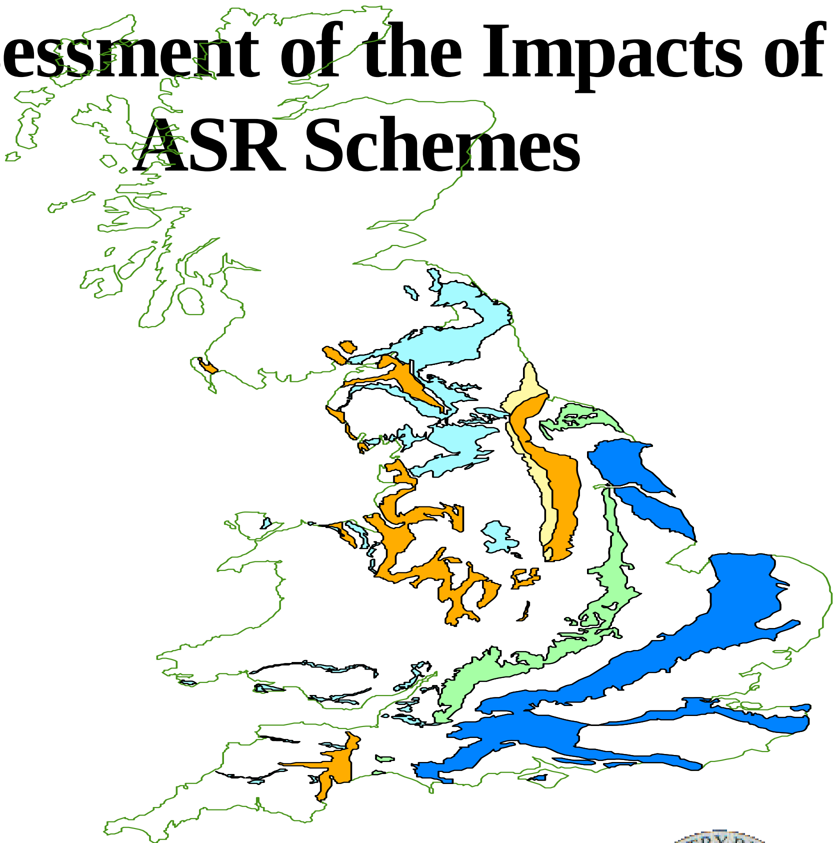
ASR - UK

Assessment of the Impacts of ASR Schemes



BRITISH GEOLOGICAL SURVEY

COMMERCIAL REPORT CR/01/153

Assessment of the Environmental Impacts of ASR Schemes

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Key words

ASR, model, hydrogeology.

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Foreword

This interim report presents the results of part of a study undertaken by the British Geological Survey (BGS) Wallingford. The project is co-funded by UKWIR Ltd and a Foresight LINK award from the Office of Science and Technology (OST). The study began in April 1999 and is due for completion in September 2001.

Acknowledgements

The need for this study was identified by the ASR-UK Steering Group who provided a forum for discussion of the nature and extent of the study as well as review of the draft report. Andrew Hughes is also acknowledged for useful discussions and Ian Gale for peer review as project leader.

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Summary

This report describes the results of modelling studies undertaken to assess the impacts of ASR on the local environment. Understanding and quantifying these impacts, in relation to other existing or proposed schemes, will be vital in the development, and subsequent licensing of any ASR scheme. As each individual scheme has its own hydrogeological and environmental setting, as well as operational requirements, an all-encompassing model cannot be prescribed. Rather, a set of models, of increasing complexity, have been run for 'typical' scenarios to illustrate their use and limitations. They are designed to act as screening tools to assist practitioners, at all stages of an investigation, to decide on the suitability of a site and to identify what additional data are required in order to proceed to the next stage. The models are appended to the report so practitioners can apply them to their specific site, as appropriate.

1 Introduction

An important aspect to be considered at the start of an ASR scheme is the effect that the scheme may have on the natural environment. Of particular concern is the impact that the scheme may have on water levels in adjacent aquifers or in an unconfined zone of the target aquifer. As ASR schemes are not designed to constitute a net abstraction from the aquifer, the normal water resources considerations about environmental impact are not relevant and different considerations are necessary. In the long-term there may be some accretion of water to the aquifer, but this should be a small volume relative to the overall resource. However, it does not necessarily follow that environmental impacts (e.g. water levels at outcrop) are negligible during the injection or abstraction phases.

A scheme which injects and then abstracts the same volume of water will, as a long-term steady state average, have no impact on water levels. However, the seasonality of the scheme means that water levels will first be increased and then reduced. The timing and absolute value of these changes are of importance to assessing the eventual impact of the scheme. The work described here investigates the short term effect on water levels and flows caused by the addition of water to the aquifer, and its subsequent removal.

Several situations have been considered and numerical and analytical models have been developed to examine these situations. The first is the case when the target aquifer is confined but is overlain (above the confining layer) by another aquifer (in this case, an unconfined aquifer). The impact that injecting water into the underlying aquifer has on the upper aquifer will be considered in terms of a change in flow between the two aquifers. Another case which has been investigated is that of a target (confined) aquifer which outcrops at some distance from the proposed ASR scheme. The effect on water levels at the point where the aquifer becomes confined has been estimated using a numerical model, for a variety of transmissivity values and aquifer dimensions. During this work consideration has also been given to the time at which the maximum effect occurs. The models which have been used to investigate these situations are included in this report and it is anticipated that these models will be used at an initial stage of any consideration of an ASR scheme, when parameters appropriate to the setting can be used. In this report, the models are described and examples of their use with parameters which might be appropriate to some UK aquifers are presented. The software is appended on disk.

It is envisaged that these ‘first-pass’ models will give one of three indications for a particular site: 1) The impact is certainly negligible 2) The impact is definitely significant and the site is not suitable 3) Further investigation is required, perhaps including detailed modelling.

2 Confined ASR target aquifer overlain by other aquifer.

2.1 MODEL DESCRIPTION

For the case of an aquifer which overlies the target aquifer and is separated by an aquitard (as shown schematically in Figure 2.1), an analytical approach can be used due to the inherent symmetry of the problem.

On initial inspection it would appear that the best way to measure the ‘impact’ on an overlying aquifer would be in terms of a change in head caused by the operation of the ASR scheme. However, further consideration suggests that this may not be the case. The first problem which is encountered is the question of where this change in head should be calculated. It is obvious that the maximum change will be directly above the abstraction point i.e. at the ASR well.

However it is equally obvious that the change at this point is not of any great interest as it will be in an area controlled by the operators of the ASR scheme and will be very much the worst case.

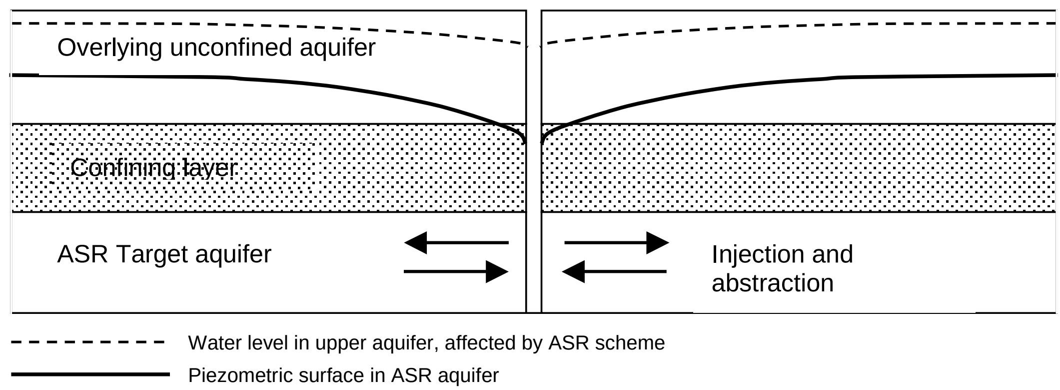


Figure 2.1 Schematic of modelled situation

The change in head at a specified distance (say 100 m), or distances, from the ASR well might be more useful and appropriate. The choice of distance should probably be influenced by the hydraulic properties of the overlying aquifer as a cone of depression will have a different shape depending on the permeability (a lower permeability aquifer will have a deeper, less extensive cone than a higher permeability aquifer). The estimated effect, in metres, at the chosen point would then be used to decide whether or not the ASR scheme would have a significant impact on the overlying aquifer. The significance of the impact would probably be compared to the natural groundwater level fluctuations expected on an annual basis, or some similar criterion. Thus the decisions which have to be taken are: a) where to estimate the change in head and b) with what to compare it.

Both of these decisions depend in part on the properties of the overlying aquifer. It would be more convenient to have a measure of the environmental impact of an ASR scheme which is independent of these properties. An ASR scheme will 'impact' on the overlying aquifer if it alters the water resources available in that aquifer. Thus a good measure of impact would be the flux created into or out of the upper aquifer as a consequence of operating the ASR scheme.

This value could sensibly be compared with the water resource available to the upper aquifer before commencement of an ASR scheme, as measured by the annual recharge rate, for instance. Alternatively, the leakage out of the upper aquifer could be considered as a new 'abstraction' within the aquifer, where the magnitude of the leakage is compared to an abstraction rate.

2.2 MATHEMATICAL DEVELOPMENT

An analytical solution to the flow equations (effectively an extension of the Theis solution) was developed and incorporated into a spreadsheet. This allows the impact of an ASR scheme operating under variable cycling regimes to be estimated. The relevant properties which are required are aquifer storage coefficient and aquitard vertical permeability, thickness and specific storage.

The assumptions used in developing the solution are:

1. The target aquifer, upper aquifer and the aquitard are effectively infinite in areal extent

2. All three are homogeneous, isotropic and of uniform thickness
3. Prior to pumping the water table and piezometric surfaces are horizontal
4. The well penetrates the full extent of the target aquifer, so that flow in the target aquifer is horizontal
5. Flow in the aquitard is vertical
6. Water removed from storage in the aquifer and the water supplied by leakage from the aquitard is discharged instantaneously with the decline in head
7. The diameter of the well is very small
8. There is no change in head in the upper aquifer.

Assumptions 1-7 are those used for the standard leaky aquifer solution for pumping test analysis. Assumption 8 is used to give a 'worst-case' for the rate of leakage from the upper aquifer into the lower aquifer. This is an appropriate assumption as it is the effect of the ASR scheme on the water resources of the upper aquifer that we are trying to predict.

The appropriate flow equations and their solution are described in Appendix 1. The approximate analytical solution has been incorporated into a spreadsheet which can be used to demonstrate the effect.

The parameters required by the solution are:

$$\tau = K_v\, t / S_s\, b^2$$

which is a dimensionless time and

$$\sigma = b\, S_s / S$$

which is the ratio of aquitard storage to storage in the lower (ASR) aquifer per unit area, where

t is the time since the start of pumping of the ASR well

K_v is the vertical hydraulic conductivity of the aquitard

S_s is the specific storage of the aquitard

S is the storage coefficient the target (ASR) aquifer

b is the thickness of the aquitard

Interestingly, the transmissivity of the target (ASR) aquifer is not a required parameter. This results from the fact that we are focussed on total leakage and not on its areal distribution. Another point of interest is that it is the properties of the aquitard which dominate. This is significant, as these parameters are often not well-known and this suggests that some effort should be focussed towards measuring these parameters. (We note that there is no database of aquitard parameters for the UK.)

2.3 EFFECT OF ONE ABSTRACTION CYCLE

Figure 2.2 shows the expected leakage from the upper aquifer to the lower aquifer for the parameters shown in Table 2.1, assuming no prior injection or abstraction

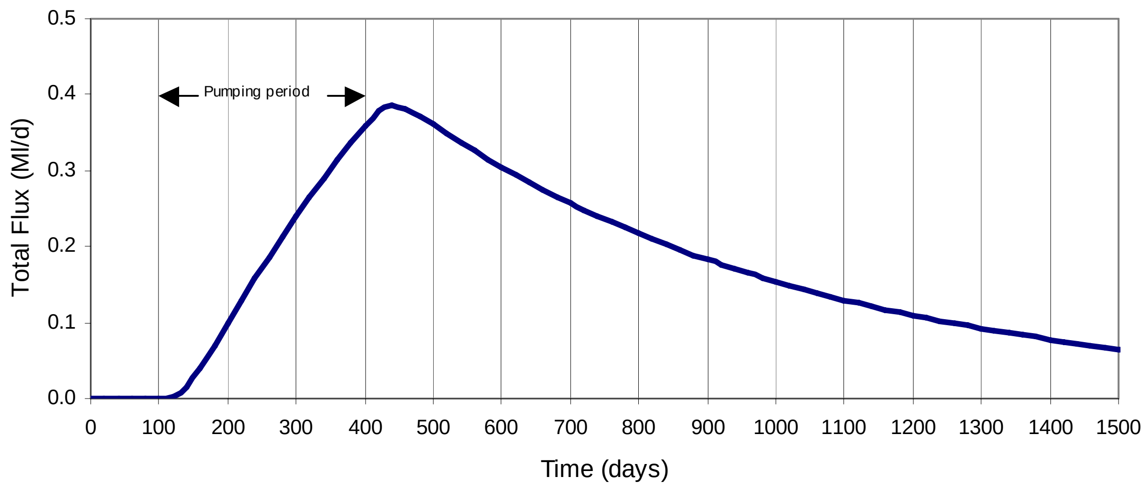


Figure 2.2 Leakage through aquitard from upper aquifer. Hydraulic properties in Table 2.1

As can be seen, leakage from the upper aquifer starts some time after the pumping starts and continues to increase even after the pumping in the lower aquifer has ceased. Leakage from the upper aquifer continues long after the end of pumping.

Table 2.1 Parameters for Figure 2.2

Aquitard vertical hydraulic conductivity (m/d)	1.0×10^{-5}
Aquitard specific storage (m^{-1})	1.0×10^{-6}
Aquitard thickness (m)	50
Aquifer storage coefficient (-)	0.0001
Pumping rate (Ml/d)	1
Time at start of pumping	100 days
Time of end of pumping	400 days

2.4 EFFECT OF FIVE ANNUAL CYCLES

The effect on the upper aquifer of an operational ASR scheme with injection and abstraction cycles is determined by adding together the response to each individual part of the cycle. This is done within the spreadsheet and the result for the 5 annual cycles described in Table 2.2 is shown in Figure 2.3.

Table 2.2 Parameters for Figure 2.3

Aquitard vertical hydraulic conductivity (m/d)	1.0×10^{-5}
Aquitard specific storage (m^{-1})	1.0×10^{-6}
Aquitard thickness (m)	50
Aquifer storage coefficient (-)	0.0001
($\tau = K\ t / S_s b^2 = 25$ d, $\sigma = b\ S_s / S = 0.5$)	
Annual ASR cycle: (5 annual cycles (360 days))	120 days injection, 90 days standing, 90 days abstraction, 60 days standing
Injection rate (Ml/d)	3
Abstraction rate (Ml/d)	4

This figure shows that the maximum leakage from the upper aquifer is about 0.3 Ml/d (less than $^1/_{10}$ th of the abstraction rate) and that this maximum leakage occurs over 60 days after the abstraction has ceased.

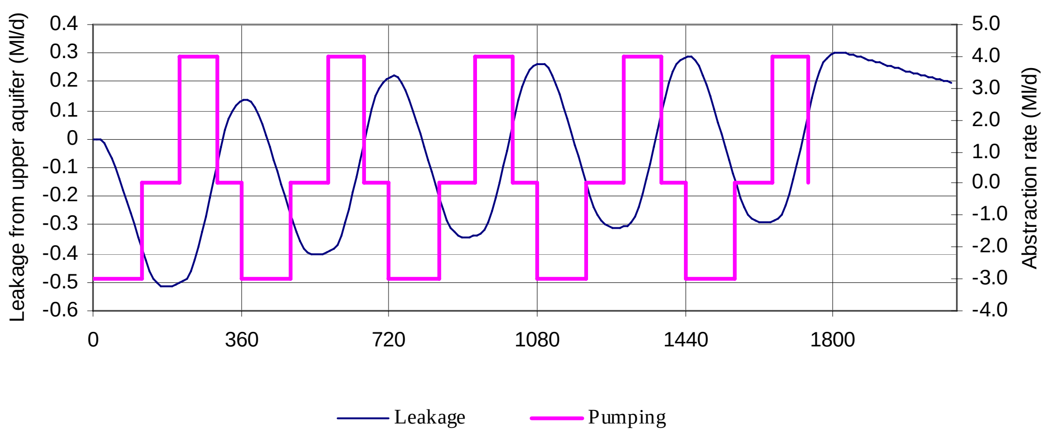
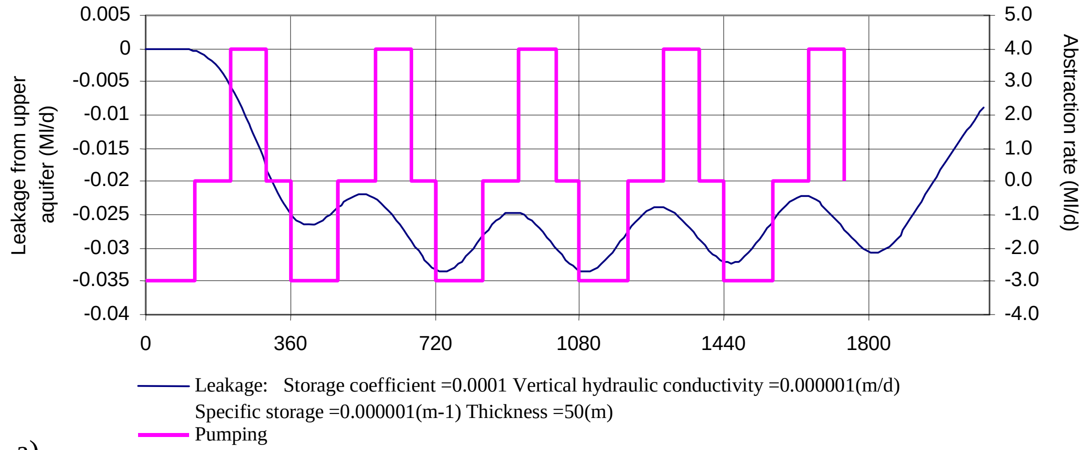


Figure 2.3 Effect of 5 annual ASR cycles on upper aquifer. Parameters in Table 2.2

Estimated leakages for a variety of other combinations of S, S_s, K_v, and b detailed in Table 2.3 are shown in Figure 2.4 a) to g).

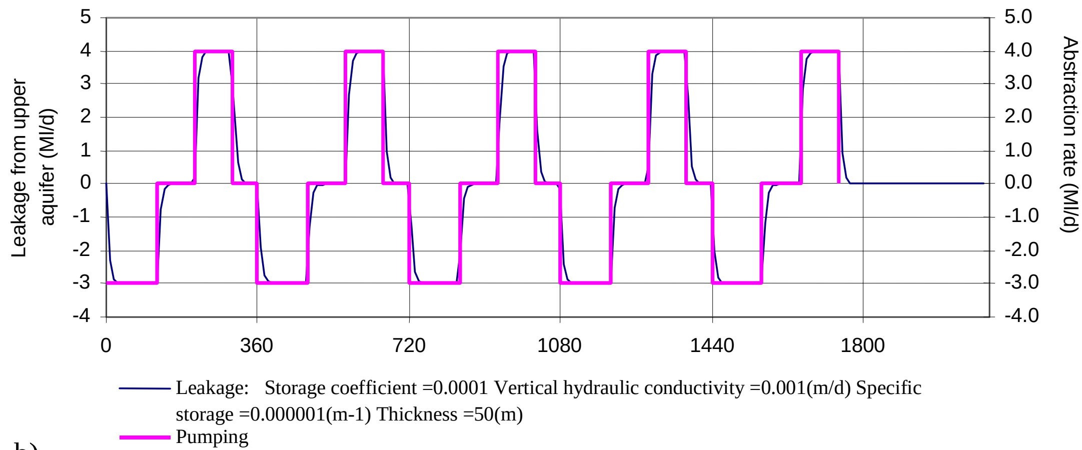
Table 2.3

	Figure	1.3	a)	b)	c)	d)	e)	f)	g)
Aquifer:	S (-)	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-3}	10^{-5}	10^{-4}
Aquitard	K_v (m/d)	10^{-5}	10^{-6}	10^{-3}	10^{-5}	10^{-5}	10^{-5}	10^{-5}	10^{-5}
	S_s (m^{-1})	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-4}
	b (m)	50	50	50	200	100	50	50	50



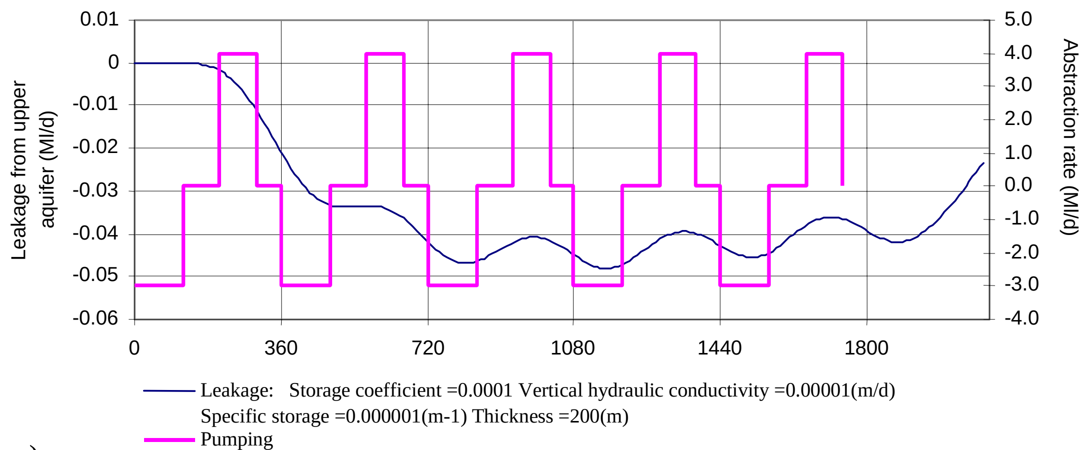
Leakage from upper aquifer (Ml/d)
Abstraction rate (Ml/d)
Leakage: Storage coefficient =0.0001 Vertical hydraulic conductivity =0.000001(m/d) Specific storage =0.000001(m-1) Thickness =50(m)
Pumping

a)



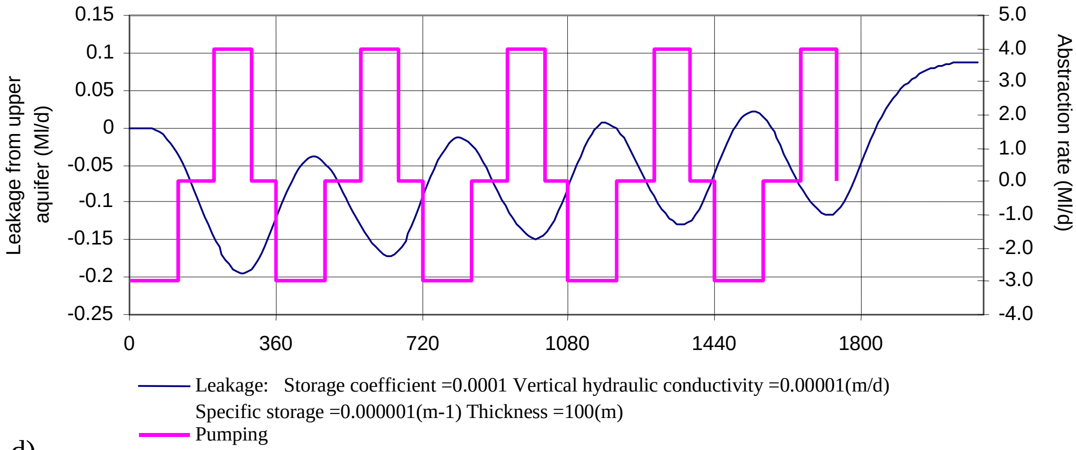
Leakage from upper aquifer (Ml/d)
Abstraction rate (Ml/d)
Leakage: Storage coefficient =0.0001 Vertical hydraulic conductivity =0.001(m/d) Specific storage =0.000001(m-1) Thickness =50(m)
Pumping

b)

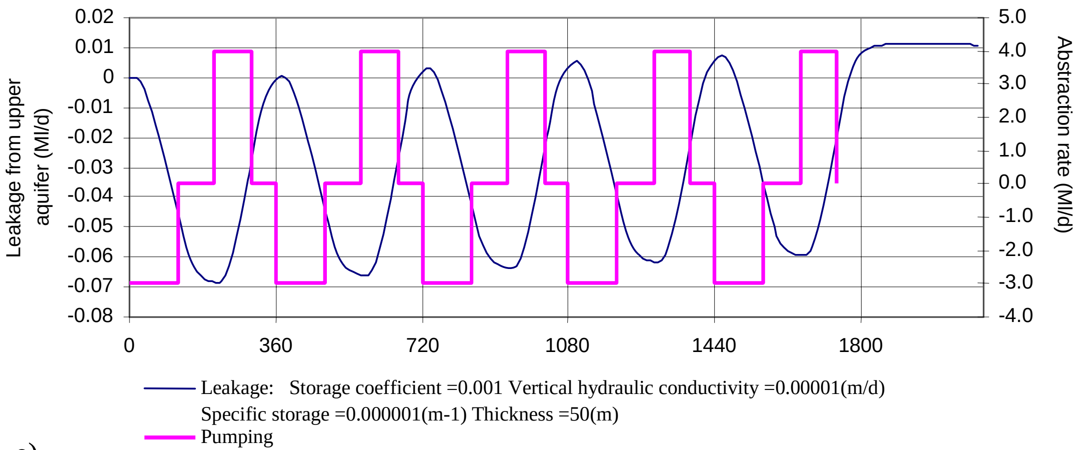


Leakage from upper aquifer (Ml/d)
Abstraction rate (Ml/d)
Leakage: Storage coefficient =0.0001 Vertical hydraulic conductivity =0.00001(m/d) Specific storage =0.000001(m-1) Thickness =200(m)
Pumping

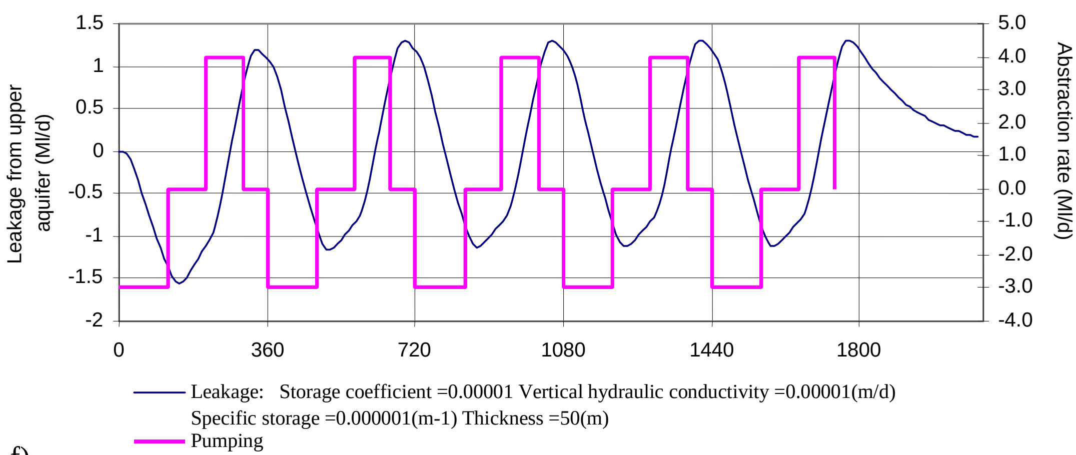
c)


0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
Leakage from upper aquifer (Ml/d)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
Abstraction rate (Ml/d)
0
360
720
1080
1440
1800
Leakage: Storage coefficient =0.0001 Vertical hydraulic conductivity =0.00001(m/d)
Specific storage =0.000001(m-1) Thickness =100(m)
Pumping

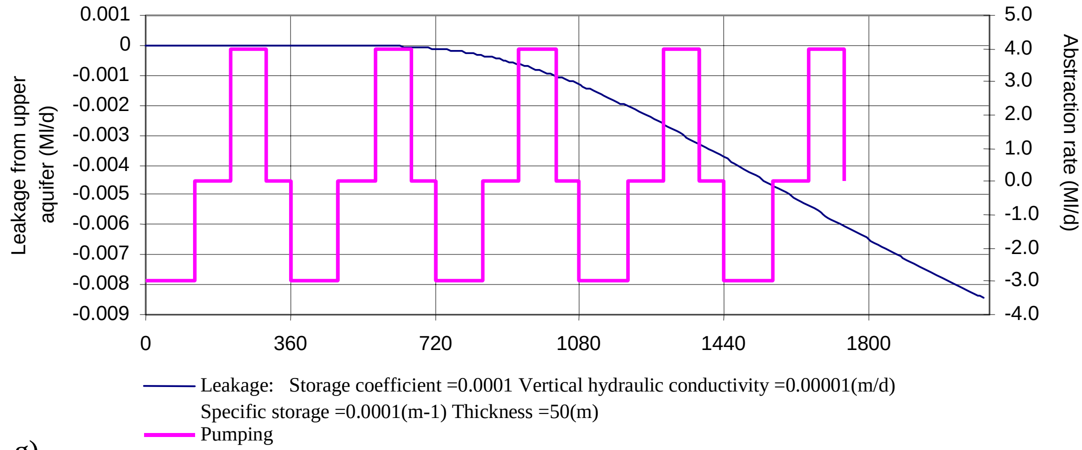
d)


0.02
0.01
0
-0.01
-0.02
-0.03
-0.04
-0.05
-0.06
-0.07
-0.08
Leakage from upper aquifer (Ml/d)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
Abstraction rate (Ml/d)
0
360
720
1080
1440
1800
Leakage: Storage coefficient =0.001 Vertical hydraulic conductivity =0.00001(m/d)
Specific storage =0.000001(m-1) Thickness =50(m)
Pumping

e)


1.5
1
0.5
0
-0.5
-1
-1.5
-2
Leakage from upper aquifer (Ml/d)
5.0
4.0
3.0
2.0
1.0
0.0
-1.0
-2.0
-3.0
-4.0
Abstraction rate (Ml/d)
0
360
720
1080
1440
1800
Leakage: Storage coefficient =0.00001 Vertical hydraulic conductivity =0.00001(m/d)
Specific storage =0.000001(m-1) Thickness =50(m)
Pumping

f)



g)

Figure 2.4 a) to g) Leakage from upper aquifer during 5 annual cycles, for given parameters

2.5 DISCUSSION

The examples here show how the spreadsheet model can be used to give an indication of how an ASR scheme might affect an overlying aquifer and also show that the effect is strongly dependant on the hydraulic properties of the aquitard.

It is also clear that the long term impact of a scheme is influenced by the nature of the last cycle. The cases shown above all end with an abstraction phase. This means that the longterm effect on the overlying aquifer is of leakage down wards into the semi-confined aquifer. If the scheme were to be terminated with a short injection phase this long-term loss from the upper aquifer could be replaced by a long term upward leakage.

The incorporation of the analytical model into a spreadsheet means that it can easily be used to demonstrate the possible effects of ASR schemes in these geological settings.

3 Effect of ASR scheme on outcrop area of target aquifer.

3.1 MODEL DESCRIPTION

A numerical MODFLOW model was developed to investigate the effect of an ASR scheme on the outcrop area of the target aquifer. This model is described in detail in Appendix 2. The design of the model was constrained by the need to be able to use it with a range of geometrical and aquifer parameters and also by the need for the model to be robust. To achieve these aims it was decided that constant head boundary conditions would be used to give a constant flow across the model. It is felt that this formulation of the model will give results which are both realisticand comparable with each other. The other options which were considered were to use a constant recharge over the outcrop area with a no-flow boundary up-stream and either a fixed head or no-flow boundary downstream. These options were not used as changing the aquifer parameters would i) lead to different positions of the confined/unconfined boundary, ii) give different head gradients and iii) could lead to unrealistic head gradients.

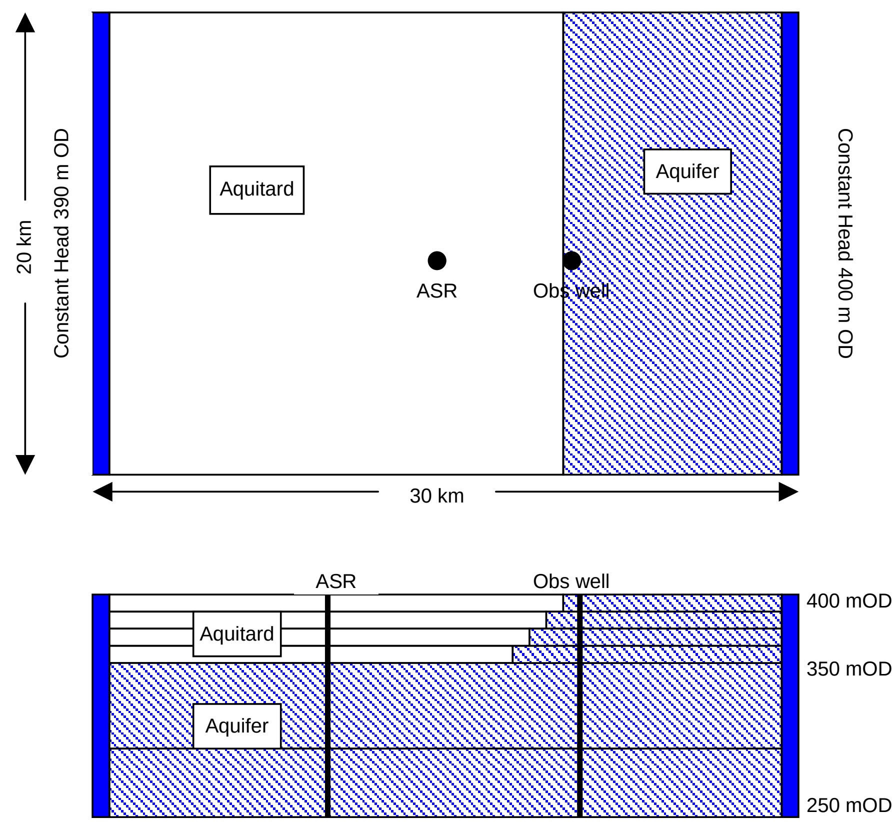


Figure 3.1 Schematic of numerical model showing dimensions and layering

Figure 3.1 shows a schematic of the model and Table 3.1 gives the parameters used for the different model runs. The effect of these parameter changes are shown in Figures 3.2 to 3.10 which are discussed briefly here.

Table 3.1 Conditions for each of the time variant simulations

Model Run	Parameters
TV12	Target aquifer hydraulic conductivity K_h =10 m/d K_z =1m/d Target aquifer storativity =0.0001 Aquitard hydraulic conductivity K_h =0.01 m/d, K_z =0.001 m/d Aquitard Storativity =0.000001 ASR well 5 km from the outcrop area
TV13	Target aquifer hydraulic conductivity K_h =1 m/d K_z =0.1 m/d Other parameters as TV12
TV14	Target aquifer storativity =0.00001 Other parameters as TV12
TV15	Aquitard hydraulic conductivity K_h =0.001 m/d, K_z =0.0001 m/d Other parameters as TV12
TV16	As TV 12 but pumping rate increased from 3000 m^3/d to 5000 m^3/d
TV17	Move ASR well to within 2 km from the outcrop area, rest as TV12
TV18	Aquitard thickness increased from 50 m to 100 m, rest as TV12
TV19	Aquifer thickness increased from 100 m to 200 m, rest as TV12
TV20	ASR well moved to 15 km from the outcrop area, rest as TV12

3.2 RESULTS

3.2.1 Initial time variant model

(TV12. Figure 3.2)

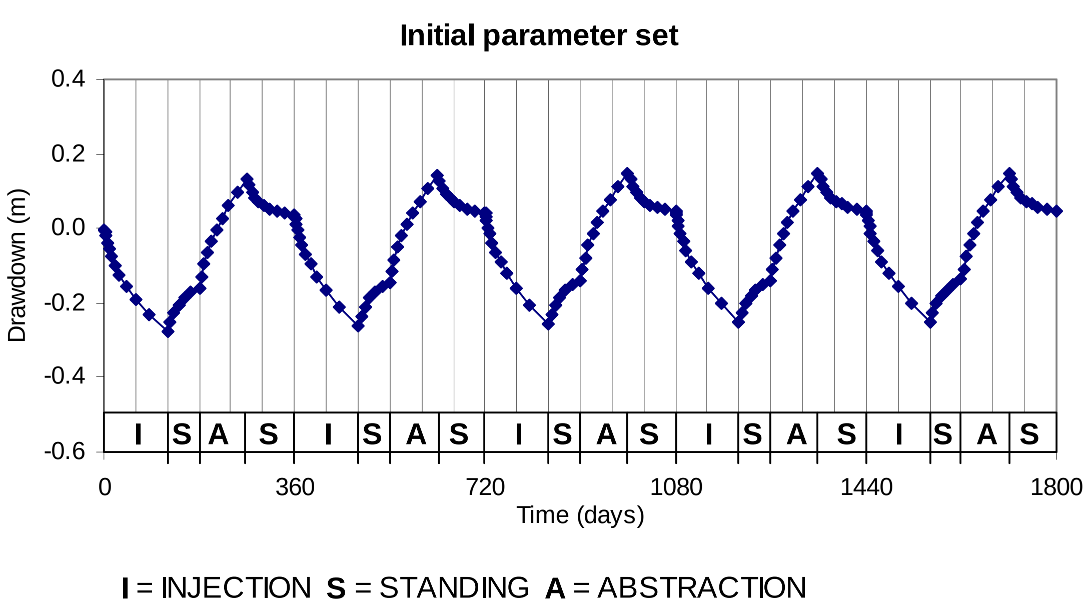


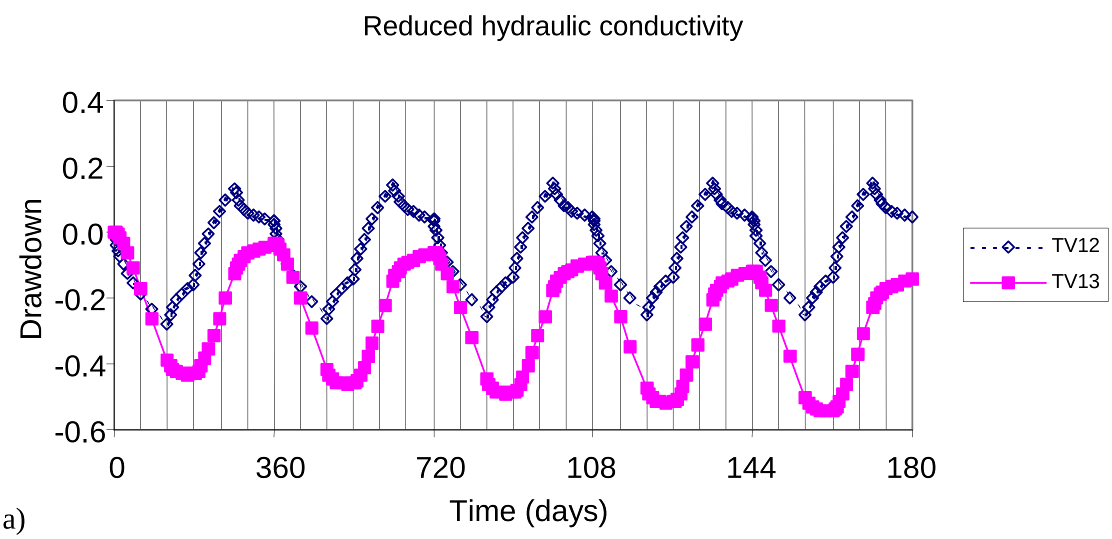
Figure 3.2 Modelled drawdown in observation well

The injection (120 days at 3 Ml/d), standing (60 days), abstraction (90 days at 3 Ml/d) and final standing period of the ASR annual cycle are illustrated in Figure 3.2. It can be seen that during injection, drawdown is negative i.e. that there is an increase in the water level within the aquifer. During the standing period which follows injection the water levels begin to fall gradually and then with the initiation of abstraction, after 180 days, the rate of fall becomes more rapid and drawdown values become positive (i.e. water levels are falling and actual drawdown is taking place). In the final standing period of the annual cycle the water level begins to recover, however it does not reach full recovery (i.e. the starting position of no drawdown) before the injection cycle of the second year begins. The maximum positive drawdown (0.148 m) occurs at the end of the last modelled abstraction and the maximum negative drawdown (i.e. water level rise) is –0.279 m which occurs after 120 days (i.e. the end of the first injection).

3.2.2 Reducing aquifer hydraulic conductivity

(TV12 vs TV13. Figure 3.3 a))

Figure 3.3a to h Drawdown estimated using numerical model



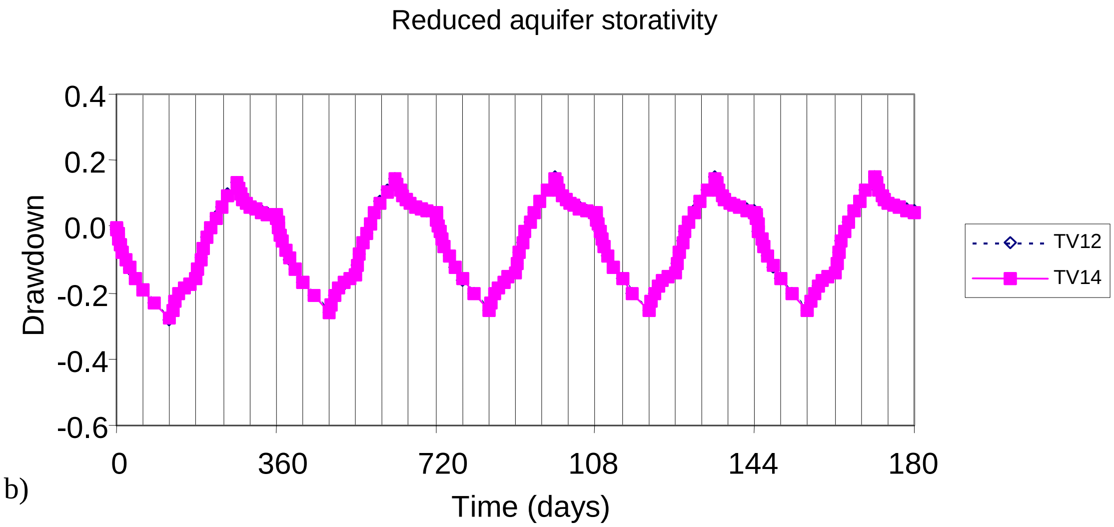
a)

Reducing the hydraulic conductivity of the aquifer by an order of magnitude from 10 m/d to 1 m/d has a significant effect on both the magnitude and the timing of the drawdown at the observation well. The maximum effect of the ASR well at the outcrop occurs just before the abstraction phase starts i.e 2 months after the end of injection. This means that an ASR scheme under the conditions modelled here could have the effect of raising water levels in the outcrop area at the time when it is assumed water levels would be lowest (i.e. when a scheme is used for supply).

As the time when the maximum impact of a pumping period occurs is a potentially significant parameter, a spreadsheet has been developed to estimate this time. This is described in detail in Appendix 3 and is discussed briefly at the end of this section.

3.2.3 Reducing aquifer storativity

(TV12 vs TV14. Figure 3.3 b))



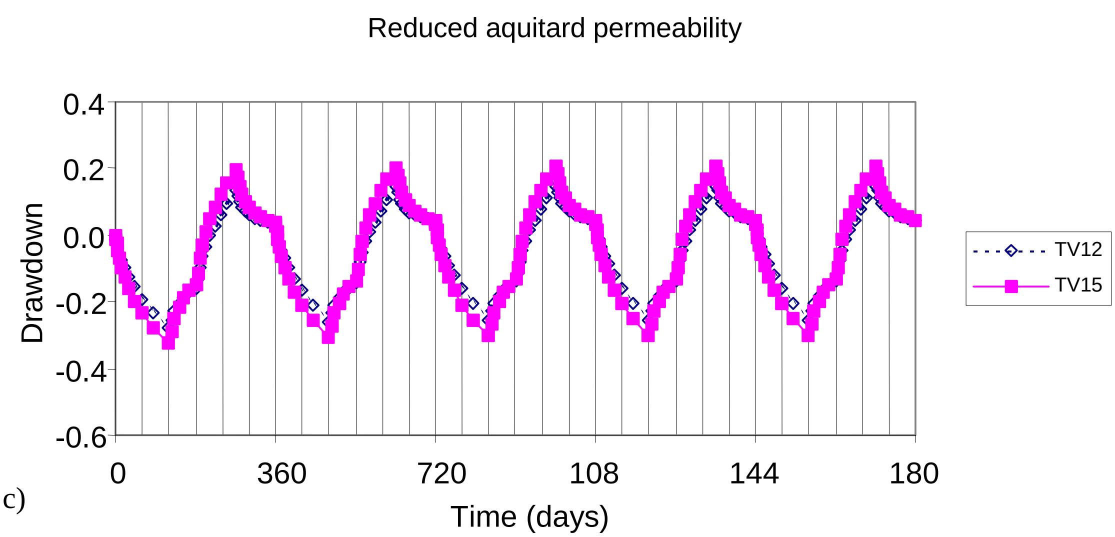
b)

Aquifer storativity was reduced by an order of magnitude from 0.0001 to 0.00001. This has very little effect on the drawdown at the outcrop

3.2.4 Reducing aquitard hydraulic conductivity

(TV12 vs TV15. Figure 3.3 c))

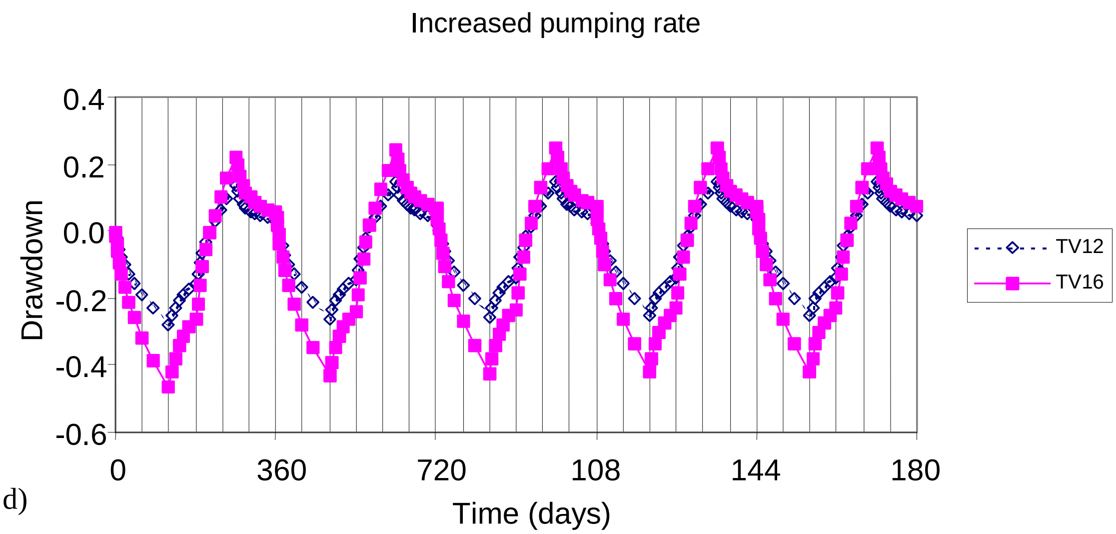
Aquitard hydraulic conductivity was reduced from 0.001 to 0.0001 m/d, which caused a small increase in the magnitude of drawdown. After 1 year at the point of maximum drawdown the water level for TV15 is 122 mm lower than for TV12, however after 4 and 5 years this difference has fallen to 114 mm i.e. the difference in drawdown decreases with time.



c)

3.2.5 Increased injection and pumping rate

(TV12 vs TV16. Figure 3.3 d))



d)

The injection and pumping rates were increased from 3000 m^3/d to 5000 m^3/d. As expected this has the effect of increasing drawdown at the observation well. During year 1 the difference at maximum drawdown was 86 mm but in year 5 difference at maximum drawdown was 97 mm (year 4 = 96 mm)

3.2.6 Changed position of ASR well

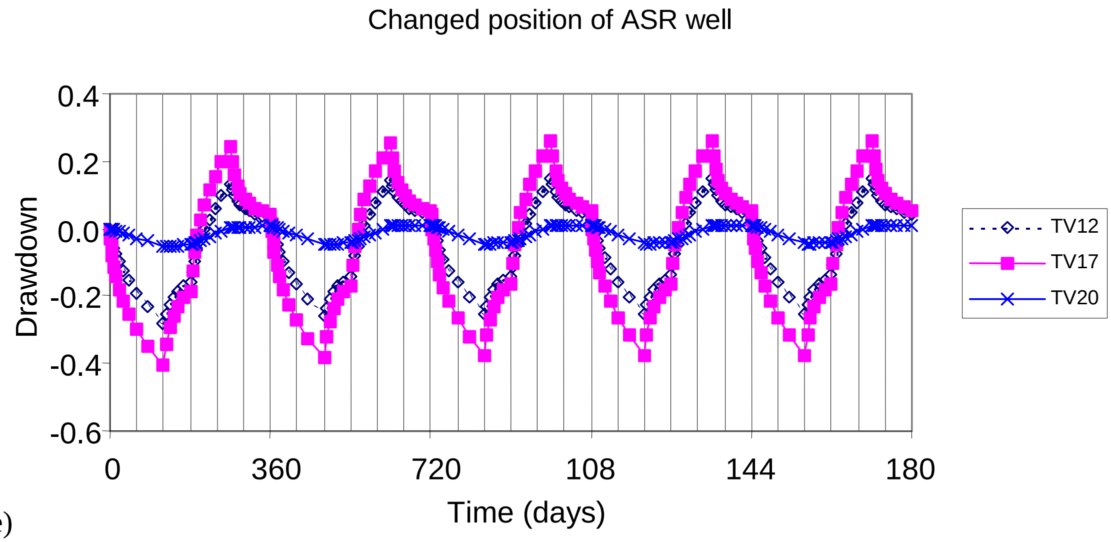
(TV12 vs TV17 and TV20. Figure 3.3 e))

Moving the pumping well from 5 km to within 2 km of the outcrop has the largest impact on water levels in the observation well (as might be anticipated). The amount of drawdwon is much greater in the observation well under the conditions of TV17.

Year 1 difference at maximum drawdown 678 mm

Year 5 difference at maximum drawdown 711 mm

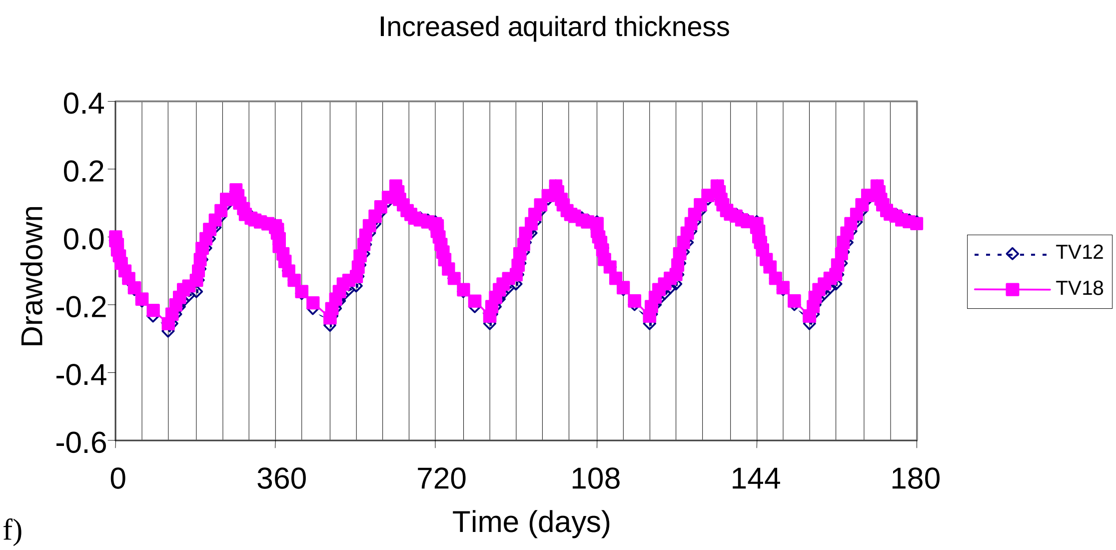
As would be anticipated moving the pumping well away from the outcrop area results in a more subdued fluctuation in the water levels at the observation well. There is also a significant time delay with the maximum drawdown (253 mm less than TV12) occurring at the end of the first annual cycle at day 360 rather than in all the other simulations when it occurs at the end of the pumping period at 270 days. For years 3 to 5 there is a 10 day time lag behind TV12 for the maximum drawdown at the observation well with the maximum drawdwon in TV20 being 277 mm less than that in TV12.



e)

3.2.7 Increased aquitard thickness

(TV12 vs TV18. Figure 3.3 f))



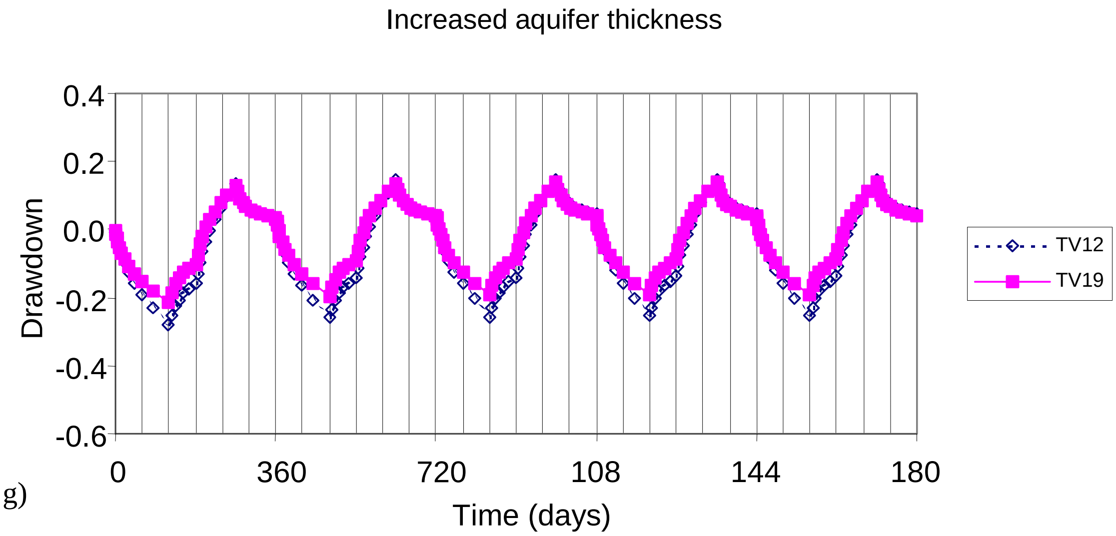
f)

Increasing the aquitard thickness from 50 m to 100 m seems to have relatively little impact on the water levels. In the model with a thicker aquitard the drawdown is 14 mm more than the thinner aquitard model, however by year 5 this difference has reduced to 9 mm.

3.2.8 Increased aquifer thickness

(TV12 vs TV19. Figure 3.3 g))

The aquifer thickness was increased from 100 to 200 m which has the effect of increasing the water levels in the observation well.as a result of the increased transmissivity. During year 1 TV19 has 4 mm less drawdown than the initial model TV12 but by year 5 this has increased to 17 mm less drawdown.

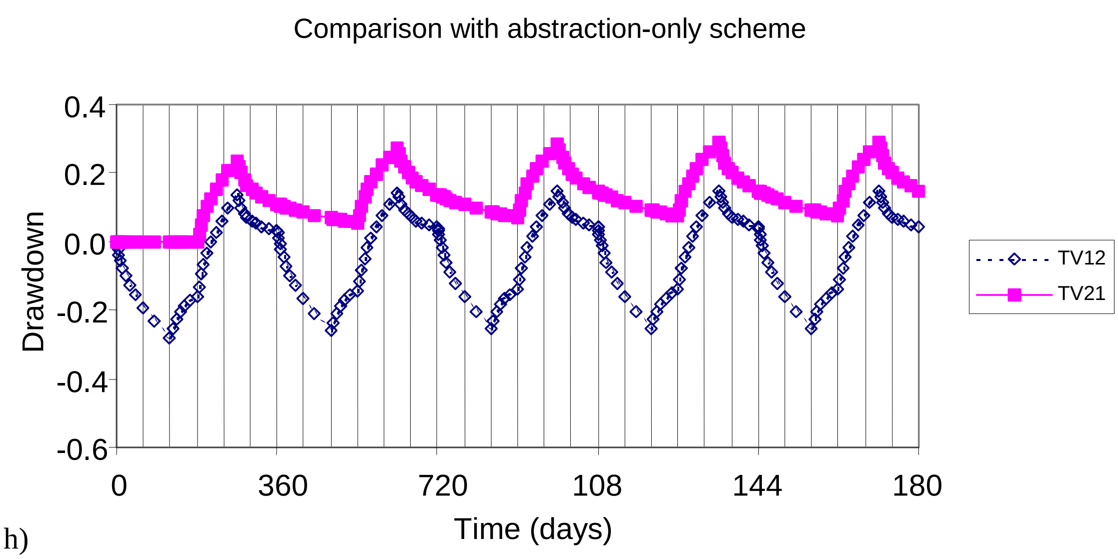


g)

3.2.9 Comparison with abstraction only scheme

(TV12 vs TV 21 Figure 3.3 h))

As can easily be seen an abstraction only scheme results in significant drawdown at the outcrop are. The maximum drawdown simulated is 0.289 m at the end of the 5th year of abstraction, the drawdown is increasing year on year. It should be noted that this aquifer recovers well between abstraction cycles but this is in part a consequence of the boundary conditions used for this model, which allows extra water to be drawn in over the simulated constant head boundaries. Thus, this simulation is very much a 'best-case'. (The model was set up with constant head boundaries as it was intended for use to simulate ASR schemes which involve no nett loss of water from the aquifer.)



h)

3.3 CONCLUSIONS

All scenarios seem to have a larger absolute effect on increase in the water level, following the phase of injection, than on drawdown after abstraction. If the pumping regime was altered so that abstraction continued for longer then increased drawdown should be expected.

Movement of the ASR well with respect to the outcrop area had the largest effect on water levels in the observation well. However even these changes induced water level changes of less than a metre.

3.4 TIMING OF MAXIMUM IMPACT

As discussed in section 3.1.2, the maximum drawdown at the edge of the confining layer due to a period of injection or abstraction, occurs sometime after the end of pumping. A spreadsheet based on the Theis solution, has been developed which gives an indication of when this maximum will occur in the case of a confined aquifer. This can be used to give a first estimate of the delay that would be expected in the situation which has been considered in this section.

The mathematical development of this solution is given in Appendix 3. Here two graphs are presented which show the delay between end of pumping and maximum drawdown for a range of aquifer transmissivity and distances. Figure 3.4 shows the estimated delays for an aquifer with a storage coefficient of 10^{-4} and Figure 3.5 shows the same for a storage coefficient of 10^{-3} (note different scale on y-axis).

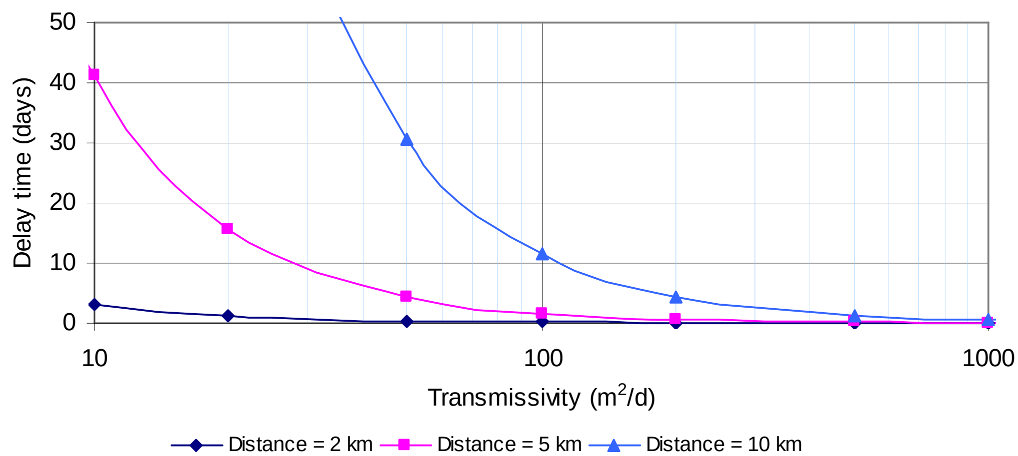


Figure 3.4 Time after end of pumping period of 60 days when maximum effect is felt ($S=10^{-4}$)

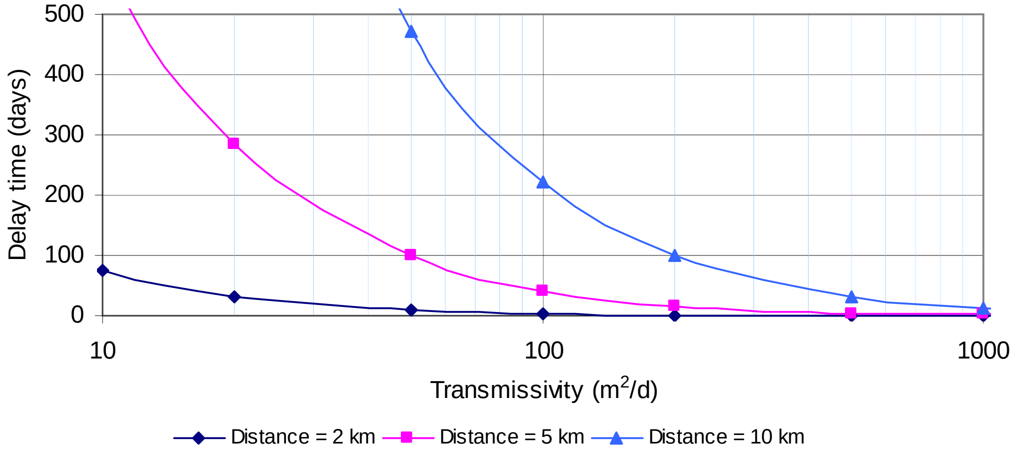


Figure 3.5 Time after end of pumping period of 60 days when maximum effect is felt ($S=10^{-3}$)

4 A leaky aquifer with a linear recharge boundary

4.1 PROBLEM DEFINITION

A analytical solution has also been developed to a scenario which is similar to that modelled using the numerical model, but which has simpler boundary conditions. The modelled scenario is shown in Figure 4.1 and the environmental impact is assessed by considering the flow across the boundary at the edge of the outcrop (the linear recharge boundary) as well as that from the upper aquifer through the aquitard.

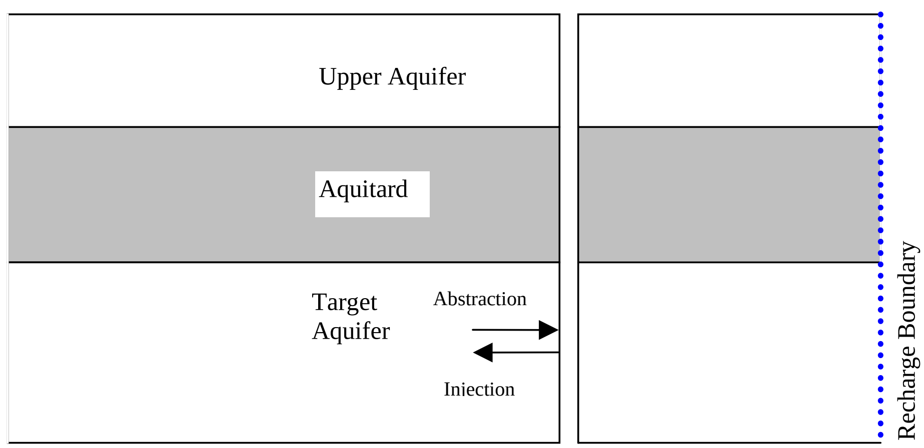


Figure 4.1 Schematic of modelled scenario

The flow equations were solved (see Appendix 4) to give the flow rate into the target aquifer at the recharge boundary and the leakage from the upper aquifer to the aquitard. Also calculated is the flow rate from the upper aquifer in the absence of the recharge boundary. This flow rate is the same as that calculated by the spreadsheet model described in Section 2 and is included so the comparisons can be made between the two scenarios.

Figure 4.2 shows the output from the model for the aquifer properties shown in Table 1.2, with an aquifer transmissivity of 100 m^2/d. The leakage from the upper aquifer shown here is identical to that shown in Figure 1.3. The leakage across the recharge boundary suggested by the model is large compared to the leakage through the aquitard. This is partly because of the way the recharge boundary is incorporated into the model. The formulation is as if the boundary is a fully-penetrating river which can supply infinite amounts of water immediately, and as such is very much a worst case of the expected actual flow across the sort of boundary that might exist in reality.

Figure 4.3 shows the *cumulative* flows across the various boundaries for the same example ASR cycles. As can be seen the cumulative effect of the ASR scheme over an annual cycle is small, but not insignificant. This figure illustrates well the difference between the model described in Section 2 and this scenario.

Further model runs not presented as they give results which are as expected with comparison to those shown in Section 2. However, the parameter which is not required in the more simple model – target aquifer transmissivity – does have a significant effect on the flow across the boundary, as would be expected. The effect of changing this parameter from 100 to 1000 m^2/d is shown in Figures 4.4 and 4.5.

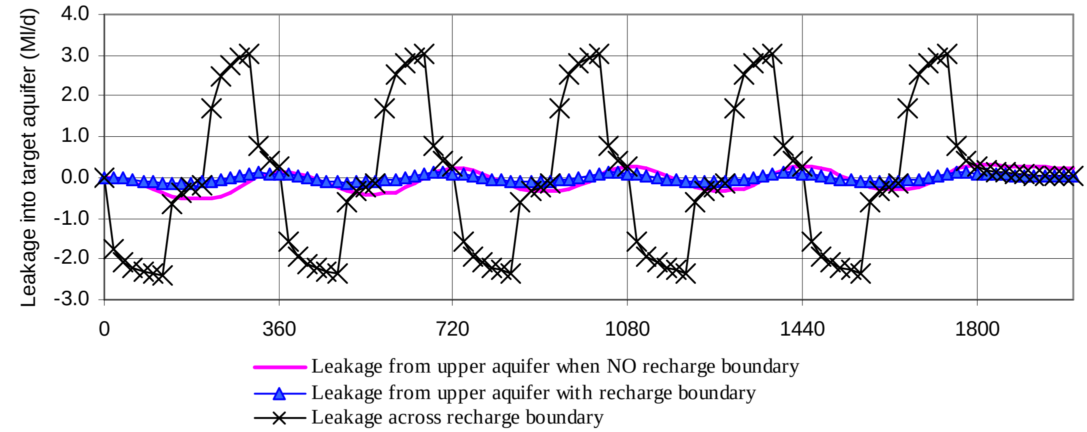


Figure 4.2 Flows into target aquifer using aquifer parameters in Table 1.2

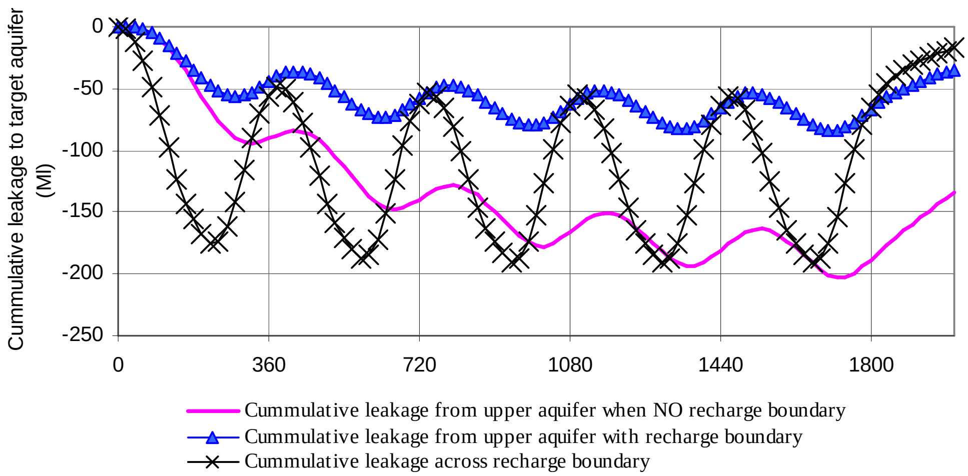


Figure 4.3 Cummulative flows into target aquifer (parameters in Table 1.2)

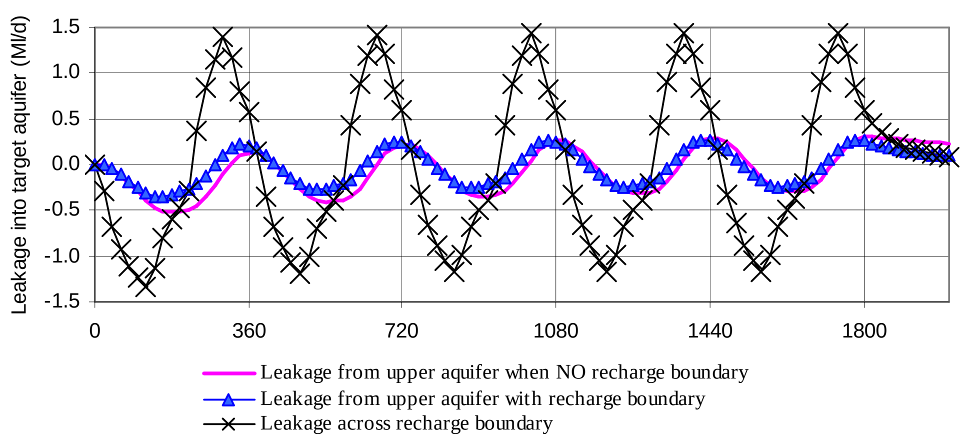


Figure 4.4 As Figure 4.2 but with aquifer transmissivity of 1000 m^2/d

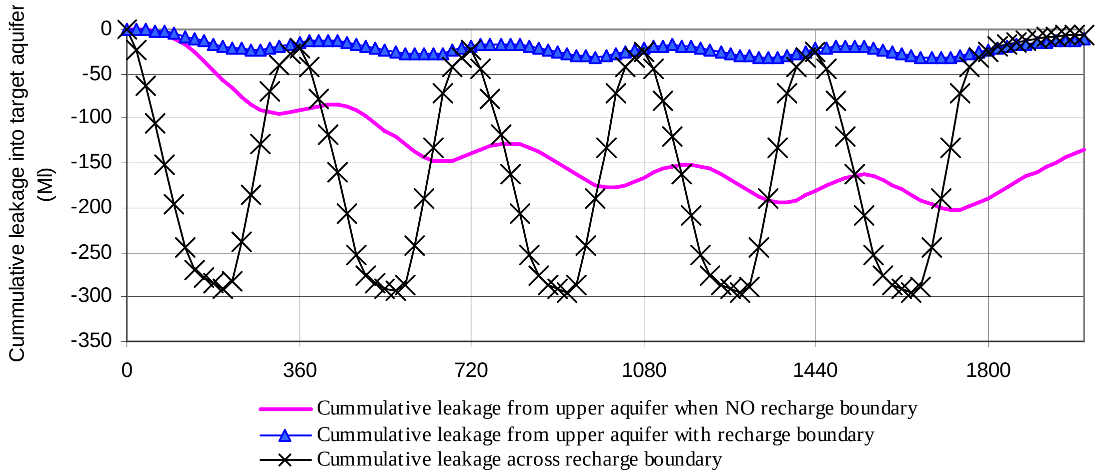


Figure 4.5 As Figure 4.3 but with aquifer transmissivity of 1000 m^2/d

4.2 CONCLUSIONS

This model, which is similar to both the previous models, gives a worst case for the flow across a boundary. It also gives a best case (i.e minimum flow) in terms of the leakage from the upper aquifer. The results can be used as a first pass tool before creating a numerical model as described in Section 3. It gives the same results as the model described in Section 1 but is slightly less easy to use as it can not be incorporated directly into a spreadsheet, instead the data produced by the Fortran program must be imported into a spreadsheet in order for the results to be graphed.

5 Discussion

Several models have been created to facilitate the examination of the environmental impact of typical ASR schemes, where the ASR well is in a lower (semi-) confined aquifer. Environmental impact has been measured in terms of fluxes or head changes in adjacent parts of the target aquifer or in adjacent aquifers. The models are presented in a form which is accessible to a practising hydrogeologist.

A variety of scenarios have been presented, to provide an indication of the likely impacts. However, the intention of this work has equally been to provide easy-to-use tools far a 'first-pass' assessment of environmental impact.

The first, most simple model is presented as a spreadsheet which incorporates a Chart which is updated as the aquifer parameters are changed. This gives a very quick indication of the fluxes expected in an overlying aquifer and will be useful at a very early stage of designing an ASR scheme.

The numerical model needs some more care to ensure that it is used appropriately, but also has much more flexibility to allow specific schemes to be examined in more detail. The numerical model presented as a Groundwater Vistas© file will allow a hydrogeologist to produce useful results in a very short time.

The final model, presented as a compiled Fortran program is intermediate between the other models both in terms of ease of use and complexity. The results produced by this model give a good insight into the dynamics of an ASR scheme.

Appendix 1

IMPACT of asr ON AN unconfined AQUIFER ABOVE AN AQUITARD ABOVE A confined AQUIFER WITH A SINGLE WELL

Mathematical Results

The flow equations and mathematical analysis is not given but the final result is that the volumetric leakage rate into (or out of) the overlying aquifer is given by

$$Q_{leak}(t)=Q_w\, G(\tau,\sigma)$$

where

Q_w is the (constant) pumping rate from the well.

$G(\tau,\sigma)$ is a function defined below

$\tau = Kt/S_sb^2$ is a dimensionless time

t is the time since the start of pumping of the ASR well

K is the vertical hydraulic conductivity of the aquitard

S_s is the specific storage of the aquitard

b is the thickness of the aquitard

$\sigma = b\, S_s / S$ is the ratio of aquitard storage to storage in the lower (ASR) aquifer per unit area.

EVALUATION OF G(τ,σ)

The following Laplace transform provides a precise (indirect) definition of G:

$$\bar{G}(p,\sigma)=\int_0^\infty e^{-p\tau}G(\tau,\sigma)d\tau=\frac{\sigma}{p\left(\sqrt{p}\sinh\sqrt{p}+\sigma\cosh\sqrt{p}\right)}$$

This can be used to evaluate G using software for Laplace transform inversion. (A Fortran program has been developed from this formula and it was used to verify results found from the spreadsheet introduced below.)

For many purposes the following alternative formula can be used

$$G(\tau,\sigma)=1-2\sigma\sum_{n=1}^{\infty}\frac{\exp(-\alpha_n^2\tau)}{[(\sigma^2+\alpha_n^2)+\sigma]\cos\alpha_n}$$

where α_n is a positive root of

$$\alpha_n \tan\alpha_n=\sigma$$

A spreadsheet has been developed based on the above summation.

Appendix 2

NUMERICAL MODEL

Development of the Steady State Model

Geometry

- The model grid was 30 km long by 20 km wide. The model grid was divided into 40 rows of 500 m length and 120 columns each 250 m long.
- The model was eventually divided into 6 layers. The aquitard was given a sloping edge (created by 4 relatively thin overstepping layers (Figure 1b)) to try to represent an inclined aquifer. The aquifer was divided into two layers. Layer bottom elevations were adjusted to alter the thickness of the aquifer and aquitard to 100 m and 50 m respectively.

Boundary conditions

Constant head boundaries were applied along the east (400 m head) and western (390 m head) boundaries of the model in order to generate a reasonable and uniform hydraulic gradient across the modelled area. It was important to generate an initial head distribution which was above the bottom of the top layer (only 12.5 m thick) of the aquitard so that the cells did not dry out.

No recharge was added

Initial aquifer properties

- Target aquifer hydraulic conductivity K_{xy} 10 m/d K_z 1m/d
- Target aquifer storativity 0.001
- Aquitard hydraulic conductivity K_{xy} 0.01, K_z 0.001
- Aquitard storativity 0.00001

Wells

- The ASR well was added, located 5 km from the outcrop, although in the first instance no pumping or injection rates were assigned.
- Two observation wells were added one adjacent to the pumping well (within the same grid cell) and a second at the edge of the unconfined part of the aquifer 5 km from the ASR well.
- The model was run without abstraction to establish the initial head distribution so that drawdown could be calculated (Figure 2). Abstraction from the ASR well was set at a rate of 3000 m^3/day and the drawdown in the observation wells was examined.

Once the steady state model was working satisfactorily a time variant model was attempted.

Development of the Time Variant Model

The time variant model was set up to follow an annual cycle programme consisting of injection for 4 months, rest for 2 months, abstraction for 3 months and rest for 3 months. It was anticipated that a five year period would be sufficient for the effects on head to be approaching dynamic balance and also that response time patterns should have become established. For each time variant simulation initial heads were imported from a steady state model with corresponding aquifer properties and geometry.

The stress period set up for one year (Table 1) is designed to simulate a typical ASR scheme annual cycle. The stress period set up was repeated for a 5 year duration.

Stress period length	Number of time steps	Multiplier	Pumping rate m^3/d
120	12	1.4	+ 3000
60	6	1.2	0
90	9	1.2	- 3000
90	9	1.2	0

Table 1. Stress period set up for one year

For each different parameter set tested, a new steady-state model must be created to give the starting conditions for a time-variant model run. Thus, for most of the model runs reported there are two GV modflow models. The exceptions are for the runs which involve changing the pumping regime and the ASR well position.

Appendix 3

DELAY BETWEEN CESSATION OF PUMPING AND MAXIMUM EFFECT

It is important to appreciate that the response to pumping for a limited period does not produce a drawdown (at distance from the pumping well) which is maximal at the end of pumping. The following analysis, based on the Theis equation, provides a means of determining when the maximum drawdown that will occur.

We introduce a well function representative of the finite period of pumping

$$s(t_R, t_P) = \frac{Q}{4\pi T} \mathrm{W}_P(\tau_R, \tau_P)$$

where

t_P is the period of pumping

t_R is the time after the cessation of pumping

Q is the volumetric pumping rate

T is the transmissivity

$$\tau_P = \frac{4Tt_P}{Sr^2} \text{ and } \tau_R = \frac{4Tt_R}{Sr^2}$$

The above well function can be expressed in terms of the Theis well function through:

$$\mathrm{W}_p(\tau_R, \tau_P) = \mathrm{W}\left(\frac{1}{\tau_R + \tau_P}\right) - \mathrm{W}\left(\frac{1}{\tau_R}\right)$$

Of particular interest is the maximum drawdown achieved; this occurs at some time after the end of pumping. We can write this in the form:

$$s_{\max}(\tau_P) = \frac{Q}{4\pi T} \mathrm{W}_P^{\max}(\tau_P) = \frac{Q}{4\pi T} \max_{\tau_R}\left[\mathrm{W}_P(\tau_R, \tau_P)\right]$$

A spreadsheet (Visual Basic) function has been prepared to find the maximum of the function W_P but this will not always be convenient for use. As an alternative, use can be made of the following approximation, which is accurate to better than about 3%:

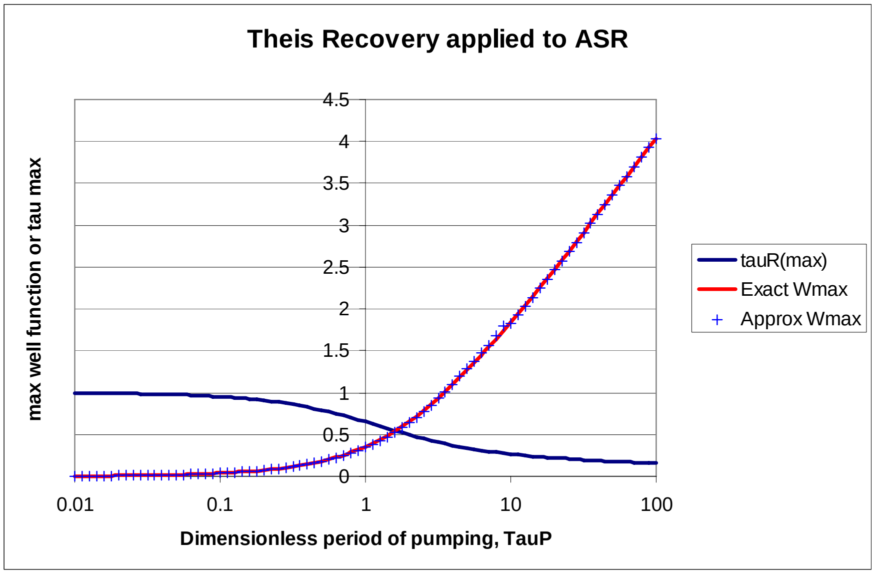
$$\mathrm{W}_P^{\max}(\tau_P) = \max_{\tau_R} \left[\mathrm{W}_p(\tau_R, \tau_P)\right] \approx F(\tau_P) = \begin{cases} \dfrac{\tau_P}{e} & \tau_P \le 0.4 \\ \dfrac{\tau_P}{e} - \mathrm{f}(\tau_P) & 0.4 < \tau_P < 10 \\ \mathrm{W}\left(\dfrac{1}{\tau_P}\right) & \tau_P \ge 10 \end{cases}$$

where

$$\mathrm{f}(x) = 0.024\, x^2 - 6.7 \times 10^{-5} x^4$$

The short (pumping) time approximation corresponds to the case of an instantaneous pulse which propagates away from the well at a rate such that the peak occurs where $\tau_R = 1$.

The long time approximation represents the situation where the period of pumping is large in relation for the time for propagation from the well to the point of interest. The maximum drawdown is then achieved shortly after the end of pumping and the drawdown at that time is approximately unaffected by the cessation of pumping: the normal Theis solution is then adequate.



Appendix 4

A LEAKY AQUIFER WITH A LINEAR RECHARGE BOUNDARY

Consider the same model as above but with a linear recharge boundary within the lower aquifer. Two quantities are of interest: the leakage between the aquitard and the upper aquifer and the flow across the boundary into the lower aquifer.

The leakage from the upper aquifer into the aquitard is

$$Q^{b}_{leak}(t)=Q_w\,\mathrm{H}(\tau,\sigma,\alpha)$$

where

$\mathrm{H}(\tau,\sigma,\alpha)$ is a function defined by the Laplace transform:

$$\overline{\mathrm{H}}(p,\sigma,\alpha)=\int_0^\infty e^{-p\tau}\,\mathrm{H}(\tau,\sigma,\alpha)d\tau=\overline{\mathrm{G}}(p,\sigma)\left[1-e^{-\alpha\xi}\right]$$

where G, τ and σ were defined earlier,

$$\xi^2 = p\left(\frac{1}{\sigma}+\frac{\coth\sqrt{p}}{\sqrt{p}}\right)$$

$\alpha=A/B$

where A is the distance to the boundary and the 'leakance', B, is given by

$B^2=Tb/K$

and T is the transmissivity of the lower aquifer.

The leakage from the recharge (constant head) boundary into the lower aquifer, induced by the pumping, is

$$Q^{b}_{bound}(t)=Q_w\,\mathrm{I}(\tau,\sigma,\alpha)$$

where

$\mathrm{I}(\tau,\sigma,\alpha)$ is a function defined by the Laplace transform:

$$\overline{\mathrm{I}}(p,\sigma,\alpha)=\int_0^\infty e^{-p\tau}\,\mathrm{I}(\tau,\sigma,\alpha)d\tau=\frac{1}{p}e^{-\alpha\xi}$$

Note the steady-state situation (at an infinite time of pumping):

$$Q^{b}_{leak}(t)=Q_w\,(1-e^{-\alpha})$$

$$Q^{b}_{bound}(t)=Q_w\,e^{-\alpha}$$

Fortran Implementation

The above functions have been implemented into a code called ImpactASR.for. (This was based on the pumping-test code PTSIM.)

The code produces the cumulative volumes as well as the volumetric flow rates, up to any given time.

The code permits complex changes in pumping rate, although the rate must be piecewise constant.

The input is from a text file (ImpactASR.dat), which is described below. Output is to a text file (ImpactASR.out), which is not described here but should be self-explanatory. It gives values for all of the above functions, and the corresponding cumulative volumes, for the times specified.

Included in the output are the steady-state ratios of Q^b_{leak}/Q_w and Q^b_{bound}/Q_w. For many parameter sets these will tend to be either unity or zero. The more interesting behaviour will occur when $\alpha=A/B$ is of the order of unity. However, consideration must be given to the time for hydraulic diffusion through the aquitard and from the well to the boundary: the output file indicates these times.

Data input

Data is input from a text file which has the structure shown in the table below. The parameters must be in consistent units. (It is recommended that the demonstration datasets provided be studied to help understand the formats.)

The first record is a title. A single record containing all of the parameter values follows this. Then a sequence of records specifies the pumping pattern. Then the simulation period and time interval is specified by a single record, further records of the same type can be used to simulate over a different time periods. The 'case' is terminated (and computation begun) by an 'end' or 'new' record (below): the latter indicates that a new case will follow.

The pumping rate can be set in a very flexible manner. There are four types of pumping data record:

- End time and pumping rate
- Period of pumping and rate (then, end time = previous end time + period)
- Repeat: the preceding sequence of periods any number of times
- A record starting with 'END', which terminates the above.

The first three types can be mixed in any order although, of course, a 'repeat' record cannot appear first. There is only one 'END' record. If the simulated period stretches beyond the specified times, zero pumping rate is assumed.

File ‘ImpactASR.dat’

Contents of Record	Format	See Notes
Title for (first) parameter set	Up to 80 characters	
T,S,K,S_s,b,a	Real numbers	1
t1, Q1	Real numbers	1,2
t2, Q2	Real numbers	1,2
tn, Qn	Real numbers	1,2
REPEAT= j k	String ‘REPEAT=’ and two integers	1,3,9
P= Dt, Q	String ‘P=’ and two reals	1,4,9
END	String ‘END’	5
ICASE, TLO,THI,DT	One integer and three reals	1,6,7
NEW or END	String	8,9
(if NEW then)		8
Title for second parameter set		
etc.		

NOTES:

1. Separated by blanks or commas.
2. Specify an end time and pumping rate

   Q1 from time 0 to t1
   Q2 from time t1 to t2
   Qn from time tn-1 to tn
3. Repeat the last j pumping periods, k times
4. Specify a pumping period and rate.
5. All pumping period specification lists are terminated with ‘END’
6. The time range (TLO to THI) and interval (DT) can be specified in four different ways, depending on the setting of ICASE:-

   ICASE=1 Linear time since the start of pumping.

   ICASE=2 Log time (log10) since the start of pumping.

   ICASE=3 Linear time since the start of recovery.

   ICASE=4 Log time (log10) since the start of recovery.

   (e.g. '2 -2 1 1 ' gives times 0.01, 0.1, 1.0 & 10.0)
7. Any number of sets of time intervals can be used for a given run.
8. Both NEW and END starts the case. ‘END’ indicates this is the last case set. Use ‘NEW’ if another case follows: any number of parameter sets can follow.
9. Text should be left justified