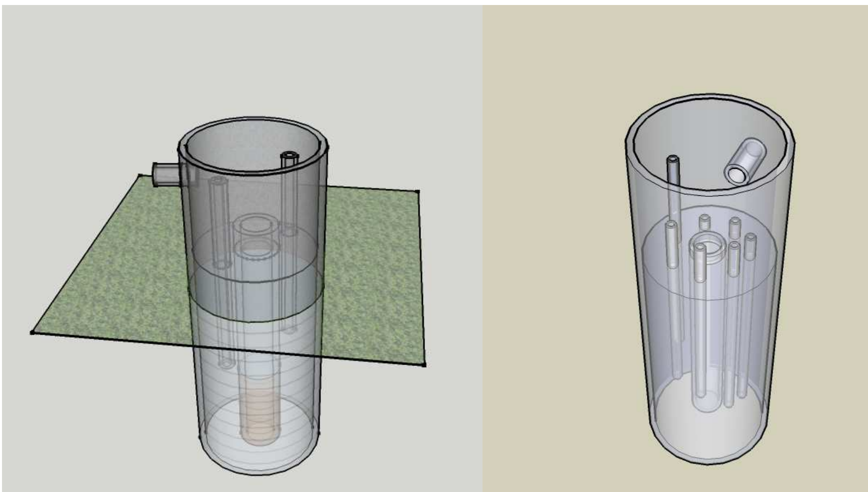
Universiteit Utrecht

Sultan Qaboos University

The application of Vertical Infiltration Systems (VIS) in the Sultanate of Oman

Part one



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September 2010

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Part one

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Glossary

Aflaj: A system of water distribution canals. Singular: Falaj.
ARS: Artificial Recharge and Storage, a MAR-type technique.
b.g.l: Below ground level.
Connate saline groundwater: original seawater deposited with the marine sediment, still present in the interstices.
Falaj: A water distribution canal. Plural: Aflaj.
MAR: Managed Aquifer Recharge.
MWR: Ministry of Water Resources.
MRMEWR: Ministry of Regional Municipalities, Environment and Water Resources.
Storm runoff: The amount of water from a high intensity precipitation event, which is not lost to bed storage or other type of infiltration and thereby gives the flood its volume.
Storm water: Storm runoff (see above).
VIS: Vertical Infiltration Systems, a MAR-type technique.

General introduction and project objective

The countries of Oman and Holland share a long history, in fact dating back to the 17th century. Nowadays the Dutch and Omani are again looking to strengthen their bonds, resulting in the Oman exhibition held in fall 2009 in the Nieuwe kerk in Amsterdam and the exhibition of etchings of Dutch painter Rembrandt van Rijn in the Grand Hyatt in Muscat. In September 2007, the Sultan Qaboos Academic Chair Quantitative Water Management in Semi-Arid Regions was installed. The fore lying thesis is a direct result of this new founded cooperation, supervised by the holder of the academic chair professor Ruud Schotting.

The Gulf state of Oman is a country eight times the size of the Netherlands, with a population size of almost 3 million in Oman, compared to 16,5 million for the Netherlands (United Nations, 2009). Both countries face major challenges concerning (ground)water management, however of a different character. In Holland, the objective is to manage water systems in such a way that, with future rising river tables, water can still be safely guided to the sea, without any floods occurring. In Oman, the challenge is to prevent any rain or surface flowing water reaching the sea directly, using it for aquifer recharge to restore groundwater balances, as well as to provide flood protection from storm runoff. Groundwater extraction in Oman has locally led to a lowering of the groundwater table of up to almost 30m. (Ashworth, 2005). Although in Arizona (U.S.) a groundwater system is considered to be sustainable if a lowering of the groundwater table is smaller that 300ft per 100 years, the Omani – like the Dutch – believe a more conservative drawdown to be acceptable (Kooiman et al, 2008). Lateral seawater intrusion in coastal aquifers is an issue in both countries, giving rise to the necessity of keeping a positive groundwater balance in these regions. This research explores a possible collaboration of both countries on the subject of Managed Aquifer Recharge (MAR-systems). In Holland the MAR-practice is considered to be a good principle for groundwater recharge and is often used. Dutch knowledge on sustainable aquifer management can be valuable to countries starting up their own MAR-systems (Kooiman et al, 2008).

Vertical Infiltration Systems (VIS) are used in the Netherlands by drinking water companies to artificially infiltrate water, to be later extracted for production. VIS techniques are now also being used to infiltrate storm runoff, preventing flooding and subsequent water inconvenience.

The objective of this thesis is to analyze if deep infiltration of storm runoff by means of a VIS-technique, is more efficient than the currently in Oman used controlled wadi flow, with respect to:

1) Combating seawater intrusion by infiltration at depth
2) The speed of infiltration of a certain volume

The controlled infiltration of storm runoff is a good method to recharge aquifers effectively. However, this thesis questions whether water infiltrated by storage dam overflow is the most effective approach to be used in Oman. The artificial infiltration by controlled wadis is hampered by the presence of extensive cemented gravel layers and clay lenses as well as a groundwater table at substantial depth (Weyhenmeyer and Waber, 2002). This research looks into the possibility of using Dutch knowledge of Vertical Infiltration Systems (VIS) to more effectively recharge Omani aquifers.

Chapter 1 gives an overview of Dutch practice regarding storm runoff. In Chapter 2, this is specified to the use of VIS techniques applied in the Netherlands. Chapter 3 addresses the Omani water practice, giving an historical view and shortly discussing current policy. Chapter 4 then presents the general geology and climate characteristics of the Northern Batinah coast, the area under interest in Oman. The effectiveness of VIS in Oman, which has been modeled and more in depth described in the second part of this report, is regarded in Chapter 5. Discussion and Conclusions are then presented in Chapters 6 and 7, respectively.

Acknowledgements

Many people supported me during my research, to whom I owe special thanks.
First I would like to thank my professor, Ruud Schotting, for giving me the chance to visit Oman and take on this project. It has been a unique experience for me and a trip I would have never embarked on, if Ruud hadn't awoken my curiosity when he called in september 2008. I would like to thank Nick Buik, hydrologist and senior consultant and my supervisor at IF Technology, for his contributions early on in the project, providing me with information about different model environments and help in outlining the subject. Rory van der Heiden, IF Technology specialist on water harvesting, retaining and storage, patiently explained to me the different well designs and underlying theory. Philip Nienhuizen, senior hydrogeologist at Waternet, helped me willingly when I reached out with my questions on pumping test data, without which I would have had a hard time concluding about the validity of the Omani dataset I took home.

Gathering the widespread information about Oman proved not always straightforward. The following people provided me with very much needed answers and considerations, for which I owe them many thanks. Dr. Abdulaziz Ali-AL-Mashikhi, working at the Ministry of Regional Municipalities and Water Resources and Saif Al-Yaroubi, hydrogeologist at PDO, supported me with important previous research and data from the Al Khawd Fan site. J.M. Ashworth, freelance water resources specialist and hydrogeologist has finished several project studies in Oman and helped me kindly with all my questions about the lithology and general hydrogeology of the Batinah plain. C. Wehenmeyer, assistent professor at Syracuse University, who shared information about her own research and fieldwork experience in Oman. Doctor S.G. Molyneux, who works for the British Geological Survey and H. Droste, who works for Shell Technology Oman, provided me kindly with articles, which I was previously unable to obtain.
I much enjoyed Professor Theodore Strobls' guest lecture about Omani dam structures and safety measures at SQU. He was so kind to send me a bounded copy of Gerhard Haimerls PhD thesis about groundwater recharge in wadi channels downstream of dams, as soon as he returned to Germany.
The people at RioNed and Geoarabia were so kind to provide me with further information for my literature review.

In Oman, many people helped me in finding my way in the new culture and surroundings, and although I can not thank all of them indivudially here, I am very grateful to all. Professor Anvar Kacimov was my never uninspiring help, always giving the best suggestions about my proceedings in Oman as well as new outlooks on the Dutch role in the second world war. Anvar, I have enjoyed our little philosophical talks and, eventhough I'm still no communist myself, you certainly made me think. My colleagues at the Water Research Centre; Mumtaz, Muaath, Chukri, Ali and Dorothy were always so helpful and full of understanding. I always smile when I think back to our in-kitchen lunches and group-dinners, you have been a joy to have around. I would also like to thank my Scotsman Gordon Aitken, who provided me with a wonderful shared appartment, snorkelling trip, british lager at dinner time and rugby at the English club. I also owe my thanks to Anke Jannie-Landman, with whom I had a great time in Oman and on our PDO trip down south. As I cannot describe everything we have been through here, I can absolutely say that it was a pleasure to share the goat at dinnerime!

Last, I would like to thank my parents, my two brothers and of course Vincent, for holding on, whenever I became a little hesitant about the road ahead.

1. Storm runoff infiltration

1.1 Dutch policy and practice

The Dutch policy regarding water management in urbanized areas is to separate runoff and effluent, constructing separate systems for both (VROM, 2003). The old combined sewerage system discharges storm runoff faster than the natural discharge would occur. The faster transport may cause rivers and streams to overflow, since there is no natural reduction in peak discharge and may damage the environment by desiccation because water is no longer infiltrated into the soil.

The capacity of a sewerage treatment plant is better used in a separated sewerage system, where the flows of effluent and cleaner rainwater never mix. As this prevents the rainwater from diluting the effluent, the volume to be treated remains less, and the treatment plant capacity is used effectively (SBR/ISSO, 2000). The separation of flows also creates the opportunity to infiltrate the relatively clean storm runoff directly where it is precipitated, thereby reducing the fast transport away from the soil system and reducing the damage water shortage due to lack of infiltration causes to nature.

Within the separated system, precipitation and storm runoff are infiltrated in several steps. The runoff is first infiltrated in the shallow subsoil. Only when rainfall intensities rise to a point where these shallow installations fall short in capacity, the water is infiltrated into deeper ground (TCB, 2009). If there is no temporary storage available, the capacity should be equal to the highest precipitation intensity (Aalten and de Kwaadsteniet, 2004).

When the precipitation flows down rooftops and streets, the water takes up contaminants and solid mass. The contaminating capacity of runoff for groundwater is therefore higher than that of the rain itself. When storm runoff contains solid particles like sand, a soil passage can reduce the sediment load and contaminant content by filtering and adsorption, before the water is used for infiltration. Concentrations of contaminants in the groundwater directly below (shallow) infiltration structures do usually not rise above the norm (Grontmij, 2002). However, contaminant loading of the soil is expected to take place over many years. When this is to be prevented for future use of the system, storm runoff water has to be pre-treated before infiltration.

1.2 Getting storm runoff to the infiltration system

To manage runoff, the flow has to be guided by a management structure. Water flowing through rain water discharge pipes, like in conventional roof drainage, is of annular nature (Lucke et al, 2007). Annular flow is characterized by the existence of a vapour phase core (air), and a film of liquid flowing along the inner walls of the pipe (Figure 1a). The air that is present in the pipe restricts the water discharge in about one quarter to one third of the pipes' cross-sectional area (Lucke et al, 2007).

In roof drainage systems, the air entrapment is overcome by using so-called syphonic drainage systems (Figure 1b). These are designed to reduce the air intake as much as possible and allow the system to become primed when necessary in case of a more intense precipitation event. The system is primed, and has switched from annular to full-bore flow (Lucke et al, 2007).

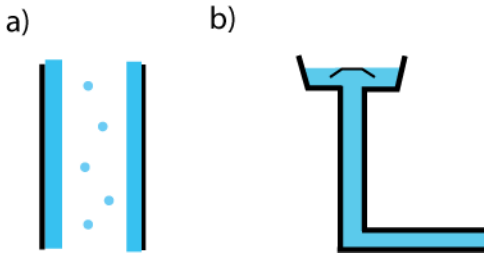

Figure 1 – a) annular flow, b) syphonic drainage system

In vertical infiltration wells too, the presence of air hampers the maximum possible flow velocity. So with the design of vertical wells, the air entrapment should be reduced or at least the air should have an easy way out, as soon as the water depth and flow velocity in the supply pipe are increasing.

1.3 Infiltration systems

The Dutch use a variety of systems to infiltrate runoff. In the Netherlands, the choice for a system is sometimes a consequence of the origin of the runoff. When infiltrating runoff from rooftops, the height difference can be used to infiltrate the water using gravity effects to elude the necessity of pumping (Lucke et al, 2007). The choice for a system also depends on groundwater depth and conductivity of the aquifer (Aalten and De Kwaadsteniet, 2004). A subdivision between systems can be based on the depth of the system with respect to the surface area.
The alternatives for infiltration are the following:

- Infiltration through hardened surface area
  - Open concrete structures
  - Half pavements
- Infiltration at surface area
  - Infiltration field
  - Infiltration basin
  - Infiltration ditch
- A combination of surface and underground infiltration
  - Wadi
- Underground infiltration, within the vadose zone
  - Vadose zone well
  - Infiltration case
  - Infiltration crate
  - Perforated sewerage pipe
- Underground infiltration, within the saturated zone
  - Injection well

Some of these alternatives are described in the next paragraphs.

1.3.1 Wadi infiltration

The Dutch wadi differ from the wadis found in Oman, where they are mostly formed naturally. In the Netherlands the term wadi is used for a small human made valley or trench. The top layer of such a wadi consists of sand mixed with humus as a requisite for vegetation. Grass type vegetation is used to make the wadi more resistant to destruction of the top-soil pack. Below the sand, a gravel pack wrapped in a geotextile is installed, where water can be temporary stored, below which sometimes a draining pipe is placed (RIONED, 2006). There is an overflow mechanism installed, which directly drains the wadi when water levels reach the opening, bypassing the improved sand bed. A schematization of the structure is presented in Figure 2.

One of the major disadvantages of wadi infiltration in The Netherlands is the amount of surface area needed to create one. In densely populated regions, usable surface area is scarce and expensive. However, when placed within the area already reserved for nature within the housing estate, no extra space is needed (RIONED, 2006).

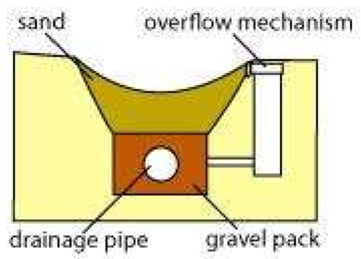

Figure 2 – Dutch wadi design

1.3.2 Infiltration-units

Infiltration-units are structures that can temporarily store water, keeping the water available for infiltration. The units are situated under ground, above the water table. This prevents the structure from getting a temporarily draining function, and being filled with groundwater, which would reduce the storage capacity for storm runoff. The infiltration case, infiltration crate and perforated sewerage pipe are all types of infiltration-units. The storage capacity of infiltration-units is rather limited. Infiltration-units are therefore often connected to a waterway, (other) sewerage systems or a deep infiltration system, in case the storage capacity falls short during a high precipitation event.

1.3.3 VIS: Vertical Infiltration Systems

Vertical infiltration systems (VIS) are typically wells, that can infiltrate runoff at deeper depths than the above mentioned infiltration systems. The water is infiltrated either to the vadose zone by a so-called vadose zone well or dry well, or to the saturated zone, by so-called injection wells (Bouwer, 2002). In contrast to the artificial wadi and infiltration-unit, the practice of vertical infiltration is not based on the principle of storage capacity but on that of the discharge capacity of the soil (van der Heiden, 2009). A schematization of the structure is presented in Figure 3.

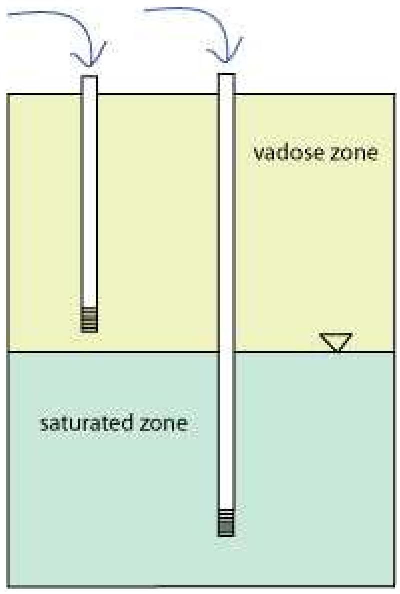

Figure 3 – Vertical infiltration wells

In The Netherlands it is estimated that the amount of storm runoff infiltrated with VIS is within the range of 0,01 – 0,05 million m^3 on a yearly basis (Cramer, 2008). However, different types of VIS are also extensively used by water companies, to infiltrate for example pre-treated river water to be used for drinking water production. In this process, the soil passage is used as treatment for viruses and bacteriological content of the water, while the infiltration of water assures the possibility of sustainable management of the aquifer itself.

The following Chapter summarizes a few designs of VIS in the saturated zone, used at IF Technology, Arnhem, the Netherlands.

2. VIS in the Netherlands: types of VIS used at IF technology

This Chapter describes VIS designed by If-Technology, Arnhem, The Netherlands. In Paragraphs 2.2, 2.3 and 2.4, three designs are discussed in detail.

2.1 General VIS design at IF Technology

One of the biggest difficulties in VIS applications is to control the airflow in opposite direction of water flow in the well. The air-entrapment hampers the flow and thus decreases the capacity, like discussed earlier in the case of roof-draining systems. During a high precipitation event, the amount of water available for infiltration per time unit becomes so large, that the well-opening might become completely filled. In this process, the air still present at a deeper location in the well becomes trapped and becomes pressurized. The well then starts 'burping', letting the trapped air out in sudden bursts (Figure 4). This happens when the rising velocity of the trapped air becomes larger than the downwards velocity of the water. Figure 6 shows the measurements of an IF test-project. The measurements show that inserting an extra pipe for the purpose of air discharge is effective in reducing negative effects of air entrapment. IF Technology designs are equipped with one or more air discharge pipes when these are necessary.

Figure 4 – A deep infiltration well 'burping' with the trapped air. The inlet of storm water is to the right. Photo Copyright IF-technology, Arnhem, The Netherlands.

VIS designed by IF Technology are usually combined with some form of a pre-treatment system. These vary from a simple sand trap and rack for the removal of solid particles and leaves, etcetera, to the more elusive several step underground soil passages for removal of solid particles and contaminants by adsorption and chemical binding (van der Heiden, 2009[2]).

The basin from where water flows into the well has been further developed over recent years. Figure 5 shows some of the steps taken in this process. The IF Technology basins are in principle now equipped with round walls to allow easier flow and prevent the water from splashing through the chamber. This should minimize the air entrapment. The next paragraphs discuss the possible design of the injection well itself.

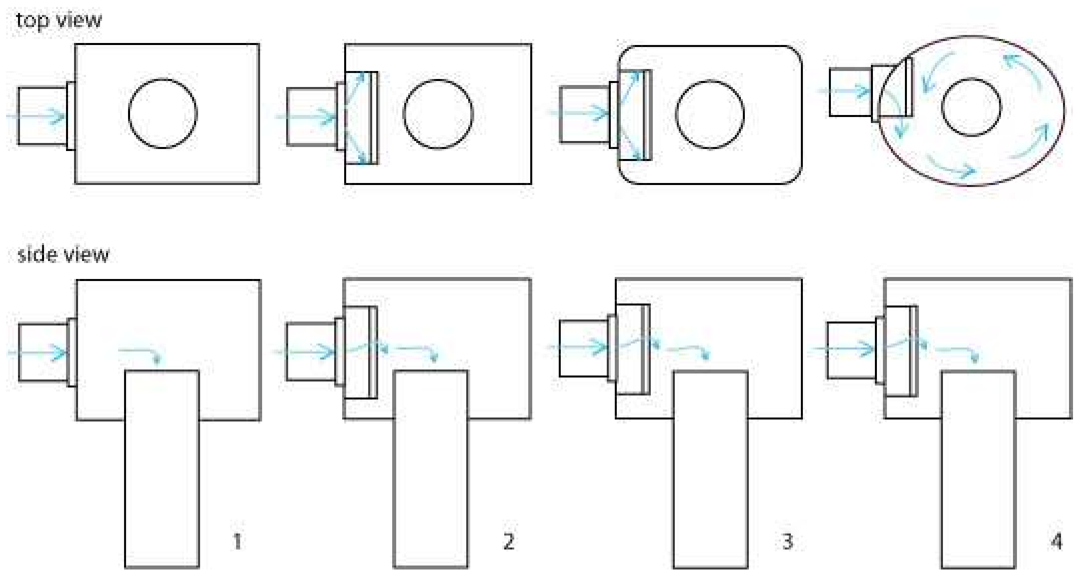

Figure 5 – development of the basin chamber seen in top view (above) and side view (below). The chamber has been developed from a total rectangular form (1), adding a screen to direct flow (2), curving the corners for easier flow (3), to a form with totally rounded walls (4) in order to create a whirlpool and allow easier flow into the injection well.

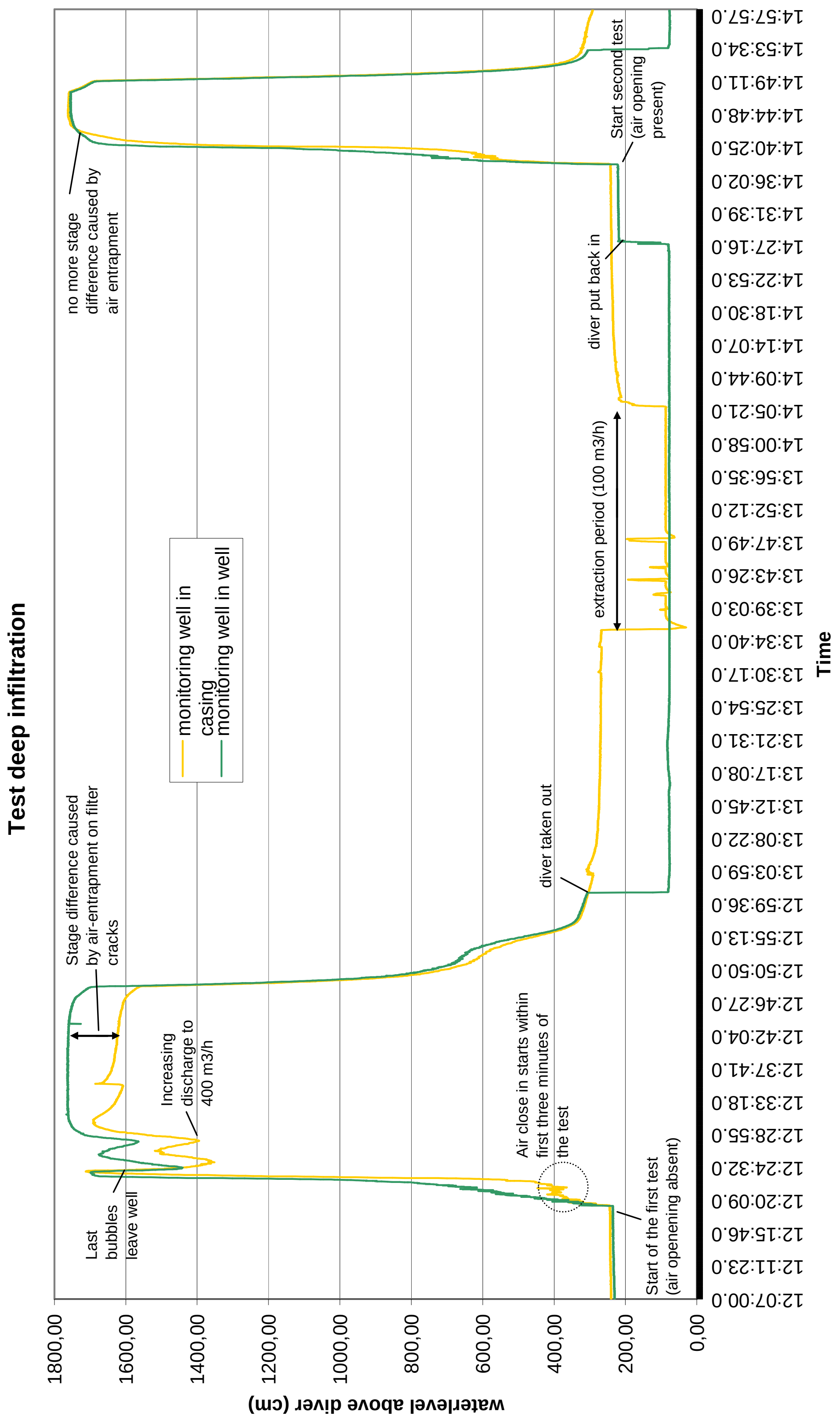

Figure 6 – test results of the installation of an air-discharge pipe in the VIS in Rijssen, The Netherlands. Notice that the difference between the levels measured in the well and monitoring well, is absent when the air-discharge pipe is added.

2.2 Organ type

This type of well is used in the cities of Rhenen and Amersfoort in The Netherlands. The organ type gets its name from its appearance. The water pipes are situated at different heights in the basin chamber, and reach different depths within the infiltration well, like the different lengths of pipes on an organ. The infiltration well itself is equipped with a filter for infiltration, while the water pipes within the well have a normal opening at the bottom of the pipe. The collecting basin is closed off from the lower part of the well by a concrete seal, letting water in only through the organ pipes (Figure 7). When the basin starts filling up in case of a high precipitation event, the lowest pipes start functioning as injection wells, while the pipes with higher elevation still function as air-discharge pipes. While injecting, the water level under the seal increases too. The different depths and heights of the pipes assure some pipes to be above water for air-discharge, up to the point where water levels below the seal reach the top, so that there is no need for air discharge anymore. The air-discharge pipes become injection wells when the water level in the above basin rises to an elevation higher than that of the pipe entrance. An increase in the amount of pipes used for infiltration, increases the injection capacity. This means that a larger inflow of storm runoff, resulting in higher water levels in the basin, results in a higher number of pipes used for infiltration and therefore an increased infiltration capacity necessary for that amount of runoff.

Figure 7 – the organ type infiltration well, with a larger pipe present in the middle

A variation at the organ type design is the organ type equipped with a so called roll seal. This design operates under the same principle as the normal organ type, but has one larger inner pipe present (Figure 8). This inner pipe is closed off by the roll seal at the top, which will open under a certain prescribed pressure originating from the water column above the seal. During a high precipitation event, the opening of the seal makes the larger inner pipe available for infiltration and thus increases the well capacity.

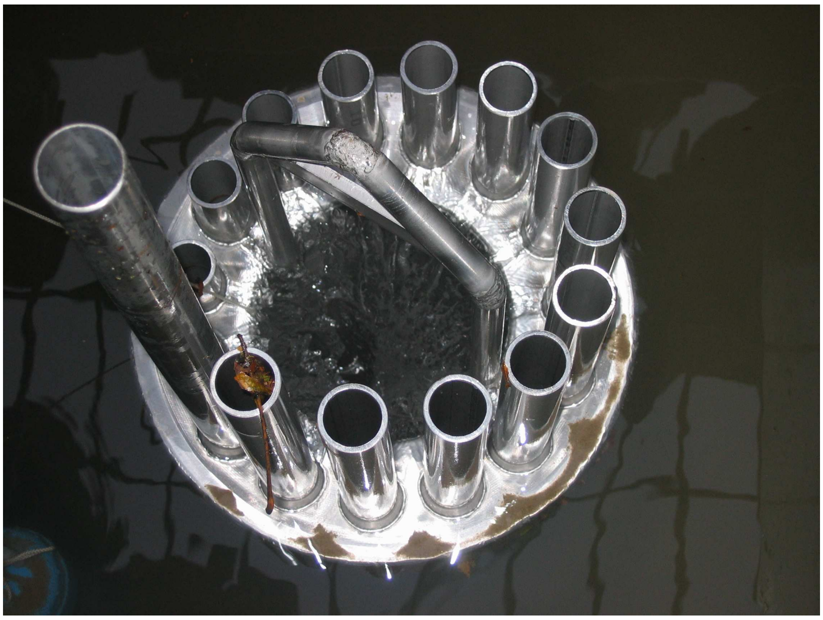

Figure 8 – The organ type well with a roll seal under working conditions. Top view

2.3 Large diameter well

This type is used in the village Rijssen, The Netherlands. The large diameter well is the original type used by IF Technology. The diameter of the borehole can reach up to 140cm and the diameter of the well itself up to 120cm. As a consequence of the size, this construction is more expensive than the other two VIS types, especially when infiltration takes place at large depths. The large diameter well is used in the Netherlands for infiltration up to 40meters below the bottom of the collecting chamber (Figure 9).

Figure 9 – The large diameter well in its construction basin. A screen is present to divert the flow from the adjoining pipeline

2.4 Inversed system

This well-type is used in the city of Sittard, The Netherlands. The inversed system is used where the groundwater depth is shallow. An inversed system well consists of two pipes, with the smaller one inside the outer pipe. The water flows from the collecting basin downwards through the inner pipe, then through the filter of the inner pipe, into the outer pipe. The outer pipe is equipped with a longer filter than the inner pipe, so that the filter slits are situated also at shallower depth. This enables infiltration at a shallower depth (Figure 10). The storage capacity of the vadose zone is thereby used to the best effect. Air-discharge in this system takes place from two or more air discharge pipes, perforating the seal of the outer tube. In contrast to the organ type design, the air discharge pipes reach to the same depth below and above the seal. The height of the air discharge pipes is such, that water will flow into the outer well, only from within the inner well, not bypassing this filter by flow through the air discharge pipes. The inner well is designed by the same principle as the large diameter well.

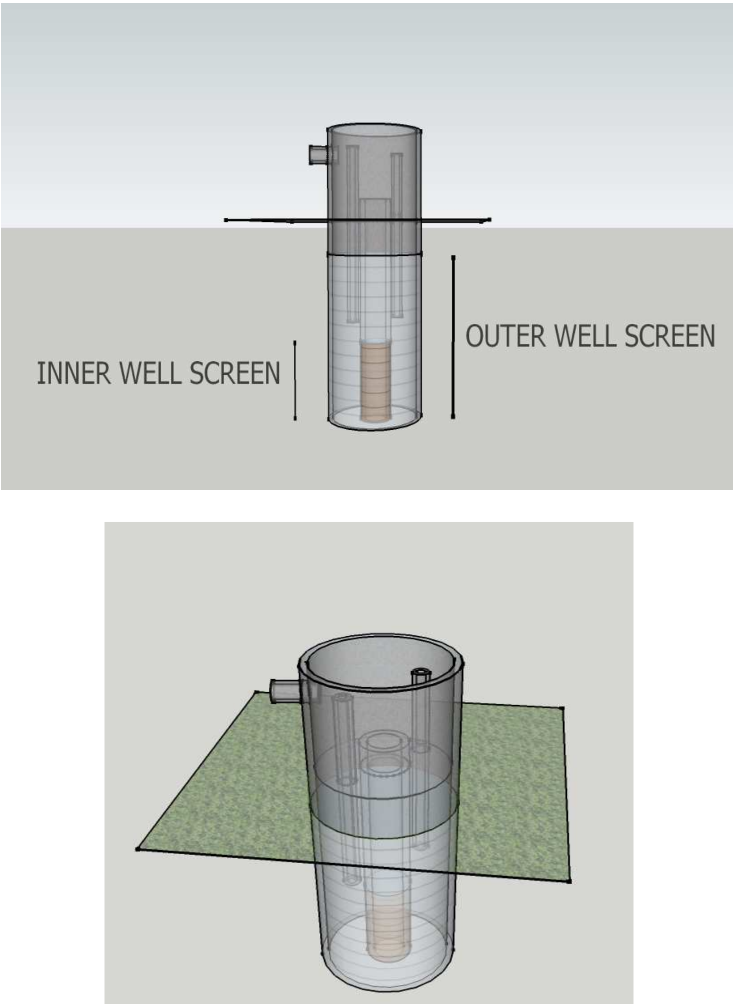

Figure 10 – The inverted system. The Inner tube (length of the line to the left) has a filter starting at the same maximum depth as the outer tube (length of the line to the right), but the outer tube filter reaches higher.

3. Water management in Oman

This Chapter first describes the challenges the Omani face within their water management system. The water management practice over the last decades is described in Paragraph 2 and 3, of which the effectiveness is discussed in Paragraph 4.

3.1 Omani water challenges

The IGRAC estimates that 50 to 75% of Omani soils do not host a significant production aquifer (IGRAC, 2009). This is defined as the cumulative extent of the zones with practically impervious rocks only or with only local aquifers of a very limited spatial extent, expressed as a percentage. Water production from groundwater resources in Oman has caused a significant demise of the groundwater table during the last decades. There are several important reasons to try to restore the water balance, especially in the more densely populated coastal areas.

One of the traditional and still used water distribution techniques is by aflaj, small transport canals. The first reason to restore the water balance is to stop the lowering or start the recovery of the groundwater tables. This will prevent further aflaj from drying out, ensuring a reliable water supply for agriculture dependent on these aflaj (Al-Ismaily and Probert, 1998).

The second reason is to stop or counter the effects of saltwater intrusion, threatening groundwater extraction for drinking water production or irrigation purposes. There are three sources of saline water in the Omani coastal area. The first saline water source originates from seawater intrusion. In a balanced coastal groundwater system, there is a net flow of fresh water flowing into the sea, creating a fresh water layer, underlain by the denser and heavier salty water (Figure 11). However in Oman there is locally no fresh water flow reaching the ocean due to extensive fresh groundwater extraction (Kacimov et al., 2009). The second source of saline water is from the deeper formations in the Omani subsoil. Connate water of the deep carbonate deposits is hyper saline (Kacimov et al, 2009 and IGRAC, 2009). This water flows upwards, probably also due to a lowering of the fresh water pressure caused by extensive pumping. The third source is formed by groundwater flow towards the coast from further inland, where it passes through Cambrian and Permian evaporates, becoming enriched in salt too.

In order to restore the water table and the fresh water lens, the Omani need to either reduce the extraction rates or increase the amount of recharge. The latter objective can be met by using the available storm runoff for infiltration.

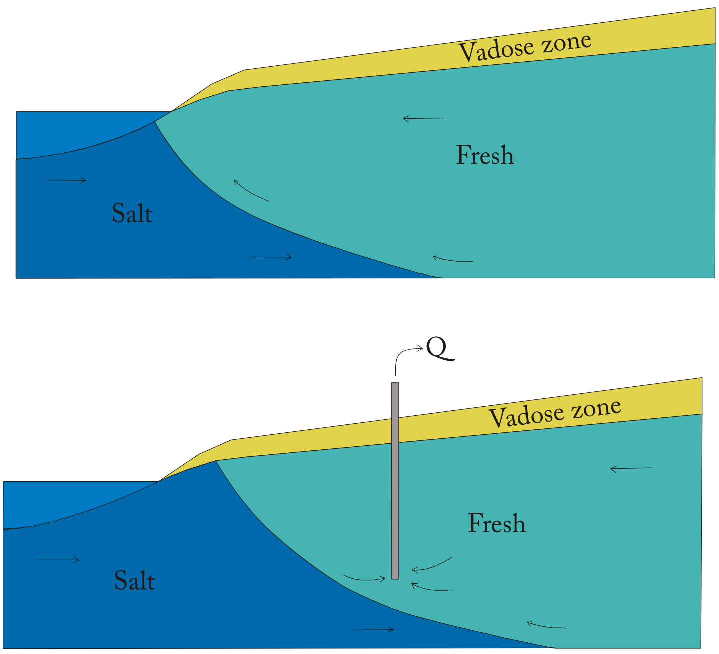

Figure 11 – Schematization of the coastal zone. The lighter fresh water is underlain by the heavier salt water (above). Due to groundwater extraction, no fresh water reaches the sea (below).

3.2 Historical water management in Oman: the aflaj system

Due to the low precipitation and high evaporation characteristic of the semi-arid climate, the water supply in Oman has constituted of groundwater resources for centuries past. This water is traditionally either harvested by small wells, or by a so called falaj (plural aflaj). A falaj is a small canal, which services sometimes as an irrigation system, not only allocating the water, but also distributing the discharge over the falaj shareholders in a fair and equitable way (Al-Marshudi, 2007). The aflaj distributes water flowing under the influence of gravity. Water rights for a falaj are sold at a water auction and can be inherited through generations. The water transported by the aflaj system is distributed between its shareholders based on a time schedule. This means that whenever a drought occurs all the shareholders receive the same amount of water as before, adjusting not the volume but the length of the distribution cycle. In a regular aflaj system, the rights are distributed over one dawran (the cycle of time allotments) which starts on Friday and ends on Thursdays, each farmer receiving its share once a week. When a drought occurs the cycle may be extended to a period of 12 days (Al-Marshudi, 2007).

There are three types of falaj in Oman. The first is the ghaily falaj type, which collects water from wadi base flow. Water flow from this falaj type can decrease significantly during the dry season, or may even seize. The second type is the daoodi or iddi-falaj type. These aflaj consist of an underground tunnelling system, with several vertical shafts for better access, transporting water to the regions where the water is needed for irrigation. The last falaj type is that of the ayni falaj, which are fed by springs and are a more reliable source of water (MWR, 1998).

3.3 Water management since the start of the renaissance

The period from 1970 onwards is known to the Omani's as the Renaissance, referring to the period of unprecedented development following the accession of His Majesty Sultan Qaboos Bin Said in the same year. Since the start of the Renaissance, Oman has changed its character, creating new opportunities for business and export. The increased industrialization and wealth as well the growing population, caused an increase in water demand. The population size in Oman is expected to increase by 50% between 1995 and 2025 (Dawood, 2005). In the year 2000, up to almost 85% of total water use was used in agriculture, 10% for domestic use and the remaining 5% for industrial purposes. Dawood (2005) shows that this differentiation is about to shift, meeting 60% agricultural water use, 25% domestic use and 15% for industrial use in 2025. By this year, the total water demand will have risen with 64% compared to the year 2000.

There are 623 well fields located in the whole of Oman, and 43 desalination plants in use. These resources yearly produce 28.65 MCM and 73.73 MCM of potable water respectively (Prathapar et. al, 2005).

The increased water demand has caused a groundwater deficit in most regions of the country, including the more densely populated Northern Al-Batinah region. Groundwater levels in this region yearly decline by 0.03 to 0.19m (Ashworth, 2006). In Shisr, in the Western Al-Wusta Desert region bordering Saudi Arabia, the groundwater table has fallen locally as much as almost 30m over a 10 to 15 year period (Ashworth, 2005). Most of these deficits are caused by extensive pumping of the aquifer. In 1991, the MRMEWR commenced on a well-inventory project, in order to get a better overview of all groundwater extractions. There appear to be more than 128.000 wells in Oman which are tapping the major aquifers (Jamrah et al., 2008[1] and MRMWR, 2010).

To create a more sustainable water supply, an effort has been made to make good use of storm water, which would otherwise be lost to the sea by fast runoff. This floodwater would better be stored underground than behind surface structures, since evaporation losses will be minimized (Bouwer, 2002 and Hut et al., 2007). The water infiltrated will recharge the aquifer. Natural recharge is defined as the difference between the input and output of water in the soil, being the amount added to the groundwater system. The total natural recharge amounts typically to about 30-50% of precipitation for temperate humid climates, 10-20% of fallen precipitation in Mediterranean climates and only 1-2% for dry climates, like in Oman (Bouwer, 2002). When artificial measures are used to recharge the aquifer, the added amount is defined as being enhanced recharge (Martín-Rosales et al. 2007). Due to the fact that the potential evapotranspiration is larger than the precipitation, recharge in semi-arid regions depends on (de Vries en Simmers, 2002):

1) rainfall events of high concentration (with following storm flows)
2) accumulation of rain water in depressions and streams
3) percolation speed and infiltration capacities, to avoid evapo(transpi)ration

Other research shows that most of the fallen precipitation in arid regions is lost to evaporation (Wheater, 2002).

In this thesis the definition of Lerner et al. (1990) found in De Vries en Simmers (2002) has been used for characterizing direct recharge: Direct recharge is that amount of water that is added to the groundwater reservoir after precipitation, in excess of soil-moisture deficits and evapotranspiration by direct vertical percolation through the vadose zone. Any water infiltrating through wadi-beds, belongs by this definition to the indirect recharge: percolation through the beds of surface water courses. The efficiency of an infiltration structure is then defined by the percentage of water actually reaching the groundwater table from the total amount of water passing through the infiltration structure.

3.3.1 Mitigation policy in Oman

Apart from enhancing the recharge, an effort is also made to reduce the water demand itself. The following is currently done in Oman to achieve either of the above.

1) Enhancing recharge by controlled wadi flow infiltration of storm water collected in storage dam reservoirs (artificial recharge) (MWR, 1998).
2) Artificially recharging treated greywater in Dhofar region (Shammas, 2007).
3) Reducing the camel population, in order to preserve tree canopy in the Salalah region. The abundance of vegetation ensures the occurrence of so-called horizontal precipitation, making the monsoonal fog available for recharge (Shammas, 2007).
4) Changing irrigation practices in agriculture (Shammas, 2007).
5) Creating new water resources: making use of desalination of seawater for drinking water purposes.

Further proposed techniques to lower water use or increase water resources in Oman are the following:

1) Jabal reforestation in the Salalah mountain region, to increase horizontal precipitation from fog.
2) Further political measures to change irrigation systems in agriculture.
3) Changing the current water allocation system to one where every client receives a prescribed amount of water. Any amount used above this value will be charged.
4) Reuse of greywater resources (Jamrah et al., 2008[2]).

3.3.2 Recharge by controlled wadi flow from temporary dam storage

To restore the groundwater balance, the Omani make use of storage dams, controlling water flow-rate over the dam into the wadi, to let as much water as possible infiltrate. Without such measures a large amount of sparsely falling precipitation would be lost to the sea. For these runoff procedures, the Omani defined a maximum period of 12 days duration (Forster et al., year unknown and MWR, 1998). This applied time limit ensures that the water stagnant behind the dam is not polluted by parasites.

3.4 Efficiency of current measures

All measures mentioned in the previous Sections depend on different principles to make them efficient. Some measures in itself might not cause the wanted difference, however, sometimes the combination proves to be fruitful.

3.4.1 Controlled wadi flow

The effect of storage reservoirs and controlled wadi infiltration on recharge in Oman is still to be better investigated (Al-Ismaily and Probert, 1998). Attempts to increase the recharge in this way have not yet been proven successful (Wehenmeyer and Waber, 2002).

The amount of effective recharge of controlled wadi flow from water stored behind dam structures depends on the evaporation during the storage phase (temporary as surface water) and evaporation from the topsoil after infiltration (Haimerl, year unknown and Bouwer, 2002). Since the subsoil is often dry or has only low moisture content when a precipitation event occurs, water is first also 'lost' to the storage in the pores itself. Any 'empty' pores will after the infiltration event remain filled to the residual moisture content. Any water remaining in these pores might later on evaporate (Wheater, 2002). However, the losses are negligible in the case of a yearly occurring flood. After dry periods of longer duration, evaporation losses can amount to 10% of the initially stored flood water (Haimerl, year unknown). Evaporation occurs up to several meters of depth (Haimerl et al., 2002).

The effectiveness of infiltration from spatially variable flow, like from a wadi channel, is hard to predict (Bouwer, 2002). Research on two recharge/flood protection dams in the Dhofar region assumed a recharge efficiency of 75% (Shammas, 2007). Research on an experimental slot in Saudi Arabia also shows that on average 75% of bed infiltration reaches the water table (Sorman and Abdulrazzak , 1993 in Wheater, 2002). Haimerl modeled recharge with and without a dam in wadi Ahin on the Al Batinah plain (Haimerl, year unknown). The results showed that the recharge more than doubled in the case of a recharge dam controlling wadi flow present. The effective recharge increases when the width of the infiltration bed or structure (in the case of a recharge dam) is increased (Haimerl, year unknown).

The 2009 resistivity survey performed by the ministry, shows local improvement in water table elevations and salinity (MRMWR, 2010).

3.4.2 Artificial infiltration of treated greywater

Near Salalah, treated effluent is artificially infiltrated by vertical wells to act as a vertical barrier for seawater intrusion (Shammas, 2007). Also the pumping of saline water from the toe of the saltwater wedge is discussed (van Dam, 1999). Recent modeling showed that this could lead to a reduction of the lateral advancement of the saltwater front by 700m (Kacimov et al., 2009). Shammas (2007) states that the artificial infiltration of treated effluent needs to be combined with measures concerning vegetation density to have a sufficient effect on the Salalah plain water balance in future.

3.4.3 Other measures

No publications on the results of the changing of the irrigation system could be found. However, this is expected to be large, since irrigation efficiencies vary between 50-60% for the most ineffective systems, up to 95% efficiency for the most effective drip-irrigation systems (Shammas, 2007). Adopting drip irrigation at root level, will reduce the water use for landscape maintenance by approximately 75% of the currently used amount (Foster and Karpiscak, 1995 in Al-Ismaily and Probert, 1998).

Measurements show that the infiltration of treated effluent in Dhofar (Salalah plain) has decreased the salinity of the groundwater and has presumably pushed back the saline front. The water levels have also risen, since the start of the artificial infiltration (Shammas, 2008).

4. Storm runoff infiltration in the Al Batinah region

4.1 (Geo) Hydrology and infiltration characteristics of the Al-Batinah plain

The Al Batinah plain comprises the coastal area, reaching roughly from Muscat in the east to the boundary with the United Arab Emirates in the west (Figure 12). The Al Batinah plain is bordered by the Gulf of Oman to the north and the Al Hajjar mountain range to the south. Groundwater in this coastal region originates mostly from precipitation over the mountain range, arriving downstream in the form of flash floods, infiltrating in the abundant wadi-channels.

Figure 12 –The location of the Al Batinah Plain. Map modified from Al-Lamki and Terken (1996).

The Hajjar mountain range consists of hard rock of different age and formation. It also comprises the Oman Ophiolite (Figure 13). Within ophiolite rocks, groundwater flow occurs mostly in the near surface zone, where the rocks are fissured (approximately 50m thick) and to a lesser extend in tectonic fractures (Dewandel et al., 2005). The ophiolite rocks have a hydraulic conductivity of 10^{-5} to 10^{-7} m/s. At the foot of the mountain range, there are at least three coarse gravel terrace levels to be found, wherein water flow is dependent on terrace thickness and the degree of cementation (Anderson, 1986). The Terraces are underlain by the calcareous rocks of the Hajjar Mountains.

Coastward of these terraces and in the centre of the Al Batinah Plain, a deep Neogene sedimentary basin is located, which is filled in with sediments belonging to the Fars Group and younger alluvium (Figure 14). The sediments are said to consist of, from top to bottom, clean gravels (also called the upper gravels), clayey gravels and cemented gravels (Young et al., 1998 and Anderson, 1986).

Another classification of the alluvial material distinguishes only the clean gravels and cemented gravels (Bhatnager and Ravencroft, 1986).

Well testing showed that the alluvium and Upper Fars consist of several, possibly interfingered, aquifers and aquitards. Average total thicknesses for the alluvium and Fars aquifers are obtained and comprise 7 and 60m respectively on the Northern Al Batinah plain(Ashworth, 2006). The total thickness of saturated aquifer at the Northern Batinah ranges between 20 and 160meters, in an area larger than 600km^2 of the plain (Young et al., 1998). For the alluvium and Upper Fars combined, a hydraulic conductivity value K of 36 m/day was obtained (Ashworth, 2006). The clayey gravels, part of the middle Fars group, are considered to be generally impermeable and locally form the base of the upper aquifers (Ashworth, 2006).

Groundwater tables on the Batinah plain show a steady drop. For the twelve catchments under consideration, Ashworth found that in only one catchment groundwater table is stable (Ashworth, 2006). In six of the catchments, water levels were falling at an increasing rate in the 1995-2004 period, compared to the 1987-1995 period. Ashworth shows that high salinity groundwater occurs near the coast and that this is the result of saline intrusion.

Infiltration of storm runoff is the most common form of recharge on the Al-Batinah plain, since the ephemeral wadis cannot supply a constant flow of water. In case of a storm event, the wadi bed is often dry, so infiltration is dependent on previous moisture conditions and soil characteristics of the wadi bed. Haimerl (year unknown) shows that the infiltration rate becomes higher, when the soil was previously dry. Because the infiltrated water has to fill the pores, before reaching the aquifer, the higher infiltration rate does not lead to a higher percolation rate and higher permeability. The permeability is increased when the pores are filled, and the domain is formed where the flow can occur (Haimerl, year unknown). The percolation velocity rises when the surface water level is increased, increasing the potential. The percolation velocity of a wet soil is twice that of a dry soil (Haimerl, year unknown).

In wadi Ahin, about 200km west of Muscat, an infiltration rate between 1.8 and 3.6m a day was measured. The infiltration test showed a 126day period was necessary for infiltrated water to reach the groundwater table, locally at a depth of 23m b.g.l., in the case of a previously dry bed (Haimerl, year unknown).

The dam basin itself, does often not contribute to the recharge. The behind-dam wadi bed becomes clogged with silt, preventing infiltration. Due to the sedimentation behind the dam, sediment load of the overflowing water is low and easily infiltrates in the gravel beds (MWR, 1998). However, Haimerl speaks of a small sediment layer deposited in the bed of wadi Ahin, stating that it will form the limiting factor for infiltration in future (Haimerl, year unknown). The design of the dam structures is such, that at maximum discharge, a wetted area sufficient to infiltrate all flood waters is available downstream (MWR, 1998).

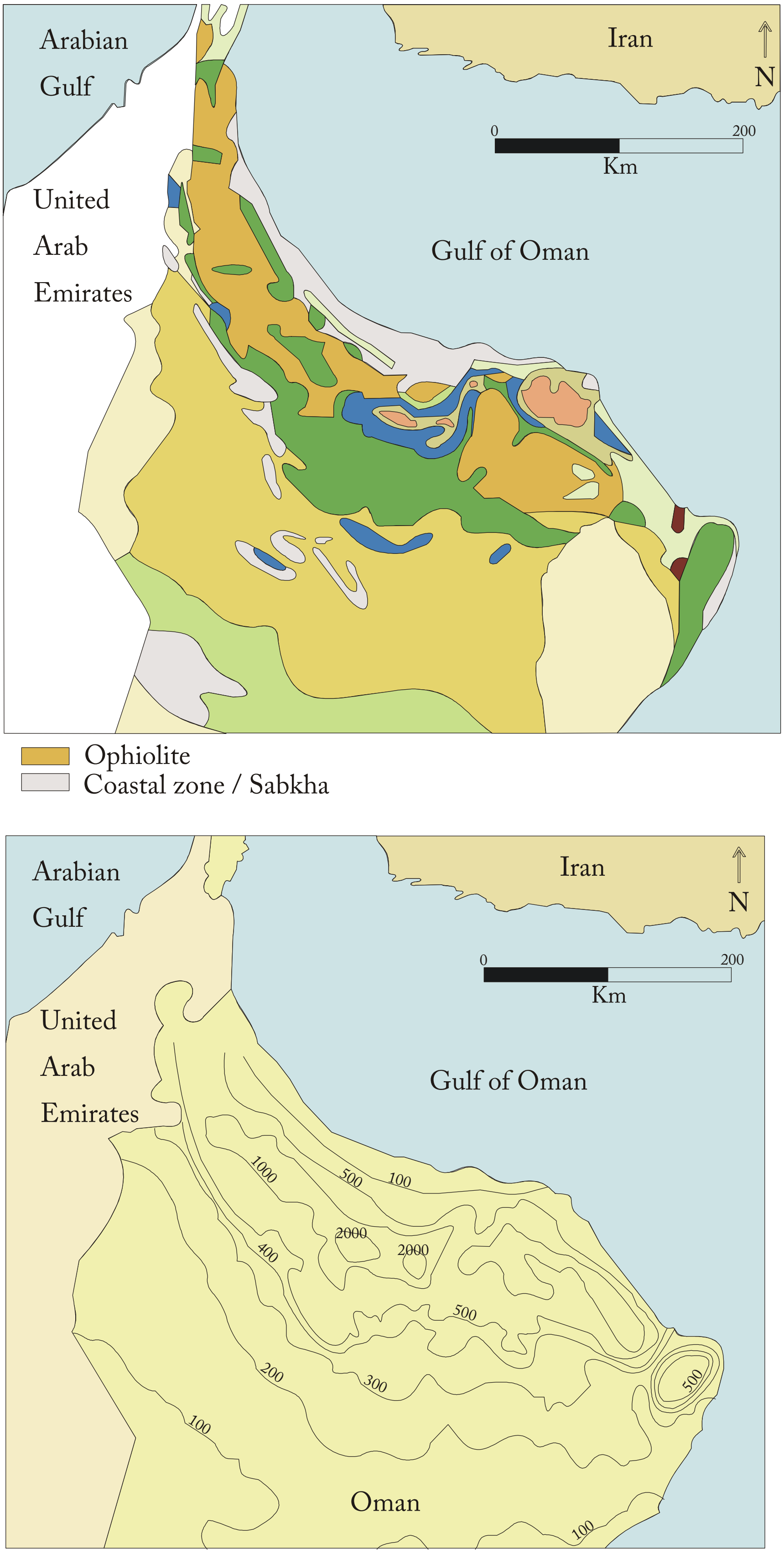

Figure 13 – Geology (above) and altitude (below) map of Northern Oman. Maps modified from Al-Lamki and Terken (1996). See for total legend of geology figure the original article.

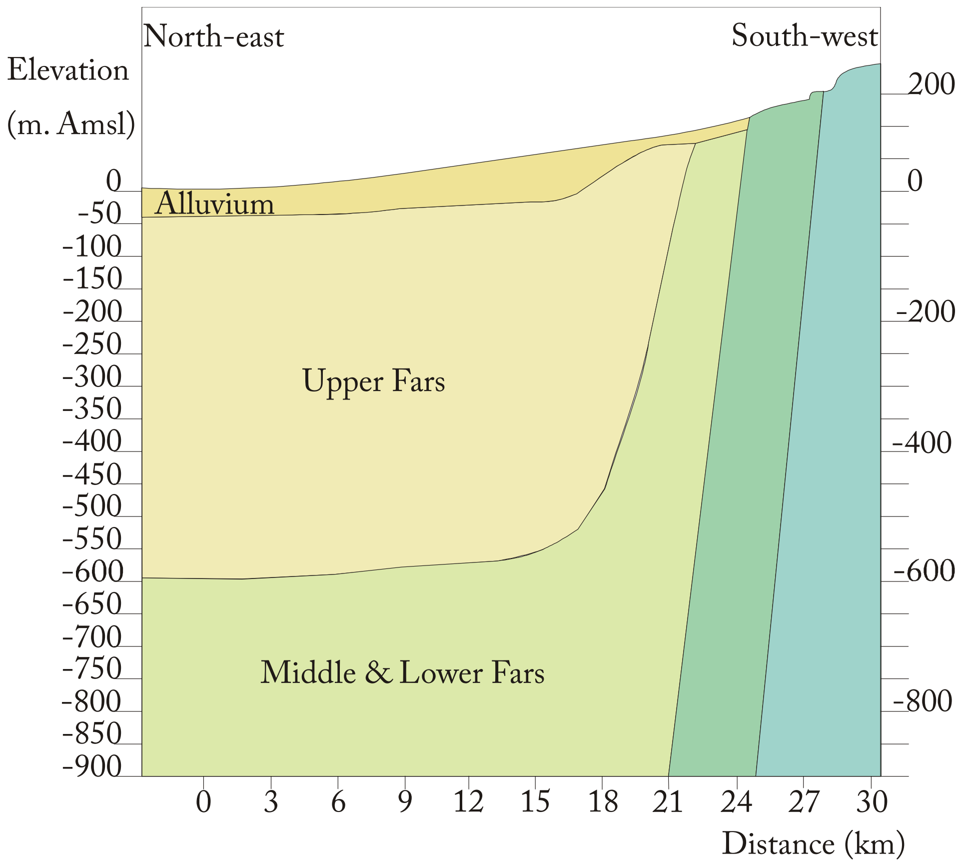

Figure 14 – Representative cross-section of the Neogene basin, as found within wadi Ahin, about 190km west of Muscat. Figure adapted from Ashworth, 2006.

4.2 Stormflow characteristics for the region

Precipitation amounts in Oman differ greatly, reaching a 75mm per year average in the Muscat Governorate and up to 340mm per year in the mountains (MWR, 1998). Precipitation events are characterized by their high variability and unpredictability in both time and space, as is common for arid and semi-arid regions (Wheater, 2002 and Martín-Rosales et al., 2007). Floods in the al Batinah region originate from the runoff of the Hajar mountain group. This mountain range is characterized by steep slopes, rising from approximately sea level at the shoreline, to locally almost 3000m at less than 75km inland (ITMB publishing, map of Oman and UAE and Andersson, 1986). Some floods occurring in Oman belong to floods with the highest peak flow/area ratio in the world (Wheater, 2002). Floods in arid regions are typically characterized by a flood wave reaching peak discharge within 15 to 30 minutes (Wheater, 2002). Changes in water load of wadi-channels may thus happen very rapidly. Figure 15 shows two typical hydrographs for both a non-jabal area and a jabal (mountainous) area.

As a flood moves downstream over a (relatively) dry bed, the flood volume is reduced as a consequence of bed infiltration. (Wheater, 2002). How the volume is reduced depends on how much water can infiltrate, percolate and be stored in the subsoil. However, a study of flood propagation in Saudi Arabia showed that the bed storage didn’t lower the flood peak as much as expected (Wheater, 2002). This effect is contributed to possible air entrapment in the pores.

The bed material of Omani wadis commonly consists of unconsolidated gravel and cobbles. High velocity flows may be able to change the stage-discharge relationship of the wadis (Zidjali et al, year unknown).

The amounts of water that pass through a wadi during a flood can be very large. In November 1997, 8.26 million cubic meters of water passed through wadi Aday, near Muscat. In March 1999 23.2 million cubic meters of water passed through wadi Al Bat'ha, near Bilad Bani Bu Hassan, to the southwest of Sur.

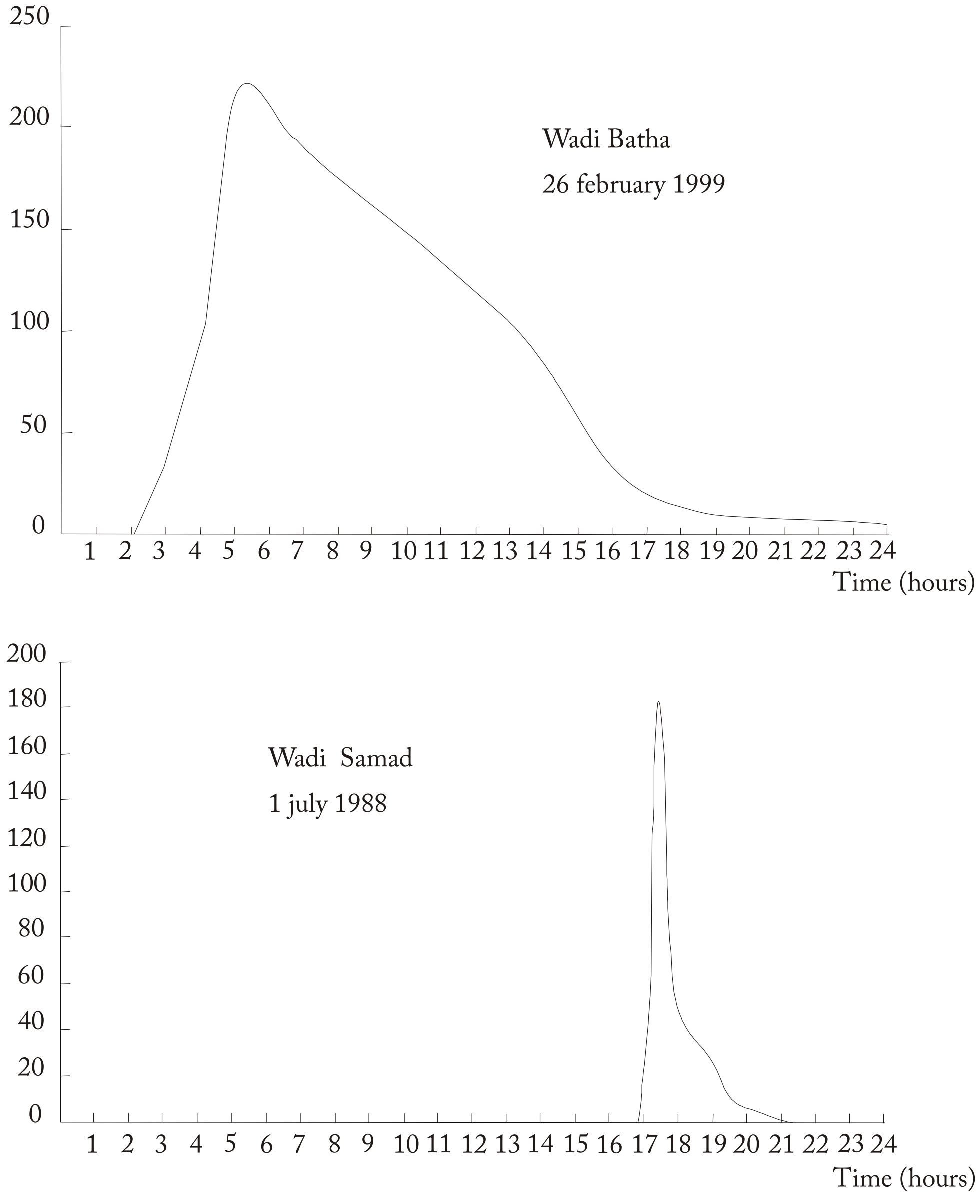

Figure 15 – Typical flood hydrographs for a non-jabal area (above) and a jabal area (below). Figures from Zidjali et al. (year unknown).

4.3 Present infiltration structures in the region

In the Gulf coast countries Saudi Arabia, Oman, Qatar and UAE combined, there were in total more than 650 dams built by the year 2000 (Al-Rashed and Sherif, 2000). The dams vary in size and function, with sizes up to several hectares in storage volume, functioning as a main storage system, as a first step for recharge, as flood prevention or all of the above combined. At present, 14 recharge dams are fully constructed on the Al Batinah coast (MRMWR, 2010).

Anderson (1986) reports that new feasibility studies were performed to build a series of low retention and buffer dams to control the recharge, but also to be able to collect some water to be directly transported to the city. Indeed most dams in Oman are low in height; the average height reaches only 10m. The length of the structure is on average 3.5km, with an average storage capacity of 4 million m^3 of water (MWR, 1998). The dams are normally designed to withstand a five to ten year flood, although there are some dams which are designed to withstand a once in a century flood (MWR, 1998).

The spillway of a dam is the dam section where water can controllably overflow to be discharged, without damaging the dam. One of the longest spillways on an Omani dam is found in wadi Ahin. It reaches a length of 4000m. The dam is designed to withstand a once in ten year flood, with an effective storage capacity of 6.8 million m^3 of water. In 1995 the dam overflowed, when a suspected 30 million m^3 of flood water flowed through the wadi. For this catchment, a flood this size belongs to the once in a century events.

5. Considerations on infiltration on the Al Khawd Fan

5.1 Hydrograph interpretation and flood size estimation

In Oman, the volume available for infiltration is determined by the flood volume stored behind the dam. When a flood moves downstream, water is infiltrating in the wadi bed. The amount of water infiltrated in this way may significantly reduce the flood intensity and can obscure the right interpretation of a flood hydrograph (Wheater, 2002). When analyzing dam efficiency with respect to flood reduction downstream, flood data at a certain location up or downstream of the dam, should therefore be handled with caution

When the peak discharge of a flood is estimated by rain gauge measurements, the estimate can significantly differ from reality. The estimation depends on the amount of raingauges present per area unit. The gauge intensity in the USA (one per 20 km^2) proved to cause errors in peak runoff estimation of up to 58% (Wheater, 2002). The gauge intensity in Oman is several times lower than that used in the USA.

The data used in this research for volume estimation of the storm runoff to be infiltrated, is obtained from measurements behind the dam structures. Any bed infiltration in the uphill flow path is therefore already removed from the volume. Infiltration through the basin floor can still occur, however no estimations of the infiltrated volume are known to this author. As a consequence of sedimentation, the infiltration at the basin floor behind the Al Khawd Dam has reduced significantly and is probably negligible on the total flood volume.

5.3 Effects of reducing irrigation ‘losses’ by a change in irrigation system

To reduce the water demand, the Omani farmers are encouraged to switch to more economic irrigation practices (Shammas, 2007). However attention should be paid to the reduction in recharge which is subsequently caused. The ‘irrigation losses’ mentioned by Omani authorities refer to the amount of irrigation which is not used for crop growth, and in that way ‘lost’ to the system. This ‘lost’ water may in fact be recharging the aquifer (Foster, Tuinhof et al, 2003). A switch to more economic irrigation measures might therefore reduce the recharge and have the opposite effect of the objective under view: restoring the groundwater balance by reducing water demand and extraction rates and increasing recharge.

5.3 Clogging of the injection well.

We set out to see if VIS could infiltrate the flood volumes on the Al Khawd Fan and to model the effects it would have on sea water intrustion. In Holland, the injection wells and drains used clogg sometimes. This often occurs when the temperature of the injection water is higher than 10 degrees Celsius, which contributes to the growth of iron bacteria (Schuurmans and Steinmetz, 1984). These bacteria are formed as strands or flakes and attach to either the inner wall of the injection well or to the filter cracks (Houdé, 2010). Temperatures of storm runoff in Oman are probably higher than the 10 degrees Celsius limit needed for bacteria growth. Based on temperature data alone, bacterial growth can therefore be expected to occur in injection wells in Oman.

When an injection well is (partially) clogged, it can help to regenerate the well by reversing the flow direction. The well is then temporarily used for extraction purposes. However, this technique alone does not regenerate the well when it is bacteriologically clogged by iron bacteria (Schuurmans and Steinmetz, 1984). The clogging of a well is measured by the resistance, which is apparent from piezometric head differences between the injection well and an observation well nearby. Tests conducted with an injection well used by a water company, show that the abandonment of the well for a period of 190 days, combined with pumping of each filter section separately, reduced the resistance to almost original levels. During this operation adding of a small amount of sodium hypochlorite appeared to have a positive effect on regeneration, but it didn't prevent the well becoming clogged again later (Schuurmans and Steinmetz, 1984).

Because flash floods in any certain wadi in Oman might occur up to 2, 3 or more times a year, while sometimes even none, it is expected that the clogging will not pose a substantial threat to the application of VIS in Oman. The time between the yearly flood periods should be sufficient to regenerate the wells when an increase in resistance is measured.

6. Preface of Part 2

This first part of the thesis presented the use of storm runoff infiltration policy currently under practice in The Netherlands and on the Omani Al Batinah coast, as well as an outlook on Omani water practice and the water challenges the Omani face in future. We discussed different VIS designs in detail.

This introduction to the subject finished, the results the of the Al Khawd Fan case study are presented in Thesis Part 2. Part two presents detailed information on the Al Khawd Fan topography, climate, geology and hydrogeology. It also describes the model workflow, set-up and results of different scenarios.

Thesis Part 2 presents the answers to the research question, investigating if deep infiltration of storm runoff by means of a VIS-technique, is more efficient than the currently in Oman used controlled wadi flow, with respect to:

1) Combating seawater intrusion by infiltration at depth
2) The speed of infiltration of a certain volume

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The application of Vertical Infiltration Systems (VIS) in the Sultanate of Oman

Part two

Heleen C. Kiela

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September 2010

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1. Introduction

A numerical model is created of the Al Khawd Fan, in order to investigate the possible use of vertical infiltration wells to recharge storm runoff in Oman. The Al Khawd fan area is located in the Southern Al Batinah region, about 45km from Muscat (Figure 16). In this 2nd part of the thesis 'The use of Vertical Infiltration Systems in the Sultanate of Oman', the modelling practice and results are presented. The results are used to estimate the possible effectiveness of the infiltration wells, with respect to
- Seawater intrusion mitigation
- Speed of infiltration

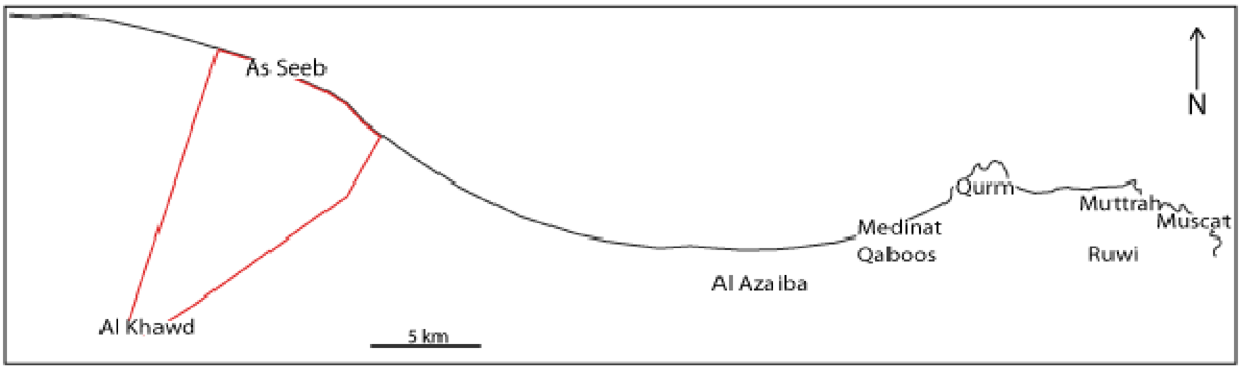

Figure 16 – Location of Al Khawd Fan

2. Study area

2.1 Location

The Al Khawd Fan catchment is located on the Southern Al Batinah coast, about 45km from Muscat city. The catchment is located between UTM 615000 and 625000 E and 259800 and 262000 N. The catchment is directed from southwest to northeast, draining the upper catchments. The Al Khawd fan discharges to the Gulf of Oman in the north.

2.2 Topography and geomorphology of the Al Khawd Fan

The Al Khawd alluvial fan forms the lower part of the Semail Catchment (Figure 17). The catchment drains the Al Hajjar mountain range, in particular the Jebel Akhdar region and covers 1720 km^2. The catchment can be divided into an upper catchment area called the Semail Basin, and the Lower Al Khawd and Seeb deltaic area (Al Ghilani, 1996). The two regions are connected by a gorge at the city of Fanja. The Al Khawd fan is bound by wadi Ar Rusayl in the east and wadi Manumah in the west.

The elevation of the Al Khawd fan catchment area varies from about 80m a.s.l. in the north, to 0m a.s.l. at the coastline. The gradient ranges from 0.012 in the most southern part, to about 0,0005 in the coastal strip (Al Ghafry, 1991).

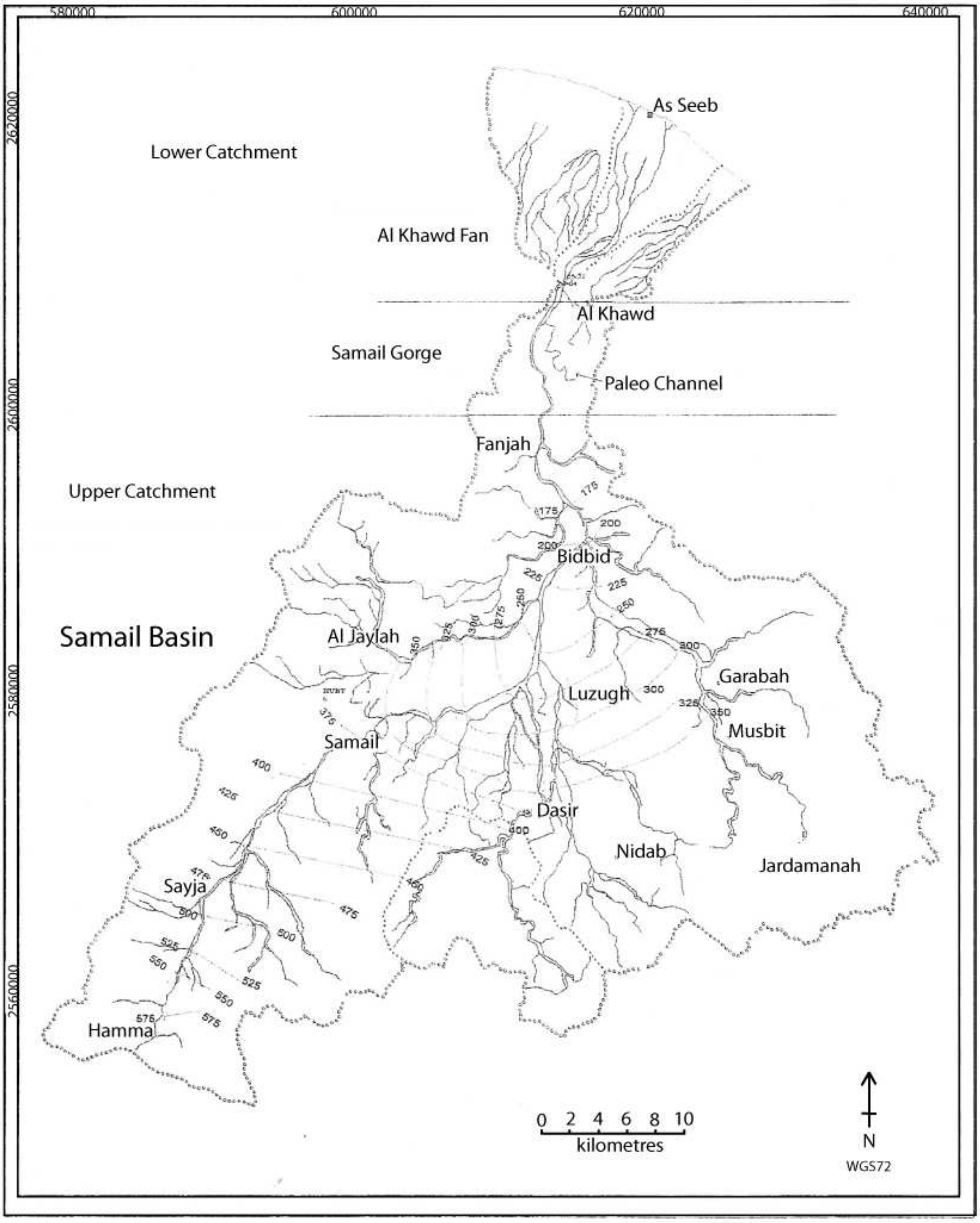

Figure 17 – Samail Catchment and Al Khawd Fan. (Macumber, 1997).

2.2.1 Well fields

There are four well fields located on the Al-Khoud fan:

i) PDO Well field
ii) Al Khawd dam Well field
iii) Seeb Well field
iv) Old Government Well field

The locations of these well fields are depicted in Figure 18. The Al Khawd Dam Well field and Seeb Well field combined are also known as the Western Well field. This is one of the three main sources for drinking water supply to the Muscat area. Other sources are found in the Wadi Aday, Eastern well field and the Al Gubrah desalination plant (Al Ghilani, 1991). The PDO Seeb well field provides potable water for the PDO buildings in the area. The Old Government Well field is out of production.

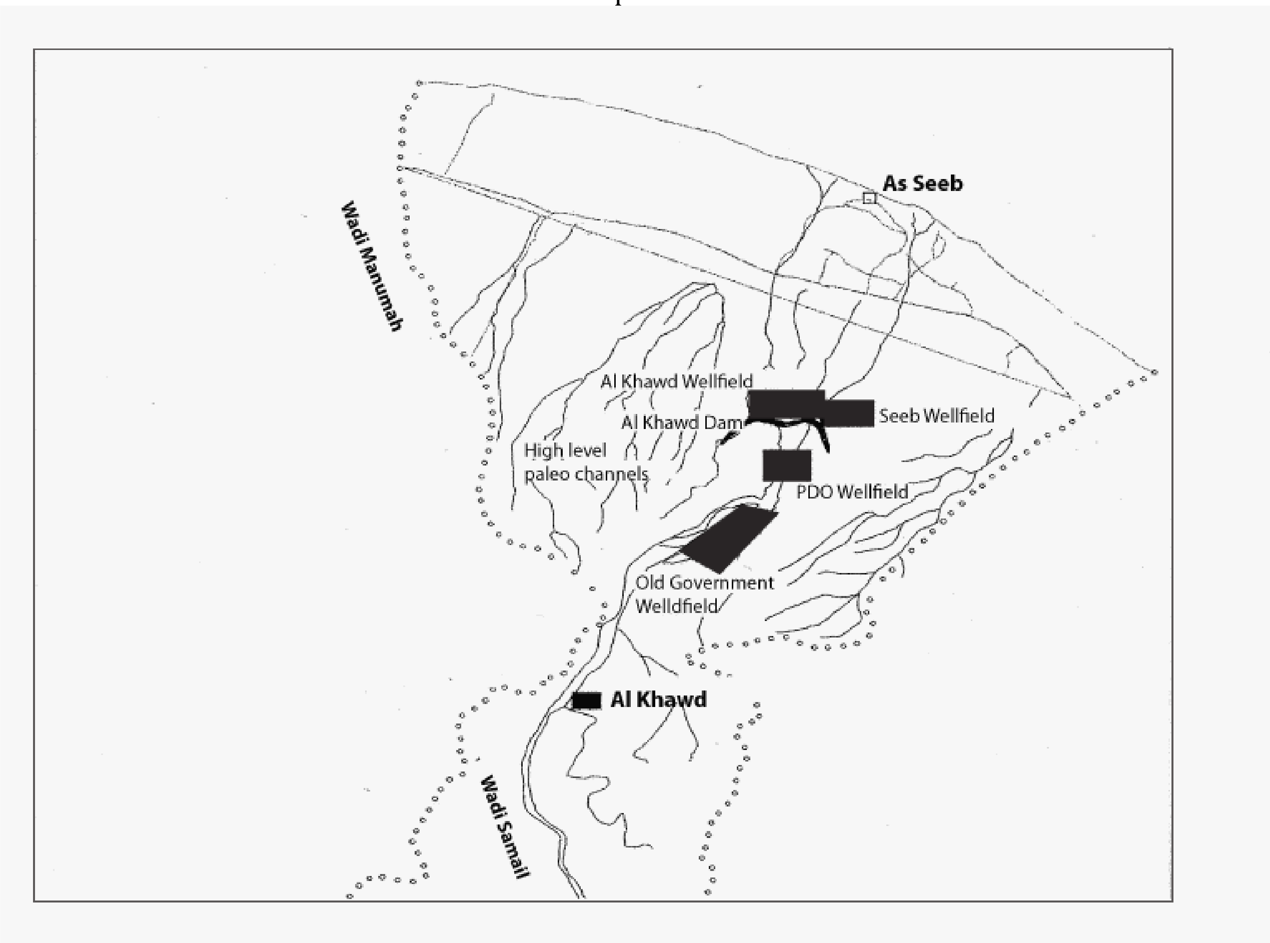

Figure 18 – Location of the well fields on the Al Khawd Fan (Macumber, 1997).

Besides these institutional wells, there are also wells which are operated for private, agricultural use. These wells are mostly localized in the near-coastal zone, where agriculture is traditionally occurring. The extent of the agricultural region has decreased over the last years, as a consequence of salinization of the production wells. In 1995, a total number of 4000 wells in the Semail catchment was reported, of which 1734 were situated in the lower Semail catchment of the Al Khawd Fan (Al Ghilani, 1991). Ghilani sums that the MEW well field produces 7MCM/yr, while the PDO well field produces 0.6MCM/yr. PDO tabulated 6.4MCM/yr for the MEW well field, 0.7MCM/yr for their own, and 15.5 MCM/yr for agricultural use (PDO, 1995).

2.2.2 Recharge dam

The Al Khawd dam has been constructed in 1985 and has until recently been the largest dam built in Oman. The dam covers the recent, active wadi channel over the Al Khawd fan. The second, inactive wadi channel is that of wadi al Manumah, which only transports water in case of major floods. The dam is situated 6km inland of the coast at the city of Seeb.

The Al Khawd dam is an earth filled dam with a maximum height of 11m. Planning is underway to reinforce the dam with concrete slabs (oral comment, Ali Al-Maktoomi, 2010). The author obtained different values for the total storage capacity of the dam ranging from 14 MCM (Phrabu, 2010), to 12.5 MCM (Al Ghafri, 1991) and 11.6 MCM (MWR, 1998). The design report considers 11.5 MCM storage (Stanley Consultants, 1981). Al Ghafri (1991) speaks of a flow rate of 12.500m/s. He also estimates that of an 11.5MCM flood, about 4.5 MCM is recharged to the aquifer. The wadi flow during the storm events lasts up to several hours and occurs a few times per year (MWR, 1998).

A photo page of the Al Khawd dam can be found in Appendix 1.

Infiltration of the flood water occurs downstream of the dam. The approximately 2 day period of retaining the water behind the dam before commencing the spill, is devised to reduce the sediment load. The sedimentation means the retention basin slowly silts up. Locally, the silt layer reaches up to 4m in thickness, prohibiting infiltration (oral comment, Ali Al-Maktoomi, 2010). Silt is also transported downwards in the soil, forming an underground silt-crust, further prohibiting infiltration. The overlying silt layer is being removed by the ministry to keep the volume of the basin as large as necessary. However, this is an ongoing process and at present the ministry is not able to remove all the silt at one time in between floods. The flash flood occurring after the Gonu event in 2007 caused the dam to overflow.

2.3 Climate and precipitation

The Omani climate is typical of an arid country. During summer in Oman, the maximum daily temperatures can easily reach 40°C or more. During spring and autumn, the maximum daily temperatures are between 25°C and 35°C. Winter temperatures are usually still more than 20°C.

Precipitation estimates from an approximately 100year record, are around 100mm/yr in the town of Al Khawd and up to 250 mm/yr in the upper Semail, north of Fanja (Al Ghilani, 1996). The MWR reports 75 mm/yr on the Batinah plain (MWR, 1998). It is believed that about 40% of the volume precipitated in the upper Semail catchment reaches the Al Khawd Fan, and that the total amount of precipitation in the whole Semail catchment amounts to 23.2% of the annual rainfall in the Muscat governorate, comprising a total 226MCM a year (Gibbs, 1986).
The evaporation at the Batinah coast is estimated to be around 2100 mm/yr (MWR, 1998).

According to the Köppen-geiger classification system, Omans climate is classified as **BWh** climate; a hot, dry desert climate with the annual average Temperature above 18°C..

3. Geology of the Al Khawd Fan

3.1. Tertiary and older formations

The highest part of the Al Hajjar mountain range drained by the Semail Catchment, is built up of rocks belonging to the Hajar Unit and Hawasinah Formation (Permian to Cretaceous age), as well as the Oman Ophiolite nappes, which provide the catchment with sediment (Al Ghafri, 1991). The ophiolite nappes are also known as the Samail Nappe, and comprises ultra mafic rocks characterized by intense fracturing and weathering. Further north these allochtonous rocks are unconformably overlain by the post-autochthonous rocks of Tertiary and Maastrichtian age. These deposits consist of interlayered limestones and marls sequence of, from top to bottom, the Dammam Formation of middle Eocene Letetian carbonate, the Rus Formation of early Eocene regressive deposit and the Umm Er Radhuma Formation of Paleocene yellow marly limestone. The Tertiary deposits are covered by possibly Tertiary and Quaternary gravels, forming the true Al Khawd fan system.

3.2 Quaternary alluvium

The Al Khawd alluvium is often vertically subdivided in an older and a recent alluvium zone, which is basically a division between cemented and non-cemented alluvium deposits (Bhatnager and Ravencroft, 1986). Because the non-cemented, recent alluvium overlies the older cemented alluvium, the distinction is often also made by dividing the fan in an upper gravel and lower gravel region. Sometimes a third, intermediate layer is recognized, the so called clayey gravel (Gibb, 1976). Figure 19 shows a profile through the Old Government Well field, drawn by Al-Ghafry (1991). It shows a 'clay' layer over the whole profile between the upper gravel, and lower, cemented gravel zones. However, this clayey gravel layer is sometimes overlying a second upper gravel layer, which in turn overlies the more cemented layer and is sometimes not present at all. It is therefore believed to be non-uniformly present and so, that the tri-diversion is to be strongly questioned (Macumber, 1997). The cementation appears to be the result of ongoing diagenesis of the peridotite and gabbroic pebbles present in the deposited gravel and sand. This causes the degree of cementation not to be determined by the depositional process of the alluvium. The cemented gravel is believed to be the further diagenetic step of the clayey gravel unit (Macumber, 1997).

In Figure 20, a conceptual diagram of the fan geology is shown (Al-Barwani, 1996 in Nasser, 1996). Figure 21 shows a top-view of the geology at the surface, constructed by Gibb, 1976.

Cementation is believed to be more extensive in the southern and eastern part of the fan (Bhatnager & Ravencroft, 1986 and Al Ghafry, 1991). This agrees with the location of the cemented gravels belonging to Terrace 1 and Terrace 2 according to the figure of Gibb. It also agrees with the profiles generated by Macumber (1976). The KWD-wells profile shows a well cemented lithology south of the KWD-2 well, while to the north, the cement is largely absent. The RGS profile (to the west of the KWD profile) shows an overall lesser cementation.

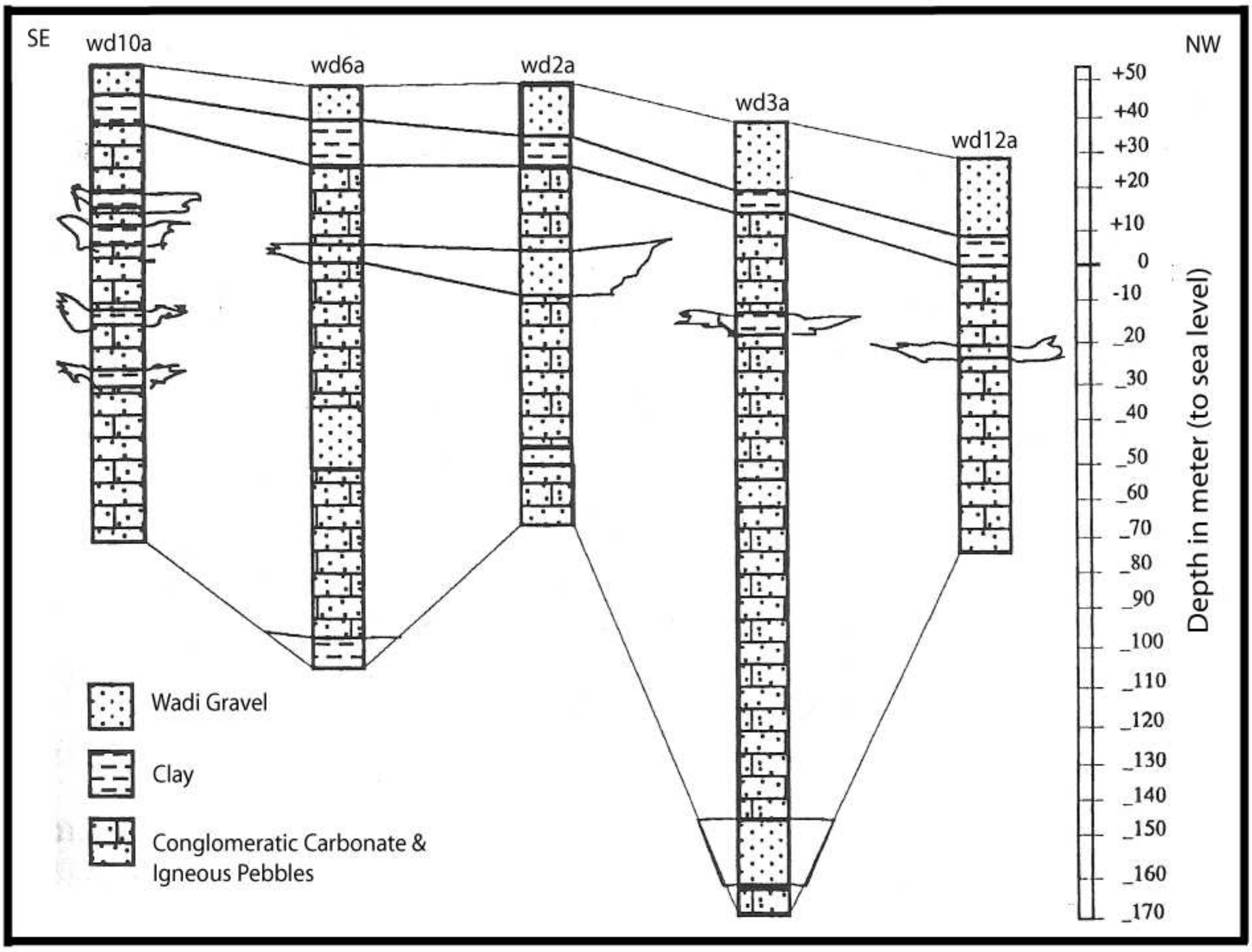

Figure 19 – Cross profile through the Old Government Well field (Al Ghafry, 1991).

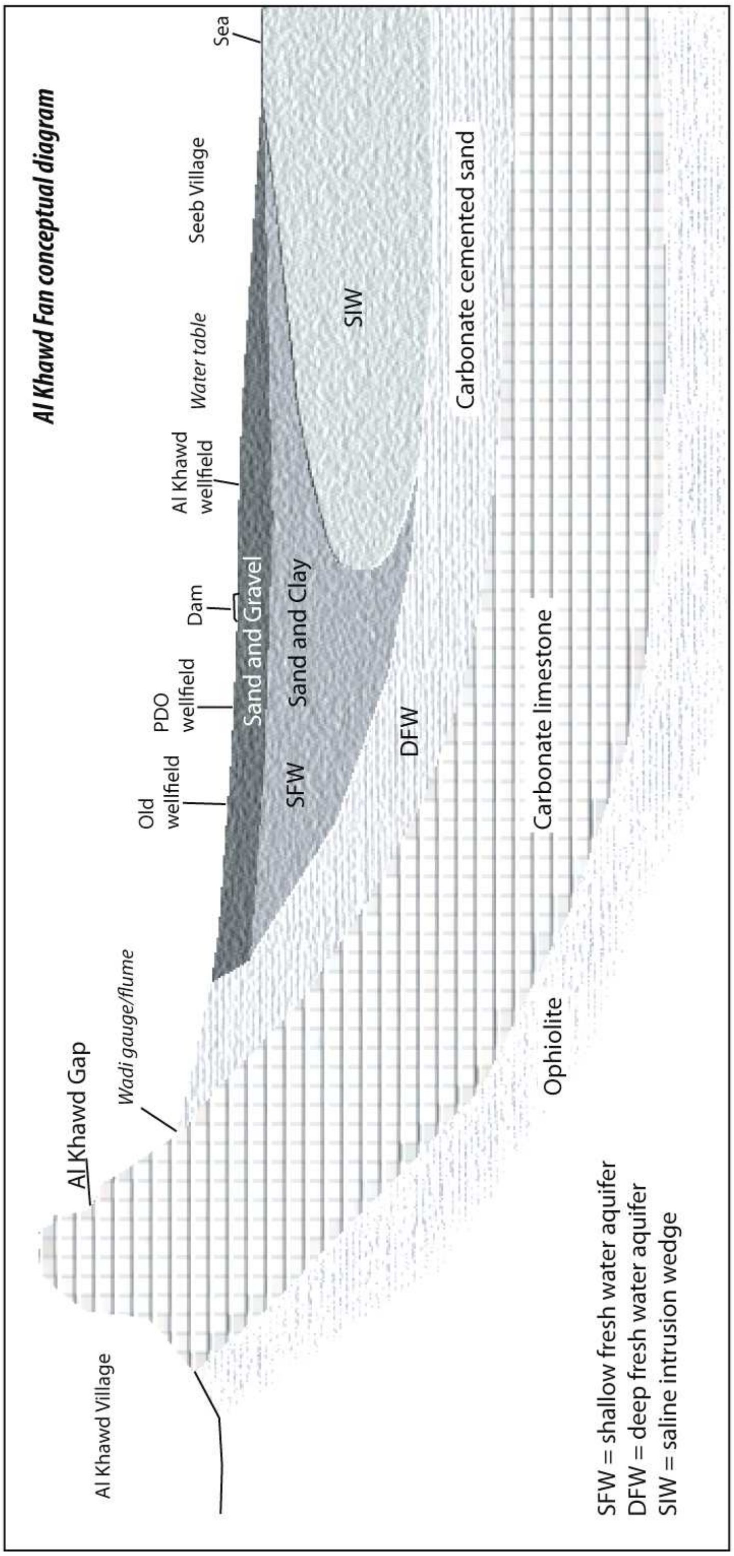

Figure 20 – Conceptual diagram of the Al Khawd Fan geology (By Al-Barwani (1996) in Nasser (1996)).

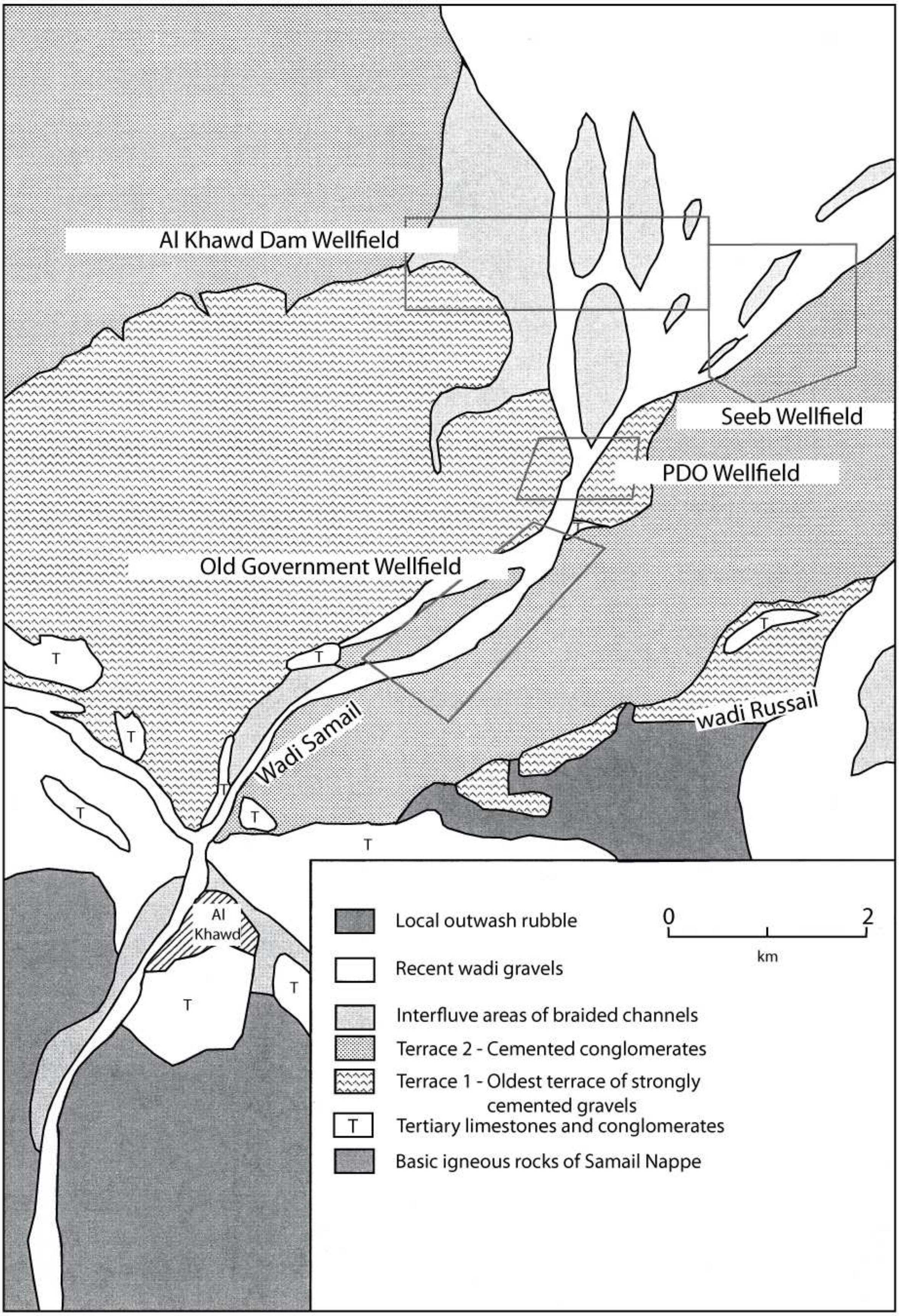

Figure 21 – Surficial geology of the Al Khawd Fan (By Gibb (1976) in Macumber (1997))

The grain size of the Al Khawd fan alluvial deposits varies from boulder size to fine silty sand and clay, with fining sequences in the northward direction of the coastline (Al Ghafry, 1991). Due to the wadi-type sedimentation and diagenesis, the fan is characterized by rapid lateral changes in sediment facies.

The alluvium thickness is believed to reach up to around 600m on an alluvial terrace, while it appears to be absent in a drilling log from the most northern part of the lower semail catchment (well SMA-51B, MWR, 1997). Over a large part of the coastal plain, the alluvial thickness is over 300m (Macumber, 1997). Al Ghafri specifies the upper gravel (UG) alluvium thickness to be of a range between 10-20m in the southern part of the fan, while about 100-120m in thickness further towards the coast. The clayey gravel (CG) is believed to be between 50-300m in thickness in the northern half of the fan if present, while the thickness of the lower gravel (LG) remains unknown, due to a lack of deep boreholes (Al Ghafri, 1991).

MWR drilled new deep bores on Al Batinah, and named them the 21-series. Two of the 21-series wells have been drilled in the Al Khawd fan. Both wells are situated on the direct downstream side of the dam. The western 21-6 drilling showed a gravel-dominated sequence up to 230m, overlying the more cement-dominated layer. The eastern 21-7 showed that the gravel-dominated layer reaches only to 80m, also underlain by the more dolomitic and cemented layer (Macumber, 1997).

The differentiation of three alluvium classes of clean, clayey and cemented gravel or alluvium does not indicate any age of the sediments. However, in the 2006 North Batinah project it was possible to define the lithology by age, discerning sediments of Quaternary and Tertiary age. They showed that on the Northern Batinah the Quaternary alluvium was situated near the piedmont zone, with the Tertiary sediments forming the majority of the fan system. These Tertiary sediments belong to the Fars Formation, which – on the Batinah plain - can be subdivided into the Upper Fars, Middle Fars and Lower Fars system (comment, J.M. Ashworth 2010). The Upper Fars is usually more cemented or consolidated than the quaternary alluvium (Ashworth, 2006). The sediment sequence represents a regression period, with the Lower Fars comprising calcerous shale and weak limestone sediments of shallow marine origin, the Middle Fars consisting of siltstones and mudstones of a still reduced environment, and the Upper Fars consisting of terrestrial dolomites, chalky limestones and dolomitically cemented conglomerates. The fact that this differentiation could be identified to the north of the Eastern Batinah, does not nescessarily make the classification valid for the Eastern Batinah and Al Khawd Fan area itself. However, the depositional processes along the Batinah coast can be expected to have been similar and thus it is likely that the above described alluvium is part of the Fars Formation.
However, Bhatnager&Ravencroft (1986) suggest the Al Khawd alluvium to be deposited during the Pleistocene pluvial period, which does not agree with the supposed Tertiairy age of the Fars Formation. Since Bhatnager&Racencroft based their research on drillings to depths at which – if the sediments belong to the formation – the Fars Formation should have been encountered, no assumptions about the age of the sediments can be made based on these two reports alone.
Since the Fars Group lithology changes countrywide depending on the depositional environment, it is not wise to try and classify lithology afterwards, only based on the often summier description of the drilling crew. It is therefore also not tried here, and we have settled for the clean and cemented gravel classification.

4. Hydrology of the Al Khawd fan

The water of the aquifers in and below the Al Khawd fan is of different ages and origin. The alluvial water at approximately 90m depth is between 5-10yrs old and originates from wadi infiltration. The deep alluvial aquifer water is more than 4000yrs old (PAWR, 1986). In the even deeper aquifers of the Fars and Umm Er Radhuma Formations, the water is still older and sometimes connate to the deposits.

4.1 Hydrogeology

4.1.1 Tertiary and older formations

It is likely that there are intensive fracture systems in the Samail Nappe ophiolite structure contributes substantially to the (secondary) porosity of otherwise fairly impermeable strata (MWR, 1995). This system is believed to recharge the alluvial fan with water infiltrated higher up in the mountains (Macumber, 1997). Although this formation probably recharges the alluvial fan with a reasonable volume of water, it is not a probable resource for drinking water production, since it is mainly stored in the secondary porosity structures.

The Tertiary limestones show limited karstic characteristics. On the Al Khawd coastal plain, not much is known about the resources from the Tertiary limestones. However, work elsewhere shows that these are generally of low porosity and contain brackish water (MWR, 1995). The waters in Tertiary aquifers are often naturally saline, due to the connate character of the water and mineral composition of the matrix (Macumber, 1997). The carbonates of the Umm Er Radhuma Formation (UER) have been reported to form an aquifer (Lamki&Terken, 1996). However, Ashworth argues that the UER aquifer characteristics are poor, as it is underlain and overlain by fairly impermeable strata and has only very small surface exposure. The UER aquifer thus lacks a source of recharge. Research showed that on the Northern Batinah, deeper aquifers are found in the Aruma and Hawasina sediments of Createaceous age. However, these aquifers are also of poor quality(Ashworth, 2006).

4.1.2 Quaternary alluvium

Due to the importance of the aquifer for drinking water production, there have been several studies on the hydrogeology of the Al Khawd fan.

On the Batinah plain to the west of the Semail coastal area, the Upper Fars Group houses reasonable aquifers, although of small extent (Ashworth, 2006).

For the schematization used in this investigation, use has been made of two alluvium layers, like suggested by Bhatnager and Ravencroft:

i) Upper, uncemented gravel (UG) – composed of clean gravel and sand, with the highest hydraulic conductivities. Comprises the main aquifer and is saturated near the coast, but becomes partially unsaturated to the south as a consequence of rising elevation. K-values expected to be between 1-55m/day.

ii) Lower gravel (LG) – cemented gravel of lower permeability. Highly cemented conglomerates, marl and clays, with k-values between 1 and 7 m/day. Although this layer is less permeable, the possible large thickness might mean this layer forms an important part of the aquifer.
It is indicated that due to secondary porosity, the cemented gravels might locally be of higher conductivity (Macumber, 1997).

Bhatnager and Ravencroft (1986) give average conductivity values of 34 m/d for the upper layer and around 1m/d for the lower, cemented layer.

The MEW well construction and upgrading program resulted in transmissivity values ranging between 10-450 m2/d for the Old Government well field, with an average of 90m2/d. A test performed in a solely dolomitic sequence, resulted in a transmissivity of 25m2/d. The Al Khawd dam well field and the Seeb well field have higher transmissivities, averaging 1500m2/d, with an average conductivity of 34m/d (Macumber, 1997 and MEW, 1985[1, 2, 3]). However, the validity of the analysis is strongly questioned by the author of this report, as is explained in the next paragraph.

4.1.1 Results from pumping tests

Over the years, several pumping tests have been performed on the Al Khawd fan. These tests are performed by different consultancy companies and/or departments of the responsible ministries. The raw data of these tests is often widespread over these organisations and not (yet) digital, so acquisition of these data appeared to be difficult.

In this research project, the pumping tests performed by M. Macdonald & Partners Limited consultancy during the well improvement works in 1985 have been studied. The analysis of the data resulted in the following observations:

1) Transmissivity values from constant discharge tests and step tests performed on the same well, sometimes differ by a factor 10.
2) The analytical analysis proved to be sometimes incorrect, disregarding the elastic storage effects of the unconfined aquifer by using a wrong part of the data curve for analysis of the constant discharge tests performed.
3) Also, the wrong dataset was plotted on the x-axis when analyzing with the unconfined Theis recovery method, using the sum of t (time since pumping started) and t' (time elapsed since pumping stopped), instead of the supposed division between t and t'.
4) For analysis of the performed step tests, use has been made of the Logan approximation. This method of analysis is described in the first edition of Kruseman & de Ridder (1970), but has been removed from the second edition (1990). The Logan approximation is in effect an approximation of the Thiem formula and uses a constant to represent the log-term. The constant is only locally applicable; MacDonald and Partners found a value of 1.32 for their analysis scheme of pumping wells in Pakistan, while Logan proposed 1.22 as 'typical' for the log ratio (Misstear, 2001). There is no report of the value used for the constant in the analysis of the 1985 MacDonald and Partners pumping test scheme on the Al Khawd fan.

All of the above raised significant questions about the usefulness of the pumping test data, existing analysis and average values. We tried to reanalyze the data, but this proved to be fairly impossible since no down-hole lithology description was available for almost all wells.

This description was needed, to determine the type of system under consideration. When plotted on a log-scale, the theoretical drawdown curve for a confined aquifer would normally form a straight line, which is known as the Theis curve. The water pumped from a confined aquifer is released by the expansion of the water and compaction of the aquifer (elastic storage), and a change in piezometric head occurs. Only when the radius of influence reaches a barrier boundary or a recharge boundary, the curve changes shape (Figure 22). In case of an unconfined aquifer, an s-shape curve would be expected due to the delayed water table response (Kruseman & de Ridder, 1990). This means that the curve in essence would follow the Theis curve, exept for a levelled part in the middle caused by the change in origin of the pumped water. An unconfined aquifer is first pumped from elastic storage, similar to a confined aquifer. The unconfined aquifer then starts producing water by dewatering, and the water table falls. The fall in water table creates a gradient which causes the flow to be non-horizontal (flat section in the curve). At later times, the flow towards the well becomes essentially horizontal again, and thus follows the Theis curve.

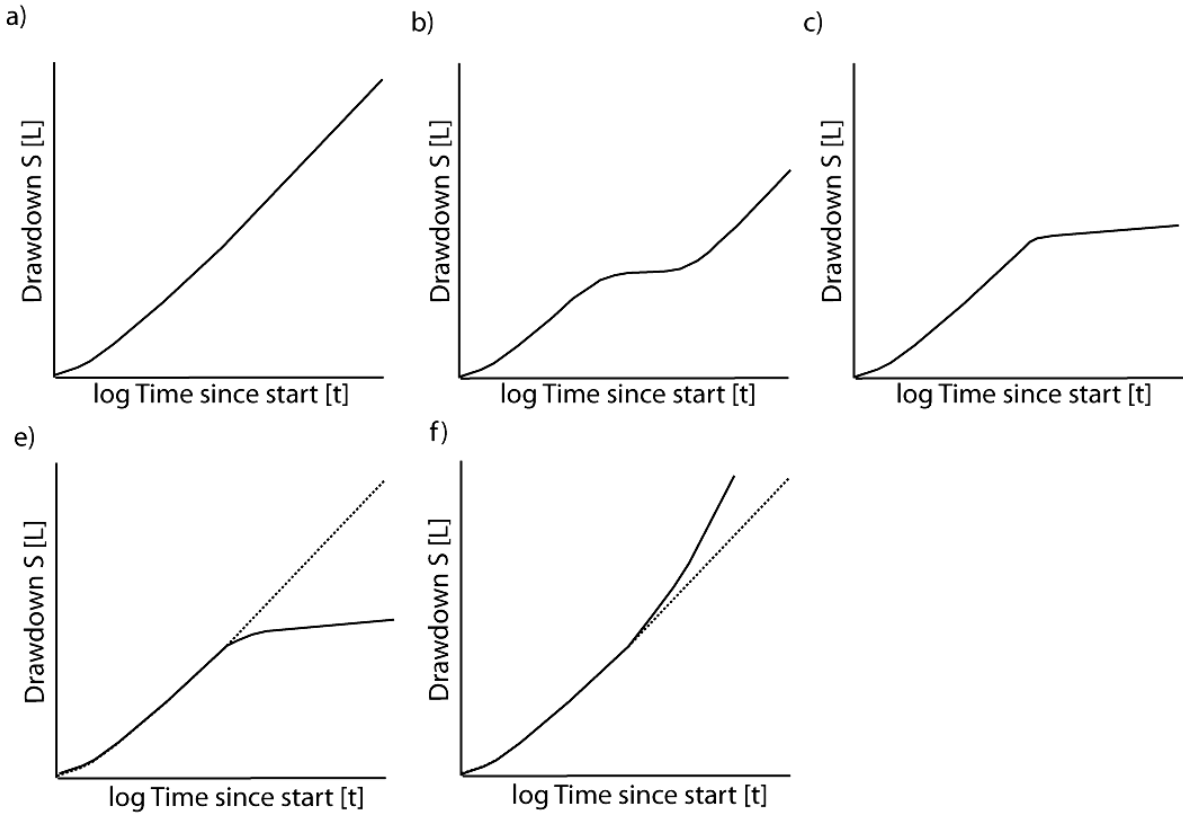

Figure 22 – Semi-log plots of the theoretical time-drawdown relationships of unconsolidated aquifers. a) confined aquifer, b) unconfined aquifer, c) leaky aquifer, d) confined aquifer with recharge boundary, e) confined aquifer with a barrier boundary.

Most of the curves from the 1985 well improvement works dataset, deviate from the straight line and level off to a plateau. This could mean that you are looking at the late-time data from the s-shaped curve of an unconfined aquifer. However, this would mean that the rising limb of the S-curve would have passed within one minute after start of pumping, and that the third, again rising part of the curve would not be seen within 2000min. after start of pumping. This would mean that the flow would not become horizontal again within 2000min., which seems unlikely for the locally high conductivity lithology type.

The well could have been in a leaky (semi-confined) or in a confined aquifer or zone. The radius of influence then might have met a recharge boundary. As there are only ephemeral streams on the Al Khawd Fan, this doesn't seem very possible. In case of a pumping test conducted shortly after a precipitation event, the ephemeral stream would have discharged the flood, but flooding would probably prevent any pumping test being performed nearby. A very possible and worrying possibility is that the water pumped for the well testing has been discharged at a too close distance to the pumped well, becoming the recharge boundary in itself.

All of the above led to the decision to disregard these pumping tests.

4.2 Aquifer flow pattern

Macumber (1997) describes different flow patterns on the Al Khawd Fan. The so-called regional flow is believed to stem from the ophiolite fracture system. Local strong upward gradients occur, with heads around 5 to 10m a.s.l. at 300m depth, while the water table is often below 1m. Stable isotope ratios are often similar to that of modern groundwater. However, some of the deep wells show a significant depletion, appearing to have been recharged at high altitude, in line with the regional flow pattern.
The deficit on the water balance, the location of the zero-head contour being further coastward than expected from the water balance deficit, strengthens the belief that there is a fracture system present in the ophiolite system. The intermediate flow is originating from processes upstream of the al Khawd dam, which are still located on the fan. On this upstream part, the wadi deeply incises into the old terraces. Base flow in this region occurs mostly through the wadi bed and causes steep gradients. The pressures are transmitted downstream and influence the local flow system. Local flow is flow which is originating from the wadi flow downstream of the dam, where it is easy to infiltrate. Here, the wadi has been able to spread over the plain and infiltrate a larger width than possible on the upstream cemented terraces. The wadi bed here not only acts as a line resource for recharge, but also increases the gradient perpendicular to the stream, so that flow occurs also in this direction.

4.3 Water table measurements

As the upper Samail catchment is so much larger than the lower fan, the piezometric response to an up-catchment precipitation event on the lower catchment is high, and hydrographs show distinctive and large peaks. The response to major flood volumes is visible in both shallow and deep piezometers. The local flow originating from wadi infiltration causes the spiky nature often seen in hydrographs of the Al Khawd Fan. A more saw-tooth like response is seen in hydrographs representing a combination of local and intermediate flow. The pressure gradient caused by recharge on the cemented terraces at the head of the fan, acts as a transmitted pressure effect downstream. After a spike response to a precipitation event, the head levels will often return to a different level than before the event, as determined by the pressure levels of the intermediate system. The pressure effect is likely to be larger when a dry period preceding a precipitation event, since the gradients are than largest. The water table fluctuation on the upstream, more cemented part of the fan is usually between 4 to 5m, except for some bores on the oldest terrace, where the fluctuation reaches a 25 to 40m. Any effects of the regional flow system are not easily identifiable, and probably subdued. It is not clear to what extend the regional flow influences the intermediate flow system, but it might be similarly impacting it, like the intermediate flow is influencing the local response.

A 10-year measurement series shows that head values declined steadily, at a rate ranging from 0.05 to 0.25 m/yr at the southern part of the fan and between zero and 0.1m/yr at the north-western area (MWR, 1995). Head levels on the western part of the catchment, and in adjacent wadis to the west of the fan, also showed a steady decline in head. The decline was less visible on the eastern half of the Al Khawd fan, but this does not necessarily mean that the process was not taking place. It is very probable that the process is slower, due to a lower permeability (Macumber, 1997).

It is apparent that a depression in groundwater heads has developed on Wadi Taww coastal plain, to the west of the fan. The depression continues to the Al Khawd Fan, with head measurements up to 0.3m below MSL in 1995 (MWR, 1995).

Abdallah and Al Rawahi (to be published, 2010) show that the zero head contour line at Al Khawd fan has recently moved towards the coast by as much as between 1 and 6.5km in 2005 from it's maximum known position in 2000 (Figure 23). The period between 1975 and 1995, when the dam was built, locally shows a coastward movement downstream of the dam, while in the western part of the fan the zero head contour line still moved inland. According to Abdallah and Al Rawahi (to be published, 2010), the withdrawl of the zero head contour line between 2000 and 2005 is the result of the reduction in pumping rate from 2003 and total cessation of pumping at the Western Water Well field in 2005. The author has been unable to verify this with the ministry.

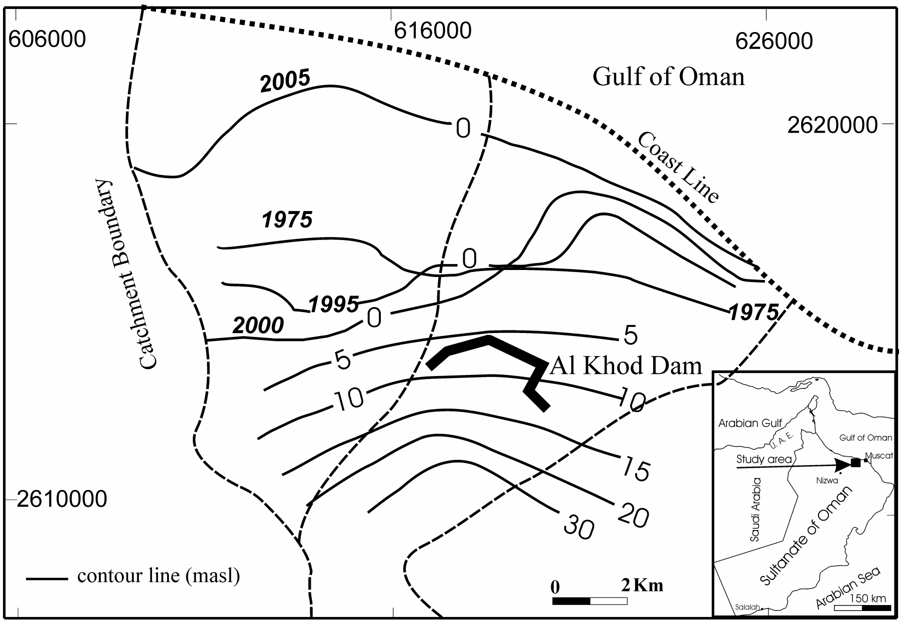

Figure 23 – Location of the zero-head contour line, from Abdallah and Al Rawahi (to be published, 2010)

4.4 Groundwater salinity

Seawater intrusion in the Al Batinah coastal region of Oman has led to the abandonment of farms, as some agricultural wells have become saline (Al Zidi, 2009; Zekri 2010). Ashworth (2006) has reported a steep salinity gradient in the northern Al Batinah region and several authors have drawn a steadily inland movement of the saline front over the past decades (Ashworth, 2006; Al Lamki, 2002, amongst others). Al Lamki and Terken (1996) and Bijjali (2005) show that there is a decrease in salinity from discharge to recharge zones.

The building of the Al Khawd recharge dam in 1985 was the first attempt to mitigate the intrusion by enhancing recharge. As for now, there is no sound proof that the dam is very effective in the mitigation of seawater intrusion. The pattern of head changes upstream, is very different from the pattern downstream of the dam (Abdallah and Al Rawahi, to be published 2010). The upstream head behaviour seems somewhat arbitrary, while the rise of water tables downstream of the dam correlates well to recharge events from the dam. This suggests that the aquifer is at least to some extent recharged by the controlled wadi flow from the dam.

Kacimov et al. (2009) show that the extensive pumping in the Al Batinah region has locally led to a coastal hydrologic situation unlike the classical one, as described by Bear (1999). In the Al Batinah case, there is locally no fresh water discharging to the ocean (Figure 24). This conclusion arises from a three year study of the fan near the town of Al Hail, at the northern coastline of the Al Khawd fan, where water table and salinity measurements were taken, as well as ohm-restitivity measurements and total chemical analysis of samples. During the three year period, no freshwater was located on top of the saltwater (oral comment, Kacimov 2010). After a precipitation event, temporarily reducing the salinity of the top layer, the salinity would bounce back to more than 10.000 ppm within one week. The transects with Ohm-restitivity measurements reached up to several hundreds of metres inland, not showing any sign of less than 10.000 ppm saline water present. The majority of these measured data has not been published, since it was lost after the unfortunate death of one of the team members.

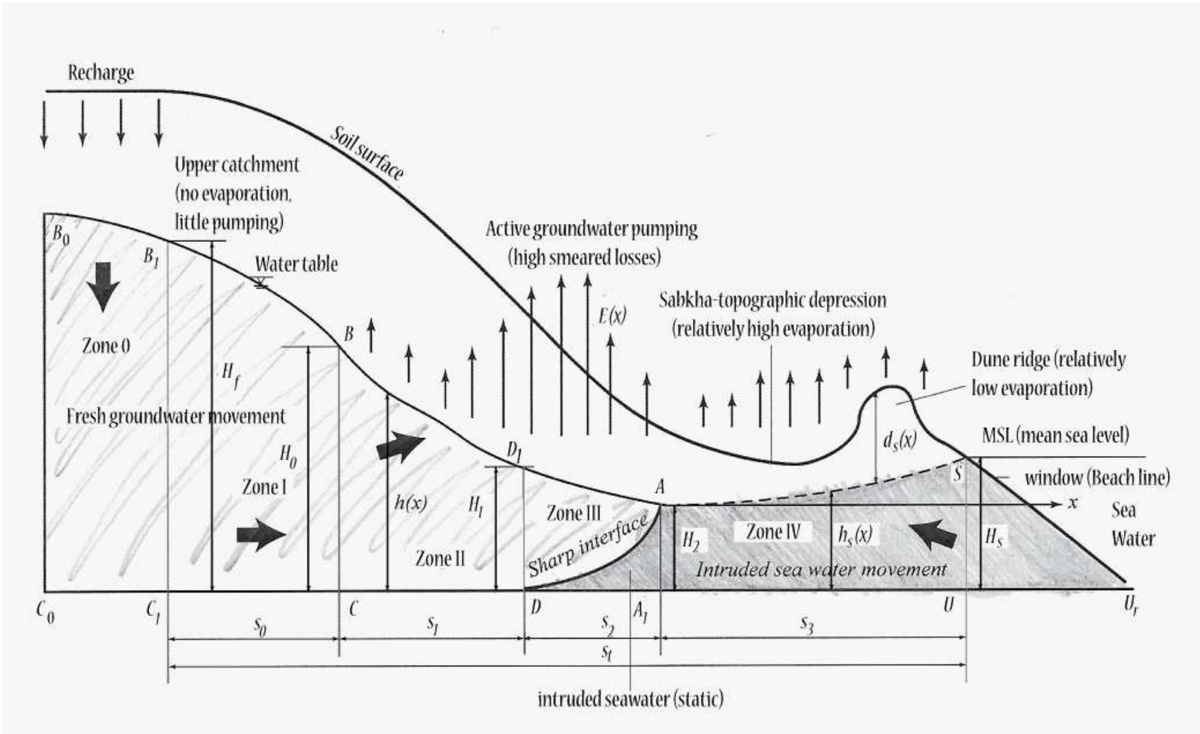

Figure 24 – no fresh water is reaching the window, no fresh water zone overlying most of the intrusion wedge (From Kacimov et. al. (2009).

As mentioned in the previous paragraph, Abdallah and Al Rawahi (to be published, 2010) show that the zero head contour line at Al Khawd fan has moved coastward. This progression of the zero head contour line shows only that the fresh water lens overlying the salt water intrusion zone, is very near to making contact with the seawater interface again, but not that the salt water intrusion zone itself has reduced in size.

Weyhenmeyer and Waber (2002) show by isotope analysis that during the late 1990s of the 20th century the saline front was still moving inland.

Salinity profiles show that in 1986 saline water (>50.000 ppm) had not yet past well kwd-2, while in 1998 it was 300m further inland (Bhatnager & Ravencroft in Al Ghilani 1996 and Al Ismaily, 1998). Until 2002 no increase in salinity measured in the PDO well field was reported (PDO, 1995).

Bhatnager & Ravencroft (1986) show with conductivity measurements in two well profiles that there are locations where water of lower salinity is underlying the saline intrusion wedge. More recently, this has been confirmed by a 1992 TDEM survey and two new drillings in 1993.

Bhatnager& Ravencroft, suggest the possible presence of a confining layer as a contributor to the density distribution. The deep fresh zone appears to be confined by a cemented gravel layer and to be under artesian pressure (MWR, 1995). However, the horizon is not found in many lithological drilling logs. The similarity in response of shallow and deep heads also suggests that these regions are in hydrological contact. It is suggested that the deep presence of fresh water is the result of fracture flow through the Tertiary and ophiolite rocks underlying the alluvial fan. The presence of upward hydraulic gradients at depth in the aquifer, existence of high temperature springs to the south of the coastal plain further along the coast and hydrochemical analysis help to confirm this theory.

Deep groundwater of the fan is believed to be between 2000 and 12000 years old and has a similar chemical composition as high elevation rainfall and groundwater on Jebel Akhdar (Bhatnager&Ravencroft, 1986, and MWR, 2000).

The following characteristics of the fan influence the shape of the intrusion wedge:

- The laterally higher conductivity zone in the western part of the fan results in a larger inland distance of the saline wedge, compared to the lower conductivity zone in the eastern part of the fan.
- Vertically lower permeability zones create a stratified seawater intrusion wedge
- The strong upward head gradients at depth cause the fresh-salt water interface to retreat (Macumber, 1997)

Saline intrusion is also influenced by the presence of the governments' and PDO well fields and wells for agricultural use, as well as (enhanced) recharge after rainfall events on the fan or on the upper Samail catchment.

5. Model

5.1 Visual Modflow and Seawat

A numerical model is run to investigate the flow pattern and possibilities of VIS in a coastal catchment like the Lower Samail Catchment. For construction purposes little use has been made of direct measurements from the Samail catchment. Instead, a generic model has been created, with similar characteristics as from the Samail Catchment.

The numeric model is built in Visual Modflow (SWS, year unknown). Visual Modflow is an interface for the Modflow environment. For the current project, the Seawat engine was used for calculations. The numeric engine Seawat makes three-dimensional, variable-density groundwater flow calculations possible. Seawat combines Modflow2000 and MT3DMS, a flow code and transport code respectively, to solve the coupled flow and transport equations (SWS, year unknown).

The governing flow equation is:

$$\frac{\delta}{\delta x_i}\left[\rho K_{fx}\left(\frac{\delta h_f}{\delta x_i}+\frac{\rho-\rho_f}{\rho_f}\frac{\delta L}{\delta x_i}\right)\right]+\frac{\delta}{\delta y_j}\left[\rho K_{fy}\left(\frac{\delta h_f}{\delta y_j}+\frac{\rho-\rho_f}{\rho_f}\frac{\delta L}{\delta y_j}\right)\right]+\frac{\delta}{\delta z_k}\left[\rho K_{fz}\left(\frac{\delta h_f}{\delta z_k}+\frac{\rho-\rho_f}{\rho_f}\frac{\delta L}{\delta z_k}\right)\right]=$$

$$\rho S_f\frac{\delta h_f}{\delta t}+\theta\frac{\delta\rho}{\delta C}\frac{\delta C}{\delta t}-\rho_s q_s$$

Where:

x, y, z	coordinate axes
i, j, k	column, row and layer indices
K	conductivity [LT^{-1}]
h_f	equivalent freshwater head [L]
ρ	salt water density [ML^{-3}]
ρ_f	fresh water density [ML^{-3}]
L	cell centre elevation [L]
Θ	effective porosity [-]
S_f	equivalent freshwater specific storage [L^{-1}]
ρ_s	fluid density of source and sink water [ML^{-3}]
q_s	volumetric flow rate of source and sink per unit volume of aquifer [T^{-1}]

The governing transport equation is:

$$\frac{\delta C}{\delta t}=\nabla\cdot\left(D\cdot\nabla C\right)-\nabla\cdot\left(\nu C\right)-\frac{q_s}{\theta}C_s+\sum_{k=1}^{n}R_k$$

Where:

C	dissolved concentration of species k [ML^{-3}]
D	hydrodynamic dispersion tensor [L^2T^{-1}]
ν	linear pore water velocity [LT^{-1}]
C_s	concentration of the source of sink flux for species [ML^{-3}]
Θ	effective porosity [-]
$\sum R_k$	chemical reaction term, rate of the reaction k of total reactions n [$ML^{-3}T^{-1}$]

In the model, the PCG solver has been used, which stands for Preconditioned Conjugate-Gradient Package. This package can simulate linear and non-linear flow conditions, and uses inner and outer iterations towards the solution. A detailed description of the package is available from the USGS (Hill, 1997).

5.2 Workflow

When starting this project, we had the ambition to create a full 3D model of the fan. However, there were some difficulties with the data, which led to the decision to only create a 2D model. It must be emphasized that although it would be physically possible to create a 3D model, the outcome of such a model would have no validity and would not lead to any reasonable conclusions. Factors contributing to the assumption that a full 3Dmodel would not be successful or valid are the following:

1. The (observation) wells are very often screened over several intervals, which would mean that difficulties would occur when trying to distinguish on a vertical plane:
    - the head distribution (head measurements become an 'average' over the screen)
    - the salinity distribution (salinity measurements become mixed over depth)
    - any flow gradients (possible short-circuiting due to the locally strong upward gradient)
2. No knowledge of the location and flow rate of the fracture systems in the Ophiolite system, providing extra recharge to the fan.
3. No substantial knowledge of the intermediate and regional flow patterns, of which flow rates are also dependent on conditions upstream of the model-area, of which no data was available to the author.
4. The rejection of the pumping test data, which made the creation of a reasonable horizontal transmissivity distribution not possible.
5. No known alluvium thickness distribution over the catchment width and very little point information on geology and lithology accessible to the author.

To only use data points which are not influenced by one of the above would mean that the total amount of usable data points would be insubstantial to create a valid 3D model of the fan.

In the 2D model, the production wellfields and agricultural extraction zone are both represented by a singular well. The VIS will also be represented by a singular well, since a multitude of VIS wells will likely be set parallel to the coast to reduce the intrusion length over the width of the fan.

5.2.1 Model objective

The model runs for a total period of 20.000 days before the VIS is switched on, to create a base scenario which is affected by seawater intrusion. With the model, different scenarios are considered, with the goal of achieving the following:

A1. To at least reduce the growth of the intrusion length for a 10 year period of using VIS starting at 20.000days, comparing the position of the interface to the position after 10years from 20.000days, without using VIS.
A2. To stop the inland movement of the intrusion wedge compared to the position at t = 20.000days during 10 years of using VIS.
A3. To reverse the inland movement of the intrusion wedge and reduce the intrusion length by using VIS for 10 years from t = 20.000days, compared to the position at t = 20.000days.

The above achievements are in order of succes, with A1 achieving the smallest effect and A3 the best. Since going from reducing future growth, via stopping the growth completely, to decreasing the size (reversing from growth to decrease), achieving A3 will mean that A1 and A2 are also satisfied.

5.2.2 Testing different scenarios

The model is first used to calculate a base scenario. This scenario runs for a period of 20.000 days. In the results of the base scenario, the location and shape of the intrusion wedge should satisfy the following conditions:

C1. Near the coastline, there is a zone where no 'fresh' water is overlying the 'salt' water at least at some point to a reasonable pumping depth, like on the Al Khawd Fan.
C2. The tip of the intrusion wedge is close to the well field (singular well in the 2D-visualization), like on the Al Khawd Fan. A distance of 2km is used as a maximum between the two.
C3. There is a zone of fresh water underlying the intrusion wedge, like suggested to exist on the Al Khawd Fan.

This first step allows for optimization of the flow rates appointed to the wells representing the wellfields and agricultural extraction zone. The models boundary conditions and hydraulic characteristics are fixed and represent the generalized Al Khawd geohydrology, as is explained in Chapter 6.

When the base scenario is satisfyingly created, different so-called mitigation scenarios are run for a period of 10 years starting from t = 20.000 days. These scenarios differ in the location and screen length of the singular well in 2D, representing the VIS wells and are tested for the three achievements stated in Paragraph 5.2.1. The model is created with two horizontal zones of different characteristic, with the above representing the clean gravel zone, and the zone below representing the cemented zone. The scenarios are tested for different locations along the profile (different x). For each location, the filter is placed to two different depths (different z). The first is to the model depth of zone 1 only, the second to 300m depth, ending in zone 2. These scenarios are tested against a scenario of no VIS during the 10year period, to compare the results objectively. The different scenarios and their characteristics are summarized in Table 1.

5.2.3 Using a local 3D model, for the 2D model calibration

We mimic the wellfields and extraction zones by singular wells in our 2D model. Since we want to infiltrate a certain volume of water by using VIS, it means we have to come up with a pumping rate for the representing well, keeping in mind that the rate can depend on the volume to be infiltrated and the number of VIS wells installed, so that the combination defines the rate at which each well infiltrates. Infiltrating with a well in a 2D model allows for the water only to be spread upstream and downstream of the well-cell, along the 2D model length. The effect infiltration has on stopping/slowing the seawater intrusion, depends on possibility of creating coastward gradients. A volume of water A, infiltrated at a pumping rate B in a 2D model can thus only flow along the model length and creates a larger gradient in the direction of the coast, than if the same volume A would be infiltrated at pumping rate B in a 3D model, where water can spread along the model length ánd width (to all four sides of the cell). This means that by infiltrating at pumping rate B in a 2D model, the effect of VIS against sea water intrusion would be overestimated.
Therefore, we re-calculate the pumping rate for the 2D model, such that infiltration has the same effect as if the well was in a 3D situation, like in reality.

We emphasize that this process is not needed for the wells representing the wellfields and agricultural extraction zone, since these pumping rates are optimized by setting the criteria that the base scenario should fit the conditions set in Paragraph 5.2.2.

We start off by creating the base model. Then, the steps taken to select a best VIS well location and depth, and allow for the recalculation are the following. This is visualized in Figure 25.

First, a comparison of scenarios is conducted, running the model with an assumed pupming rate for the VIS wells which is lower than maximum recharge known from other VIS installations over Europe. The pumping rate is set to only 150m^3/h, while the maximum pumping rate possible known to the author is 500m^3/h. Next, a 'best' scenario conserning well location and depth is chosen under these conditions, satisfying the requirements of Paragraph 5.2.2 as well as possible.

Second, a local 3D model is created. The term 'local', refers to the fact that this 3D model is constructed using the characteristics at the VIS x-location of the 2D 'best' scenario found in the previous step. This means that ground surface gradient, water table gradient, conductivity and storativity characteristics are appointed to the model, similar to the location of the VIS well. This local 3D model simulates the water table elevation after a VIS infiltration period of 12 days. The 3D model has a size of 1500x1500m of surface area, and has no density data incorporated, so calculates only flow. The 3D model is used to optimize the distance between the wells, to infiltrate as much water as possible of the assumed available dam storage volume within 12 days, in the assumed area available for constructing wells. This means it is tried to infiltrate 4MCM of water in 12 days, over a width (y-direction) of 1500m.

Third, we try to mimic the resulting water table gradient along the 3D model length in a simplified 2D model, by optimizing the pumping rate appointed to the well. Pumping rate Q is optimized in such a way that the amount of water flowing to the upstream and downstream cell from the cell containing the well, is similar to the amount flowing up and downstream in the 3D model. This simplified 2D model also lacks density effects and is shorter compared to the original 2D model (1500m instead of 12520m).

To legitimate the use of a 3D model for calibrating the 2D model, the principle of creating a symmetrical flow pattern, with flow being perpendicular to the coastline, was tested on the Modflow model. This is explained in Appendix 2.

Finally, the optimized flow rate from the simplified 2D model, is used as input for the original 2D model and the final model results are obtained.

Table 1 – Characteristics of the different first step mitigation scenarios

Title	Distance along x , from beachpoint(m)	Max. depth of filter , from surface[1] (m)	Recharge[2] (m^3/h)
No VIS	-	-	0
VIS 5000-1	5000	210	150
VIS 5000-2	5000	300	150
VIS 3000-1	3000	225	150
VIS 3000-2	3000	300	150
VIS 1000-1	1000	230	150
VIS 1000-2	1000	300	150

[1] The depth is chosen as the depth of zone 1 or a max of 300metres, ending in zone two.
[2] The recharge stated is the daily volume appointed to the well. In all scenarios, the VIS-well only infiltrates for a period of 12 days every year, with $0m^3/d$ recharge for the other 353days.

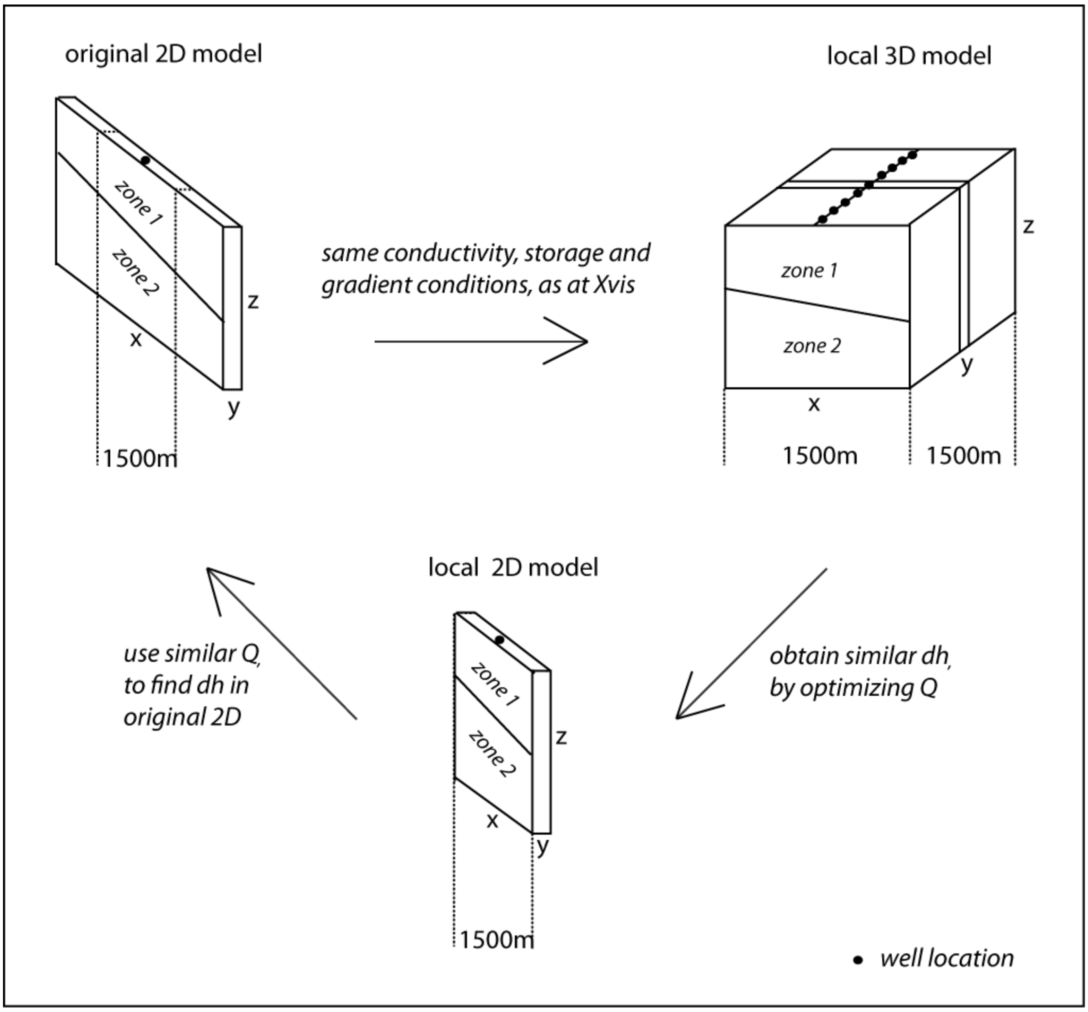

Figure 25 – the process of using the 3D model for 2D model calibration

6. Model setup

In the following paragraphs, the model setup is discussed as used for the base scenario. It was tried to meet the requirements set in Paragraph 5.2.2, by optimizing the discharge volumes, well locations and filter depths as well as number of total pumping wells, representing the pumping locations on the fan. Any changes to the values and/or schematizations listed below, needed to calculate the mitigation scenarios are summed in Paragraph 6.5

6.1 Grid and Lithology

The model was created as 2D, with a cell width of 10m. A grid size (length x height) is used of 80x80m. The model mimics the land section from the south of the Al Khawd fan to the beachpoint in the north, and a part of the sea to the north of this beachpoint. A beach slope has been created in this model for the purpose of visualization only. The beach slope appointed in this model is 0.044, like suggested by McLachlan et. al. (1998).

The total model length is 13440m, with 920m to the North of the beach point used for visualization of the beach slope and sea depth at distance. The model length representing the surface area thereby becomes 12520m. The elevation of the top layer is ranging from 70m in the South to 0m in the North, creating a gradient of 0.0056. This gradient is similar to the overall gradient on the Al Khawd Fan, calculated with the height from well SMA-51B and assuming the beachpoint as 0m elevation. The model was appointed 6 layers, resulting in a total model thickness of 480m.

Two zones of different hydraulic parameters are appointed, representing the cemented and uncemented region of the alluvial fan. The zonation is based on the generalized known lithology of some boreholes distributed over the actual fan, with cementation occurring practically at surface elevation in the southern part of the fan (the old alluvial terrace) and deepening towards the coast. The hydraulic parameters associated with the two lithological zones are described in the next paragraph.

6.2 Hydraulic parameters and well locations

For the sake of simplicity, the upper zone representing the uncemented part of the fan is named hereafter zone one. The lower zone representing the cemented fan part is hereafter called zone two.

6.2.1 Transmissivity and conductivity

The pumping test data from M.Macdonald&Partners (1985) are rejected, as previously explained in Paragraph 4.1.1. However, it is assumed that the order of magnitude of the difference in conductivity between zones one and two as suggested by Gibb(1976) is legitimate. His average values for conductivity are used. The appointed conductivities are stated in Table 2. An anisotropy is assumed, such that k_z is 0.01*k_x in both zones. This anisotropy is similar to the value used in a 2009, Northern Batinah wadi study by Al-Zidi (Al-Zidi, 2009). Genesis of both wadis can be expected to be similar.

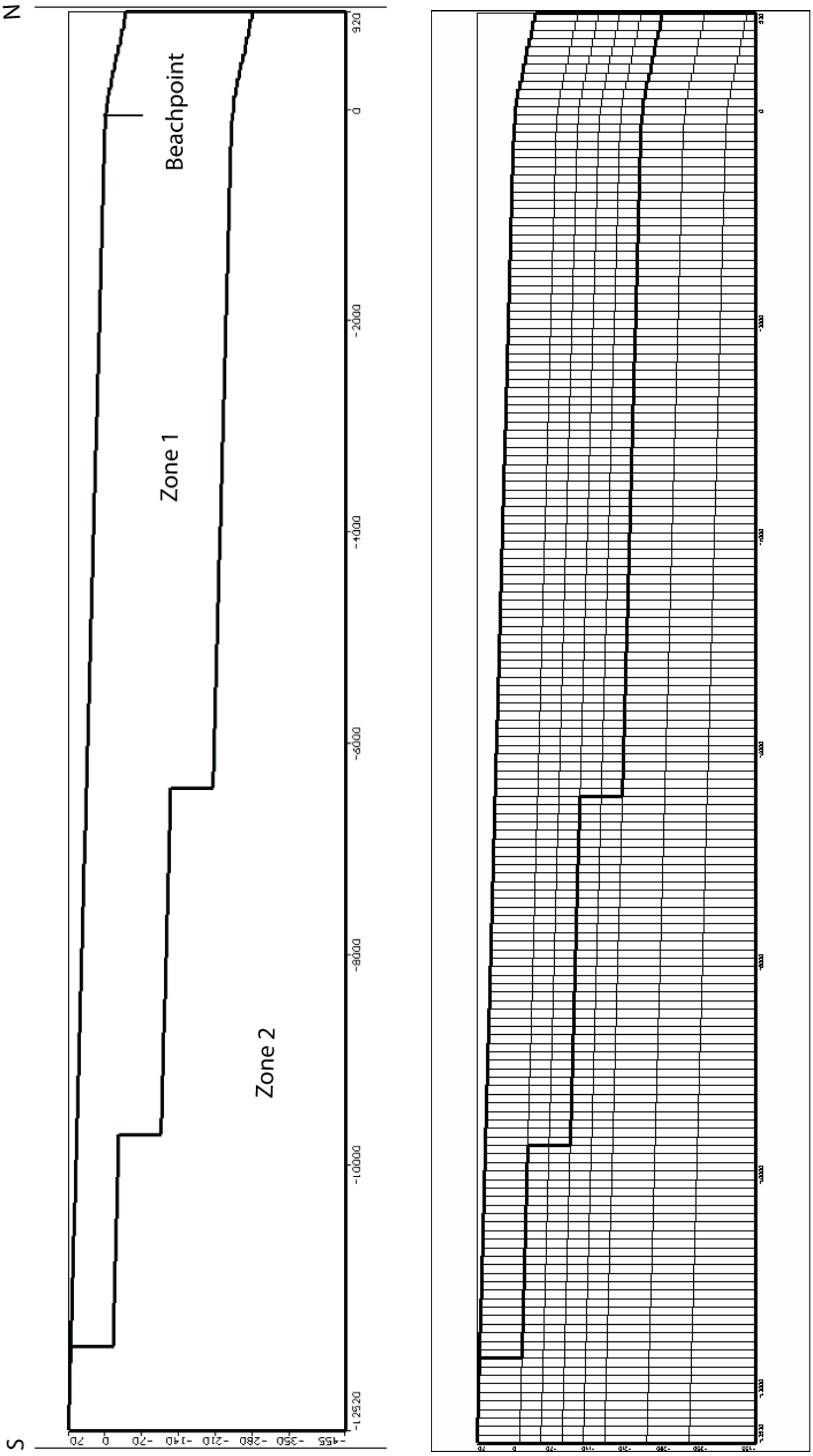

Figure 26 – a) Lithology zones and beach point, b) grid spacing of 80x80m. Grid appears non- square since the vertical distance is exaggerated by a factor 5

6.2.2 Aquifer type of each model layer

For each model layer in Visual Modflow the aquifer type has to be assigned. An unconfined aquifer type can only be appointed to the most upper model layer (SWS, year unknown). For the layers below, a confined type or dependent type can be assigned. For this model the first layer is appointed unconfined characteristics, for the other layers the third, dependent type is chosen. This means that Modflow will check the head value in these layer cells to determine if it is behaving as confined or not (SWS, year unknown). For storage calculation the specific yield Sy is used in case of unconfined aquifers and specific storativity Ss for confined aquifers.

6.2.3 Storage parameters and porosity

The storage parameters of the Al Khawd alluvial fan are not widely investigated and unknown to the author of this report. Al Zidi (2009) uses values of 0.0075 for the specific storage Ss, a specific yield Sy of 0.15 and a value of 0.15 for effective porosity n_e in a numerical study of wadi Al Hawasinah, more to the west on the Batinah coast. These values are originating from a drilling test on one borehole of a largely cemented and silty nature. His values are therefore here used for zone two only. According to Fitts (2002) is the specific yield Sy in coarse grained materials typically more than half of the total porosity n. The coarser the material, the more Sy would approach n. According to the Visual Modflow manual (Table 2), the specific yield Sy of sand and gravel is generally equal to the porosity. The table of Johnson (1967, presented in Fitts, 2002) shows that a specific yield value of 0.15 appears to be low for the uncemented alluvial sequences, since it is the same magnitude as from sandy clay, silt and lime- and siltstone sediments. Because the grainsize of the alluvial sediments is very diverse, it is assumed that the specific yield is less than the porosity. Combining the above, a value of Sy is 0.2 is assumed for zone 1 (Table 2). A value of specific storage Ss of 0.01 is appointed to zone one, also higher than that of zone two. The effective porosity n_e of zone 1 is also chosen to be larger than that of zone two, while total porosity n values are assigned equal.

Table 2 – overview of parameter values appointed to the two zones of different lithology.

Zone	k_x (m/d)	k_z (m/d)	Ss	Sy	n_e	n	α_L (m)	D* (m^2/s)
1	34	0.34	0.01	0.2	0.2	0.3	18.306	10^{-9}
2	5	0.05	0.0075	0.15	0.15	0.3	18.306	10^{-9}

6.2.4 Dispersion coefficients

For the calculation of saline front movement, values for hydrodynamic dispersion are appointed. These are also tabulated in Table 2. The hydrodynamic dispersion is dependent of the local dispersivity, flow velocity and diffusion by the following relation:

$$D_L = D^* + \alpha_L v$$

Where D_L is the hydrodynamic disperion, D_* the effective diffusion coefficient, α_L the longitudinal dispersivity and v the average linear velocity in the direction of $\alpha_.$

Due to pumping, flow will be the major solute transport system and due to the difference in order of magnitude, diffusion coefficient D* will be only of minor importance. A generic value of D* is 10^{-9} was appointed to both zones.

A precept in hydrology is that the longitudinal dispersivity can be estimated by taking 1/10 to 1/100 of the problem length. The problem length is determined by the distance between the well representing the potable water production well fields, which is between 4 and 4.5kilometers. This would mean that dispersivity would range between 40 and 450metres, which seems too large. The value of the longitudinal dispersivity is thus estimated with the method of Xu and Eckstein (1995) as described in Fetter (1999). Unlike other formulas based on stochastic analyses, this method is valid for problem lengths also larger than 3500m. The dispersivity is calculated by:

$$\alpha_m = 0.83\ (\log L_s)^{2.414}$$

where α_m is the apparent longitudinal dispersivity and L_s the problem length. The calculated dispersivity becomes 18.3m.
Transversal dispersivity α_t is assumed to be 0.1* α_L. Vertical dispersivity is assumed to be 0.01* α_L.

6.2.5 Wells

Two pumping wells are introduced in the model to represent two zones of intense pumping on the Al Khawd Fan. The first is a well representing the potable production well fields close to the dam. This well was assumed to fully penetrate zone 1 and is located at x = -4800m from the beachpoint (x=0). This well was appointed a pumping rate of Q= -290m^3/d. The second well represents the agricultural area, and is thus located at x=-1700m. The fact that the water table in the agricultural zone is shallower and the assumption that the agricultural wells are less professionally constructed, result in the conclusion that these wells are less penetrating and thus the representative well reaches a depth of only 100m below ground surface. This well pumps at a rate of Q= -100m^3/d. The pumping rates of both wells are resulting from optimizing the base scenario to fit the conditions of Paragraph 5.2.2.
From estimates of the 1980's by PDO and the ministry, it appears that the agricultural wells would extract more than the production wellfields combined. However, it was not possible to create a base scenario which would fit all the conditions if the agricultural pumping rate was set larger compared to the wellfield pumping rate.

6.3 Initial conditions

6.3.1 Initial heads

The initial heads appointed are following a gradient of 0.004, assuming a head of 50m a.m.s.l. in the south and 0m at the beach point. The cells to the east of the beachpoint are also appointed a 0m head, representing sea surface elevation.

6.3.2 Initial concentration

The cells representing the sea boundary are appointed an initial concentration of seawater salinity (35000 mg/l). The rest of the model domain was appointed an initial concentration of 0mg/l TDS. The model recalculates the salt-water heads of initial input and calibration to fresh water heads.

6.4 Boundary conditions

6.4.1 Constant head boundary

For simplicity, the constant head boundary at the southern end of the model is from here onward called the upper boundary; the northern end boundary at the beachpoint is called the lower boundary.

The relative heads of the upper and lower boundary determine the average groundwater gradient and thus influence the flow velocity. The lower boundary head is established at 0m above sea level. Without pumping, the natural gradient in the Batinah region is from the southern mountains towards the sea in the north. This means that when the gradient is locally reversed as a consequence of pumping, a lower upper boundary head causes a smaller gradient and causes the saline water to intrude easier than in a higher upper head case.

The upper boundary was appointed a head value of 50m. The lower boundary was appointed a head value of 0m at beachpoint and to the cells 'under the sea'. Depending on the head value compared to the nearby water table, these two boundaries will act as a source or sink.

Tidal effects are not included in this model exercise. The influence of these effects on the water table usually reaches up to about 100m inland, beyond which the water table remains stable (Li et al, 1999 in Al Maktoomi, 2007).

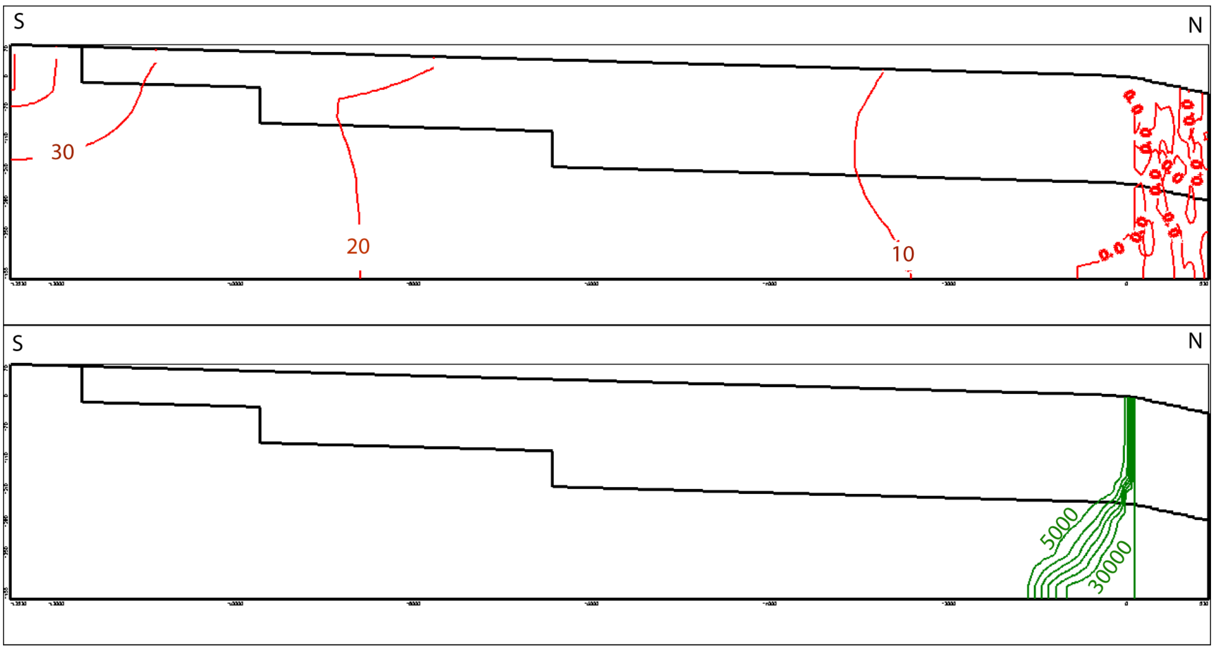

Figure 27 – a) head distribution and b) concentration distribution for 20.000 days without pumping

6.4.2 Constant concentration boundary

A constant concentration boundary was appointed to the north of the beachpoint, representing the sea. The concentration appointed was 35000 mg/l. Figure 27 shows that, without pumping, the model does distribute the salinity in a wedge shape, as would be expected due to density differences of the water. The figure shows the result at t = 25000 days of unsteady state flow.

6.4.3 Recharge and evapotranspiration

The in this Pragraph reported recharge values are only valid for the base scenario of the model. The recharge values used in the mitigation scenario are discussed in Paragraph 6.5. The evapotranspiration and recharge conditions are in Modflow appointed to the upper model layer. The cells of this layer can 'drain', and be characterized by a head lower than the surface elevation of the cells. The changes in head caused by evapotranspiration and recharge are simply added to or subtracted from the head value.

Values for evapotranspiration are spatially different and appointed in three separate zones, depending on the assumed land surface cover (Table 3). It is assumed that the coastal strip used for agriculture reaches 2km inland, like on the Al Khawd fan. This means that cells upstream of this 2km boundary are appointed a lower evapotranspiration and the cells in the agricultural strip a higher value to account for transpiration of the date palm trees. The extinction depth for this zone is deeper than in the non-agricultural zone. To the sea-side of the beachpoint, an evaporation value was appointed of 0mm/yr.

Table 3 – Evaporation boundary condition distribution.

Evaporation zone	1	2	3
Distance x-direction	-12520m to -2000m	-2000m to 0m	0m to 920m
characteristics	Upstream of agricultural area	Coastal agricultural strip	Sea surface
Evaporation E (mm/yr)	1800	2400	0
Extinction depth (m from surface)	3	10	0

1 = upstream from the agricultural region, 2 = agricultural region, a higher evapotranspiration rate and deeper extinction depth, due to the date plantations, 3 = seaside, no significant losses to evaporation.

The surface recharge was also spatially distributed along the model domain, but was appointed in four recharge zones (

Table 4). This distribution is based on the assumed dam location at approximately 4.5km inland. Upstream of the dam, the appointed recharge is based on the annual average recharge from local rainfall only (MWR, 1998). Downstream of the dam a division had been made between the non-agricultural zone and the agricultural strip, to account for irrigation. To the sea-side of the beachpoint, the appointed recharge was set to zero.

Table 4 – Recharge boundary condition distribution

Recharge zone	1	2	3	4
Distance x-dir.	-12520m to -4500m	-4500m to -2000m	-2000m to 0m	0m to 920m
Characteristics	Upstream of dam	Downstream of dam	Downstream of dam, agriculture	Sea surface
Recharge R (mm/yr)	80	500	2500	0

The recharge caused by the dam is calculated with the average yearly volume of water behind Al Khawd dam, from the period 1985 to 2006. This amounts to 4.8709 MCM. The area downstream of the dam is estimated as 15km^2, which means that per meter domain length the recharge is 325mm/yr. Add to this the 80mm/yr local recharge and the recharge would amount to more than 405mm/yr. Because the 2007 cyclone Gonu and 2010 cyclone Phet values are not used to calculate the yearly average, it is decided to enlarge the recharge to 500mm/yr. The recharge from irrigation is calculated by assuming that a hectare of date palm trees in Oman needs 25000m^3 of water yearly, like in Jordan (FAO). The agricultural area of 10m model width and 2km length recalculates to 2500mm/yr recharge, see the calculation below.

$$10m * 2000m = 20000m^2 = 2ha$$

$$25000m^3 / yr * 2 = 50000m^3 / yr$$

$$\frac{50000m^3 / yr}{20000m^2} = 2.5m / yr = 2500mm / yr$$

As can be seen in Table 5, from 0 to 5000days, the recharge on both the area downstream of the dam and the agricultural area is set to 80mm/yr, assuming no large scale irrigation taking place. From 5000d to 11000d, the agricultural area is appointed a recharge value of 2500mm/yr, as calculated above. During this period, the area downstream of the dam, but upstream ofthe agricultural area is still appointed 80mm/yr; the natural recharge. From 11000d to 20000d then, the agricultural zone is appointed 3000mm/yr and the between dam and agriculture area 500mm/yr. This increase by about 400mm/yr in recharge is calculated assuming the 4.8MCM average yearly stored behind the dam is available for infiltration on the 15km^2 downstream, see the calculation below. The increase from 325 to 420mm/yr is to account for the fact that the average does not include cyclonic events, like occured in 2007 and 2010, so the figure is rounded off upwards.

$$\frac{4870900m^3 / yr}{15000000m^2} = 0.325m / yr = 325mm / yr$$

6.5 Changes to the above, necessary to run the mitigation scenarios

The water needed for infiltration with a VIS system, is originating from the total assumed amount yearly stored behind the dam. The amount of water appointed as surface recharge boundary condition is also stemming from this resource. This means that when using VIS in the model, the amount of surface recharge appointed downstream of the dam should be reduced. The final model is corrected for this issue, and the surface recharge boundary condition being appointed 100mm/yr, as 4MCM of the total volume available is infiltrated by VIS. For calculation of the scenarios in between the surface recharge from dam-flow is reduced to 250mm/yr.

6.6 Stress periods

To account for different processes and their changes in magnitude, the model uses a total of five stress periods. For ease of use, the changes between stress periods are summed in Table 5 .

Table 5 – Parameter values for the different stress periods. The changing parameters are underlined.

Stress period	Zone 1	Zone 2	Wells	BC
1 t 0-50 days	$K_x = K_y = 34$ $K_z = 0.34$ $S_s = 0.01$ $S_y = 0.2$ $N_e = 0.2$ $N_t = 0.3$ $\alpha_L = 18.306$	$K_x = K_y = 34$ $K_z = 0.34$ $S_s = 0.01$ $S_y = 0.2$ $N_e = 0.2$ $N_t = 0.3$ $\alpha_L = 18.306$	Q_a = -100 m3/d Q_w = 0 m^3/d	R_u = 80 mm/yr R_d = 80 mm/yr R_i = 80 mm/yr R_s = 0 mm/yr E_u = 1800 mm/yr E_d = 1800 mm/yr E_i = 1800 mm/yr E_s = 0 mm/yr
2 t 50-5000 days	As above	As above	Q_a = -100 m3/d Q_w = -290 m^3/d	R_u = 80 mm/yr R_d = 80 mm/yr R_i = 80 mm/yr R_s = 0 mm/yr E_u = 1800 mm/yr E_d = 1800 mm/yr E_i = 1800 mm/yr E_s = 0 mm/yr
3 t 5000-11000 d			Q_a = -100 m3/d Q_w = -290 m^3/d	R_u = 80 mm/yr R_d = 80 mm/yr R_i = 2500 mm/yr R_s = 0 mm/yr E_u = 1800 mm/yr E_d = 1800 mm/yr E_i = 2400 mm/yr E_s = 0 mm/yr
4 t 11000-20000			Q_a = -100 m3/d Q_w = -290 m^3/d	R_u = 80 mm/yr R_d = 500 mm/yr R_i = 3000 mm/yr R_s = 0 mm/yr E_u = 1800 mm/yr E_d = 1800 mm/yr E_i = 2400 mm/yr E_s = 0 mm/yr
5 t 20000-30000			Q_a = -100 m3/d Q_w = -290 m^3/d	R_u = 80 mm/yr R_d = 250 mm/yr R_i = 2750 mm/yr R_s = 0 mm/yr E_u = 1800 mm/yr E_d = 1800 mm/yr E_i = 2400 mm/yr E_s = 0 mm/yr

Explanation of subscripts used in columns 'Wells' and 'Boundary Conditions':
A = agricultural well zone, W = well field zone, U = upstream of dam, D = downstream of dam, I = irrigation area/agricultural area, S = sea

7. Selecting the best mitigation scenario

7.1 Base Scenario

For the base scenario, all hydraulic parameters are used as described in chapter 6. Figure 28 shows the results after 20.000days. It can be seen that a salt water intrusion wedge has developed, which fulfils the requirements set in Paragraph 5.2.2 as well as possible. It can be observed that a fresh water zone has developed, underlying the intrusion tip. It is also clear that there is no fresh water to be found close to the coastline, not even at shallow depth.

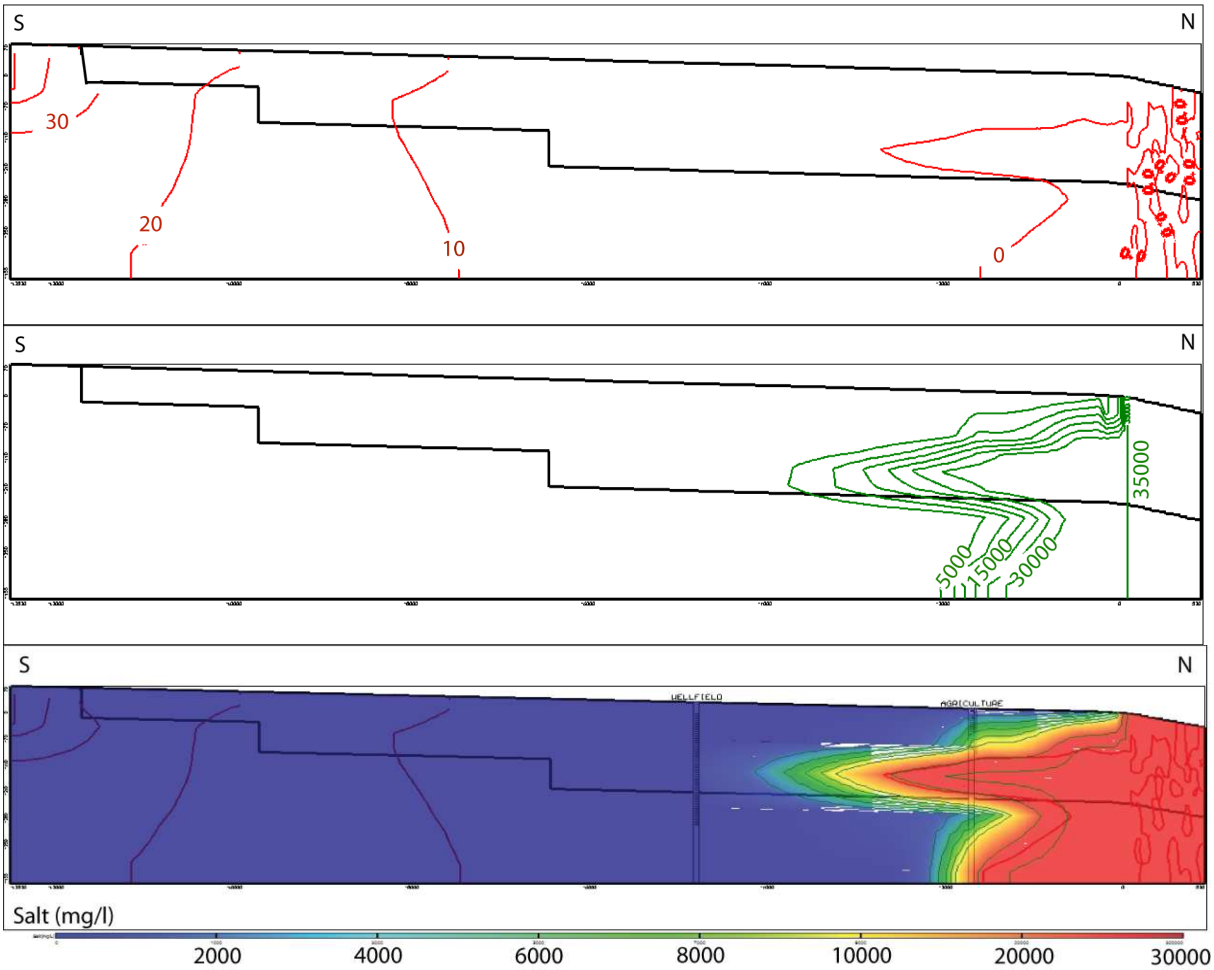

Figure 28 – a) head distribution and b) concentration distribution contour lines and c) concentration distribution color shading of the results from base scenario at t = 20.000 days

7.2 Mitigation Scenarios: optimizing x_{well} and z_{well}

For the sake of simplicity, the characteristics of the different mitigation scenarios are again summed in Table 6. The results after 20.000days+10years of VIS running are presented in Figure 29.

To choose the best scenario, we used the achievements listed in Paragraph 5.2.1. All mitigation scenarios fulfil the first achievement by reducing the inland movement of the wedge in the 10 year mitigation period, compared to a situation where no further measures are taken. However, the VIS 1000-1 and 1000-2 scenarios do not prevent the centre of the saline plume with the highest salinities from reaching the production well field, so these are considered not suitable. The VIS 5000-1 and 5000-2 scenarios, show the saline plume to be very similar to the position at t=20.000, indicating that the plume is somewhat stabalized, fulfulling the second achievement. However, the intrusion length is not effectively reduced, compared to the situation at t=20.000days. The VIS 3000-1 and 3000-2 scenarios show that the original plume is reduced in size, although a small plume is still travelling towards the wellfield. However, it appears that after pumping 20 years, the travelling plume is completely removed by the wellfields and that the original plume towards the coast has stabalized. The VIS 1000-1 and 1000-2 scenarios are therefore considered to be the most effective. Because the depth of the VIS does not appear to have a big influence, the VIS 3000-1 scenario is chosen as a start point for the local 3D model, to calculate a representative Q.

Table 6 - Characteristics of the different first step mitigation scenarios

Title	Distance along x , from beachpoint(m)	Max. depth of filter , from surface[1] (m)	Recharge[2] (m^3/h)
No VIS	-	-	0
VIS 5000-1	5000	210	150
VIS 5000-2	5000	300	150
VIS 3000-1	3000	225	150
VIS 3000-2	3000	300	150
VIS 1000-1	1000	230	150
VIS 1000-2	1000	300	150

[1] The depth is chosen as the depth of zone 1 or a max of 300metres, reaching into zone two.
[2] The recharge stated is the daily volume appointed to the well. In all scenarios, the VIS-well only infiltrates for a period of 12 days every year, with $0m^3/d$ recharge for the other 353days.

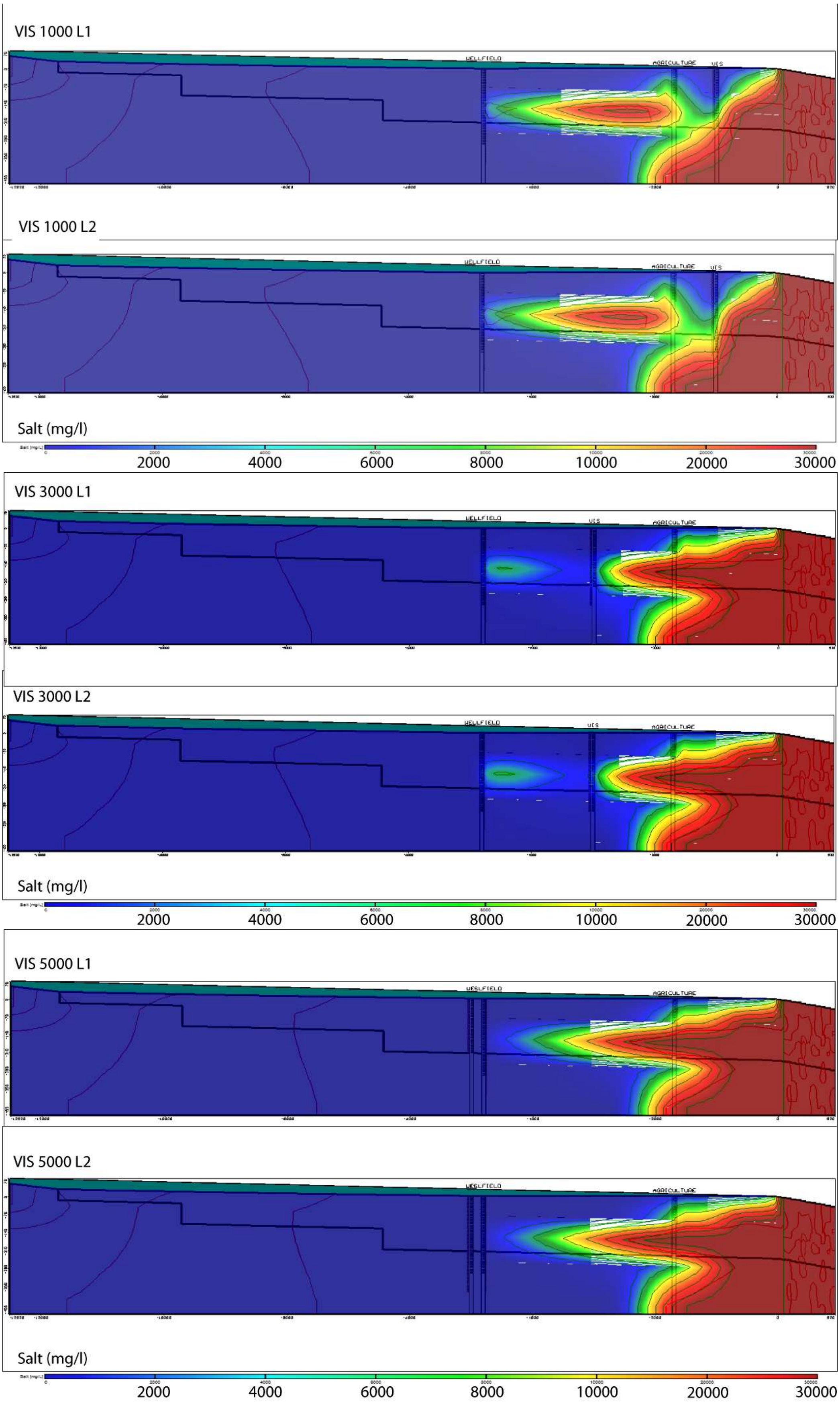

Figure 29 – Results of the first step mitigation scenarios, for t = 23650 (10 years mitigation, Q150m^3/d).

8. Results and Analysis: effectiveness of VIS against seawater intrustion

8.1 Testing for an advantage of VIS in combating seawater intrusion: infiltrating the yearly average available volume

The total volume of water behind the dam is averaged to 4.87 MCM yearly (1985-2006). In the model, the possibility of infiltrating 4MCM of this volume within 12 days with the VIS system was investigated. Assuming a flow rate of 500m^3/h for each well, 28 wells are needed to achieve the desired result.

The variable most influencing the water table elevation when infiltrating with the constants above, is the inter-well distance. From the investigation to create similar flow to a 2D model, within the 3D model, described in appendix 2, it became clear that even with a 10m well distance the water table would be below ground surface, after 12 days pumping at 500m^3/h. The thickness of the unsaturated zone thus does not imply a limit to infiltration. Out of spatial consideration, it was decided to assume a maximum width of 1500m for the well zone, keeping well within limits of the dam width. This set the maximum inter-well distance to 53.5m. This configuration was tested in the 3D local model (Figure 30). The head distribution after 12 days of pumping is shown in Figure 31. The maximum water table elevation is 5m.

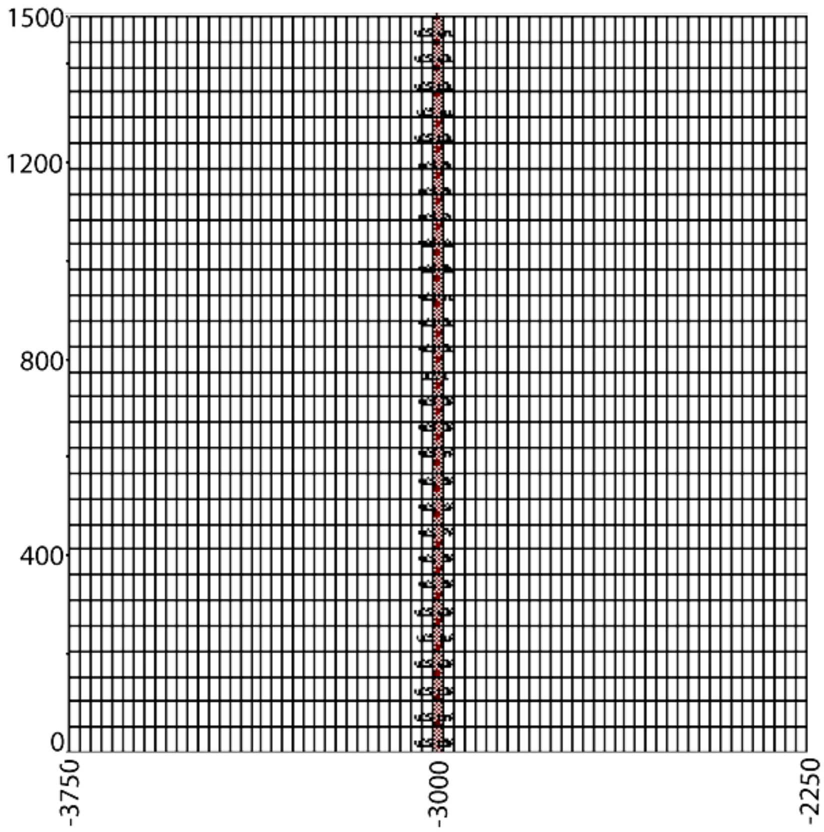

Figure 30 – Configuration of the well locations (Top view).

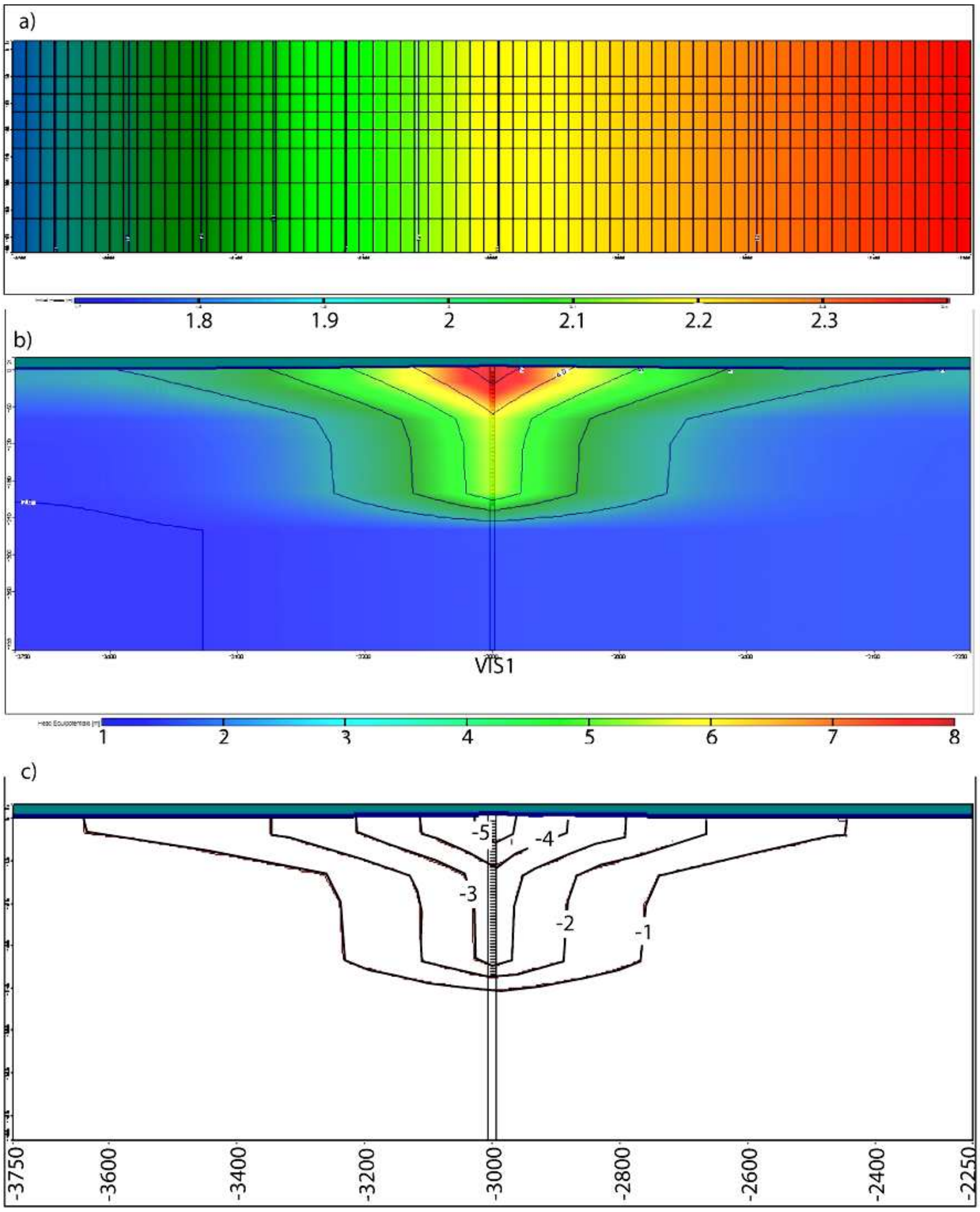

Figure 31 – a) initial heads (profile through 3D model), b) heads after 12days of pumping at 500m³/h for all wells, c) drawdown after 12days of pumping at 500m³/h for all wells.

The water table elevation of 5m is reconstructed in the local 2D model. The pump rate needed to achieve a 5m elevation is 2250m³/d. This pump rate was then used as input for the original 2D model in the best scenario. The VIS well pumps 12 days every year. The results after 10 years are depicted in Figure 32. A small plume of saline water is, in time, still reaching the well field. Comparing the results after 20 years of pumping at 2250m³/d with the no mitigation scenario, it is apparent that the salinity upstream of the infiltration well is reduced. However, the light-blue color in Figure 32 indicates that the water pumped after 20 years of mitigation is still above 4000mg/l, which is too high for the Omani drinking water standards. Water in Oman is resolved to be suitable for potable use if the TDS is less than 1500mg/l (Ashworth, 2006[2]). It was not tested if this stretched part of the plume seizes to exist any time after 20 years. The figure also shows that the concentration distribution in the area downstream of the VIS well does not

change significantly in shape, between 10 and 20 years of using VIS and thus that the plume downstream of the infiltration well appears to be stable.
Typically, the accepted Global Mass Balance discrepancy provides that the model is numerically 'valid' when the value is as minimal as possible. The discrepancy of the model is less than 1%.

From the model, it was concluded that by using VIS, the growth of salinity intrusion wedge part with salinities higher than 4000mg/l, was halted, compared to the location at 20.000 model days (modelled present situation). It was also apparent that for these salinities, the intrusion length will decrease, compared to the situation where VIS would not be implemented in the modelled 10 and 20 year mitigation periods and that no water of salinity higher than 4000mg/l is intruding further than the chosen VIS location, about 1.5km downstream of the Al Khawd dam. However, the below 4000mg/l stretched part of the plume, reaches the production wellfield and creates the nescessity of continued desalination of the extracted water, for a duration longer than the modeled 20 years of mitigation. The concentration distribution downstream of the VIS location, appears to be stable and does not significantly increase in salinity over the 20 year mitigation period modelled.

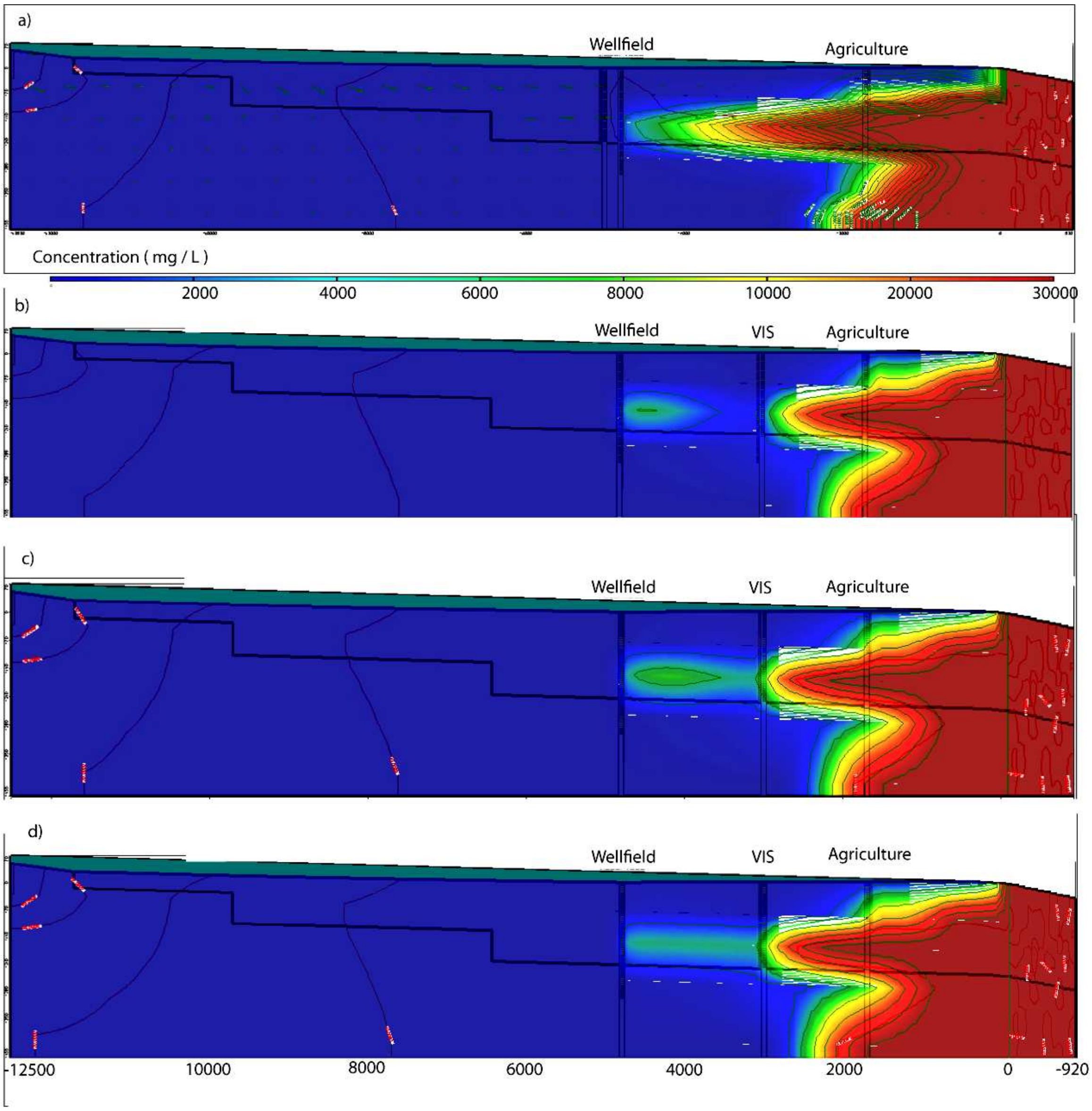

Figure 32 – a) Model results for 2D original model after 10years of no mitigation, b) results after 10 years pumping at 3600m^3/d (scenario selection), c) results after <u>10</u> years pumping at 2250m^3/d, d) results after <u>20</u> years pumping at 2250m^3/d.

8.2 Testing for an advantage of VIS in combating seawater intrusion: infiltrating the maximum dam capacity volume

So far, the volume assumed available for the model infiltration was 4MCM, calculated as the rounded off, 1986-2006 average. In this paragraph however, the volume was substantially increased to full dam capacity; a total of 12,5 MCM.

Again assuming a maximum infiltration capacity of 500m^3/h per well, more wells than the 28 needed in the average dam volume infiltration (Paragraph 7.3) are necessary. A total of 87 wells are used to infiltrate the volume. The 87 wells are divided in three parallel lines, with 29 wells on each, along a total width of 2000m downstream of the dam location, remaining within the dam width. The inter-well distance in x and y direction is set to 69m, and first modelled in the 3D model (Figure 33). As the maximum width as well as the length of this model is only 1500m each, only 63 wells are actually put in the model.

The results after 12 days of pumping are shown in figure Figure 34. The largest water table elevation is encountered at the middle row of wells, and arises to more than 10m, with the heads remaining well below the 5m depth from ground surface limitation.
To attain this water table elevation in the local 2D model, a recharge of 1600m^3/d was needed per well. As can be seen, the configuration of wells along three separate rows is also maintained in the 2D models. The distance between the wells is again set as 69m.

The results of using the 1600m^3/d pumping volume in the large, original 2D model are shown in Figure 35. The figure shows that the infiltration of the full dam capacity is more efficient than infiltration of 4MCM, calculated in the previous Paragraph. The difference is in fact smaller than expected, since the plotted results are after ten years of infiltrating the triple volume, each year. The figure shows that after infiltrating at the maximum dam capacity, two separate salinity plumes have developed, so that it can be expected that the >2000mg/l would not reach the wellfield any more when the ‘free’ plume has been pumped out. However, this scenario is unrealistic, as it is unrealistic to expect full dam capacity for 10 years in a row.
Most importantly, this model run shows that it is possible to infiltrate the maximum dam capacity at a relatively small area.

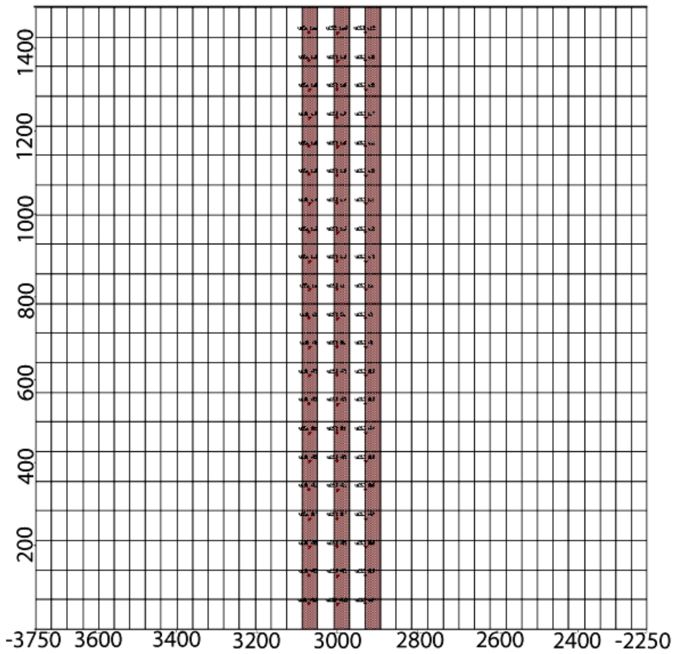

Figure 33 – Configuration of well locations (top view)

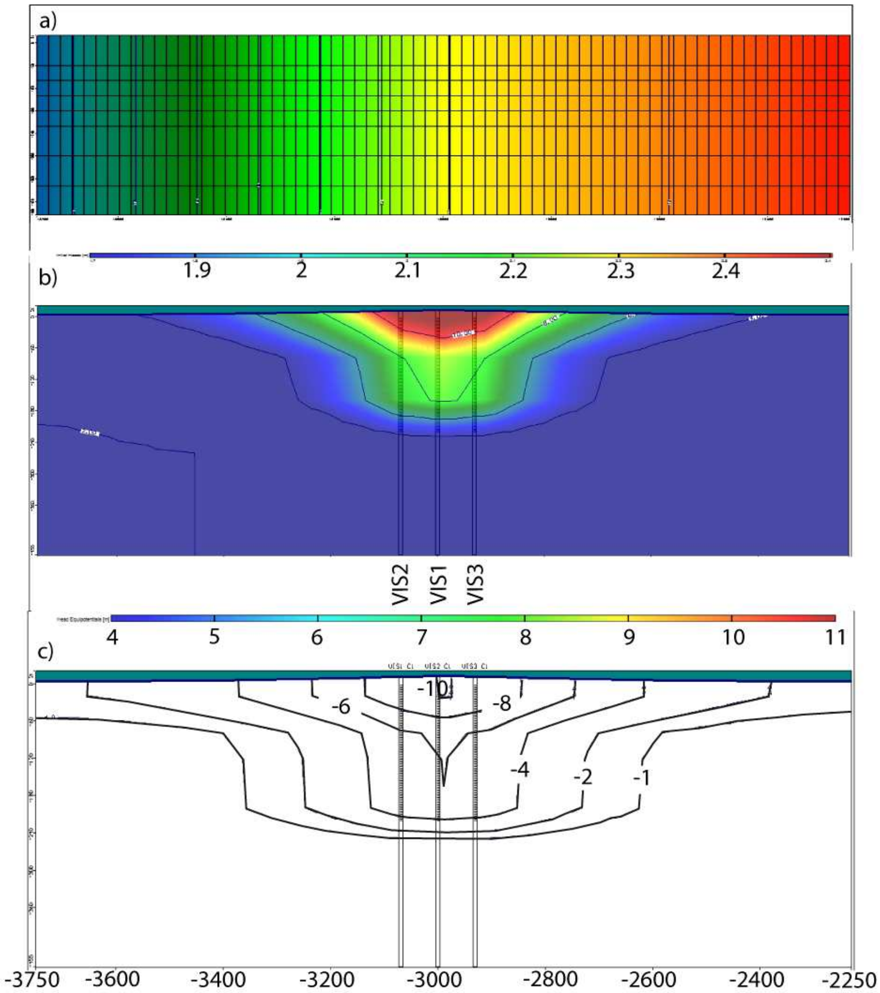

Figure 34 – a) initial heads (profile through 3D model), b) heads after 12days of pumping at 500m³/h for all wells, c) drawdown after 12days of pumping at 500m³/h for all wells.

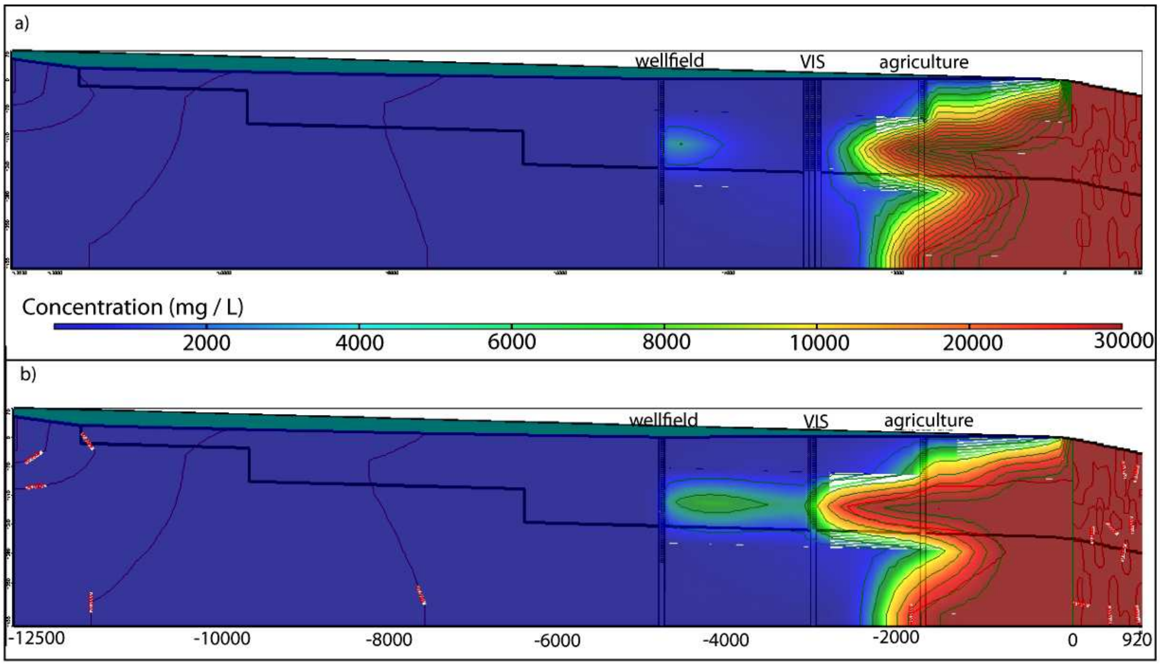

Figure 35 – a) model results after 10 years pumping with wells at 1600m³/d each, assuming total volume for infiltration 12.5 MCM, b) results after 10 years pumping with wells at 2250m³/d, assuming total volume for infiltration 4MCM (results previous paragraph).

9 Discussion

The importance of the sea water intrusion wedge shape

The achievements set in Paragraph 5.2.1 are discerning on the size of the intrusion wedge. However, no strict requirements were set. The question thus arises if 'succes' is achieved only if the total plume is pushed back or if a reduction of the highest salinity zones alone would be sufficient too. If a residue salinity zone arises, which can be removed by extraction at the production wellfields , like with the VIS-3000 3600m^3/h pumping rate scenario (Paragraph 7.2), the use of VIS could be considered succesful: After a certain period of extracting water of higher salinity, potable water will be available continuously if the aquifer system is kept in balance. Water is considered potable if TDS is <1500mg/l, but for farm animals, the tolerance for higher salinity water is higher. FAO reports that poultry is tolerant of TDS <2800mg/l, pigs for TDS <4300mg/l, and horses for TDS <6400mg/l water (FAO, 1986 in Ashworth, 2006).

However, if the residue salinity zone does not separate from the main intrusion wedge, the extracted water at the production wellfield will be of salinity higher than allowed for potable use for years to come. In this case the situation the adverse effect of pumping saline water can certainly prove the implementation of this specific VIS configuration less desirable.

The VIS-3000 scenario was selected as best, before further optimizing. However, a VIS location between 3000 and 5000m from the coast might give better results, as the earlier rejected VIS-5000 scenario (with well location 5000m from the coastline, close to the recharge dam) might in retrospect prove better, as it stabalized the plume after a 10year mitigation period when modeling for the selection of the best well location. It might be that in such a configuration no residue salinity zone develops. However, this scenario was only modeled with the assumed and as appeared later, too high pumping rate. It should yet be investigated if the plume would also be stabilized when pumping at a lower, more realistic pumping rate. More modeling is needed to investigate this possibility.

Mixing of overlying salt and underlying fresh water zones

The stability of a denser liquid plume depends on horizontal flow velocity, contaminant leakage rate and density difference, hydrodynamic dispersion and transverse dispersivity (Quoted from different authors by Al Maktoomi, 2007)
Schotting (1998) shows that when an originally horizontal fresh-salt interface coinciding with a discontinuity in permeability, mixing occurs by fingered flow of saline water into the fresh water zone (Figure 36). Al Maktoomi (2007) shows the same principle, though with an aquitard separating the two layers with saline and fresh water respectively.
There exists a minimum critical wave length in the fingering pattern. Fingers with width smaller than (half) the critical wavelength are dissipated by diffusion and/or dispersion and decay in time (Schotting, 1998). As the vertical and horizontal velocities increase, dispersion becomes larger, and the instability occurs later.
In the current model, no signs of fingered flow are visible. Schotting (1998) shows that the grid size close to the interface influences the development of fingers in a numerical model; with a larger grid size, fingers develop later and are less prominent (Figure 37). The grid size of the current model is probably too large for fingers to develop.

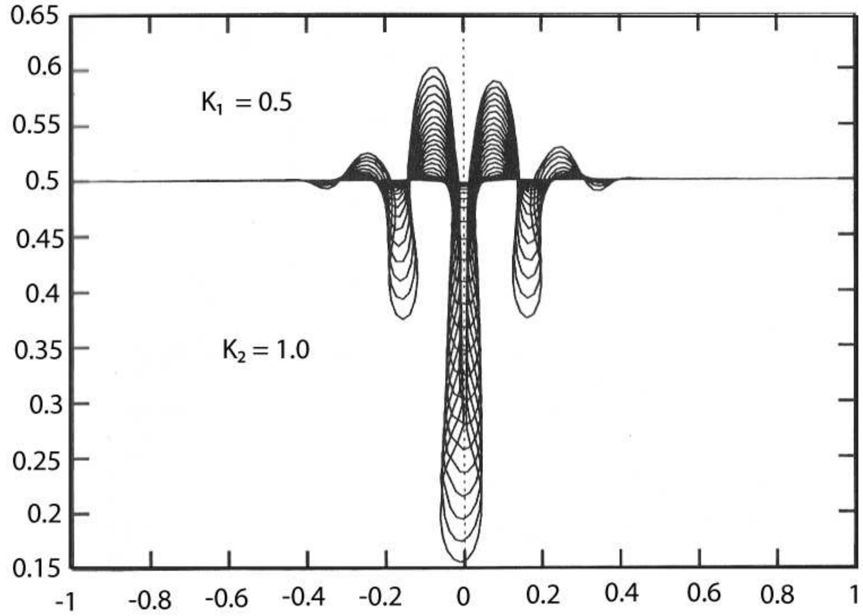

Figure 36 – Development of fingers at an initially horizontal discontinuity in permeability and salinity. The upper layer is initially saline; the lower layer is initially fresh. Figure shows the time evolution of the interface (from Schotting, 1998).

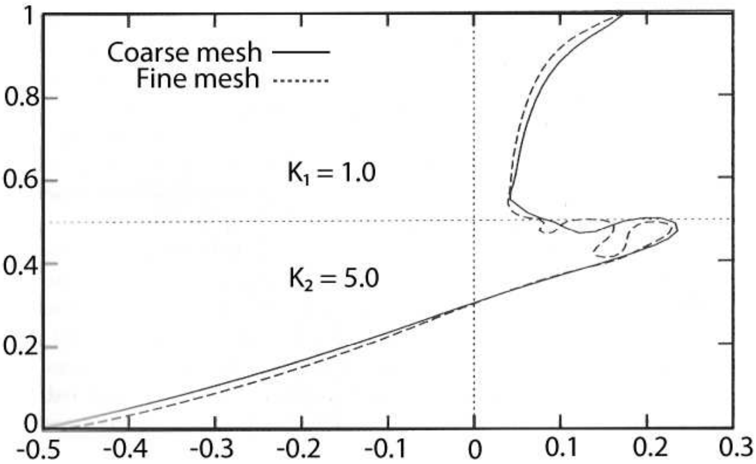

Figure 37 – Fingers develop less easily in the numerical model if the grid discretisation is coarser

Vertical limitation of the upper constant head boundary

The upper boundary condition (UB) is set with only two constant head cells, overlying cells of a no flow boundary. The reason for this set-up is because in reality the wadi channel entering the Fan is believed to be of limited depth, and probably even smaller than the grid size. Two constant head cells thus seems to be too much to represent the wadi channel and shallow subsurface flow. However, when only using 1 cell as a constant head boundary, the flow velocity downstream in the aquifer becomes too small, and the intrusion is occurring much faster than has happened in reality. The flow occurring from the second cell thus solves an apparent deficit on the water balance. In literature this deficiency is suggested to be the result of flow from fracture zones in the base rock. Since there is no indication as to where these fracture zones occur, and as the distance to the zone of interest of the model is large, the current representation with two constant head cells is considered reasonable.

In literature, the occurrence of water of high age at depth in the fan is blamed on water flow from these fracture systems, as these allow for long travel times. In the current model, the set-up ensures that the flow velocity in the bottom-south part of the profile is very small at least, to almost absent locally. The flow pattern within the aquifer itself, might thus be contributing to the age of old water locally occurring in the aquifer.

Pumping rates appointed to the well field and agricultural well

The pumping rates of the Well field and Agricultural wells in the original 2D model are obtained by trial and error. The requirements summed in paragraph 5.2.2 needed to be fulfilled within 20.000 days, as day 1 was set as steady state. This means that within 55years, the current saltwater wedge should be modelled. Literature shows that, at least at some point in history, the volume extracted at the agricultural zone would have been larger than the volume extracted at the well field(s). However, in the model, the pump rate of the well field is considerably larger. It has appeared to be not possible, to have a higher agricultural than well field extraction and still get the saline wedge to intrude further than the agricultural well, almost to the dam location. No (recent) pump rates for the agricultural zone are known to the author. The reduction of the depth of the agricultural well might proof fruitful, although several tests showed that, despite becoming of almost similar size of the well field flow rate, the agricultural well flow rate remained smaller.

Creating a beach slope in the model

In the current model, the beach slope was purely created for visual considerations, illustrating the limited depth of the sea-aquifer interface at distance from the beach point (paragraph 6.1). From Figure 38 it becomes clear that only at a distance of about 25km from the Seeb coast, the seabed is about 400m depth. This means that the aquifer continues to a total modeled fan depth for around 25km from the coast.

Al Maktoomi (2007) shows that a beach slope ranging between 0 and 45 degrees for an Omani coastal aquifer model, results in changes in the intrusion length significantly smaller that the currently used grid size. Thus, the (representative) gradient of the beach slope does not significantly influence the presented results. The model might thus be produced without a beach slope, and a constant head cell at the beach point, with hydrostatic pressures below, similar to the now appointed boundary conditions at the beach point.

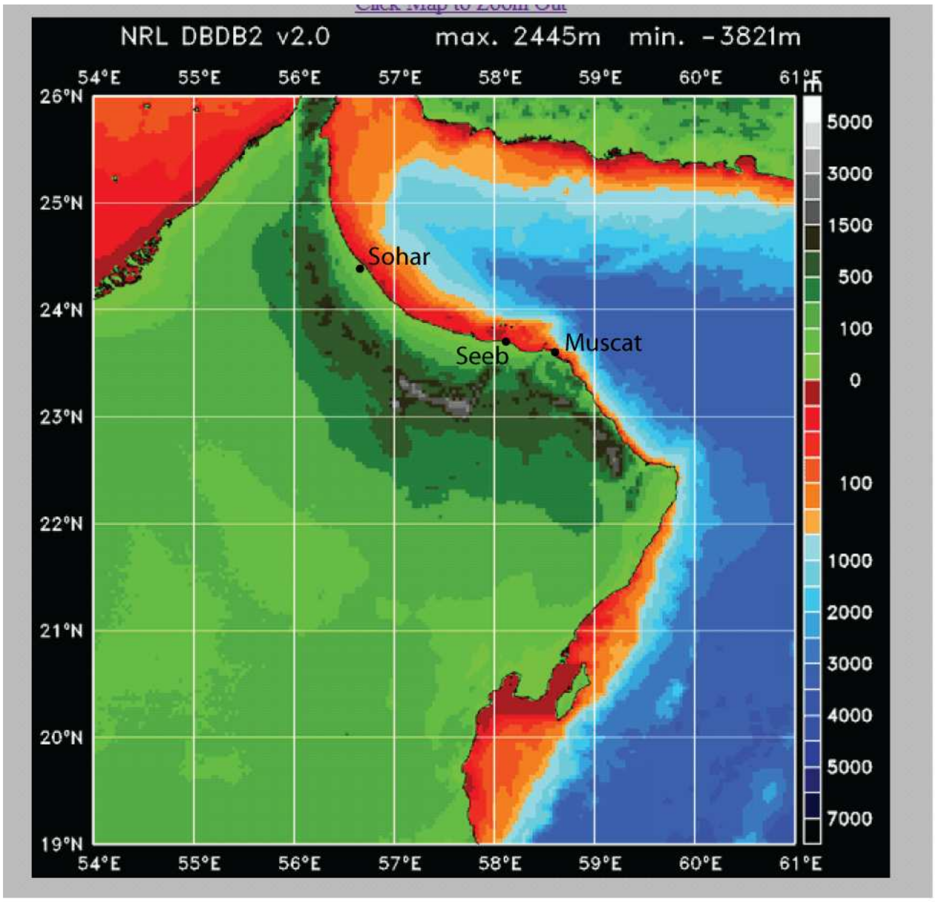

Figure 38 – Bathymetry along the northern Omani coast. Map from NAVOCEANO, the US Naval Oceanographic Office

Selecting a 'best' scenario

As explained in Paragraph 5.2.2, a best scenario was selected based on three requirements. These requirements only concern concentrations within the aquifer and do not take into account the ease of construction of the VIS. The now selected best scenario, thus locates the wells 3000m from the coastline, which is considerably downstream from the assumed dam location at little more than 4.5km inland. This would mean that the water would have to be transported this distance, however, it is not necessary to do this by piping.

The design of the infiltration wells allows for a local collector point, so that the water can flow from the dam to the infiltration site by itself. Of course, in 3D, the wells are spaced at a distance and, especially for minor floods, are thus not all located within an active wadi bed. It should be possible to design the infiltration structure in such a way, that several infiltration wells are connected to one collector point, distributing the runoff over the several wells, up to maximum capacity.

Other use of VIS in Oman

This thesis investigated the use of VIS in Oman for mitigation of the seawater intrusion. However, VIS systems can certainly be used on a small scale too, by locally infiltrating excess water. VIS systems can for example be usefull near roads which are prone to flooding, or near areas where standing bodies of water remain on low lying, hardened areas.

10 Conclusions and recommendations

Similar challenges concerning water management exist in The Netherlands and in Oman. Both countries need to effectively reduce flood risks, and both countries are under subject of seawater intrusion and try to manage their aquifers to attain a sustainable groundwater system. Whereas in Holland, the high precipitation allows for reasonable natural recharge, precipitation in Oman is low and storm runoff is lost to the sea as a consequence of fast runoff. Attenuating the storm runoff by recharge- and flood protection dams, and infiltrating the stored storm runoff to recharge the aquifer is of high importance to reduce seawater intrusion along the Al Batinah, and other coasts. The use of a Vertical Infiltration System (VIS) was modelled to see if this could contribute to the infiltration and if the seawater intrusion could be reduced.

We set out to analyze if deep infiltration of storm runoff by means of a VIS-technique, is efficient for infiltration with respect to:

1) Combating seawater intrusion by infiltration at depth
2) The speed of infiltration of a certain volume

The tested well location 3000m from the coastline and well length to the depth of the clean gravel zone appeared to give the best results, reducing the size of the intrusion wedge. The residue salinity upstream of the VIS installation would be pumped out by the production well field, allowing potable water to be extracted at the wellfield within ten years after start of pumping the residue saline water of TDS up to 7000mg/l.

As we optimized this VIS-3000 scenario by infiltrating with the average yearly volume stored behind the dam, it appeared that the pumping rate was in reality smaller than used for selection of the well location. This resulted in a reduction of the high salinity zone of the intrusion wedge, but did not prevent a tail of >4000mg/l TDS to continuously reach the wellfield, even after a 20year mitigation period. It was not tested if this tail would diffuse after an even longer continuation of VIS. The concentration distribution downstream of the VIS well location appeares to be stable, as it does not increase or decrease in salinity from 10 to 20 years of using VIS.
When comparing the final VIS-3000 scenario intrusion length and intrusion wedge shape, with a no-mitigation scenario, it shows clearly that the VIS influences the wedge shape and pushes back the highest salinity zones to downstream of the VIS location. With respect to our set achievements of reducing the future growth, stopping the intrusion growth and reducing the intrusion length, we can say that VIS has a positive effect compared to no mitigation/surface infiltration, and that it accomplishes all three of these achievements. In that respect, using a VIS matches perfectly with the goal to mitigate saline intrusion along the Al Batinah coast. However, the fact that water of higher salinity than allowed for potable use reaches the extraction wellfield makes the implementation of this specific VIS configuration less desirable.This does not mean that no other configuration of VIS downstream of the dam can be perfectly succesfull. Extra modelling should be undertaken to investigate this.

When more VIS wells were installed, it proved to be possible to infiltrate the full dam capacity volume within the set 12day limit.

General recommendations

When drilling projects on the Al Khawd Fan are commenced in future, special care has to be put to the lithological classification of the underground, to try and correlate the Al Khawd facies with existing Formations known to exist elsewhere on the Batinah. Downhole geophysical logging can assist in this analysis.The classification of sediment in classes of clean, clayey and cemented gravel is outdated and does not allow for comparison with other Batinah wadis where new drilling programs have categorized facies to Formations.

From a hydrological analysis point of view, it is best to construct production and observation wells with only one filter. Production wells preferably need to be constructed with filters not intersecting several identified less permeable strata, so that pumping tests easier to interpret. Observation wells should be constructed with limited filter length, to allow for better discrimination of differences in head, salinity and other parameters. For the best results, observation wells should be constructed taking into account the need for not only horizontal but also vertically spaced data points. Several observation wells in one hole, all with a filter at different depth are especially useful to analyze vertical stratification.

Existing pumping test data from the Al Khawd Fan should be reviewed, to check if similar errors have been made, as in the MacDonald&Partners 1986 analysis.
When performing new pumping tests on the Al Khawd Fan, the use of existing or construction of new observation wells is advised. The test data from these observation wells will help in correctly analyzing the test data and developing more nuanced conclusions about the hydrogeology of the fan.

The Omani ministries have been busy to get a better grip on water flows country wide, by installing water meters, by issuing laws about construction of new wells and keeping track of existing wells, as well as measuring salinities and head values along the coast. To allow for better estimation of mitigation effects on seawater intrusion measures, a longer record is needed and ongoing observation with good spatial density is nescessary.

This thesis presents a first deliberation on using VIS in Oman. However, more modelling work will be needed, before any VIS can be implemented to combat seawater intrusion along the Al Batinah coast.

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LIST OF APPENDICES

Appendix 1

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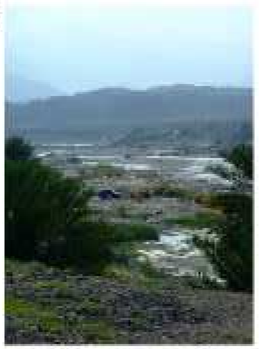

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Wadi Al Khoudh and Al Khoudh dam

1. view on dam basin
2. silt crust in dam basin
3. stilling ditch to reduce flow velocity downstream of dam
4. culvert spilling into stilling ditch
5. wadi downstreem of dam, seeb city in the background
6. wadi channel upstream of dam, new highway built in the background
7. view on wadi channel upstream of dam, near al Khoudh town
8. steep wadi wall, scoured by wadi flow. Orange vehicle in background

Appendix 2

From 2D to 3D and back

To legitimate the use of the local 3D model, it was tested if a symmetrical configuration of wells along the width of the model would indeed create flow perpendicular direction; so only flow along the length of the model, like it would along the length of the 2D profile. Figure 39 shows the result of 12 days pumping with a singular well in the 3D model. Since streamlines are perpendicular to the equipotential lines, in this situation flow is on a 360° radius.

New wells are inserted in the 3D model, all at the same distance along the profile (same x), but at different locations along the width of the 3D model (y direction). Each well gets its own grid cell. The distance between each well, and the two wells to the left and right is always 10m, so that grid size in y direction is also 10m. This means that at 5m to the left and right of each well (so at the cell boundary), a flow of similar magnitude is met, forming an artificial 'no flow' boundary (principle of mirror images). Thus all flow from the well will only be able to flow upstream and downstream from the cell, so along the x-direction of the model. This principle is visible in Figure 40: The equipotential lines are completely parallel to the coastline to the right and thus flow is completely perpendicular.

When the inter-well distance is such, that not every cell along the row (y-direction) gets its own cell, it depends on the number of cells 'open' up and down the row being first of all equal and second of all limited (y direction) to create equipotentials parallel to the coastline. If this is not the case, the flow is not symmetrical and the flow within the column is not in x-direction. Figure 41 shows this principle. On the left side of the figure, there is one cell in between each well-cell. The top view (above) shows that the equipotentials are nearly parallel to the coast, and that flow is thus almost perpendicular to the coast, shown by the velocity vectors (below). The cross view (middle) shows that there is flow from each well is entering the 'empty' cells to the left and right, but only at negligible velocities. To the right is an irregular spacing depicted; between every four cells containing wells, is one empty cell. The top view (above) shows that the equipotential lines are now not parallel to the coast anymore and that (below) the flow is not only in the x direction as becomes clear from the velocity vectors.

The flow and heads in each of the columns of the 3D local model, with no cells in between well-containing cells, would be similar to a 2D model profile. The jump from 3Dlocal to 2D local to 2D original model is here needed, because in the 3D model no other processes are at work: there are no other wells, there is no evaporation set, and there is no saline intrusion and no density difference. Since the 3D model is only run for 12 days pumping, any effects that these processes would have can be neglected.

Concluding, by using a local 3D model to test for different inter well distances, it is possible to obtain a needed flow rate for your more complicated 2D model to mimic the 3D situation. It is best to use cells of a width similar to the well distance. Thus each well is in the centre of a cell and flow is only perpendicular to the coastline. In effect you create a number of 2D models put together to form a 3D visualization – there is no flow between columns.

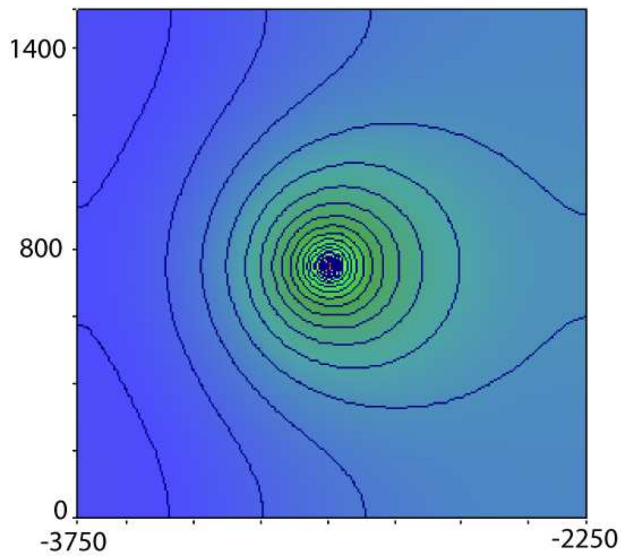

Figure 39 – A singular well in the 3D model (top view). Water table gradient is naturally to the right.

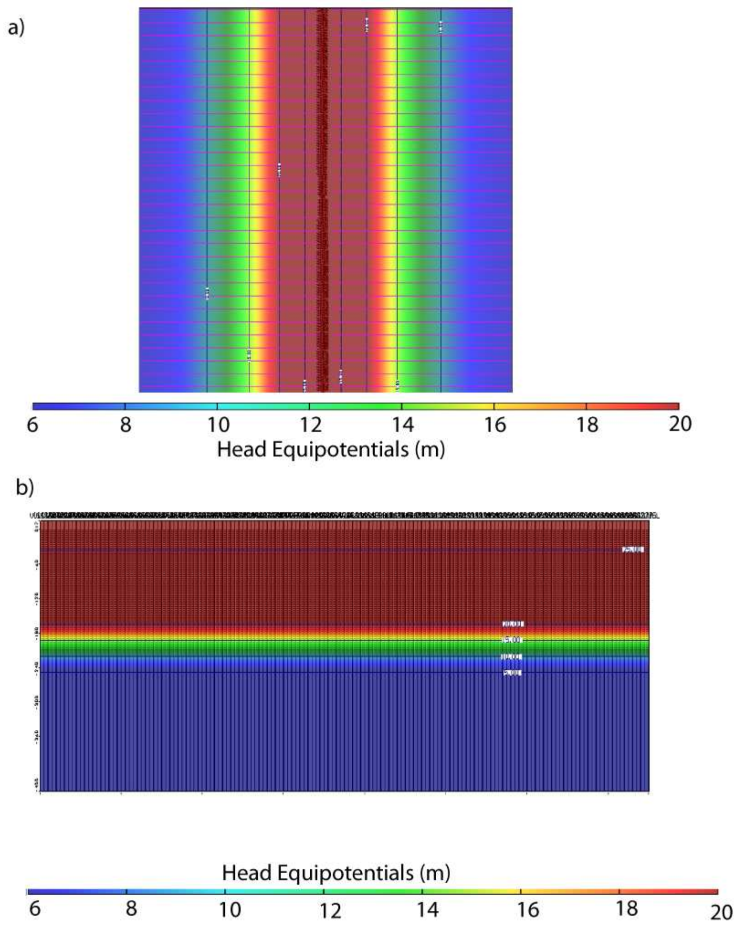

Figure 40 – The well distance being constant, water flows only from left to right (pink lines in figure a are the velocity vectors) like in a 2D profile of 10m width. a) Top view, b) cross view.

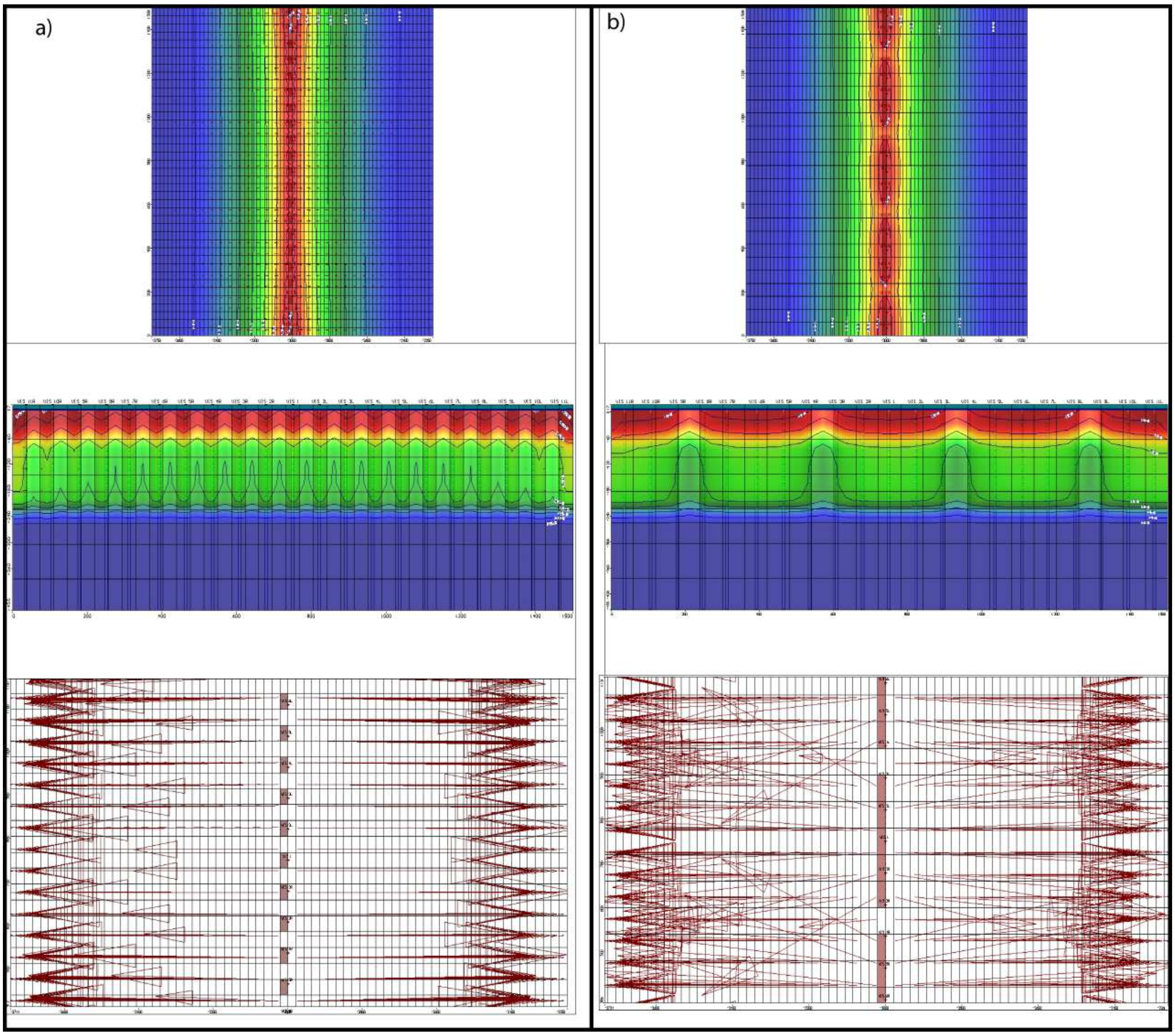

Figure 41 – a) one cell in between each well-cell, b) Irregular spacing; between every four cells containing wells is one empty cell.