Modelling the Pumping Scheme Associated with an Impervious Wall to Avoid the Groundwater Pollution from a Landfill

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ABSTRACT

The best way to make use of all the available data and to optimize the means that have to be mobilized to protect an aquifer from a landfill pollution, is undoubtedly the numerical simulation of the polluted water flow in the porous geological medium. The application described here, consists in a standard case of a landfill, leaking water polluted with organic compounds, in an underlying sandstone aquifer.

A F.E.M. simulation is completed including a detailed calibration procedure due to the heterogeneous properties of the aquifer that have been measured in situ.

The main results and conclusions of this study are commented and illustrated in order to show how the numerical simulations have helped the decision-makers in their environmental concern.

INTRODUCTION

The selection of a site in view of waste dumping is mainly function of the geological and hydrogeological conditions. Indeed, the main environmental concerns are :

- the pollution risk of the groundwater aquifers,
- the slopes stability and any modification of the natural conditions induced by the dump site,
- the landscape aspect,
- the different nuisances: odors, rats, papers and eventually truck traffic.

The two first problems are more fundamental and depend principally on the geological and hydrogeological characteristics. The third and especially the fourth ones, are functions of the landfill setting and operating conditions and consequently in normal cases, it is easier to palliate them [1] .

The risk of pollution of the groundwater is linked to the leakage of the infiltration water through the waste products. In Belgium, the annual averaged rainfall is about 750 mm (or 750 liter/m2). In a normal vegetal environment, 500 to 550 mm/year correspond to the evapotranspiration by plant roots and transpiration. From the remaining 200 to 250 mm, another part is immediately driven away by the surface runoff and only the water left is really infiltrated into the aquifers.

These values are reliable in Belgium, for a natural ground covered with vegetation but not for an active dump or disposal site. In this last case, no vegetation is present and the surface runoff is very weak. After evaporation, the remaining water infiltrates and percolates through the waste products and, of course, becomes highly polluted. In Belgium, the percolating volume of water is estimated to be 4000 to 5000 m3/km2/year, corresponding to 400-500 mm/year of infiltrating water for a landfill in full activity. For a closed disposal site with a restored vegetal cover, a mean percolating volume of about 2000 m3/km2/year is estimated and can be reduced to a few hundred if the dump has been covered previously by an impervious geotextile (Monjoie, 1990).

In any case, the percolating water is to be collected in integrality to avoid the pollution of the groundwater. If the actual volume of collected water does not reach the approximations mentioned here above, the landfill is certainly leaking and is likely provoking a pollution of the aquifers.

It is very important that the potential landfill sites would be characterized by a satisfying imperviousness ; it is evident that the naturally impervious sites are particularly propitious.

In these conditions, the best sites are often small valleys or vales in impervious layers consisting of shales or clays. The imperviousness is ensured by the impervious substratum and the superficial drainage by the altered colluvial mantel driving the collected polluted water downwards in the valley, where an adequate treatment can be done. Moreover, these sites are generally characterized by their weak agricultural or forest lumbering income, encouraging their selection as future disposal site for household or municipal wastes.

Another solution consists in embankments over an impervious substratum (figure 1), a draining trench cutting the superficial layers or the colluvial mantel is then necessary to collect the percolating polluted water. This drain can be extended upwards by superficial drains ensuring a better collection of the percolates in the embankments, contributing also, in this way, to the consolidation and to the stabilization of the waste products [1].

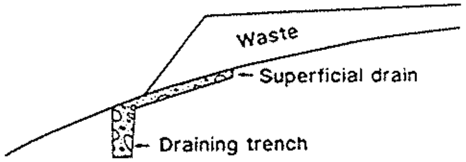

Fig.1 Waste disposal in embankment with an underlying superficial drain and a draining deep trench located downwards.

Usually, the old open-pit sites, except maybe the clay-pit sites, do not present the required characteristics of natural imperviousness. A sand-pit on a shaly substratum, limestone or sandstone old quarries or open-pits (figure 2) are rarely affected by a total lateral and/or vertical imperviousness. These different potential waste disposal sites have to be improved by artificial impermeability systems. More often, it consists in using impervious geomembranes (or geo-textiles). But in case of awaited instabilities (e.g. differential compactions) an additional clayey bedding is recommended.

Despite all these precautionary measures, in fissured limestones and chalks or in calcareous sandstones, where aquifers are very productive but also highly vulnerable, it is still not recommendable to place a disposal site for waste products. The choice of such a site can be considered for the dump of inert waste products at the unavoidable condition that all the percolating water could be collected adequately (by pumping wells or underlying galleries) in the case of accidental spill concerning other products than those which were prescribed (figure 3) [1].

In Belgium, unfortunately, many disposal sites have been chosen previously, without taking into account all these recommendations. The cost of cleaning up the most severe cases of induced contamination can be estimated to millions of dollars.

Choices have to be made between the measures to be taken in order to improve and eventually to restore the groundwater quality. Various types of remedial measures can be developed and the use of models should provide the opportunity to compare the different alternatives for their effectiveness.

As mentioned by Fetter [2], there are mainly two broad categories of remedial measures: one must either remove or isolate the source on one hand, and pump and treat the groundwater on the other hand.

MAIN CHARACTERISTICS OF THE STUDIED CASE

The case described here is relevant to the optimization of the isolation scheme and of the pumping system for a landfill located in an old open-pit in calcareous sandstones.

The first local geological and geotechnical studies have revealed that a marl layer was lying under the calcareous sandstones. One of the main questions arising was to know if this layer was sufficiently continuous and thick. In this last case, it could probably limit the polluted water migration into the underlying aquifers.

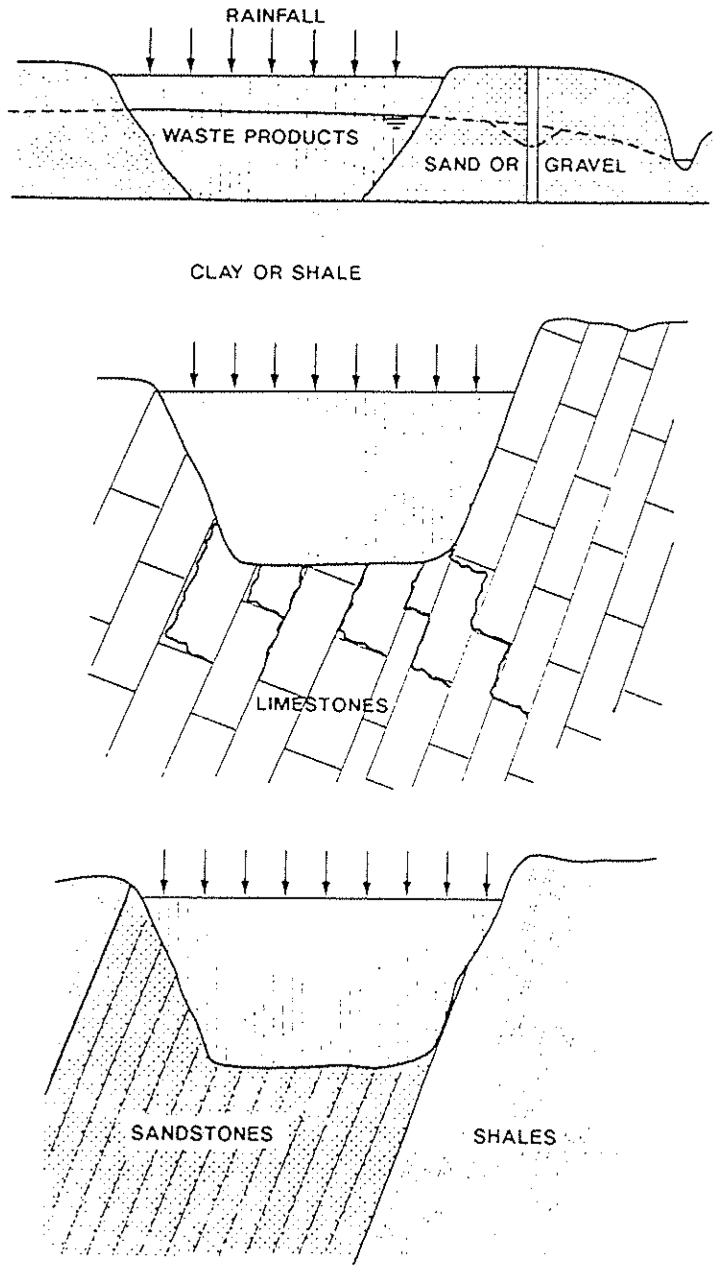

Fig.2 Different cases of abandoned open-pits, used now as waste disposal sites.

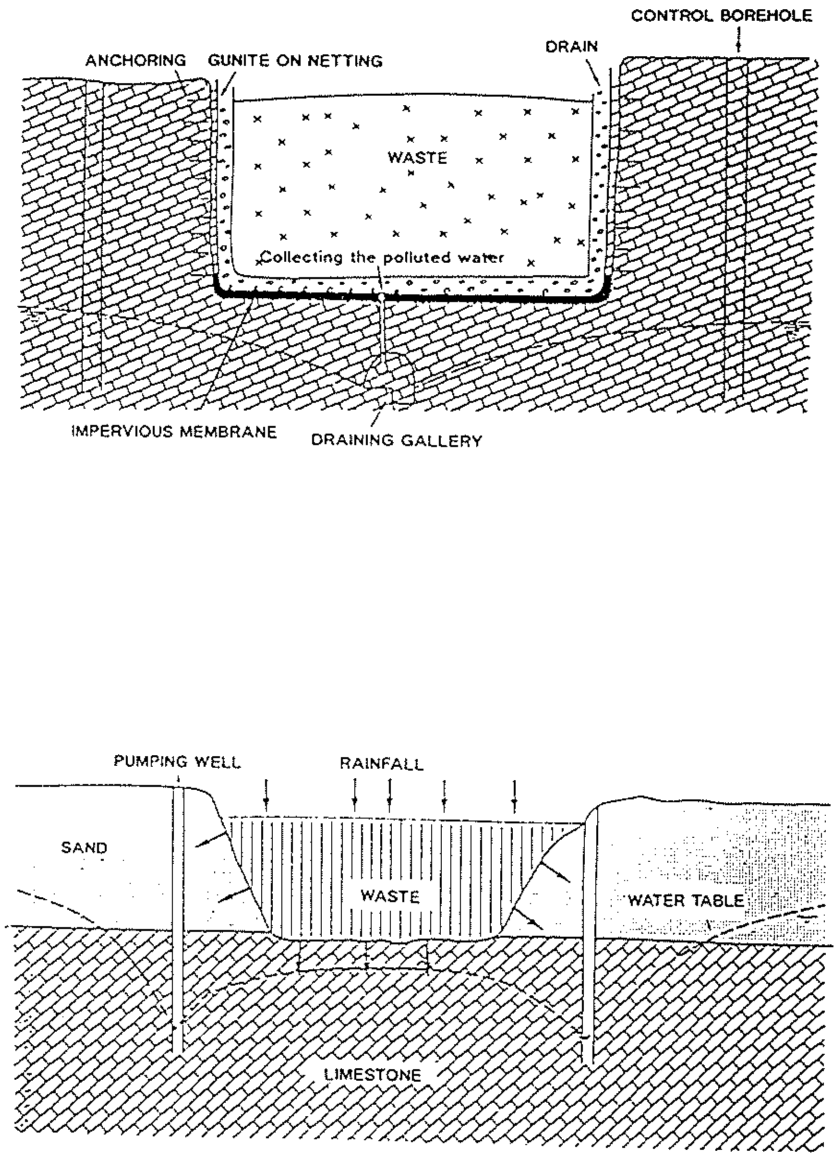

Fig.3 Adequate collecting systems for waste disposals sites located in unpropitious geological conditions (from Monjoie [1]).

A detailed study of the site has lead to define accurately the hydrogeological conditions and has determined: (1) the extension and the thickness of the marl layer at the site and around the site; (2) its permeability by Lugeon tests; (3) the chemical and the geomechanical behaviours of the organic waste products; (4) the degree of contamination of the groundwater.

It has been found that the marl layer is located between two sandstone aquifers. Its very low permeability and its thickness determined between 10 and 15 meters, by electrical resistivity geosounding correlated with boreholes, reveal that we can consider that it plays the role of a very effective screen or impervious layer through which the polluted water can probably not leak more deeply. Moreover, the underlying confined aquifer is characterized by higher piezometric heads than those in the superficial water table aquifer. In these conditions, if a flow exists in the marl layer it must be an uplift flow, so that no polluted water can reach the underlying confined aquifer.

On the contrary, the water table aquifer is highly polluted by the percolating water from the landfill deposits. This aquifer shows a gradient mostly directed westwards (figure 4), and in the western part, it is in complete connection with an alluvial aquifer. Indeed, the last one is also contaminated.

The organic compounds found in the content of the polluted water are not of high toxicity and concentrations in heavy metals are still lower than the legal standard for industrial water in Belgium. However, a collecting system and the treatment of the polluted water have been prescribed.

Fourteen boreholes have been drilled and equipped as piezometers in the upper sandstone aquifer and in the alluvial aquifer. The conductivity, the temperature and the pH of the water are regularly measured. Water samples are analyzed to determine their pollutants contents. Hydraulic conductivity values of this water table aquifer have been determined on basis of five pumping tests using in each case the other boreholes as piezometers. The transmissivity defined as the product $T = K.e$, with e being the saturated thickness is found in different zones of the studied area.

The results of the chemical analysis on the waste products, the percolating waters and the aquifer groundwater show that the contamination of the aquifer is relatively weak in respect of the chemical nature of the waste products. Three main reasons can be invoked explaining this fact:

- the low permeability coefficient of the peaty and clayey waste product itself, inducing a very weak percolation;
- the basic pH of the percolating waters reducing the metals solubilization;
- the dilution of the percolates in the sandstone aquifer characterized by high permeability coefficients.

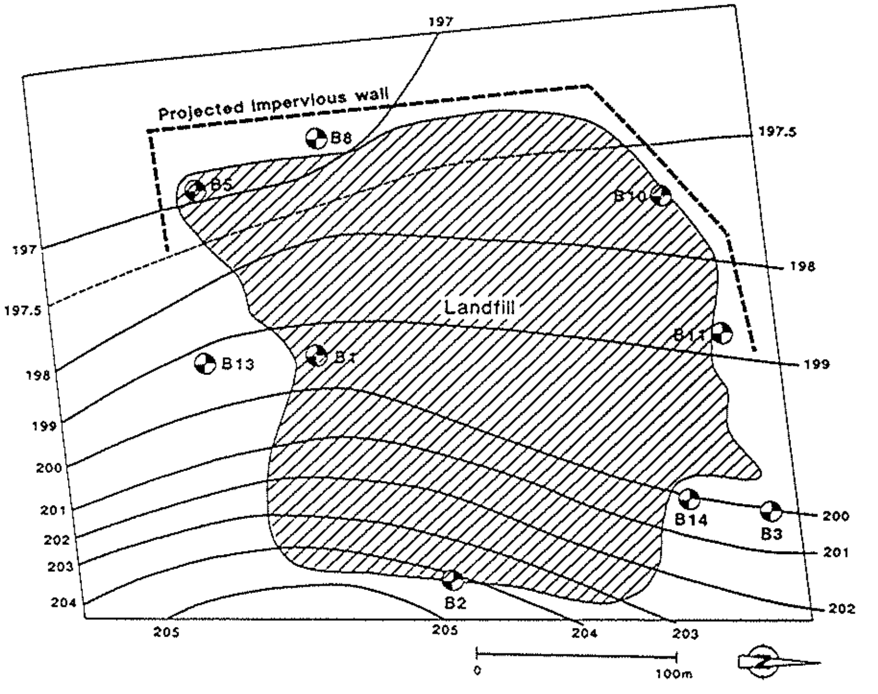

Fig.4 Measured piezometric map of the aquifer in the zone of the studied landfill site.

DATA AND CONDITIONS OF THE MODEL

On basis of all the collected data, a groundwater flow model using the Finite Element Method has been elaborated ([3],[4],[5],[6] and many others). The program used is a very common code running on IBM PC-386. A 2D approach is realized considering that the aquifer thickness (i.e. the saturated thickness) is implicitly taken into account in the transmissivity value affected at each element of the meshing network [7]. Of course, this vertical integration introduces a non linearity of the transmissivity, solved numerically by successive iterations on its value [8].

The existing boreholes (figure 4), located in the studied zone should be chosen preferably as permanent pumping well for the polluted water. One or two additional boreholes could be chosen optimally in conjunction with the design of an impervious flushed wall, located in the downward part of the aquifer and just upwards from the connection with the alluvial aquifer (figure 4).

The finite element discretization (figure 5) includes the whole landfill zone and is prolonged on the different sides in order to carry the boundary conditions forward from the

main stressed area [9].
The lateral boundary conditions are prescribed piezometric heads, as no impervious boundaries can be found in this hydrogeological environment (described above). An impervious basement of the model is assumed as the marl layer is continuous and characterized by permeabilities certainly 10^{-4} to 10^{-5} lower than in the water table aquifer.

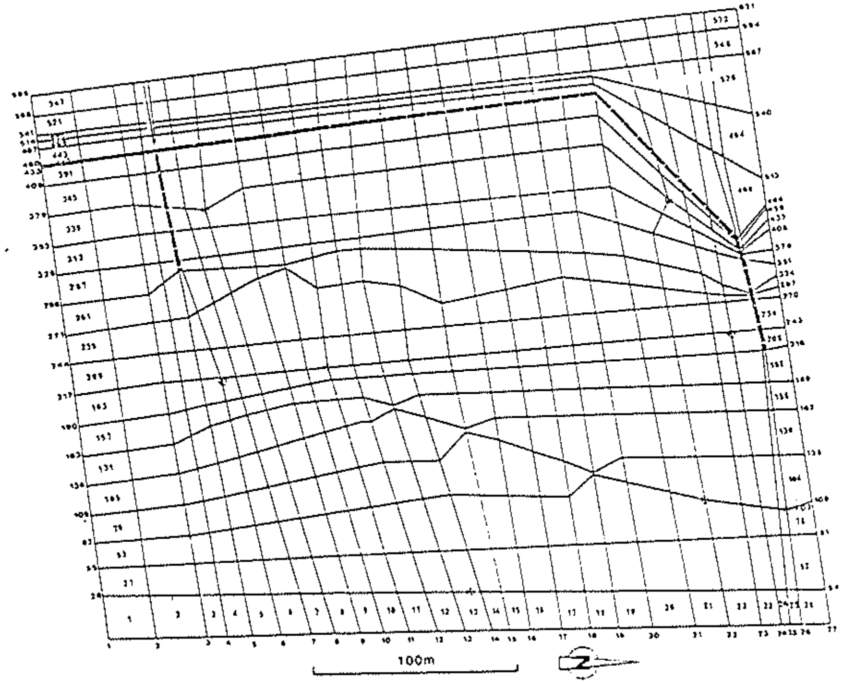

Fig.5 Finite element discretization of the studied area.

Infiltration due to the rainfall (as explained above) is taken to a maximum value of 600 mm/year in agreement with the hydrological studies in the region. The infiltrating water leaking through the waste deposits is the pollution source and the flushed walls assorted with the pumping wells have to be designed in order to collect all the contaminated waters.

CALIBRATION

A calibration procedure has been completed adjusting the values and the spatial distribution of the transmissivity, so that the computed piezometric heads would be very close to the measured heads for a same stress of the system. This procedure requires good experience of the hydrogeologist in order to obtain a reliable calibration ([10],[11]). In this case study, the procedure has begun dividing the area of the model in three main zones characterized by different transmissivities (figure 6): zone I (upwards) $T = 1.3 - 1.9\ 10^{-5} m^2/s$, zone II (central) $T = 2.0 - 6.0\ 10^{-3} m^2/s$, zone III (downwards) $T = 1.9\text{-}2.7\ 10^{-3} m^2/s$. The first runs of the calibration have shown that the aquifer has mainly the behaviour of a confined aquifer under the clayey waste products.

The calibration procedure has been completed step by step and finally it has taken more than 40 runs to obtain the computed piezometric map shown at the figure 7 corresponding to the following transmissivity values :

- zone I A : $T = 1.0\ 10^{-4}\ m^2/s$
- zone I B : $T = 2.0\ 10^{-5}\ m^2/s$
- zone I C : $T = 1.0\ 10^{-3}\ m^2/s$
- zone II A : $T = 5.0\ 10^{-4}\ m^2/s$
- zone II B : $T = 4.0\ 10^{-3}\ m^2/s$
- zone III A : $T = 1.0\ 10^{-3}\ m^2/s$

The upward zone is characterized by highly heterogeneous values of the transmissivity. Very high values have to be distinguished in the zone I C in order to compute a piezometric map reasonably in agreement with the measurements [12]. If any additional investigation is to be foreseen in the future, it is in this particular zone that the borehole should be located.

RESULTS OF THE SIMULATIONS

Many simulations have been completed varying the wall geometry and the pumping scheme. Only some of the results are summarized in the following lines :

A) The simulation is realized with the impervious wall as described before (figure 4) and a total pumping of 450 m^3/day corresponding to 120 m^3/day in both boreholes B5 and B1, 100 m^3/day in B8 and 25 m^3/d in B10 and B11. An additional pumping of 60 m^3/d is localized at the node 424 of the discretized mesh (figure 8).
The computed piezometric map (figure 8) shows that near the southern boundary of the model, some streamlines not converging to the pumping wells can be drawn. In this case, it is then possible that after the vertical leakage through the landfill, the polluted water joining the aquifer would not be collected by the pumping system. For the next runs, this pumping scheme is changed in order to avoid such a situation.

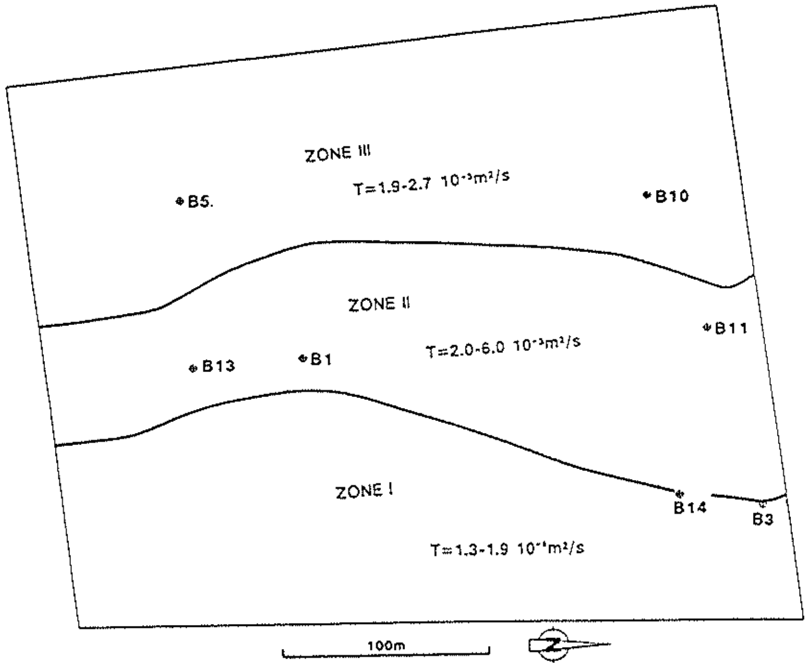

Fig.6 Initial transmissivity values and their distribution.

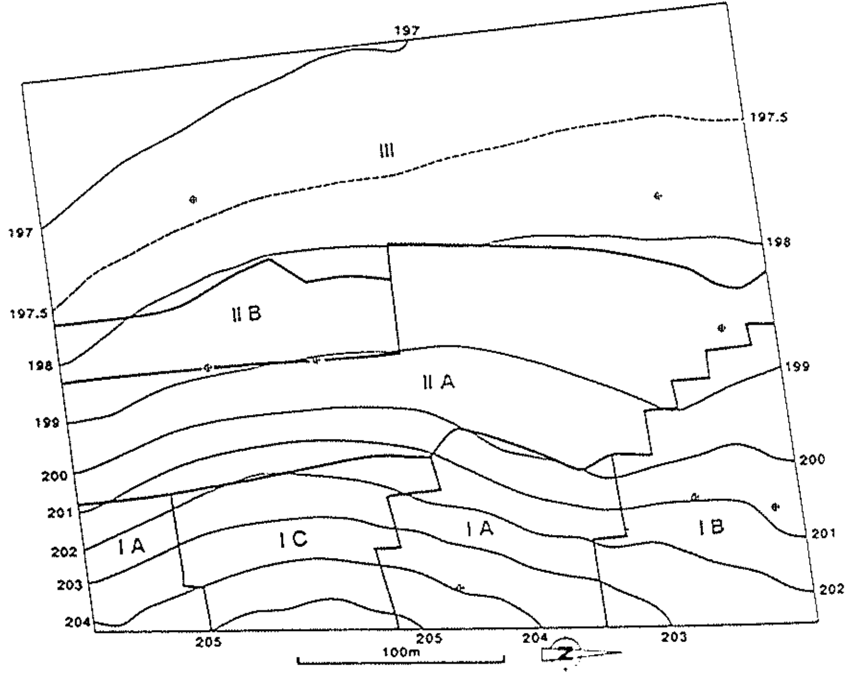

Fig.7 Final values and distribution of the transmissivity after calibration.

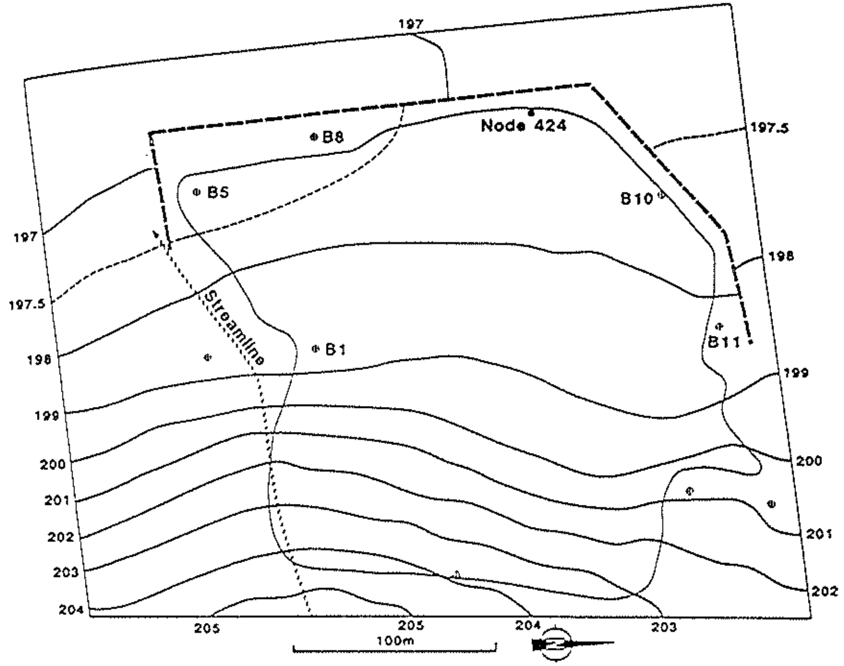

Fig.8 Results of the run (A), some streamlines passing under the landfill can escape from the pumping wells.

B) With a total withdrawal of 475 m^3/d, corresponding to 200 m^3/d in B1, 150 m^3/d in B5, 100 m^3/d in B13 and 25 m^3/d in B12, the second simulation shows clearly a better situation than in (A) (figure 9).
No streamline can be drawn under the disposal site without being directed to the pumping wells. This scheme which consists in concentrating the main values of pumping on B5 and B1, located in the high permeability zone, is apparently propitious to collect the maximum of polluted water.

C) With another geometry of the wall and a withdrawal concentrated on B5 (300 m^3/d), B1 (150 m^3/d) and B11 (25 m^3/d, the computed piezometry (figure 10) is also propitious for collecting the contaminated water.
However for the same order of pumping, the total length of the wall is increased in comparison with the solution (B).

D) The wall is prolonged eastwards (figure 11) closing nearly completely the high permeability zone located near the boreholes B13 and B1. The pumping scheme can be limited to 200 m^3/d in B5, 125 m^3/d in B1 and 25 m^3/d in B11 (figure 11) keeping an acceptable solution for the collection of the contaminated water. It is shown that this prolongation of the wall allows to pump less in the wells.

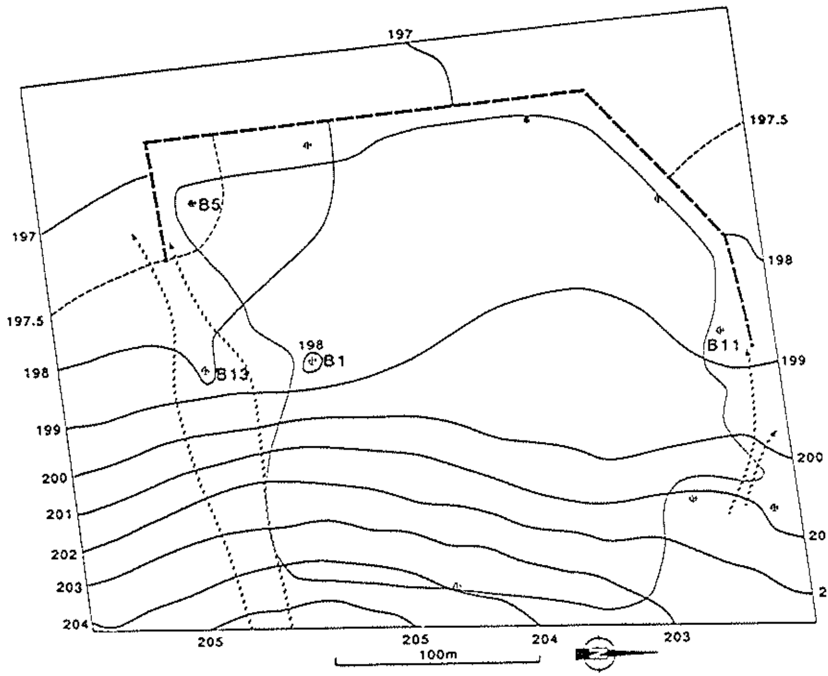

Fig.9 Results of the run (B).

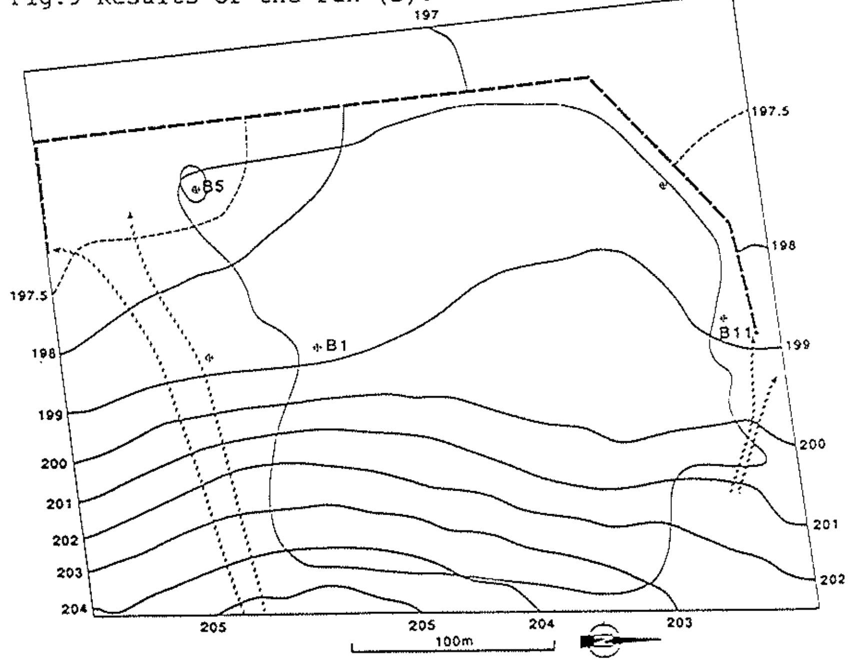

Fig.10 Results of the run (C).

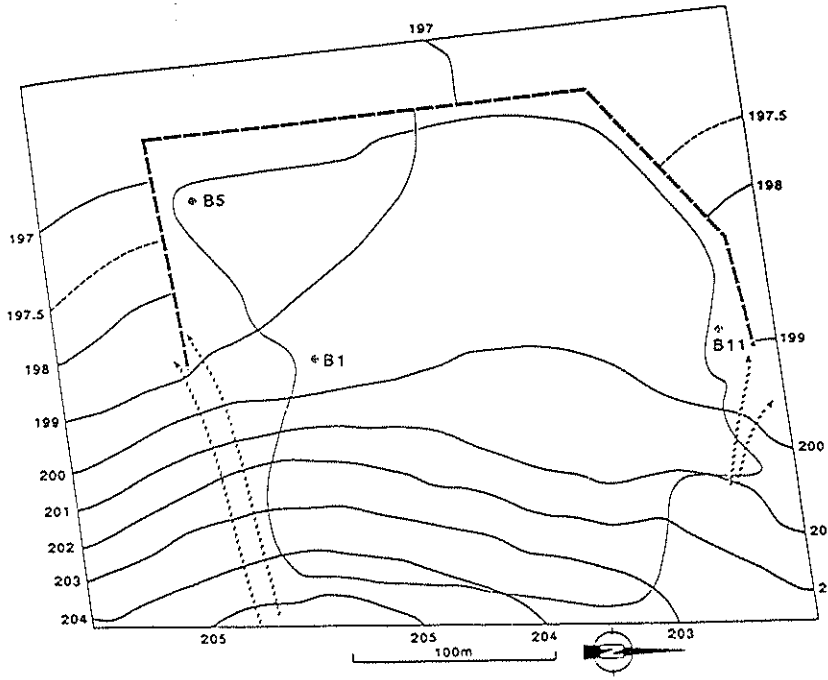

Fig.11 Results of the run (D).

Many other simulations with various pumping schemes and different geometries of the wall did not bring more advantageous situations than those shown in the cases (B), (C), and (D).

CONCLUSIONS

It has to be recalled that, during the calibration procedure, the upwards zone of the studied area appeared more heterogeneous than thought before. In a particular zone (I C), a transmissivity value of about 1.0 10^{-3} m^2/s has to be chosen in order to match the measured piezometric levels. This very high permeability affecting elements located near the prescribed head boundary of the model, induces large flow entering in the model along this zone. These flows are probably larger than in the reality and this fact gives to our computations an additional security factor.

Even if this model is limited to steady conditions and does not take into account the eventual diffusion of pollutant, the results give a good idea of the situation.
On basis of the different pumping schemes and the wall geometries that have been simulated, general trends can be deduced : (1) it will be necessary to pump about 450 to 500 m^3/d; (2) the optimal pumping scheme depends strongly on the chosen geometry of the wall and conversely.

Of course, additional data would lead to more accurate results, but the results shown here above, bring the necessary information that decision-makers need before beginning the important remedial works. In this sense, we can consider that this modelling exercise was particularly beneficial as the task remaining now consists in choosing between two or three collecting systems, considering from that moment only the financial aspect.

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