Annex 8C Calculations with the aid of an analog model (Teledeltos paper)

1 Summary

The test problem has been solved by the use of a specially prepared type of paper with a thin conductive layer (commercially available under the name of Teledeltos paper). This type of paper can be used to indicate how many wells are required for a dewatering scheme and recharge scheme, and especially where the wells should be sited in order to obtain the desired results. The method may be used for both confined and unconfined aquifers, and also in the case of semi-confined aquifers in which the resistance of the surrounding clay and peat layers is sufficiently high.
$\lambda = \sqrt{kDc}$ must be approximately 2.5 – 3 x the largest relevant distance from the dewatering scheme.
The analog method discussed below is based directly on the required situation, from which it then follows where the wells must be sited in order to achieve the object in question, the discharge of the wells having been selected beforehand on other than hydrological grounds alone.

2 Method

The problem of the optimal siting of wells around an excavation and a recharging scheme can be solved in a straightforward manner by the use of Teledeltos paper (a special paper covered with a thin layer of conductive material). The discharge of the wells can be selected beforehand on other than purely hydrological grounds.
The model works as follows. The paper is a two-dimensional horizontal representation of the water-bearing formation. Differences in the piezometric level are simulated on the paper by differences in electric potential.
A boundary with constant potential may be obtained by constructing an electrode out of conductive silver paint. A copper wire is soldered to the paint, and the electrode hooked up to a voltage supply. Lines which in reality connect up points with the same groundwater elevation are represented in the model by lines of equal electric potential; these may be traced with a voltmeter.
Once the path of the lines of equal groundwater elevation (isohypses) has been established the path of the stream lines may be traced. This is done by drawing lines cutting the isohypses at right angles, thus producing a rectangular network. A preferable method, however, consists of solving the 'inverse' problem. This is done by converting the isohypses into stream lines and vice versa, so that the stream lines

can be traced with the voltmeter as isohypses. This is the method used here.
The test problem concerns a water-bearing layer fed from above through a semi-permeable clay or peat layer, i.e. a semi-confined aquifer. The Teledeltos paper cannot of course be fed from above, so that in this respect the model is not quite accurate. The extent to which the model does nevertheless produce accurate results is taken up below.
The drawdown resulting from the abstraction of groundwater from a semi-confined aquifer may be found by the following equation:

$$s = \frac{Q_0}{2\pi kD} \; Ko \; (\frac{r}{\lambda}) \qquad (1)$$

If $r < 0.3$ to $0.4\ \lambda$, this formula simplifies to:

$$s = \frac{Q_0}{2\pi kD} \; \ln \frac{(1.123\lambda)}{(r)} \qquad (2)$$

In the case of a confined aquifer, which the Teledeltos paper does represent accurately, the formula becomes:

$$s = \frac{Q_0}{2\pi kD} \; \ln \; (\frac{R}{r}) \qquad (3)$$

Good results can therefore also be obtained with Teledeltos paper in the vicinity of the centre of a dewatering scheme provided that the largest relevant dimensions of the dewatering (or recharge) scheme are no more than approx. 0.4 λ and if a fixed boundary is assumed at a distance $R = 1.123\lambda$ from the centre of the project (Fig. 1).

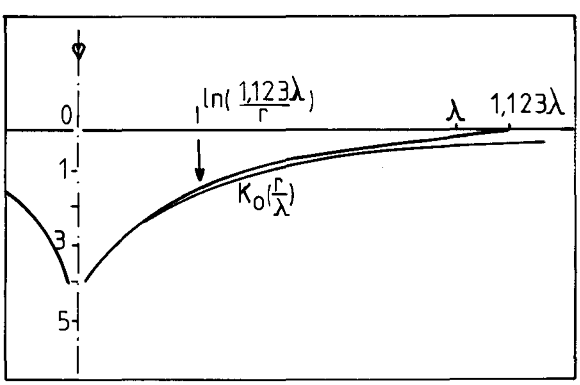


Figure 1 Logarithmic approximation of Bessel function K_0.

3 Specific electric resistance of Teledeltos paper

The specific electric resistance must be known in order to determine the relationship between the size of the electric current in the model (found with the aid of an ammeter) and the size of the groundwater flow in the real situation. If the resistance is known the quantities of water to be abstracted, recharged or discharged can then be measured.

The specific electric resistance of the paper can vary when measured along axes in different directions, i.e. the paper may not be isotropic. Anisotropy can be compensated for by using different length and width scales. It is then assumed that the specific resistances in these two directions are the maximum and minimum values or vice versa. For the sake of certainty measurements should be taken in at least three directions.

The current has been measured by taking two sample strips of 100 x 100 mm^2 and subjecting them to a voltage difference of 10 V (see Fig. 2). The respective current strengths were:

Along the length of the paper roll: $q_{e1} = 5.17$ mA (4)
Along the breadth of the paper roll: $q_{e2} = 4.38$ mA

ρ_1 and ρ_2 may then be calculated as follows:

$$\rho_1 = \frac{V}{q_{e1}} \qquad \text{so that}$$

$$\rho_2 = \frac{V}{q_{e2}} \qquad \frac{\rho_1}{\rho_2} = \frac{q_{e2}}{q_{e1}} \tag{5}$$

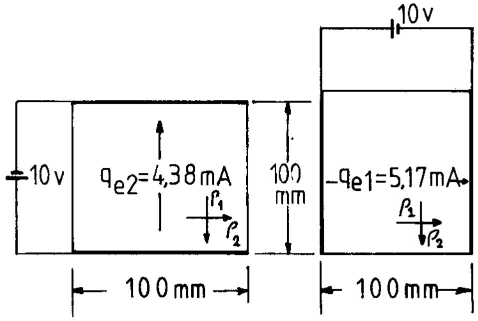


Figure 2 Sample strips in two directions.

4 Length scales used

The paper is anisotropic in that the strength of the current through the two sample pieces varies. In order to compensate for this variation, a different scale has been used across the width of the paper from that along the length of the roll. The relationship between the two scales must be such that if a part of the water-bearing formation is represented on the Teledeltos paper, the strength of the current remains constant irrespective of where that representation is drawn.

If a square section of the water-bearing formation of edge L is drawn on the paper with the sides parallel to the directions of the specific resistance, a rectangle will then be obtained with sides S_1L and S_2L, with S_1 and S_2 being the scale factors for these two axes (in this instance parallel to the lengthwise and crosswise direction of the paper respectively).

If the square section of the water-bearing layer is imagined as being bounded by two impermeable walls, and if a constant difference ΔH in groundwater elevation is maintained between two opposite sides, the total discharge will be:

$$q_w = \frac{\Delta H}{L} . L . kD = \Delta H . kD \tag{6}$$

irrespective of the two opposite sides between which the difference ΔH applies.

Since the paper is not isotropic, this must be taken into account. Along one axis the specific current will be:

$$q_{e1} = \frac{\Delta V}{S_2 L} . S_1 L . \frac{1}{\rho 1} = \frac{\Delta V}{\rho 1} . \frac{S1}{S2} \tag{7}$$

while in the other direction:

$$q_{e2} = \frac{\Delta V}{S_1 L} . S_2 L . \frac{1}{\rho 2} = \frac{\Delta H}{\rho 2} . \frac{S2}{S1} \tag{8}$$

The relationship between the two scale factors may be obtained by equating (7) with (8) and then substituting (5) and (4):

$$\frac{S1}{S2} = \frac{\rho 1}{\rho 2} = \frac{qe2}{qe1} = \frac{4.38}{5.17} = 0.92 \tag{9}$$

Given the width of the paper roll of 0.785 m, a scale of 1:2700 has been selected in the lengthwise direction and one of 0.92 x (1:2700) = 1.2935 in the crosswise direction of the paper. This makes the diameter of the circle of radius R = 1.123λ = 1123 m, which borders the flow model, (2 x 1123)/2935 = 0.77 m, which just fits within the width of the paper.

5 Conversion of mA in the paper into real discharges

Since the specific resistance of the paper used was not homogeneous, the actual situation had to be portrayed by the use of differing scales. By substituting (11) in (7) and (8) and filling in the measured values for q_{e1} and q_{e2} for $\Delta V/\rho_1$ and $\Delta V/\rho_2$ respectively (see (4)), it may be seen that q_{e1} and q_{e2} have been made equal as a result of the respective scales used:

$$q_{e1} = 5.17 \,.\, 0.92 = 4.76 \text{ mA} \qquad (10)$$
$$q_{e2} = 4.38 \,.\, 0.92 = 4.76 \text{ mA}$$

This applies irrespective of the size of the square represented on the paper (see 6)) given a voltage difference of 10V.
If the value of kD (750 m^2/d) is filled in in (6) together with a groundwater elevation variation of 10 m (so that 1 m of groundwater elevation difference corresponds with 1V of voltage difference), it follows from (10):

$$q_w = \Delta H.kD = 10 \text{ m}.\ 750 \text{ m}^2/\text{day} = 4.76 \text{ mA}$$
$$\text{so that } 1 \text{ mA} = 1580 \text{ m}^3/\text{day}. \qquad (11)$$

6 Withdrawal and infiltration discharges

Fig. 3 illustrates the schematized measurement model, showing the specially prepared conductive boundary of the area with radius $R = 1.123\lambda = 1123$ m and the discharges from the excavation and at the object to be protected by recharge. Since the electric potential along the boundary of the object to be protected and the boundary of the area as a whole must be kept equal (= zero), the object and the boundary have been short-circuited by means of a connecting wire.

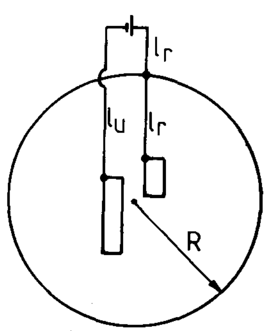


Figure 3 Discharge measurement.

Currents I_u an I_r, which are maintained by a 10 V power supply, have been measured with an ammeter. I_u is the abstraction discharge, i.e. the total discharge to be pumped up from the excavation. A reading of 24.9 mA was obtained for I_u, which may be converted directly into the abstraction discharge by means of the equation arrived at in the previous section.
I_r is the total discharge for recharge purposes, for which a reading of 14.2 mA was registered. $I_u - I_r$ is the discharge flowing across the boundary towards the excavation, i.e.
$I_u = 39300\ m^3/day\ (= 14.3\ .\ 10^6\ m^3/a = 1640\ m^3/h)$
$I_r = 22500\ m^3/day\ (= 8.22.10^6\ m^3/a = 940\ m^3/h)$
That is, 57% of the water must be recharged.

7 Pumping wells

The pumping wells selected have an individual tapping capacity of 60 m^3/h. Given a well diameter of 0.5 m, these wells achieve a drawdown of:

$$s = \frac{Q}{2\pi kD} \ln \left(\frac{1.123\lambda}{r}\right) \qquad (12)$$

$$s = \frac{60 \times 24}{2 \times \pi \times 750} \ln \left(\frac{1123}{0.25}\right) = 2.57 \text{ m} \qquad (13)$$

Twenty-eight of such wells are required in order to abstract the required total discharge of 1640 m^3/h.

8 Recharging wells

The intake capacity (i.e. discharge) of the recharging wells has been arbitrarily fixed at half that of the pumping wells. This means that 32 recharging wells with a 30 m^3/h capacity are required to achieve the total discharge of 940 m^3/h.

9 Well siting

After the isohypses were drawn (see Figs. 4 and 5), the problem was reversed in order to determine the stream lines. This is done by converting the potential lines into stream lines. By cutting the paper out along the boundary of the excavation and the 5 V potential lines, these lines automatically become stream lines (see Figs. 5 and

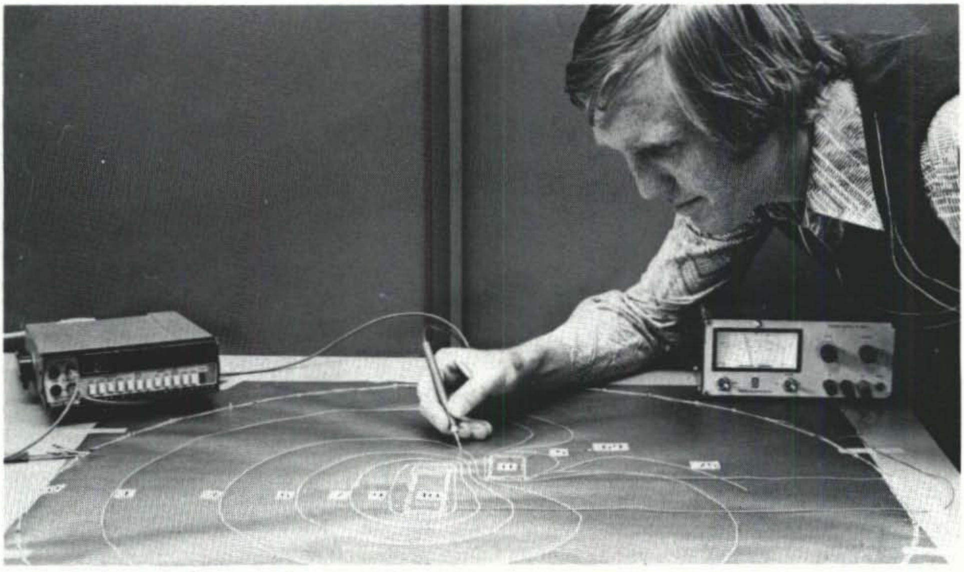


Figure 4 Drawing the isohypses.

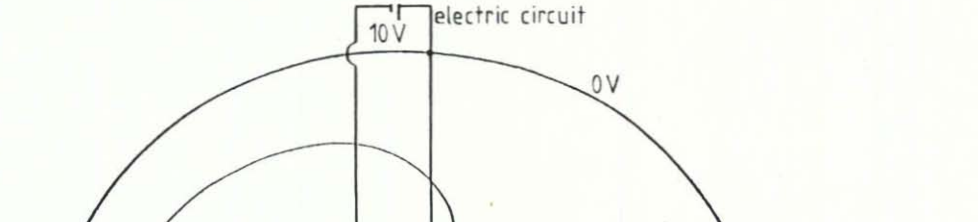


Figure 5 Isohypses with electric circuit.

6). The paper is then cut along a stream line which has been estimated as best as possible. The two edges formed by cutting are made conductive, after which a voltage difference is applied between them. Since the number of wells required around the perimeter will have been determined as discussed in the previous sections, a useful device is to make this potential difference equal to the number of wells or to a constant number thereof. (The total voltage difference must however be kept below 30-40 V in order to avoid electric shocks.) The stream lines from the original problem are next traced with a voltmeter, and one (or more) well sited between each pair of stream lines. The same is then done for the recharge project, thus solving the problem of the optimal location of pumping and recharging wells. Figures 4-6 illustrate the working methods, while all the measured stream and potential lines have been brought together in the one diagram in Fig. 7.

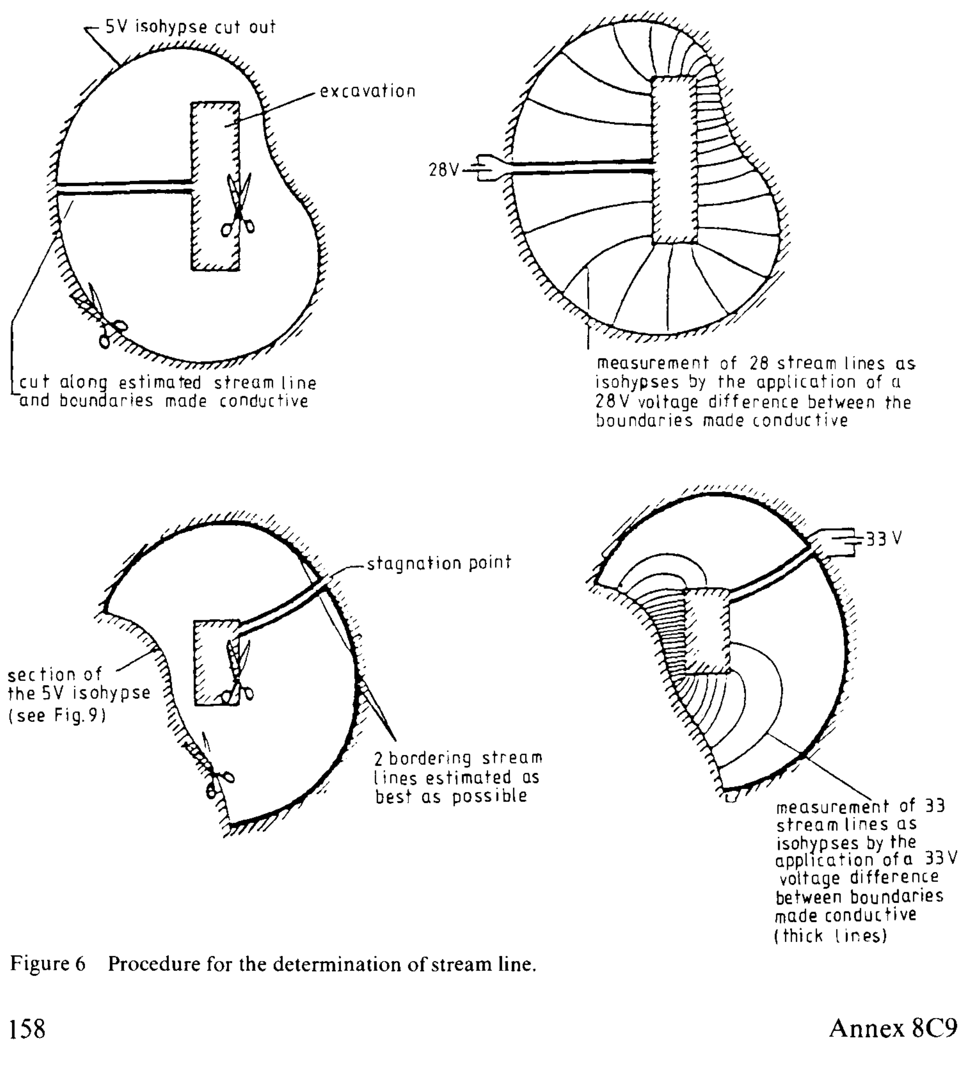


Figure 6 Procedure for the determination of stream line.

10 Conclusion

A well-nigh optimal design for a dewatering/recharge project can in certain situations be arrived at fairly quickly (within one day) with the aid of Teledeltos paper, some conductive silver paint, copper wire, a power source, voltmeter and ammeter. The method may be regarded as producing an optimal result because it will provide the correct number and location of pumping and recharging wells required to achieve the desired object. This can moreover be done in such way that each well will in fact abstract or inject the discharge selected for it on other than hydrological grounds alone. Subject only to the constancy and value of the geotechnical parameters, not a well too few or too many will be drilled, or be drilled in the wrong place. The work required for obtaining the final result can moreover be appreciably reduced by confining onself to a single isohypse and the points of intersection of the stream lines with the excavation and the object being protected by recharge.

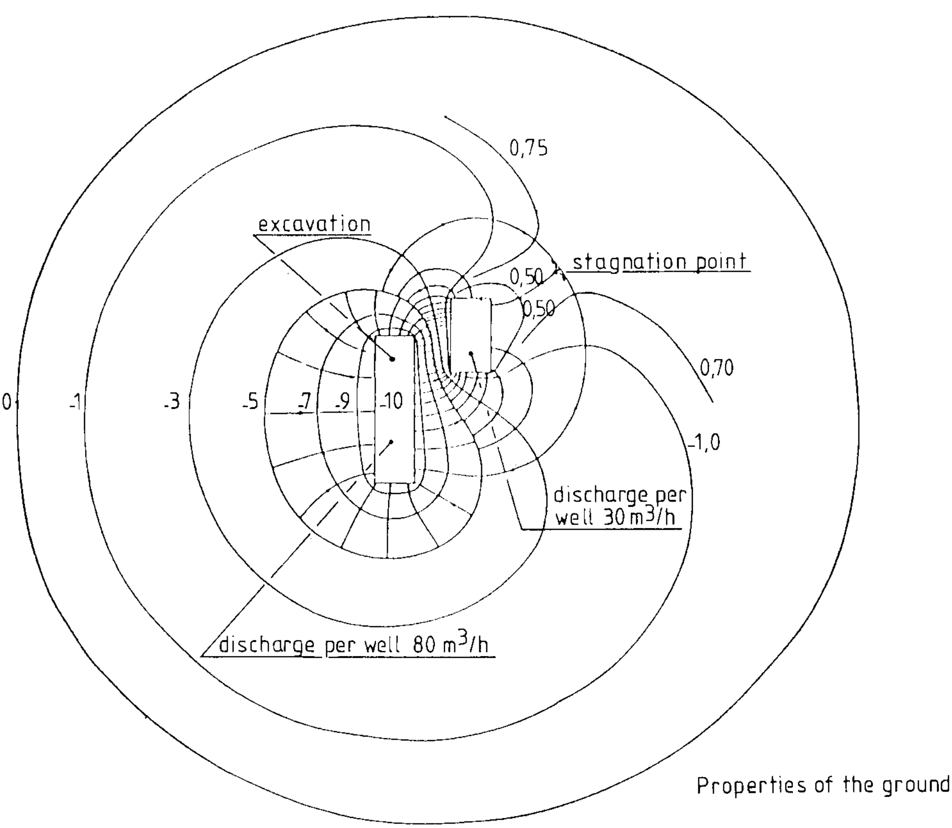


Figure 7 Results.

Annex 8D Calculation by means of the finite element method

A. Leijnse

Annex 8D Calculation by means of the finite element method

Introduction

In the finite element method the region to be studied is subdivided into a large number of sub-regions or elements. The shape and dimensions of the elements are a matter of choice, the selection depending on the geometry of the region under consideration and the degree of accuracy required. Where there are large differences in the hydraulic head (e.g. around pumping wells), the area will have to be broken up into many small elements in order to obtain the required degree of accuracy.
The corner points of the elements are taken as the modal points or nodes.
Assuming a two dimensional stationary flow equation

$$\frac{\partial}{\partial x}(kD\frac{\partial \varphi}{\partial x}) + \frac{\partial}{\partial y}(kD\frac{\partial \varphi}{\partial y}) \times Q = 0 \qquad (1)$$

and assuming there to be N nodes, an approximate solution Φ_b may be defined for Φ as a linear combination of N Basic functions:

$$\Phi_b = \sum_{i=1}^{N} \Phi_i L_i(x, y) \qquad (2)$$

The functions L_i are chosen in such a way that:
$L_i = 1$ at the node $j = i$
$L_i = 0$ at the node $j = i$
The coefficients Φ_i are therefore the values for the approximate solution Φ_b at the nodes i. These coefficients are now determined in such a way that the approximate solution Φ_b is as close as possible to the true solution Φ.

Mathematical formulation

Two methods, viz. the variational principle and the weighted residual method, may be applied in order to determine the coefficients Φ_i in equation (2) in such a way that the approximate solution Φ_b is as close as possible to the true solution for equation (1). Both methods have been described in detail in the literature (Zienkiewicz, 1971; Finalyson, 1972; Norrie et al, 1973).

In principle these methods work as follows:
a. Variational principle
A function

$$F \left(\frac{\partial \Phi}{\partial x}, \frac{\partial \Phi}{\partial y}, \Phi, x, y\right)$$

may be found so that the integral of F over the area under consideration G:

$$I = \iint_G F dG \tag{3}$$

is minimal for the solution Φ of equation (1). In this equation F takes the following form:

$$F = \tfrac{1}{2} \left\{ kD \left(\frac{\partial \Phi}{\partial x}\right)^2 + kD \left(\frac{\partial \Phi}{\partial y}\right)^2 \right\} - \Phi Q \tag{4}$$

If we substitute the approximate solution Φ_b in F, the integral I may be calculated by carrying out integration element by element.
I is now minimalized with the aid of the coefficients Φ_i by assuming

$$\frac{\partial I}{\partial \Phi_\iota} = 0 \quad i = 1,N \tag{5}$$

(5) forms a set of N linear equations in the N unknowns Φ_i. A solution may now be obtained by means of well-known methods.

b. Weighted residual method
Substitution of the approximate solution Φ_b, in (1) gives:

$$R = \frac{\partial}{\partial x}\left(kD \frac{\partial \Phi_b}{\partial x}\right) + \frac{\partial}{\partial y}\left(kD \frac{\partial \Phi_b}{\partial y}\right) + Q \tag{6}$$

In general R will not be equal to zero. The coefficients Φ_i of the approximate solution Φ_i, are now determined in such a way that R is minimal.
This may be done in many ways. The best known method is that of Galerkin, in which R is minimalized by making R orthogonal to the basic functions L_i, i.e.

$$\iint_G RL_i\, dG = o \qquad i = 1,N \tag{7}$$

Integration is once again carried out element by element.
(7) forms a set of N linear equations in the N unknowns Φ_i. A solution may now be obtained by means of well-known methods.

Boundary conditions

Boundary conditions can be readily incorporated into the finite element method. If a point has a fixed potential, the discharge Q can then be calculated from the equation for that point. If the discharge Q is given, the potential for that point can be calculated.

Recharge

The finite element method, using triangular elements, may be applied to the test recharge problem (Fig. 1).

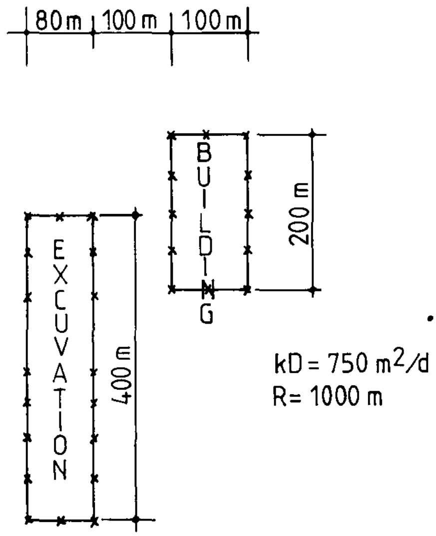


Figure 1 Site plan.

The test area was a 2 x 3 km site with the excavation and building in the centre of the area. The boundaries of the area were maintained at a constant potential of Q = o. The area was divided into 628 triangular elements with 337 nodes, the network being made a good deal finer around the excavation and the building. Twenty pumping wells were sited along the perimeter of the excavation, with the potential

being held constant at −10 m, while twelve infiltration wells were sited around the building, with the potential being held constant at 0 m.
The results of the calculations indicated a total required abstraction discharge of 41740 m^3/d and an infiltration intake of 23670 m^3 d. The distribution of the discharges over the various wells is shown in Fig. 2.
Fig. 3 plots the isohypes in a portion of the test area. The fact that the −10 m lines has not been drawn along the boundary of the excavation is a result of the interpolation technique used for the calculation of the isohypses. The same applies to the 0 m line along the edge of the building.

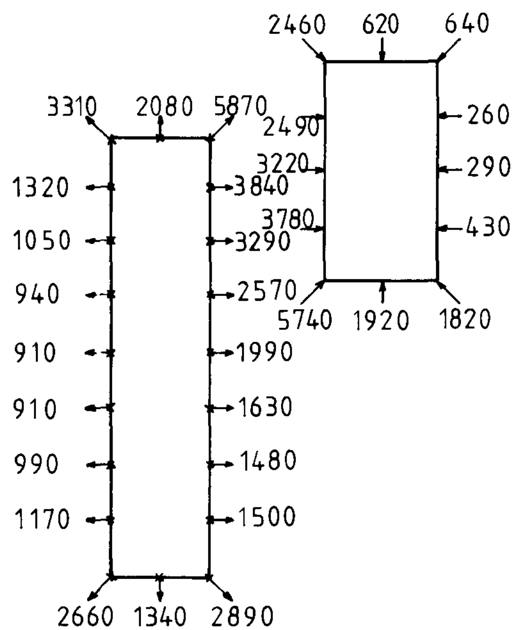


Figure 2 Discharge distribution along excavation and building.

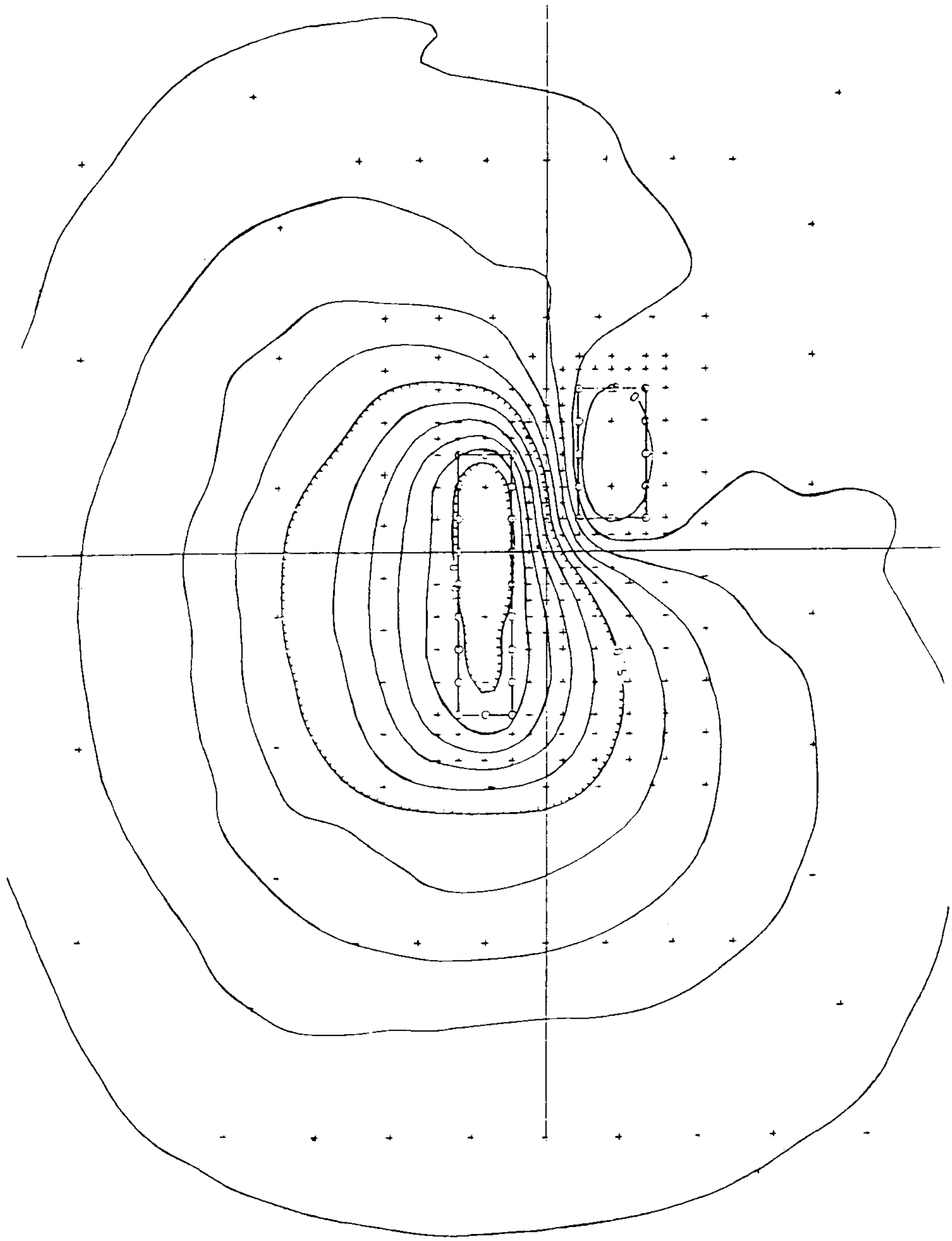


Figure 3 Isohypses in portion of model region.

Annex 8E Calculation by means of the well method

T.N. Olsthoorn
A.N.G. de Vogel

Contents

Symbols

Φ	= piezometric level	(m)
Φ_o	= undisturbed piezometric level	(m)
S_i	= change in piezometric level (drawdown) at point i as a result of wells in use	(m)
Q_j	= discharge of well j	(m^3/day)
$\vec{q_{ij}}$	= specific discharge at point i as a result of well j in direction ij	(m^2/day)
$\vec{r_{ij}}$	= distance between point i and well j (vector)	(m)
R	= distance from the well at which the latter no longer affects the piezometric level (confined aquifer)	(m)
λ	= $\sqrt{\text{VkDc}}$ = characteristic length (semi-confined aquifer)	(m)
kD	= coefficient of transmissability of water-bearing layer	(m^2/day)
c	= resistance of semi-pervious layer	(day)
a_{ij}	= coefficient	(day/m^2)or (^{1}/m)
$\vec{n_i}$	= normal vector on potential line at point i	
α_i	= angle between $\vec{n_i}$ and positive x-axis	
θ_{ij}	= angle between $\vec{r_{ij}}$ and positive x-axis	
$K_0(\tau)$	= Bessel function	
$K_1(\tau)$	= $-d\,Ko\,(\tau)/d\tau$	

Annex 8D Calculation by means of the well methods

1 The well method in general

The test problem has been solved by a method termed by its authors as the 'well method'.
The well method is the result of the attempt to find a way of directly calculating the well discharges required in order to obtain the stipulated piezometric level at a number of selected points.
The well method consists of an analytical calculation technique based on the superposition principle. It may be used to solve problems for both confined and semi-confined aquifers.

If certain minor adjustments are made the programme can also be used to tackle problems relating to unconfined aquifers, as well as non-stationary problems. Because the method is analytical it is subject to the same advantages and disadvantages as those applying to other analytical methods. Boundaries of all shapes can however be readily introduced. In this respect the method is comparable to that of line sources: see Annex 8F.

2 Development of the method

Provided that R is significantly higher than (r_{ij}), the groundwater elevation at point i may, in the case of a confined aquifer, be written:

$$\Phi_i = \frac{Q_l}{2\pi kD} \ln \left(\frac{r_{il}}{R}\right) + \ldots \frac{Q_j}{2\pi kD} \ln \left(\frac{r_{ij}}{R}\right) \ldots \frac{Q_n}{2\pi kD} \ln \left(\frac{r_{in}}{R}\right) + \Phi_n \qquad (1)$$

(Φ_n = Φ resulting from rainwater alone; boundary conditions are created with 'wells'.)
In the case of a semi-confined aquifer the equation becomes:

$$\Phi_i = \frac{Q_l}{2\pi kD} Ko \left(\frac{r_{il}}{\lambda(2)}\right) + \ldots \frac{Q_j}{2\pi kD} Ko \left(\frac{r_{ij}}{\lambda}\right) + \ldots \frac{Q_n}{2\pi kD} Ko \left(\frac{r_{in}}{\lambda}\right) \qquad (2)$$

(all boundary conditions are created with 'wells').
For both types of groundwater systems we may therefore write:

$$\Phi_i = a_{i1}\, Q_1 + a_{2i2}\, Q_2 + \ldots\ldots a_{ij}Q_j + \ldots\ldots a_{in}\, Qn + \Phi_n \qquad (3)$$

The coefficients a_{ij} may be calculated directly by computer.
In the case of prescribed piezometric levels the only remaining unknowns are the discharges $Q_1 - Q_n$. (In the case of the test problem for a semi-confined aquifer $\Phi_n -$ 0).
One possible approach is to prescribe the piezometric level to be achieved at n points. This would then give rise to n equations with n unknown discharges, which could be solved directly by computer. The solutions obtained in this way are not, however, necessarily what was sought. The solutions might result in the stipulated piezometric levels, but only at the expense of the groundwater elevation in other places. The result is that wells with enormous abstraction discharges are often found side by side with wells with exceptionally high infiltration discharges, thus producing a net result in line with the stipulated conditions.

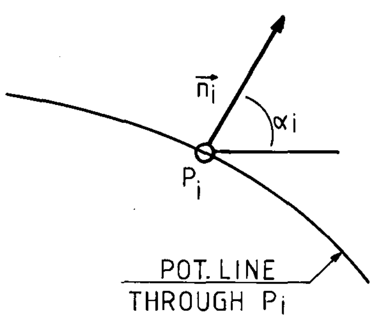


Figure 1.

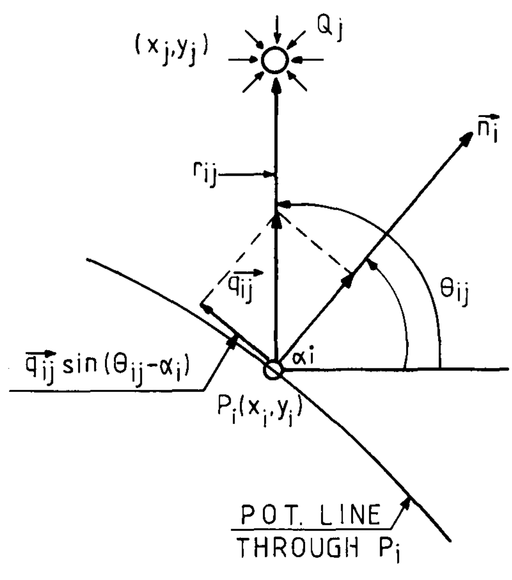


Figure 2.

It was consequently necessary to introduce a further constraint in such solutions. The constraint selected was the direction of the potential line at a number of set

points. This produced good results (see also (1) and (2)). This direction is given by the angle α_i made by the normal vector $\vec{n_i}$ through the point P_i on the potential line through P_i with the positive x-axis. The constraint imposed is that the component of the vector which indicates the specific discharge $\vec{q_i}$ at P_i at right angles to $\vec{n_i}$ should be equal to zero.
The formula is defined:

$$\sum_{j=1}^{n} \vec{q}_{ij} \sin (\Theta_{ij} - a_i) = 0 \qquad (4)$$

in which i = the number of the point and j = number of the well.

$$\Theta_{ij} = \text{arctg} \frac{(y_j - y_i)}{x_j - x_i} \qquad (5)$$

$$\text{Since} -\frac{\pi}{2} \leq \text{arctg} (\tau) \leq \frac{\pi}{2} \qquad (6)$$

Q_{ij} must be increased by π if $(x_j - x_i) < 0$, i.e. for the second and third quadrants. In the case of a confined aquifer, the specific discharge $\vec{q_{ij}}$ through well j at point i amounts to:

$$\vec{q}_{ij} = \frac{Q_j}{2\pi \; \vec{r}_{ij}} \; \vec{e}_{ij} \qquad (7)$$

in which $\vec{e}_{ij}$ represents the unit vector in the direction i-j. In the case of a semi-confined aquifer this becomes:

$$\vec{q}_{ij} = \frac{Q_j}{2\pi\lambda} K_1 \; \frac{\vec{r_{ij}}}{\lambda} \vec{e_{ij}} \qquad (8)$$

By substituting (7) or (8) in (4), equations are obtained with the same form as (3) in which zero must be substituted for Φ_i and in which the coefficients $a_{0_{ij}}$ satisfy the condition:

$$a_{ij} = \frac{1}{2\pi \left| \vec{r}_{ij} \right|} \; .\sin (\Theta_{ij} - a_i) \qquad (9)$$

for a confined aquifer; and

$$a_{ij} = K \frac{\left(\frac{\vec{r_{ij}}}{\lambda}\right)}{2\pi\lambda} \, .\sin(\Theta_{ij} - a_j) \tag{10}$$

for a semi-confined aquifer.

3 Arriving at solutions

Solutions are now in principle obtained by siting n wells, stipulating the potential at (n-k) points and prescribing the direction of the potential line at k points.
As far as the constraints are concerned, these points may be allowed to coincide. So far we have always selected $k = n/2$. We have also seen that $k = 0$ does not produce good results for the test problem.
In order to solve the test problem 30 wells have been distributed along the perimeter of the excavation and the building (see also Table 2). As shown in Fig. 3, a point has

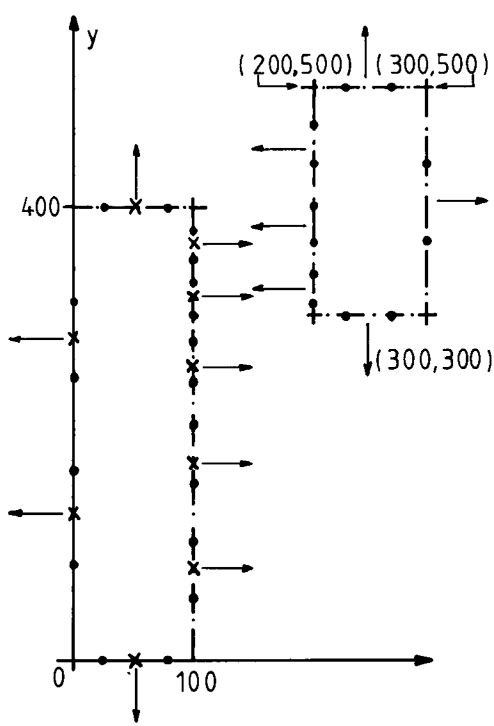


Figure 3 Distribution of the wells (.) and points (x) = normal vector.

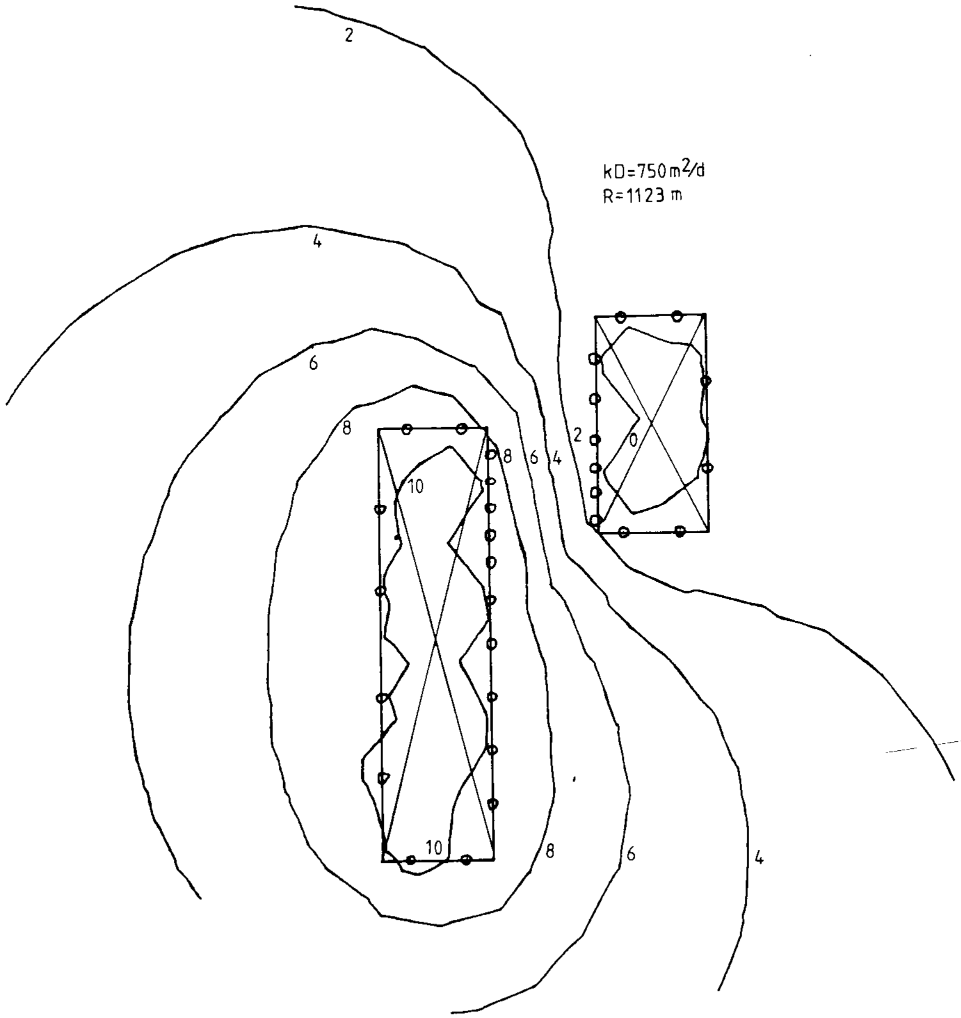


Figure 4 Drawdown contours (m) drawn by computer and calculated by well method, for a confined aquifer.

been selected between each pair of wells, the piezometric level and the direction of the potential line being prescribed at those points (by means of the normal vector). Calculations have been carried out for both confined and semi-confined aquifers. In the case of a confined aquifer $\lambda = 1000$ m has been replaced by $R = 1.123\lambda = 1123$ m.

4 Checking the results obtained

It will not always be certain that the desired solution has been obtained in each case, for which reason it is essential for the results to be checked. This is best done by filling in the calculated discharges in the basic equations (1) and (2), on the basis of which the isohypse profile can be shown diagrammatically or be plotted by computer. This will immediately expose any mistakes or irregularities. Checks of this kind were carried out and are shown in Figs. 4 and 6.

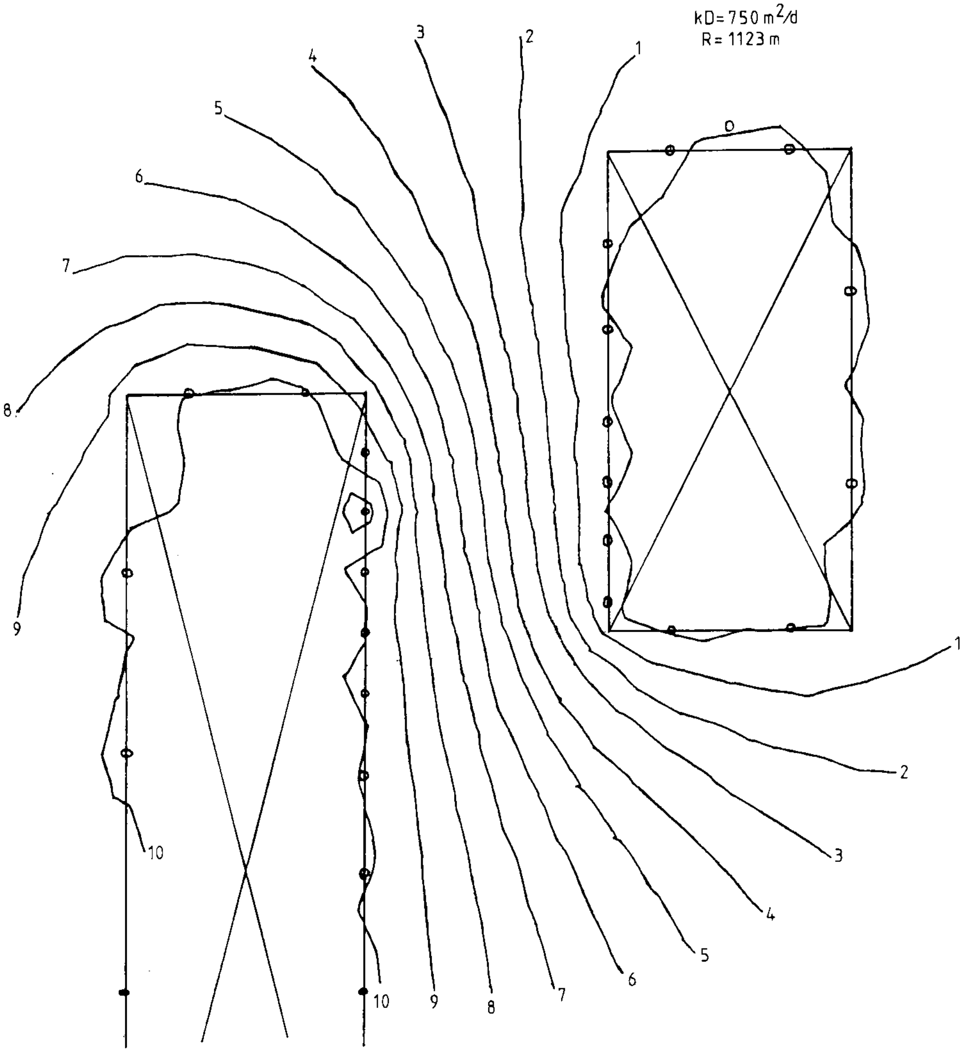


Figure 5a. Detail of Fig. 4.

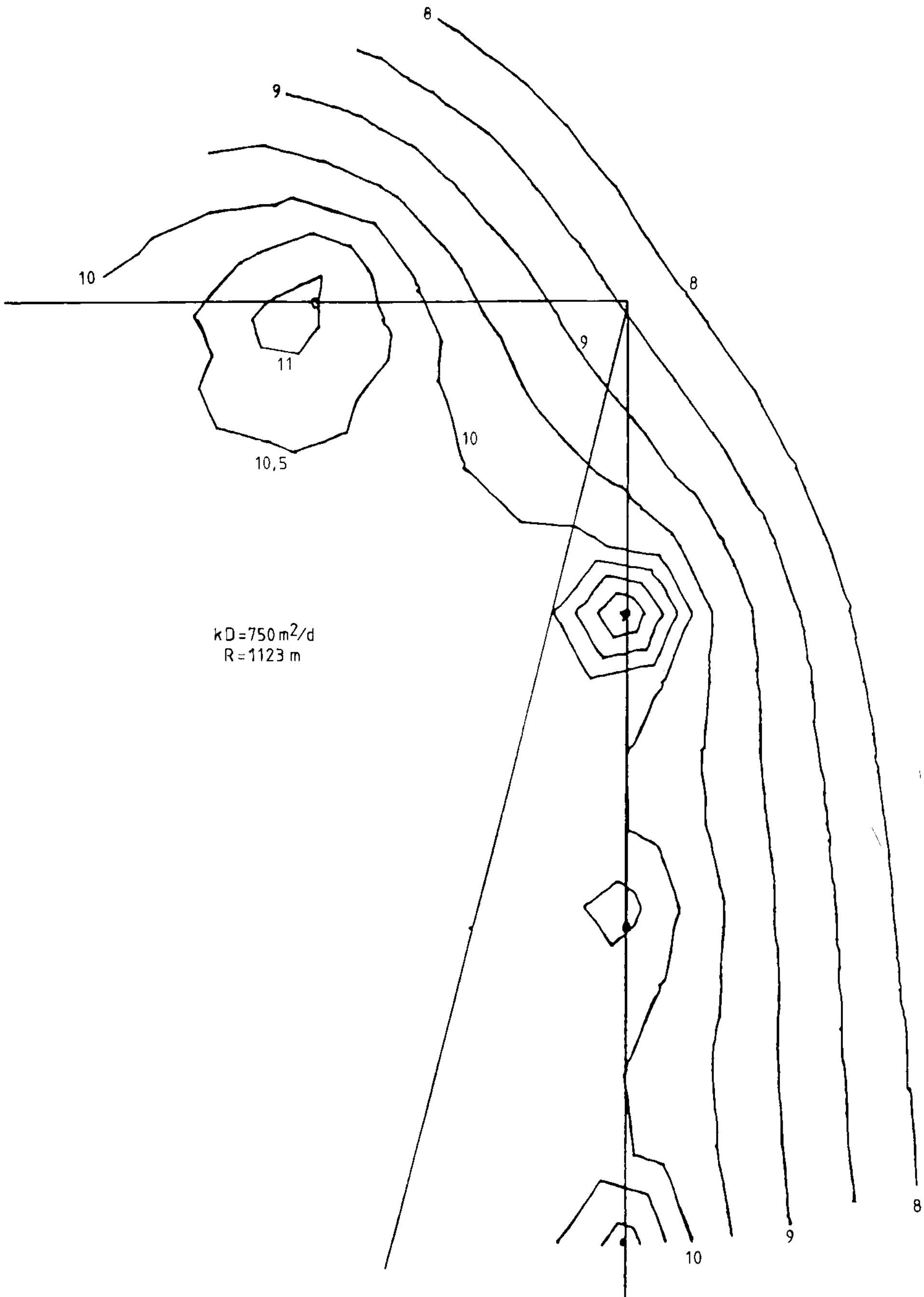


Figure 5b Detail of the piezometric surface in the top right-hand corner of the excavation (see Figs. 4 and 5a).

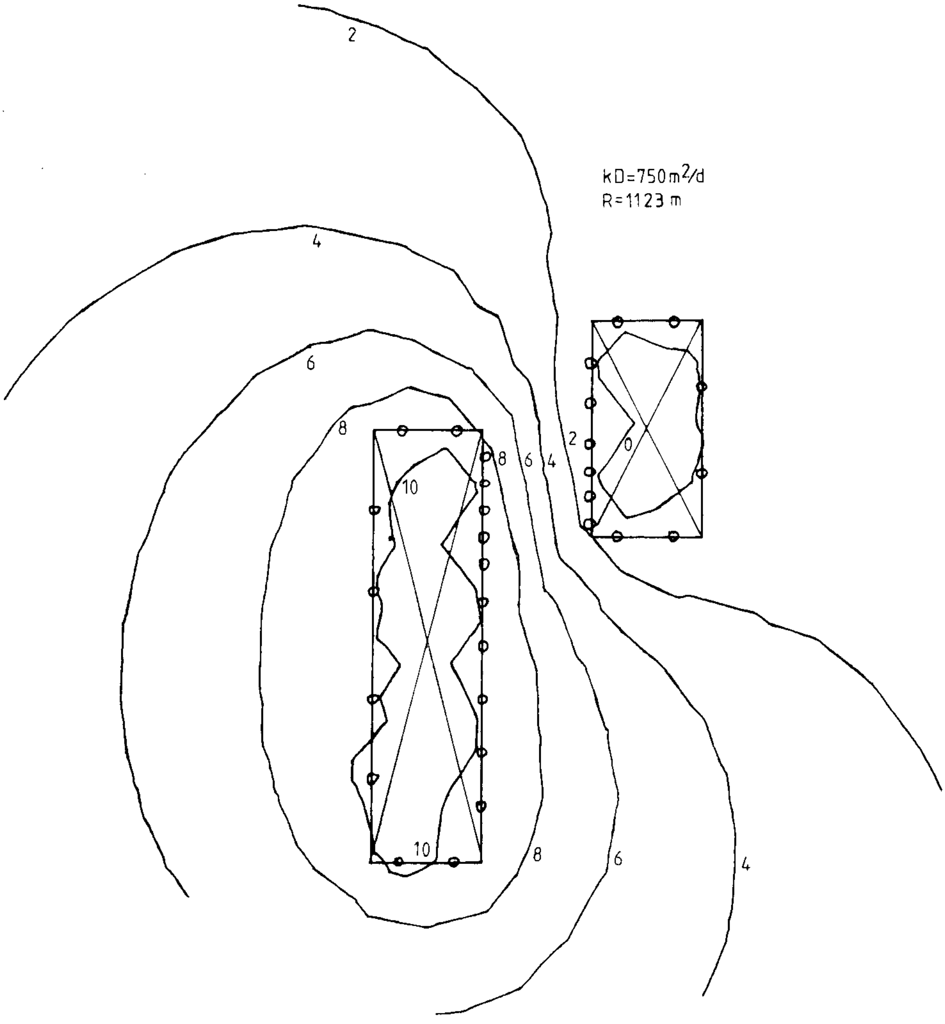


Figure 6 Drawdown contours (m) drawn by computer and calculated by well method, for a confined aquifer.

Figure 7 Detailed section of the drawdown contours around the top of the excavation (see Fig. 6).

It must be borne in mind that some of the irregularities may be due to the plotting programme. Any doubts or questions about the solution or plotting can be overcome by the enlargement of details, when faults in the plotting programme become steadily less important.
This procedure is possible because the method of calculation is an analytical one, whereby the piezometric surface is a continuous function (i.e. one which can be calculated for each point).
Detailed sections of the upper right-hand corner of the excavation are shown in Figs. 5 and 7.

5 Results of application to test problem

The results of the calculations are set out in the following tables and diagrams. Table 1, column 1 and Figs. 4 and 5 set out calculations for the test problem in the case of a confined aquifer in which $R = 1.123\lambda = 1123$ m. Table 1, column 2 and Figs. 6 and 7 concern the same problem, but in this case calculated for a semi-confined aquifer with $\lambda = 1000$ m. The differences between the two cases turn out to be very small:

	*Total abstraction**	*Total infiltration**	
confined aquifer	39000	22800	$R = 1123$ m
semi-conf. aquifer	39600	23700	$\lambda = 1000$ m

*) discharges in m^3/day

The more detailed drawings (Figs. 5 and 7) reveal the drawdown in the top right-hand corner of the excavation to be 8.5 m instead of 10 m. This could be compensated for in practice by the installation of another well or by distributing the calculated discharges over a greater number of wells. Either of these would improve the discharge profile along the perimeter.
The discharge profile along the perimeter may be derived directly from the results of the calculations by dividing the discharges obtained by the length of perimeter to which they refer. Once the discharge profile has been obtained the solution can be optimalized by redistributing the wells, as done in the case of Annex C where the test problem was solved by analog methods.

Bibliography

(1) **Veer, van de, P.** Calculation Method for Two-Dimensional Groundwater Flow. Delft Prog. Rep., 2 (1976) pp. 35-49.

Table 1 Calculated well discharges in the case of confined and unconfined aquifers.

No.	calculated for a confined aquifer with R = 1123 m Disch. (m^3/day)	calculated for a semi-confined aquifer with = 1000 m. Disch. (m^3/day)
1	2592	2623
2	2638	2644
3	1943	1954
4	1501	1519
5	1632	1708
6	1925	1954
7	1809	1830
8	1900	1919
9	1493	1510
10	1914	1929
11	2090	2102
12	3345	3355
13	4355	4345
14	3657	3644
15	1804	1877
16	1375	1468
17	1334	1436
18	1694	1789
Total	39001	39606
19	−3225	−3246
20	−2446	−2518
21	− 462	− 669
22	− 262	− 503
23	− 647	− 874
24	−1357	−1523
25	−2344	−2364
26	−2523	−2525
27	−1832	−1826
28	−2024	−2018
29	−2112	−2106
30	−3535	−3528
Total	22769	23700

Table 2 Coordinates of the wells. Centre of the axes system in left-hand bottom corner of excavation.

No.	Coordinates x (m)	y (m)
1	25	0
2	75	0
3	100	50
4	100	100
5	100	150
6	100	200
7	100	240
8	100	275
9	100	300
10	100	325
11	100	350
12	100	375
13	75	400
14	25	400
15	0	325
16	0	250
17	0	150
18	0	75
19	225	300
20	275	300
21	300	360
22	300	440
23	275	500
24	225	500
25	200	462
26	200	425
27	200	387
28	200	362
29	200	337
30	200	312

(2) **Veer, van de, P.** The Pattern of Fresh and Salt Water Flow in a Coastal Aquifer. Delft Progr. Rep., 2 (1977) pp. 137-142.

Annex 8F Calculation by means of the line sources method

H.G. Luger

Annex 8F Calculation by means of the line sources method

1 Summary

In this calculation method well galleries are schematized into line sources.
If a piezometric level Φ_j is required at N points and a reference point with a reference piezometric level is given or assumed, the specific discharges (i.e. discharge per unit of length) σ_i of N positionally fixed line sources may be calculated, together with an integration constant Φ_0.
The method results in a set of $N = 1$ equations with $N + 1$ unknowns.
The method differs from the finite element method in that the equations for solution are obtained from the boundary conditions rather than from the situational descriptions in a large number of finite elements. The number of equations and the number of unknowns are much smaller in the case of the line sources method than in the finite element method. The matrix of the system of equations is nevertheless completely filled.

This analytical calculation method is suitable for calculation of unconfined or confined aquifers. The numerical approach in the case of semi-confined aquifers is outlined in section 4 of this annex.

2 Complex potential

If a complex potential function Ω is defined as $\Omega = \Phi + i\Psi$, in which Φ is the potential and Ψ the stream function, and if Ω is made an analytic function of z (= x + iy), Φ and Ψ will then satisfy Laplace's equation. Ω (z) describes a two-dimensional stationary liquid flow through a homogeneous isotropic porous material.
The complex potential Ω may describe the flow in either an unconfined or a confined aquifer.
This is done by altering the definition of the potential Φ, viz:
- $\Phi = k.D.\o_2$ for a confined aquifer
- $\Phi = \frac{1}{2}.k.\o$ for an unconfined aquifer

3 Line sources

The complex potential around a source is described by:

$$\Omega - \Phi_0 = (Q/2\pi).\ln(z\text{-}z_w) \tag{1}$$

in which z_w is the location of the source in the complex z-plane, Q the discharge obtained, and Φ_0 the integration constant.
The complex potential describing a horizontal line source may be found by integrating this function for a line segment.
This gives:

$$\Omega_j - \Phi_0 = (\sigma_j/2\pi) \int_{-l_j/2}^{+l_j/2} \ln(z\text{-}s)\,.ds$$

$$\Omega_j - \Phi_0 = (\sigma_j/2\pi)\,\{(z\text{-}l_j/2)\ln(z\text{-}l_j/2) - (z+l_j/2)\ln(z+l_j/2) + l_j\} \tag{2}$$

in which σ_j is the discharge per unit of length of the line source, which is assumed to be constant.

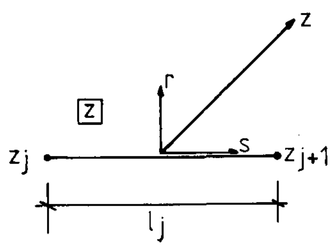


Figure 1 Line source along the real axis.

Consider an arbitrarily located line source.
Select a local set of coordinates $z^* = s + ir$ in such a way that the origin is in the middle of the line source and the real axis lies in the direction of the vector $(z_{j+1} - z_j)$.
The point z is represented in the local set of coordinates by $z^* = (z\text{-}m_j).\exp(-i.\alpha_j)$.

The complex potential for an arbitrary line source may then be found if $z^* = (z\text{-}m_j).\exp.(-i.\alpha_j)$ and σ_j are substituted for z and Q respectively and if equation (1) is then integrated for $-l_j/2$ to $+l_j/2$:

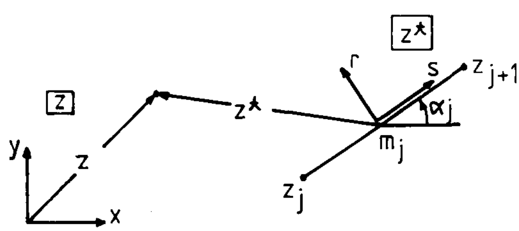


Figure 2 Arbitrarily located line source.

$$\Omega^*_j - \Phi_o = (\sigma_j/2\pi) \int_{-l_j/2}^{+l_j/2} \ln((z\text{-}m_j).\exp.(-i.a_j)-s).ds$$

$$= (\sigma_j/2\pi)\{((z\text{-}m_j).\exp(-i.a_j)-l_j/2).\ln((z\text{-}m_j).\exp(-i.a_j)-l_j/2)$$

$$-((z\text{-}m_j).\exp(-i.a_j) + l_j/2).\ln((z\text{-}m_j).\exp(-i.a_j)+l_j/2)+l_j\} \tag{3}$$

If this is now written as

$\Omega^*_j - \Phi_o = \mathrm{Coef}_j(z)^* \sigma_j$, the potential Φ at a point i in a region with n line sources is described by:

$$\Phi_i = \Phi_o + \sum_{j=1}^{n} \mathrm{Re}\{\mathrm{Coef}_j(i)\}^* \sigma_j \tag{4}$$

In an example with n line sources the above may be written in matrix form, reading $\mathrm{Re}(\mathrm{Coef}_j(i))$ for $\mathrm{Coef}_j(i)$

$$\begin{bmatrix} \mathrm{Coef}_1(1) & \mathrm{Coef}_2(1)\ldots & \ldots\mathrm{Coef}_n(1) & 1 \\ \mathrm{Coef}_1(2) & \mathrm{Coef}_2(2)\ldots & \ldots\mathrm{Coef}_n(2) & 1 \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & & \cdot \\ \mathrm{Coef}_1(n) & \mathrm{Coef}_2(n)\ldots & \ldots\mathrm{Coef}_n(n) & 1 \\ \mathrm{Coef}_1(\mathrm{ref}) & \mathrm{Coef}_2(\mathrm{ref})\ldots & \ldots\mathrm{Coef}_n(\mathrm{ref}) & 1 \end{bmatrix} \cdot \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \cdot \\ \cdot \\ \cdot \\ \sigma_n \\ \Phi_0 \end{bmatrix} = \begin{bmatrix} \Phi_1 \\ \Phi_2 \\ \cdot \\ \cdot \\ \cdot \\ \Phi_n \\ \Phi_{ref} \end{bmatrix}$$

Ready-to-use subroutines are available for most computers for solving the above. The following example makes use of the IBM fortran subroutine GELG (Gauss elimination).

4 Semi-confined aquifers

In the case of a semi-confined aquifer a two-dimensional description of the groundwater flow in the horizontal plane does not satisfy Laplace's equation and cannot therefore be represented by a complex potential Ω which is an anlytical function of z.
The impact on the piezometric level ($\Delta\Phi$) of a well in an infinite plane may be described as:

$$\Delta\Phi = \frac{Q}{2\pi kD} K_0 \left(\frac{r}{\lambda}\right)$$

in which r is the distance to the well.
The impact of a line source on the piezometric level at a point with coordinates x_i and y_i may be found by integration along the line source. If $\sigma = Q/l$ and $\Delta\Phi = \sigma$*Coef(i), it follows for a line source along the x-axis (see also Figure 3):

$$\text{Coef}\,(i) = \frac{1}{2\pi kD} \int_{x=-l/2}^{x=+l/2} k_0 \left\{\frac{\sqrt{(x_i - x)^2 + y_i^2}}{\lambda}\right\} dx$$

since $r_i = \sqrt{(x_i - x)^2 + y_i^2}$.

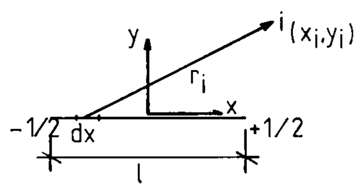
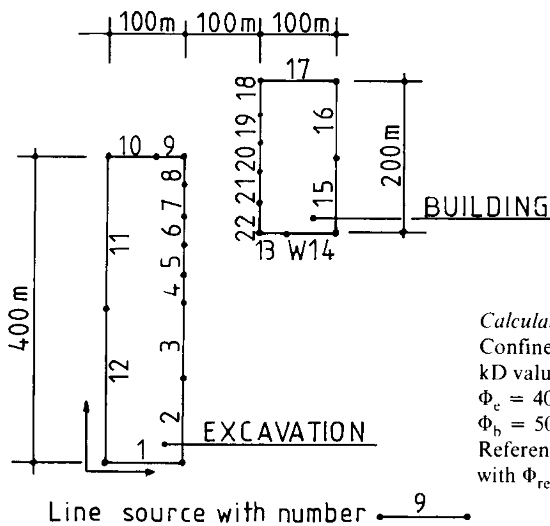


Figure 3.

An analytical solution for the integral of values of $y_i \neq 0$ has not yet been found. The line source method can nevertheless be applied to a semi-confined aquifer provided that the coefficients Coef (i) for each i are determined by means of numerical integration along the line source.



Calculation (test problem)
Confined aquifer.
kD value = 750 m^2/day
Φ_e = 40 m.
Φ_b = 50 m.
Reference point (1425.-975)
with Φ_{ref} = 50 m.

Figure 4 Site plan of test problem.

5 Test problem

Having been measured and calculated in a number of different ways, as discussed in the preceding annexes, the test problem may now be approached by means of a set of 22 line sources, 12 located around the excavation and 10 around the building (see Figure 4 and Table 1 for the coordinates).
Piezometric levels of Φ_e and Φ_b are prescribed for the mid-points of the line sources around the excavation and building respectively. For the calculation results see Table 2.

Table 1 Coordinates of the line sources.

Line source No.	Starting point		Finishing point	
1	0.0	0.0	100.00	0.0
2	100.00	0.0	100.00	100.00
3	100.00	100.00	100.00	200.00
4	100.00	200.00	100.00	240.00
5	100.00	240.00	100.00	280.00
6	100.00	280.00	100.00	320.00
7	100.00	320.00	100.00	360.00
8	100.00	360.00	100.00	400.00
9	100.00	400.00	60.00	400.00
10	60.00	400.00	0.0	400.00
11	0.0	400.00	0.0	200.00
12	0.0	200.00	0.0	0.0
13	200.00	300.00	240.00	300.00
14	240.00	300.00	300.00	300.00
15	300.00	300.00	300.00	400.00
16	300.00	400.00	300.00	500.00
17	300.00	500.00	200.00	500.00
18	200.00	500.00	200.00	460.00
19	200.00	460.00	200.00	420.00
20	200.00	420.00	200.00	380.00
21	200.00	380.00	200.00	340.00
22	200.00	340.00	200.00	300.00

Table 2 Calculations results.

Line source	Specific discharge ($m^3/s/m$)
1	0.4120693E-03
2	0.3573671E-03
3	0.3367194E-03
4	0.4750681E-03
5	0.5869337E-03
6	0.7293397E-03
7	0.8389605E-03
8	0.1225632E-02
9	0.9214291E-03
10	0.6336500E-03
11	0.2010681E-03
12	0.1809266E-03
13	–0.8996418E-03
14	–0.4806262E-03
15	–0.1478337E-03
16	–0.8316216E-04
17	–0.2107604E-03
18	–0.6653934E-03
19	–0.5975172E-03
20	–0.7293110E-03
21	–0.8328585E-03
22	–0.1222351E-02

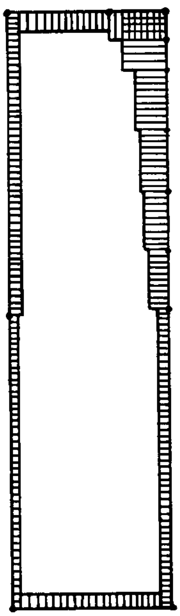
Required pumping capacity   0.1498060E-04 cub.m/hour
Required infiltration capacity   –0.9752246E-03 cub.m/hour
Return infiltration 65.099%

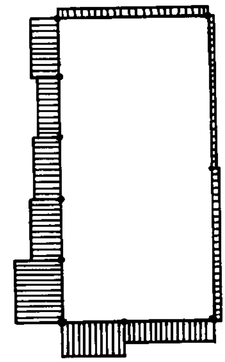
Cf. the values calculated for the same case by means of electric analog:
Required pumping capacity: 1640 m^3/hour
Required infiltration capacity: 938 m^3/hour
Return infiltration: 57%

Bibliography

(1) **Asgian, Margaret,** I. Line Sources. Department of Civil & Mineral Engineering, Univeristy of Minnesota, Minneapolis, October 1977.

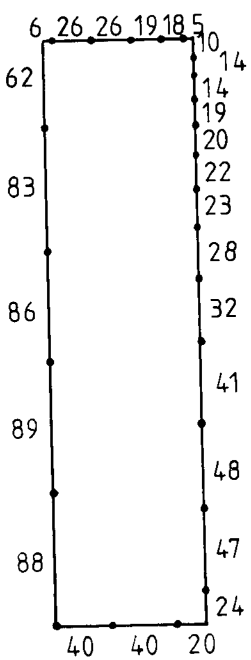
Diagrammatic representation of calculated line source strengths (test problem)

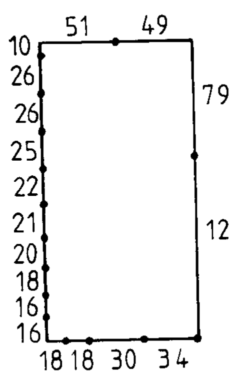




Dewatering arranged inwards.
Infiltration arranged outwards.
Scale: *:2500

$= 10^{-3}\ m^3/s/m$





Well diagram

Designed on the basis of the line source strengths on p. 5-2-1
All wells with a capacity of 60 m^3/hour
25 pumping wells = 1500 m^3/hour
16 infiltration wells = 960 m^3/hour
All measurements in metres.

Annex 10A The elimination of methane from groundwater: some estimates

J.A. Wesselingh

Annex 10A The elimination of methane from groundwater: some estimates

1 Summary and conclusions

Groundwater pumped up in excavations often contains significant quantities of dissolved gas, especially methane. If this water is returned to the soil in a recharge system, gas bubbles can be released and clog up the system. The major problems to which this can give rise may be avoided by eliminating the methane. A number of estimates have been made in this annex of the dimensions of various pieces of equipment that might be used:

1. A spray column
2. A bubble column
3. A packed column
4. A settlement basin

See Figure 1.

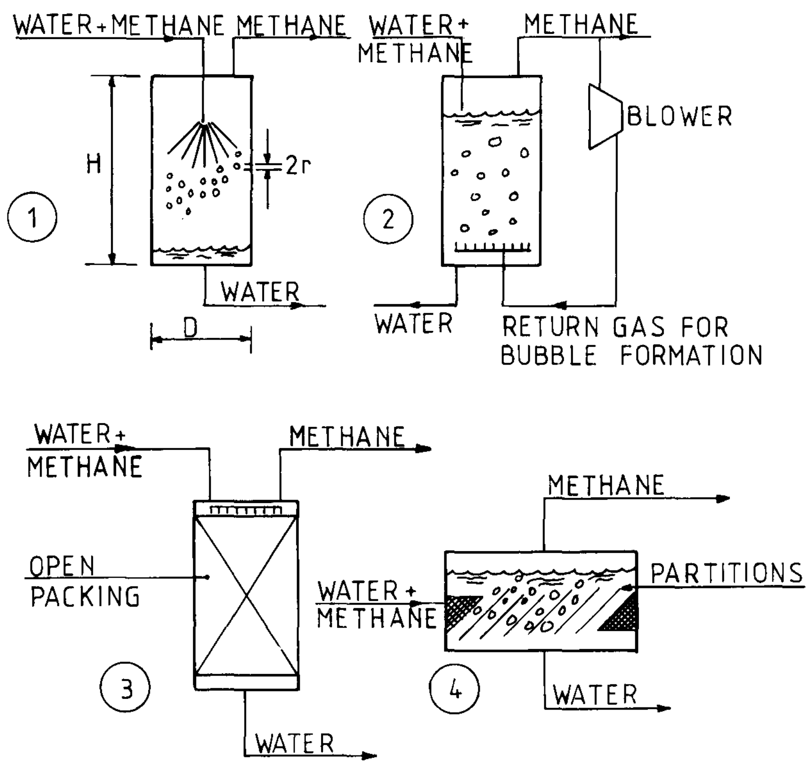


Figure 1.

All these appliances are based on the principle of providing sufficient contact between the water and a gas phase, so that the methane over-saturation can escape to the gas phase. This has been achieved artifically in types 1, 2 and 3; in type 4 it is achieved by bubble formation in the liquid itself.

In all four cases the required volume of the container has been estimated for the treatment of 1 m^3/s of water, with the over-saturation being reduced to approx. 1/10 of the original level. In the case of system 1 several thousand m^3 are required; for systems 2 and 3 around 100 m^3, while for system 4 a flat tray some 2 m deep and 20 m^2 in surface area is required. The last system would therefore appear the most attractive.
It should be stressed that these are order of magnitude estimates; on no account should a large-scale appliance be constructed without further tests.

2 Calculation of the transfer of methane from the liquid to gas

The local material flux of methane through a liquid surface is given by:

$$J = \beta (x - x^*)$$

Where β is a 'material transfer coefficient' (m/s)
x is the actual concentration of methane in the liquid
x^* is the concentration of the liquid which would be in equilibrium with the gas phase.

(I.e. transport becomes **zero** when the liquid has reached equilibrium with the gas.)
Since the gas in all four systems consists of methane at approx. 1 bar, x^* is fixed ($x^* = 50 \times 10^{-3}$ kg/m^3). The value of x varies within the appliance from the input concentration ($x_0 = 3\text{-}10\ x^*$) to a value just above x^*.
The value of the material transfer coefficient varies for the different systems.

1. For the transfer of droplets into gas:
   $\beta \geqq 5D/r$
   where D is the diffusion coefficient of methane in water ($= 10^{-9}$ m^2/s)
   where r is the radius of the droplets.
   With $r = 10^{-3}$ m $\rightarrow \beta = 5 \times 10^{-6}$ m/s.
2. For the transfer of water into bubbles $\beta \simeq 3 \times 10^{-4}$ m/s.
3. In this case the water flows as a film over the packing; the transfer coefficient here is in the order of $\beta \simeq 5 \times 10^{-5}$ m/s.
4. See 2.

The transport of methane is equal to the integral of the flux over the available interface A between the gas and the liquid. The interface in the various appliances is approximately as follows:

1. Interface per unit of volume of the appliance:

$$a = \frac{3\,\varepsilon\,d}{r}$$

where ε d is the volume fraction of the droplets (< 0.1) and r is the radius of the droplets (= 10^{-3} m).

$$a = \frac{3 \times 10^{-1}}{10^{-3}} = 300\ m^2/m^3$$

2.3. The calculated interface is of the same order of magnitude.
In the case of 4 it is difficult to specify the size of the interface, although something may be deduced from the behaviour of individual bubbles.

3 Calculation of container volume for 1, 2 and 3

Appliances 1, 2 and 3 achieve a particularly high degree of remixing of the liquid phase. This means that the liquid concentration will be much the same everywhere, and equal to the outflow concentration.

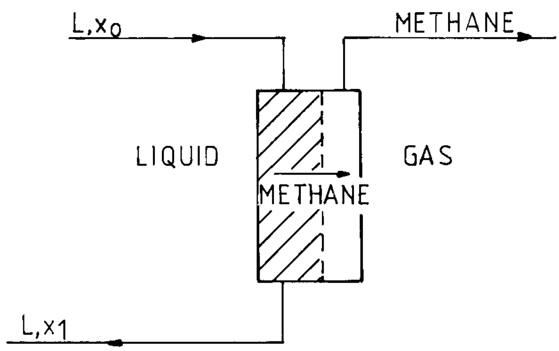


Figure 2.

If M m^3/s of water flows through the appliance the methane transfer is $L(x_0\text{-}x_1)$, which must be equal to β A (x_1-x*). From this it follows:

$$\frac{x_1 - x^*}{x_0 - x^*} = \frac{1}{1 + N}$$

$$\text{where } N = \frac{B\,A}{L} = \frac{\beta\, a\,(\text{container volume})}{L}$$

In order to reduce over-saturation by a factor of 10, N must have a value of 9. If L = 1 m^3/s the container volume for spray column 1 is as follows:

$$\text{container volume} = \frac{9L}{\beta a} = \frac{9 \times 1\ m^3 s}{(5 \times 10^{-6}\ m/s)\ (300\ m^2/m^3)} = 6{,}000\ m^3.$$

While the required container volume does depend heavily on the estimated droplet size, very large tanks will nevertheless be required. In the case of bubble column 2:

$$\text{container volume} = \frac{9L}{\beta a} = \frac{9 \times 1\ m^3/s}{(3 \times 10^{-4}\ m/s)\ (300\ m^2/m^3)} = 100\ m^3$$

No remixing in the liquid occurs in the packed column.
In this instance over-saturation declines as follows:

$$\frac{x_1 - x^*}{x_0 - x^*} = \exp(-N)$$

(To simplify matters the derivation of this formula has been passed over.)

If $\frac{x_1 - x^*}{x_0 - x^*} = 1/10$, N = 2.3, so that:

$$\text{container volume} = \frac{2.3L}{\beta a} = \frac{2.3 \times 1\ m^3/s}{(5 \times 10^{-5}\ m/s)\ (300\ m^2/m^3} = 150\ m^3$$

i.e. the same order of magnitude as the bubble column.

In the case of the settlement basin (no. 4) it has been assumed that very small bubbles are dragged along in the channels between the partitions in the container. These bubbles grow as a result of over-saturation and achieve an increasing slip velocity in relation to the liquid. When the bubbles reach the left-hand edge of the channel they float upwards against the partition and are released.

To begin with we may calculate the growth and the height of rise of a bubble in relation to the liquid. For convenience sake over-saturation may be assumed to have a constant value

$$\frac{x - x^*}{x^*} \simeq 1/2$$

The material transport to a bubble determines its rate of growth:

$$\beta A_{Bub.}\,(x - x^*) = y.A_{bub.}\,\frac{dr}{dt}$$

whereby the bubble surface area A bub. disappears from the equation. The other symbols are as before; y is the concentration of methane in the gas phase ($y \simeq 30\, x^*$).

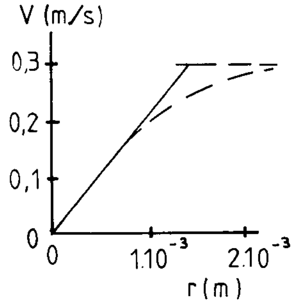


Figure 3.

If the equation is solved for an assumed constant value for the over-saturation ($x - x^*$), we obtain the following formula for the radius of a bubble as a function of time:

$$r = \frac{\beta\,(x - x^*)}{y}\,t$$

With $\frac{x - x^*}{x^*} \simeq 1/2$ and $y = 30\,x^*$, $\beta = 3 \times 10^{-4}$ m/s,

$r - (1/2 \times 10^{-5}\ \text{m/s})\ t.$

The rate of climb of small bubbles is given by:

$$V = \frac{2}{9}\,\frac{\rho l g r^2}{\eta}$$

where $\rho l = 10^3$ kg/m^3 (fluid density)

$g = 10$ m/s^2 (acceleration of gravity)

$\eta = 10^{-3}$ Ns/m^2 (fluid viscosity) therefore $V = (2/9 \times 10^7\ 1/3)\,(r/m)^2$

In the case of bigger bubbles the rate of climb rises to a constant value of approx. 0.3 m/s (see Figure 4). This stage is however less relevant for out observations.

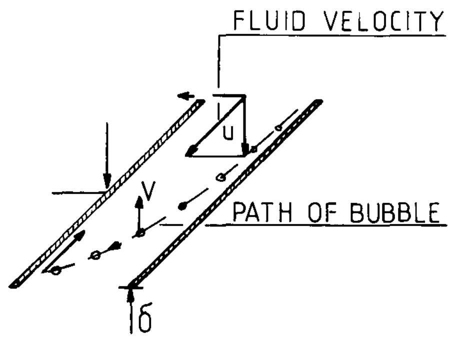


Figure 4.

This makes the height of rise of a large bubble in a stationary liquid:

$$s = \int_0^t v\,dt = \int_0^t \left(\frac{2}{3} \times 10^7\right) r^2 dt = \int_0^t \frac{1}{18} \times 10^{-3})\, t^2 dt = \frac{1}{54} \times 10^{-3} t^3$$

Given a settlement basin with parallel inclined partitions, the bubbles must be able to traverse the vertical distance between the partitions ($\simeq 5 \times 10^{-2}$ m) during the time which the liquid spends in the container. This means that the required time spent in the container is:

$t = (54 \times 10^3 \times 5 \times 10^{-2})^{1/3} - 20s$
while the partition volume required is 20 m^3.
In view of its attractiveness this solution has been worked out in more detail.

The flow between the partitions must not be turbulent, i.e.:

$$\frac{\rho u \sigma}{\eta} \lesssim 500$$

Where u is the vertical liquid velocity and σ is the vertical gap between the partitions.
Thus:

$$u \lesssim \frac{500 \times 10^{-3}\ Ns/m^2}{10^3\ kg/m^3 \times 3 \times 10^{-2}\ m} = 0.07\ m/s$$

The settlement basin therefore requires a vertical corss-section of 1 m^3/s – 15 m^2, which would seem perfectly feasible.

In the series of Rijkswaterstaat Communications the following numbers have been previously published

No. 1. *Tidal Computations in Shallow Water*
Dr. J. J. Dronkers† and prof. dr. ir. J. C. Schönfeld
Report on Hydrostatic Leveling across the Westerschelde
Ir. A. Waalewijn, 1959

No. 2. *Computations of the Decca Pattern for the Netherlands Delta Works*
Ir. H. Ph. van der Schaaf† and P. Vetterli, Ing. Dipl. E. T. H., 1960

No. 3. *The Aging of Asphaltic Bitumen*
Ir. A. J. P. van der Burgh, J. P. Bouwman and G. M. A. Steffelaar, 1962

No. 4. *Mud Distribution and Land Reclamation in the Eastern Wadden Shallows*
Dr. L. F. Kamps†, 1962

No. 5. *Modern Construction of Wing-Gates*
Ir. J. C. le Nobel, 1964

No. 6. *A Structure Plan for the Southern IJsselmeerpolders*
Board of the Zuyder Zee Works, 1964

No. 7. *The Use of Explosives for Clearing Ice*
Ir. J. van der Kley, 1965

No. 8. *The Design and Construction of the Van Brienenoord Bridge across the River Nieuwe Maas*
Ir. W. J. van der Eb†, 1968

No. 9. *Electronic Computation of Water Levels in Rivers during High Discharges*
Section River Studies. Directie Bovenrivieren of Rijkswaterstaat, 1969

No. 10. *The Canalization of the Lower Rhine*
Ir. A. C. de Gaay and ir. P. Blokland†, 1970

No. 11. *The Haringvliet Sluices*
Ir. H. A. Ferguson, ir. P. Blokland† and ir. drs. H. Kuiper, 1970

No. 12. *The Application of Piecewise Polynomials to Problems of Curve and Surface Approximation*
Dr. Kurt Kubik, 1971

No. 13. *Systems for Automatic Computation and Plotting of Position Patterns*
Ir. H. Ph. van der Schaaf†, 1972

No. 14. *The Realization and Function of the Northern Basin of the Delta Project*
Deltadienst of Rijkswaterstaat, 1973

No. 15. *Fysical-Engineering Model of Reinforced Concrete Frames in Compression*
Ir. J. Blaauwendraad, 1973

No. 16. *Navigation Locks for Push Tows*
Ir. C. Kooman, 1973

No. 17. *Pneumatic Barriers to reduce Salt Intrusion through Locks*
Dr. ir. G. Abraham, ir. P. van der Burg and ir. P. de Vos, 1973

No. 18. *Experiences with Mathematical Models used for Water Quality and Water Problems*
Ir. J. Voogt and dr. ir. C. B. Vreugdenhil, 1974

No. 19. *Sand Stabilization and Dune Building*
Dr. M. J. Adriani and dr. J. H. J. Terwindt, 1974

No. 20. *The Road-Picture as a Touchstone for the three dimensional Design of Roads*
Ir. J. F. Springer and ir. K. E. Huizinga (also in German), 1975

No. 21. *Push Tows in Canals*
Ir. J. Koster, 1975

No. 22. *Lock Capacity and Traffic Resistance of Locks*
Ir. C. Kooman and P. A. de Bruijn, 1975

No. 23. *Computer Calculations of a Complex Steel Bridge verified by Model Investigations*
Ir. Th. H. Kayser and ir. J. Brinkhorst, 1975

No. 24. *The Kreekrak Locks on the Scheldt-Rhine Connection*
Ir. P. A. Kolkman and ir. J. C. Slagter, 1976

No. 25. *Motorway Tunnels built by the Immersed Tube Method*
Ir. A. Glerum, ir. B. P. Rigter, ir. W. D. Eysink and W. F. Heins, 1976

No. 26. *Salt Distribution in Estuaries*
Rijkswaterstaat, Delft University of Technology, Delft Hydraulics Laboratory, 1976

No. 27. *Asphalt Revetment of Dyke Slopes*
Committee on the Compactions of Asphalt Revetments of Dyke Slopes, 1977

No. 28. *Calculation Methods for Two-dimensional Groundwater Flow*
Dr. ir. P. van der Veer, 1978

No. 29. *Ten Years of Quality Control in Road Construction in the Netherlands*
Ir. C. van de Fliert and ir. H. Schram, 1979

No. 30. *Digital Large Scale Restitution and Map Compilation*
J. G. van der Kraan, H. Rietveld, M. Tienstra and W. J. H. IJzereef, 1980

No. 31. *Policy Analysis for the National Water Management of the Netherlands*
(Netherlands Contributions, related to the PAWN-study, for the ECE-seminar on Economic Instruments for the Rational Utilization of Water Resources – Veldhoven, Netherlands – 1980)

No. 32. *Changing Estuaries*
Dr. H. L. F. Saeijs, 1982

No. 33. *A Bird's Eye View of the Shipping Traffic on the North Sea*
North Sea Directorate, 1982

No. 34. *Contributions to Remote Sensing: Applications of Thermal Infrared*
H. W. Brunsveld van Hulten (ed.), P. Hoogeboom, A. F. G. Jacobs, C. Kraan, G. P. de Loor and L. Wartena, 1984

No. 35. *On the Construction of Computational Methods for Shallow Water Flow Problems*
Dr. ir. G. S. Stelling, 1984

No. 36. *Wrong-way driving*
Ir. G. A. Brevoord, 1984

No. 37. *The Use of Asphalt in Hydraulic Engineering*
Technical Advisory Committee on Waterdefences, 1985

No. 38. *Recycling of Road Pavement Materials in the Netherlands*
J. J. A. Gerardu and C. T. Hendriks, 1985.