Modelling sand storage dams systems in seasonal rivers in arid regions. Application to Kitui district (Kenya).

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UNESCO-IHE
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Master of Science Thesis

by

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Note 1: Some of the graphs in the present report require a colour view to be interpreted.

Note 2: Photo in the cover by L.Borst & S.A. de Haas.

Abstract

Different kinds of groundwater dams have been found to be an effective way to supply water in many developing countries during the dry seasons or even during dry years. These structures store water in subsurface reservoirs for livestock, domestic use and minor irrigation. The sub-surface reservoirs are recharged through flash floods originated from rainfall events. In the Kitui district in Kenya hundreds of sand storage dams have been built. The Kitui case has been successful, and the possibility to upscale this experience to other regions has been considered.

In this research, based on the sand storage dams constructed in the Kitui district, several hydrological models were created in Modflow to study the hydrology of these structures. The purpose was to gain insight on groundwater dynamics around sand storage dams in the long term and large scale.

The influence of some of the parameters involved in the hydrologic processes around the dams in the performance of each individual dam was studied. This influence could determine the success or failure of the project.

In the Kitui region, hundreds of sand storage dams were built in the Kiindu river and its tributaries. The conditions under which a system of multiple dams in the same riverbed behaved as multiple individual dams or as a connected network with global effects were analyzed.

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1. Introduction

1.1. Problem identification and justification

The current research is a Master Thesis prepared in cooperation between the Water Resource Management Department of Delft University of Technology and the UNESCO-IHE Institute for Water Education. This project aims at modelling and understanding the hydrology of sand storage dams and it is focused on the particular case of sand storage dams built in the Kiindu river catchment, located in the Kitui area, Kenya (see Figure 1.1).

Many developing countries with problems for supplying water are located in arid regions where rainfall is seasonal and erratic. Groundwater dams have been found to be a good way to store water from the rainy season to the dry season and from years with high rainfall to dry years. Groundwater dams are built across the stream bed. Their principle is to obstruct the groundwater flow and to store water below the ground surface. Evaporation losses and contamination risks are thus reduced. There are two types of groundwater dams, sub-surface dams and sand storage dams. Sub-surface dams are built below ground level and use the aquifer beneath the river to store water. Sand storage dams stand a few meters above ground level and enlarge the natural aquifer by retaining sediments carried by the river.

The interest in using groundwater dams for water supply has increased during the last few years and several research projects are planned or have already started in Asia, Africa and South America. In the Kitui district (Kenya) a total of almost 500 sand storage dams have been built in the Kiindu river and it's tributaries by the local NGO Sahelian Solutions Foundation (SASOL). This experience has been successful; water is available for a longer time in the dry season and people (women) don't have to travel such long distances to fetch water[1].

It is desirable to gain understanding of the hydrologic processes involved in these structures. This knowledge can then be used to optimize existing structures and to predict favourable locations for new ones. Studies of the Kitui area case will increase the chance of success in other areas. Most existing hydrological studies for this type of dams are based on water balance models and tend to ignore spatial and temporal variation in hydrological patterns [2]. Mathematical models of single dams have also been investigated to understand water flow effects in very close proximity to a dam [3]. However, the large scale effects, the long term effects and the interactions of several dams have not been studied.

[1] For instance, during the dry season (March to November) women typically had to walk one hour from Kiindi river to Nzeeu river and one hour back [2].

Figure 1.1 Location of Kutui region in Kenya

1.2. Background

The interest at Delft University of Technology in using groundwater dams for water supply has increased during the last few years in connection with rural developments projects (in Voi, Kitui and Machakos regions in Kenya). Several research projects have been carried out, and thesis and books have been written about it. Recently (11/01/2007) a seminar on "Small water retaining structures" was celebrated, in the Water Resources Management Department, Faculty of Civil Engineering and Geosciences, Delft University of Technology.

Different studies have been done on the design and management of water retaining structures. "Groundwater Dams for Small-scale Water Supply" by Ake Nilson [1] is a publication about design and constructions of groundwater dams. This study deals with issues such as:

- Advantages of using groundwater dams in many developing countries which are located in arid climatic regions.
- Suitable location physical conditions for the success of groundwater dams
- Historical review of several cases where groundwater dams constructions have been identified all over the world.

"Recharge Techniques and Water Conservation in East Africa – Up-scaling and Dissemination of the good practices with the Kitui sand storage dams" is a project by the Acacia Institute and SASOL which aims at using the experiences of the sand storage dams in Kitui to upscale the construction of sand storage dams in other regions. A first component of this project is the Master thesis "Hydrology of Sand Storage Dams - A case study in the Kiindu catchment, Kitui District, Kenya" by L.Borst and S.A de Haas [2], and focuses on the hydrology of sand storage dams in the Kitui case.

Some attempts have also been made to study the performance of the structures. Rolf Hut et al. 2006 [3] developed a mathematical model designed in a Matlab environment based on Bousinesq equation to predict the groundwater flow in close proximity of a sand storage dam. This model should be able to predict the flow in arbitrary situations. This research was based upon two different areas in Kenya where sand storage dams are present, Kitui and Voi.

1.2.1. General literature

Groundwater dams obstruct the flow of groundwater and stores water below the ground surface. They can raise the groundwater table in an aquifer with a limited flow of groundwater which is thus made accessible for pumping.

Groundwater dams may be of two types: sub-surface dams and sand storage dams. A sub-surface dam is constructed below ground level and arrests the flow in a natural aquifer, whereas a sand storage dam impounds water in sediments caused to accumulate by the dam itself. In sand storage dams by the construction of a weir of suitable height across the streambed, sand carried by heavy flows deposits and fills up the reservoir with sand. This artificial aquifer will be replenished each year during the rains, and water will be stored for use during the dry season. A sketch of a sand storage dam is shown in Figure 1.2.

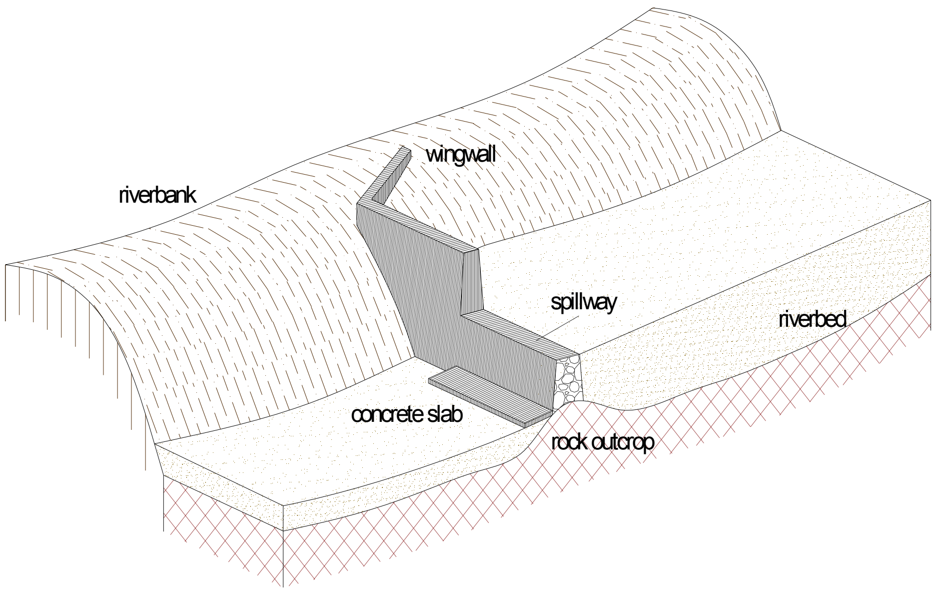

Figure 1.2. Sand storage dam sketch (Source: Borst and de Haas)

The main advantages of using groundwater dams instead of common surface dams are:

- Evaporation losses are reduced
- The water stored is less susceptible to pollution and health hazards such as mosquito breeding are avoided
- The land above the stored water can be used for other purposes

Groundwater dams have been used since Roman times and examples can be found all over the world. Recently various small scale groundwater damming techniques have been developed in India, Southern and East Africa and Brazil.

Generally it can be concluded that the present experience is positive; if properly sited and built, groundwater dams definitely serve their purpose. The hydrogeological conditions at the site have to be known and proper investigations should be carried out. Costs have to be kept to an absolute minimum and so investigations should be done with simple and unexpensive methods. Generalisations within areas with similar conditions should be used whenever possible.

Groundwater damming is not a universally applicable method for water supply. It can be applied only if certain physical conditions are at hand.

1.2.2. Hydrology of Sand Storage Dams in Kitui

In October and November 2005 a field research was carried out in the Kitui District in the context of a project by the Acacia Institute and SASOL which aimed at using the experiences of the sand storage dams in Kitui to upscale the construction of sand storage dams in other regions. A number of catchments, both with and without sand storage dams, were visited. As a case study the Kiindu catchment was selected. In and around this catchment measurements and observations of meteorology, soils and groundwater were carried out. The results of this experience were reflected in [2].

Figure 1.3. Sand storage dam in Kitui (Source: Borst and de Haas)

Results

The average annual rainfall in the Kiindu catchment was found to be 920 mm. Rainfall is strongly concentrated into two rainy seasons: the so called ‘short rains’ from October to February and the ‘long rains’ from March to May. In the dry period from April to September on average only 17 mm is recorded.

Average annual potential evaporation was determined at 1552 mm.

The soils on the riverbanks consist mainly of a top layer of silts and clays, under which a layer of weathered rock or sand is found until the hardrock is reached. The riverbed consists of coarse sand.

The response of the groundwater in the riverbed on rainfall and runoff is very rapid. A delayed rise of the groundwater levels in the riverbanks is observed. Within a few weeks however, both the sediments of the riverbed and the riverbanks are fully saturated. The effect of the sand storage dams on the water table in the riverbed is obvious since a step difference between the groundwater levels upstream and downstream can be seen.

Based on observations in the field and conversations with local people it was found that the reservoir upstream of a recently constructed dam fills up solely with coarse sand. Silt and clay particles which settle behind the dam are taken away with the river water when the river is flowing, or are blown away by wind. After 5 to 7 years the reservoir behind a dam is fully filled with sand and the dam is 'mature'.

During floods the river deposits a layer of silt and clay on the riverbed of the mature reservoir. Since the silt layer forms when most of the water has already infiltrated in the riverbed the infiltration is not obstructed. This layer is also taken up and discharged by subsequent river floods and blown away by wind.

When the groundwater level in the riverbed has increased it induces a sideward groundwater flow to the riverbanks where the groundwater level is still low, until the water level in the riverbed and riverbanks has levelled out.

The water level then starts to decrease due to evaporation from the sand, groundwater flow trough the hard rock, leakage trough and around the dam, and the use of water by people and animals. When the groundwater level in the river decreases below the groundwater level in the banks, the water starts to flow from the riverbanks towards the riverbed.

A part of the water that has infiltrated on the hill slopes will flow down to the river as lateral baseflow and will recharge the groundwater in the riverbed. Better land management can increase the amount of infiltration, which leads to more potentially harvestable water.

The leakage of water through the hard rock and around the dam can be considered a loss in the case of a single dam. In case of a cascade of dams however this water will replenish the water in a downstream reservoir.

A water balance was set up for a segment of the catchment that drains between two dams. This was done to determine the effect of one dam without the influence of its location in the catchment and the effect of local differences such as rainfall or vegetation in upstream areas.

All inputs and outputs of the catchment segment are determined or estimated. It was found that the amount of water that is retained by a dam is only about 2% of the total amount of runoff, so the remaining 98% discharges as surface water through the river.

Since the water stored in the riverbanks makes up an important part of the total amount of water retained (about 40%), the amount of water retained by a sand storage dam is much higher than just the amount of water stored in the sand of the riverbed.

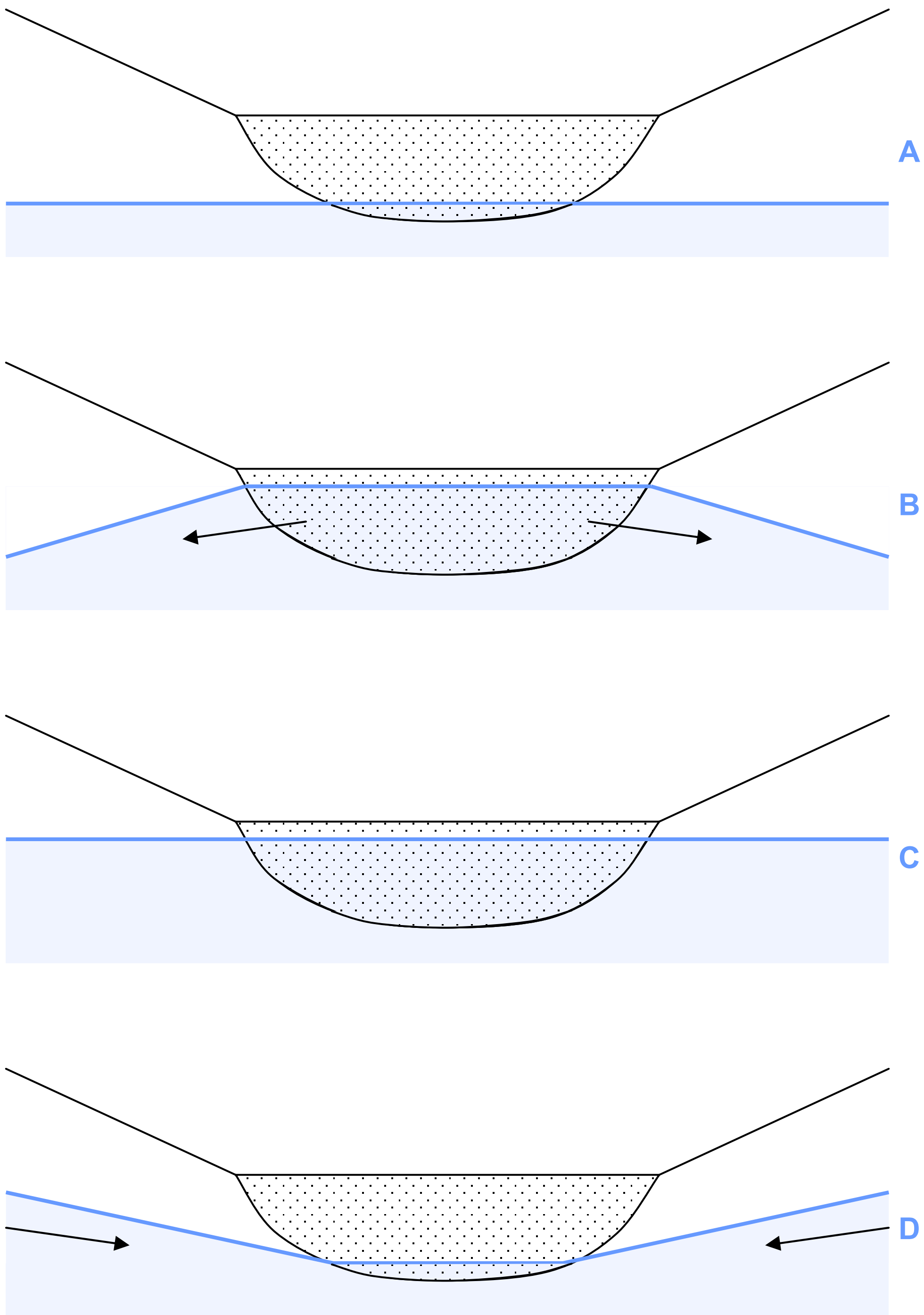

Figure 1.4. Schematic cross-sections over the riverbed: low water level (A), recharge from the riverbed to the banks (B), high water level (C) and recharge from the banks to the riverbed (D). (Source: Borst & de Haas)

Modelling sand storage dams

Hut et al. designed a dedicated mathematical model to study the hydrological processes and flows around a sand storage dam. The model, based on Bousinesq equation, was designed in a Matlab environment and it was focused on the study cases of Kitui and Voi in Kenya. The aim was to understand water flow effects in close proximity to a dam on the long run, when flows between dam reservoir and riverbanks and interactions between recharge and water extraction become important. With the model different scenarios were defined to study under which circumstances sand storage dams do or do not function as intended.

The modelling results indicated that sand storage dams can be an effective means to raise groundwater levels in landscapes surrounding seasonal rivers. However, which longer terms effects can be expected depends strongly on the way the water is used. It was shown that household water use and riverbanks infiltration can go hand in hand. Household water used is the case in the Kitui region. When water in the dams is used in irrigation as it is done in Voi, the infiltration effect is minimal.

This model presented several limitations. The first limitation is related with Boussinesq equation, which is an approximation. This can be a problem when events lead to abrupt increases or decreases in groundwater levels, for example when pumping is considered. Secondly, this is an explicit model which able to express groundwater levels at certain time step as a function of the groundwater levels in the previous time step. This introduces and error which can be minimized by using an implicit model as Modflow does.

The authors of this research suggested using the model to study the behaviour of networks of dam on regional scale as for example exists in Kitui. However for the present Master Thesis it was decided to do it in Modflow for several reasons. The first reason is that Modflow is a widely use software by groundwater modellers. Secondly, as Modflow is specific software for groundwater modelling, it is ready to easily implement the changes required for different scenarios. A Modflow user might wonder: "why should one spend time programming in Matlab a code with an explicit model when it is already done in Modflow with an implicit model where introducing changes is easy and user-friendly?"

In order to validate the Matlab model, before starting to work in the contents of this paper, the author of this Master Thesis in cooperation with Rolf Hut made a comparison between the Matlab model and Modflow for different test cases. The results of this research will be soon reported and it is plan to be published in a different paper independent of this Master Thesis.

1.3. Goals of the study

It is desired to better understand the groundwater dynamics in sand storage dams on the long term and large scale. In this context the following goals were proposed for this Master Thesis:

- Short term goals:
  - To validate the studies carried out in this field so far.
  - To study the case of one dam taking into account the seasonal variation of the hydrological inputs.
  - To gain qualitative insight of the parameters affecting the structure (hydraulic conductivity, hydraulic gradient, water use …). The values that these parameters take can determine the success or failure of the project and so are crucial in the selection of future dam sites.
  - To analyze the groundwater dynamics of dams' networks, discerning under which conditions several dams behave as a connected system or behave as individual structures. It must be determined when there is an interaction between the dams and when the effects of each dam are localized and don't affect each other.
  - To explore the advantages and disadvantages of Modflow as applied to this problem.
- Long term goals:
  - To lay the foundations of future Modflow studies which include other factors such as detailed rainfall modelling or detailed geomorphology modelling.

- To lay the foundations of future Modflow studies of several dams.

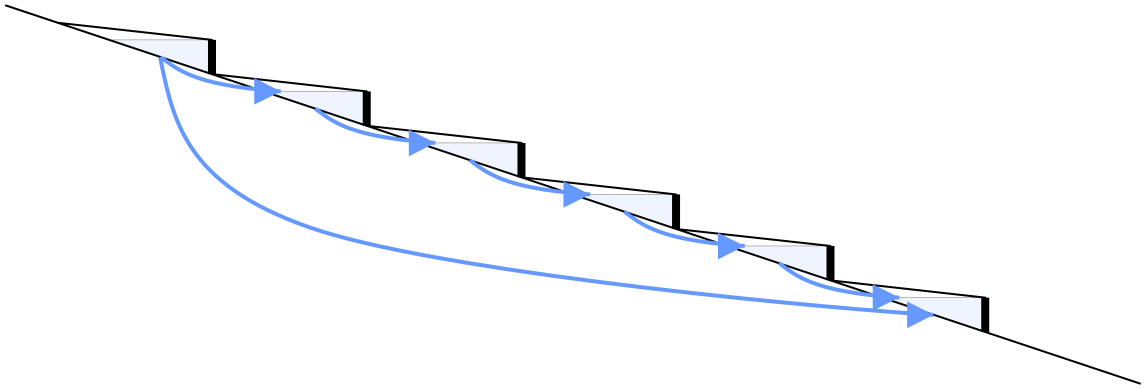

Figure 1.5. Sketch of a dams' network (Source: Borst and de Hass)

1.4. Activities

In order to achieve the previous goals several cases and scenarios were created in Modflow.

Firstly a certain geology in which the impermeable rockbed could be found at shallow depth was assumed. As a result the aquifer was a few meters thick (2 m), which could be considered as a *shallow system*. For this *shallow system* three cases were studied: the natural situation previous to the construction of the dams, a single dam, and a network of three dams.

In each system, several scenarios were defined. The first considered scenario was the base case. For the rest of scenarios some of the involved parameters were changed.

The organization chart shown in Figure 1.6 presents a scheme of the simulated cases in Modflow.

After the study of the *shallow system*, different geological conditions were assumed. This time an extra layer 50 m thick below the shallow aquifer mentioned above was considered. Again, the same three cases were studied: the natural situation previous to the construction of the dams, a single dam, and a network of three dams.

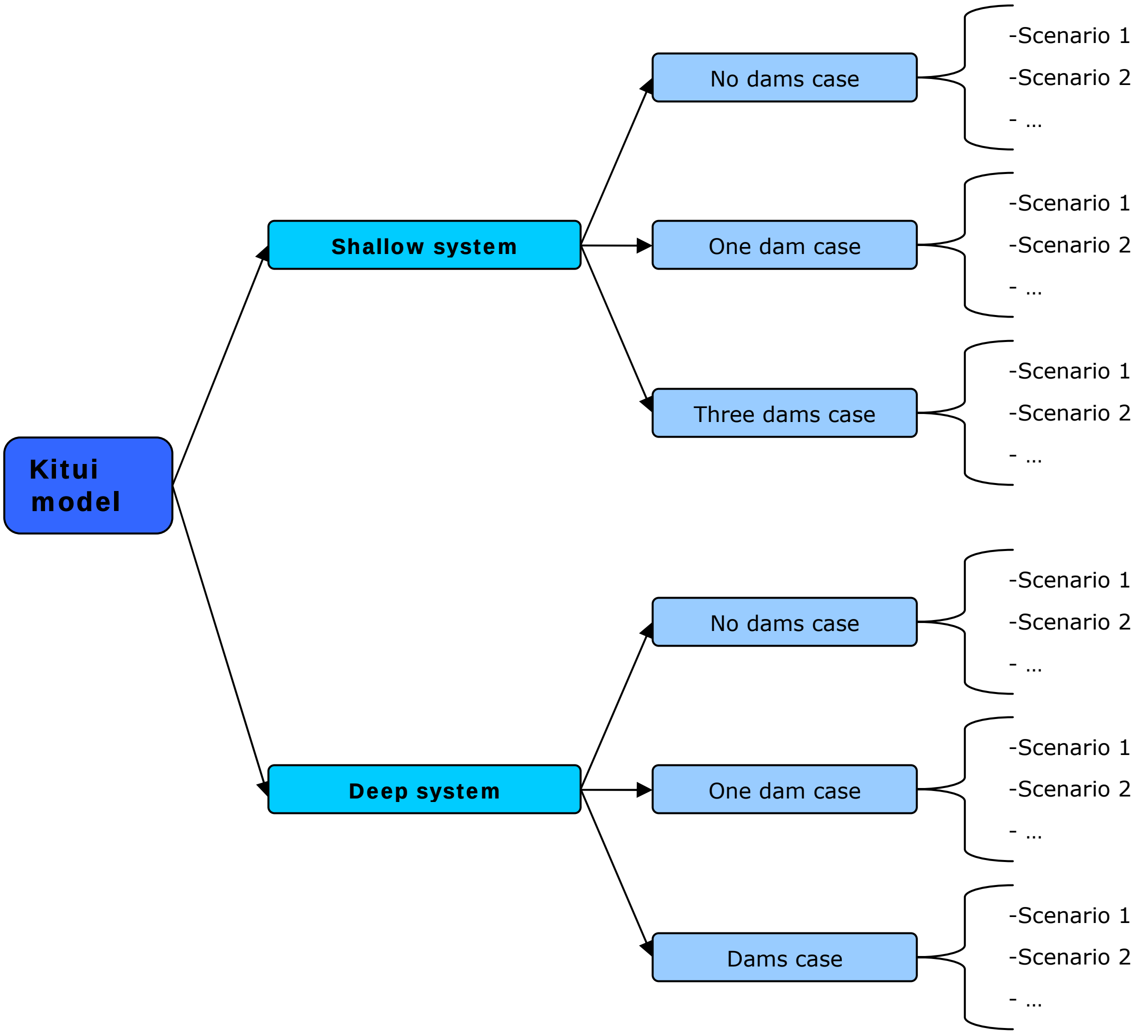

Figure 1.6. Organization chart of the model, systems, cases and scenarios created in Modflow

2. Methods

The purpose of this chapter is to describe the cases and scenarios created in Modflow to study the groundwater dynamics in sand storage dams' surrounding areas.

2.1. Geometry and soil properties

2.1.1. Shallow system

The model grid spanned 2600 m in the length profile and 400 m to each side of the river. The segment of the river was assumed to be straight and the model was simplified due to the symmetry along the river axis. Thus, the most upstream point of the river was located in the South-West corner of the grid and the most downstream point of the river was in the North-West corner. The river flowed throughout the west end of the model grid. The riverbed width was assumed to be 10 m, so that due to the symmetrical conditions the model included 5 m of the river. In Figure 2.1 (a) the geometry described above can be seen.

Concerning the geomorphology, in the bottom of the system an impermeable rock layer was assumed. The aquifer lied on top of this rock layer. The aquifer was composed of one sand layer (*Layer 2*). On top of the sand, in the riverbanks, a clay layer (*Layer 1*) was assumed. The rock layer had a 1 % slope in the length direction and a 1 % slope from the banks towards the riverbed. So the highest point of the model was located in the South-East corner and the lowest in the North-West corner (see Figure 2.1 (a)). The sand aquifer layer had a constant thickness of two meters throughout the entire domain. The second model layer was composed of clay and had a thickness of three meters except in the riverbed. In the riverbed the second model layer was composed of sand and its thickness varied between zero and three meters at the location behind the dam.

To model this, the grid distribution used was the following:

- In the riverbed direction: 260 rows
- From the riverbed to the riverbanks: 45 columns with widths of five or ten metres

As said before, the natural situation without dam was simulated first. In this case all the cross sections throughout the domain were the same (Figure 2.1 (b) presents one of these cross sections).

For the dam case, the dam itself was modelled as a wall located in the riverbed blocking the entire riverbed and with 5 m wing walls on both sides. The wall was modelled as an impermeable horizontal flow barrier without thickness which extended from the rock layer to the top of the clay layer. This means that the height of the dam above the riverbed was 3 m, the same as the thickness of the clay layer.

Behind the dam a wedge of sediments carried by the river and deposited upstream of the dam was assumed (see Figure 2.1 (c)). This was a reasonable assumption for the Kitui case since the dams were built more than ten years ago, so the sedimentation process can be assumed to be completed. The dimension of this sand wedge depended on the slope of the river and on the dam height: the steeper the slope the smaller the wedge in the river length direction; the higher the dam, the larger the wedge (both in height and length).

The single dam was located 800 m upstream of the Northern model boundary (x_D = 1800 m)..

The soils were considered to be homogenous and isotropic. The soil properties for each layer were taken from [3] and are shown in Table 2.1.

	Sand	Clay
k (m/day)	0.864	0.00864
S_y (-)	0.3	0.08
S_s (-)	0.0001	-

Table 2.1. Model layer property values

The assumed boundary conditions were:

- Northern and Southern boundaries: horizontal constant head of 1 m above the rock layer at the riverbed.
- Eastern and Western boundaries: no flow boundaries.

The dimensions of the grid and the distance between the dam and the boundaries were taken large enough to assure there was no influence of the boundary conditions on the observations next to the dam location. Concerning the Western boundary, there was no inflow coming towards the riverbed from it. This was assumed based on the fact that most of the water flowing into the seasonal riverbeds in Kitui comes from rainfall of an upstream area where the rainfall is larger, so that the runoff from the bank to the river is not that important. The East boundary was a no-flow boundary.

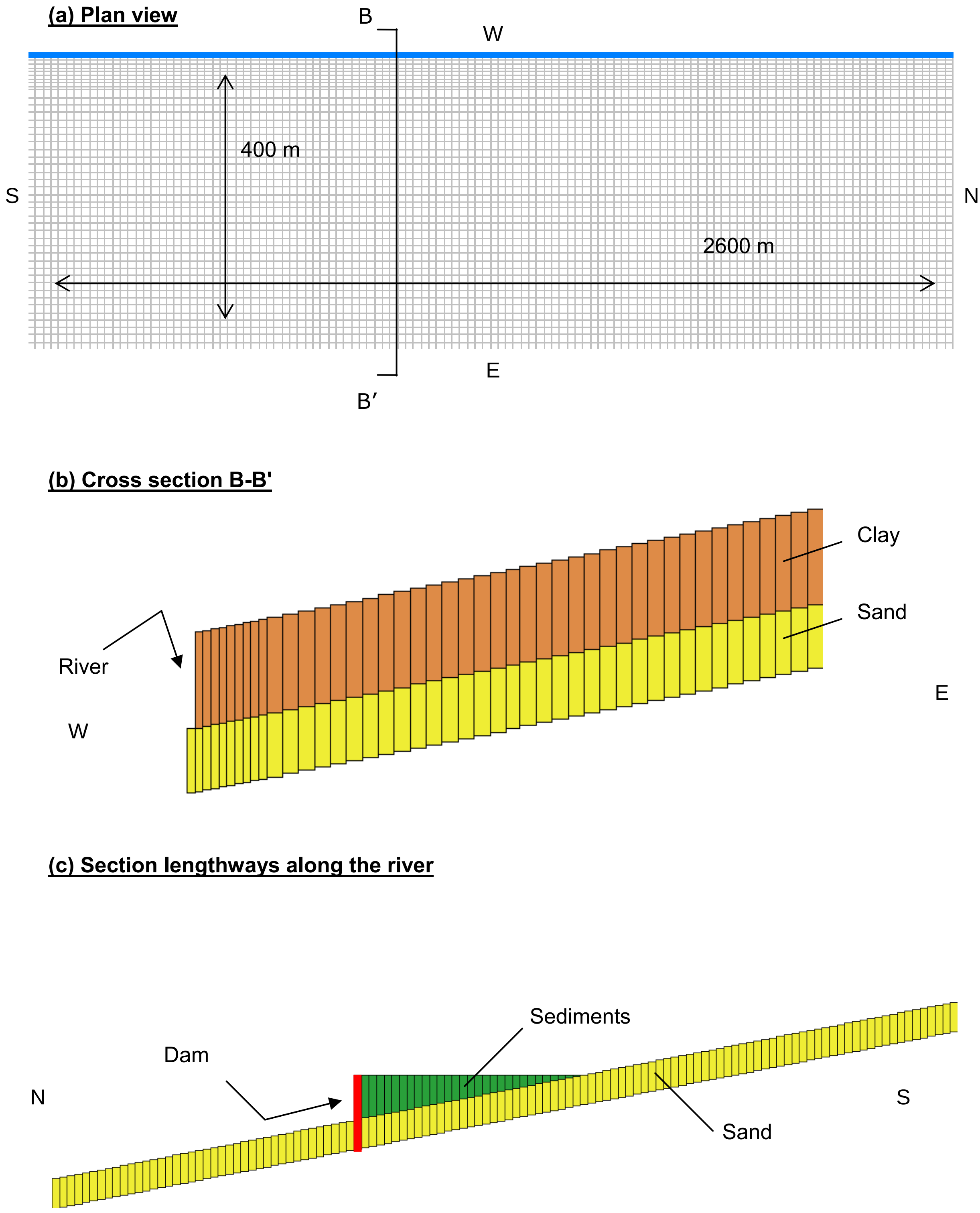

Figure 2.1. Model geometry

For the three dams case, the first dam (*Dam 1*) was located 800 m upstream of the Northern boundary (x_{D1} = 1800 m). The second (*Dam 2*) and third (*Dam 3*) dams were upstream of the first one with a distance between dams of 500 m (x_{D2} = 1300 m, x_{D3} = 700 m), which is considered the average distance between dams in the Kiindu river by [2].

2.1.2. Deep system

The geometry of this model was the result of adding an extra layer (*Layer 3*) between the sand layer and the bedrock to the three dams' model in the *shallow system*. This extra layer was 50 m thick, thus enlarging considerably the aquifer.

The dams' height remained the same as in the previous model, that is, it extended from the top of *Layer 1* to the bottom of *Layer 2*.

The soil properties for *Layer 1* (sandy) and *Layer 2* (clayish) remained equal to what they were in the *shallow system*. *Layer 3* was assumed to be sand as well.

2.2. Events

Kitui district is one of the arid and semi-arid areas of Kenya. Rainfall in Kitui is seasonal with two rainy seasons, one from October to February and one from March to May. During a rainy season flash floods which saturate the riverbed and the reservoir behind a dam are common. For this study two rainy seasons were considered:

- February: from the 1st February to the 28th
- November-December: from the 1st November to the 31st December

Each season was assumed to be a different stress period in Modflow. As a result each year had four stress periods. In all the simulation time steps were 5 days long.

Rainy seasons were modelled as saturation of the riverbed and the reservoir behind a dam. Accordingly during all the time within the mentioned rainy seasons, cells in the riverbed remained full.

This was implemented in Modflow by using the following model properties during the rainy seasons in the cells located in the riverbed:

- A large recharge value to instantaneously saturate the riverbed.
- A drain located at the riverbed surface level with a very large conductance to prevent the heads from rising above the top of the riverbed.

Evapotranspiration in Modflow is defined by assigning the following parameters to each vertical column of cells:

- Maximum ET Rate (R_{ETM}) [LT^{-1}],
- Elevation of the ET Surface (h_s) [L],
- ET Extinction Depth (d) [L],
- Layer Indicator (I_{ET}) [−]

The Modflow Evapotranspiration package removes water from the saturated groundwater regime based on the following assumptions:

1) When groundwater table is at or above the elevation of the ET surface h_s, evapotranspiration loss from the groundwater table is at the maximum ET Rate R_{ETM};
2) No evapotranspiration occurs when the depth of the groundwater table below the elevation of the ET surface exceeds the ET extinction depth d; and
3) In between these two extremes evapotranspiration varies linearly with the

groundwater table elevation.

The assumed values for the previous parameters were:

- R_{ETM} = 3 mm/day
- h_s: 20 cm below the ground surface
- d = 2 m
- I_{ET} : the highest active layer in each vertical column of cells

2.3. Scenarios

2.3.1. Scenarios for shallow system

For the two considered models of the *shallow system* five scenarios were analyzed:

1. *Scenario 1*: The natural system with no water uses (pumping wells). In this case the only stresses for the system were the rainy seasons and the evapotranspiration.
2. *Scenario 2*: The previous scenario adding pumping wells for household water use.
3. *Scenario 3*: The same system described in *Scenario 1* but changing the sand hydraulic conductivity to 0.0864 m/day, which is ten times lower than the value assumed in *Scenario 1*.
4. *Scenario 4*: The natural system described in *Scenario 1* but changing the hydraulic conductivity to 8.64 m/day, which is ten times larger than the value assumed in the original case.
5. *Scenario 5*: The natural system described in *Scenario 1* but changing the slope in the downstream direction to 5 % instead of the original 1 %.

The pumping rate value assumed was based on the use communities give to the water provided by dams in Kitui district. This water is mainly used for household purposes and a dam like the one modelled would serve approximately ten families. Farmers in Kitui region do not pump to irrigate their fields.

The values assumed as initial heads were:

- *No dam cases*: in each row initial heads were 1 m above the rock layer at the riverbed in this row except in the riverbed cells which were saturated.
- *Dams cases*: heads obtained at the end of the longest dry season for the case without dam.

The study period was 2 years, but in order to suppress the influence of the initial conditions (which were somewhat arbitrary), 38 additional years were run at the start of the simulation. This was observed to be enough time for the model to reach a steady state where the results only depended on the seasonal variations and were independent of the initial conditions.

2.3.2. Scenarios for deep system

Two scenarios were considered for the *deep system*:

1. *Scenario 1*: The natural system with no water uses (pumping wells). In this case the only stresses for the system were the rainy seasons and the evapotranspiration.
2. *Scenario 2*: The natural system described in *Scenario 1* but changing the hydraulic conductivity of certain area of Layer 3 into a low value. This

situation might represent the case where a sort of natural underground dam in the form of low permeability bar improve the natural groundwater conditions for sitting a sand storage dam. A natural dike may have damming effect which would be enhanced by the construction of groundwater dams. The hydraulic conductivity for those cells located downstream of *Dam 1* in *Layer 3* was changed from the initial 0.864 m/day value to 0.0864 m/day.

2.4. Modflow flow packages used

The Modflow flow packages used to create the model described above were:

- Drain package: to model the rainy seasons [5].
- Recharge package: to model the rainy seasons [5].
- Evapotranspiration package [5].
- Horizontal-Flow Barriers package: to model the dam [5] [3].
- Wetting Capability package: to allow cells to convert form dry to wet [5].
- Well package: to model the effect of the water use in the system [5].

3. Results

The purpose of this chapter is to show the results of simulating in Modflow the models described in the *Methods* chapter. First of all the results of analyzing the *shallow system* (*One dam model* and *Three dams model*) are presented. Secondly the obtained results for the *deep system* are shown.

Head-time curves are very helpful to understand the hydrology of the system. In order to obtain these curves several rows of boreholes were set in the models to keep track of the heads in the borehole location over time. Borehole rows named *R1* were located immediately upstream of the dams in the dams cases and in the same location in the cases without dam. In the *Three dams model* a *R1* borehole row was located upstream of each dam. As a results three *R1* boreholes rows named *R1_1* (upstream of *Dam 1*), *R1_2* (upstream of *Dam 2*) and *R1_3* (upstream of *Dam 3*) were set. In each borehole row there were three boreholes with the following locations (see Figure 3.1):

1. Borehole located in the riverbed
2. Borehole located 25 m from the riverbed towards the riverbanks
3. Borehole located 75 m from the riverbed towards the riverbanks

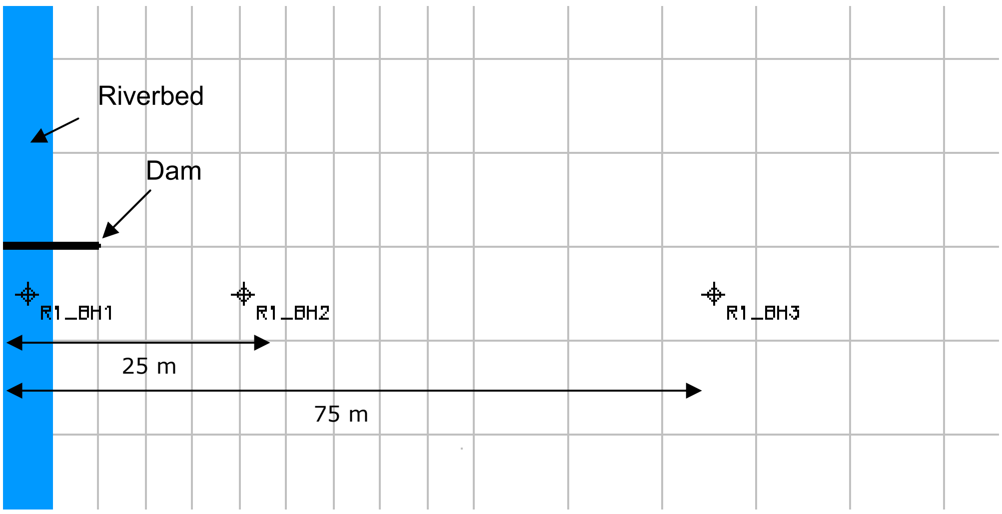

Figure 3.1. Boreholes in R1

3.1. Shallow system

3.1.1. No dam case

Scenario 1

A forty year simulation was run. The simulation started with the initial heads described in the previous chapter and the first head rise was due to the first year's February rainy season (see Figure 3.2 (a)).

The colour code used in all the figures was:

______ borehole in the riverbed

______ borehole located 25 m from the riverbed towards the riverbanks

______ borehole located 75 m from the riverbed towards the riverbanks

In the *no dam case*, head-time curves behaviour was the same for all the cross sections due to the fact that there wasn't any alteration in the river length direction. Thus, for certain time step, the water thickness didn't change throughout any column in the model domain, except in the vicinity of the Northern and Southern boundaries.

The observed head levels presented seasonal oscillations. The further the borehole was from the riverbed, the more damped the seasonal oscillation were. A part from the seasonal oscillations, a general rising trend was observed. After 10000 days (27 years) the general rise stabilized. From that moment on, the heads could be considered to be independent from the initial conditions. Thus the results in the two last years of the forty years simulation were not influenced by the initial heads. These results are shown in Figure 3.2 (b). The maximum head difference between lowest and highest head throughout the year in the riverbed was 50 cm, whereas in the borehole located 25 m towards the riverbanks this value was 20 cm. In the third borehole, 75 m towards the riverbanks, these oscillations could be neglected.

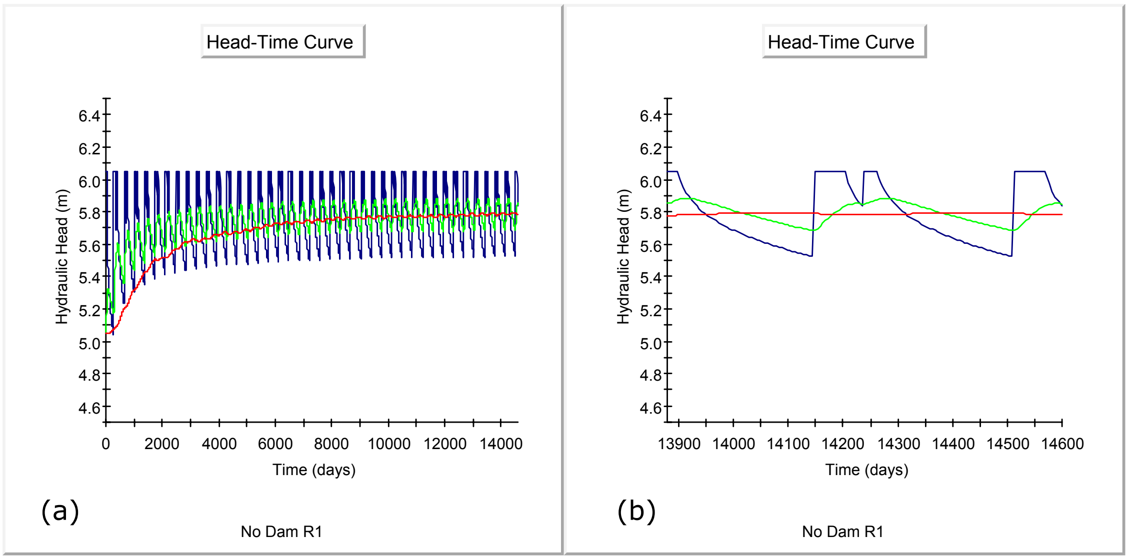

Figure 3.2. Head-time curves for No dam case, Scenario 1

Scenario 3

The current scenario considered the natural system described in *Scenario 1* changing the sand hydraulic conductivity to 0.0864 m/day, which is ten times lower than the assumed value in the original case.

As it was said before after certain time, the results could be considered to be independent from the initial conditions. The lower the hydraulic conductivity, the longer the time needed to reach this situation. For this case a 70 years simulation was run to give time enough to the system to reach certain equilibrium (see Figure 3.3 (a)).

Lower hydraulic conductivity values (*k*) gave higher inertia to the system, which means that the system responded slower to changes. During rainy seasons a lower water volume infiltrated in the riverbanks, whereas during dry seasons the water flow from the banks towards the riverbed was also lower.

The observed heads in this scenario were lower than in *Scenario 1* (compare Figure 3.2 (b) with Figure 3.3 (b)).

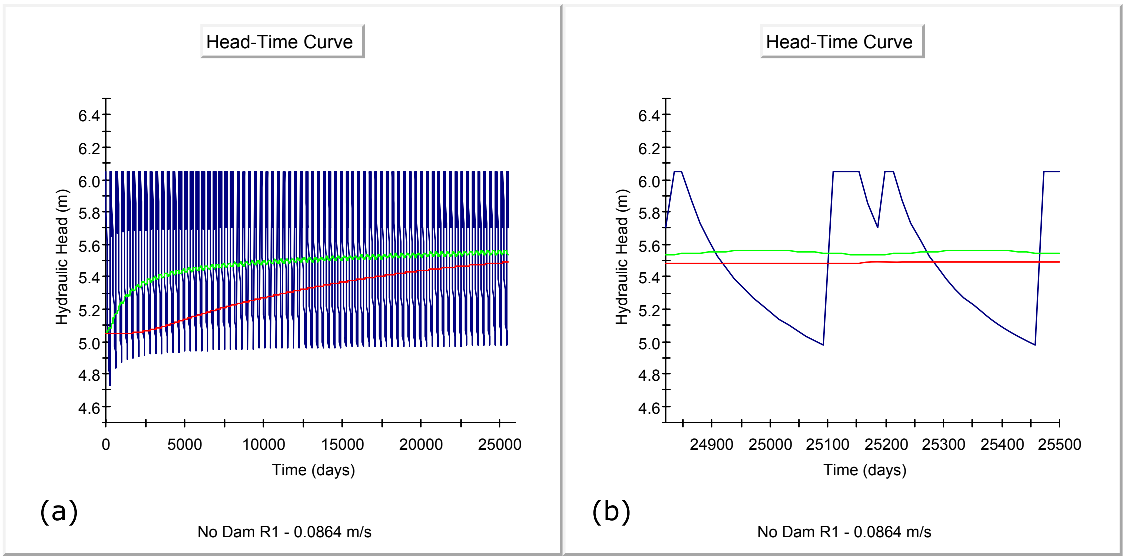

Figure 3.3. Head-time curves for No dam case, Scenario 3

Scenario 4

As said before, this scenario considered the system described in the *Scenario 1* changing the sand hydraulic conductivity to 8.64 m/day, which is ten times the value assumed in the original case.

Due to the higher permeability, the process until the heads could be considered to be independent from the initial conditions was faster, as can be seen in Figure 3.4 (a) where the heads in a forty years simulation in the three mentioned boreholes are shown.

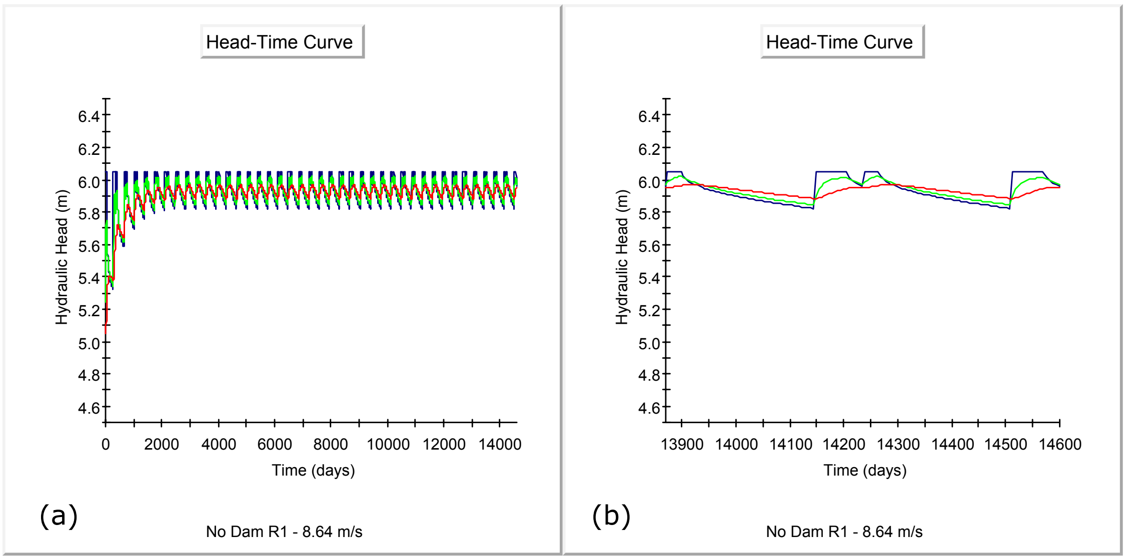

Figure 3.4. Head-time curves for No dam case, Scenario 4

Compared with the prior scenarios in this case during the rainy seasons a larger water volume infiltrated towards the riverbanks reducing the difference between the water levels in the riverbed and the riverbanks. Similarly, during dry seasons water from the bank was flowing easily towards the riverbed reducing again the difference between the levels in the riverbank and in the riverbed. As a result, the observations in the three boreholes were closer to each other than for lower *k* values (see Figure 3.4).

For larger *k* values, the water table level in the system was higher in every time step throughout a year (compare Figure 3.2, Figure 3.3 and Figure 3.4).The wet area was larger; the distance towards the riverbanks reached by water infiltrated form the riverbed was longer.

Scenario 5

In the current scenario the longitudinal slope in was changed from the 1 % slope of *Scenario 1* to a 5 % slope.

In this case the rock layer elevation in the riverbed cell in the boreholes row was 30.25 m. Accordingly the riverbed surface elevation was 32.05 m and the bank elevation was 35.05 m.

The head level values were much higher than in *Scenario 1* due to the higher elevation of the rock layer in this case. Nevertheless, the water thickness in the aquifer in this scenario was lower than in *Scenario 1* due to the steeper slope.

Before building a dam, the steeper the slope was, the lower the water table levels in the system were (refer to Figure 3.5). This is logical since the downstream flow was higher and the water volume infiltrated in the riverbanks was lower.

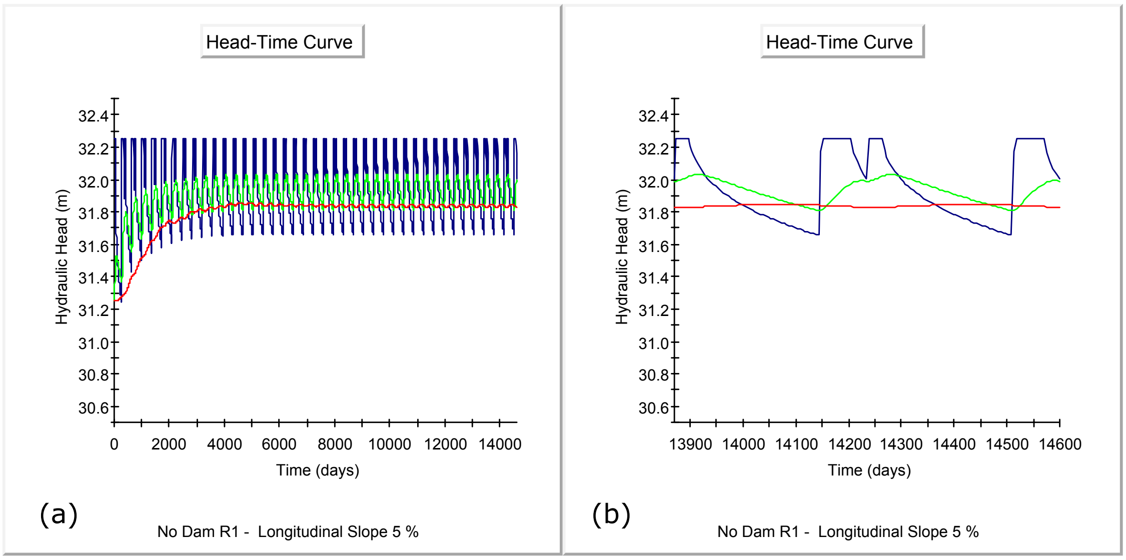

Figure 3.5. Head-time curves for No dam case, Scenario 5

3.1.2. Dam case

Scenario 1

The construction of the dam, as desired, raised the water levels in the area. The maximum rises took place immediately upstream of the dam, where the borehole row *R1* was placed. The observation of the head-times curves in the boreholes helps to analyze this rises in the direction perpendicular to the river length. These curves can be seen in Figure 3.6.

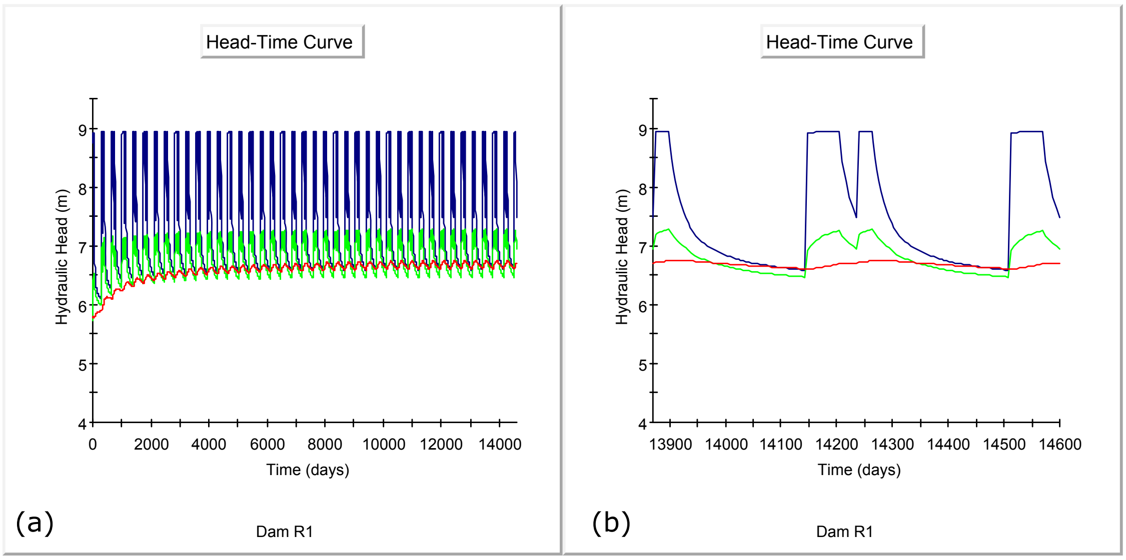

Figure 3.6. Head-time curves for One dam case, Scenario 1

At the end of the longest dry season, due to the dam the rises in the heads in the three boreholes location were:

- Borehole in the riverbed: 1.1 m
- Borehole 25 m towards the riverbank: 0.8 m
- Borehole 75 m towards the riverbank: 0.8 m

As expected, the maximum rises took place in the borehole just behind the dam, and the further towards the riverbanks, the lower the rises. This can be seen in Figure 3.7, where drawdowns for the boreholes mentioned before are represented. It should be taken into account that the drawdowns were measured from the initial heads. Therefore the values on the graph represent the difference between the heads measured in the boreholes at the end of the longest dry season in case with no dam (initial heads), and the heads observed in each time step. Thus the maximum values observed in the curves indicate the end of the longest dry season since these values represent the moments when the heads were lower. The negative drawdowns represent the rises.

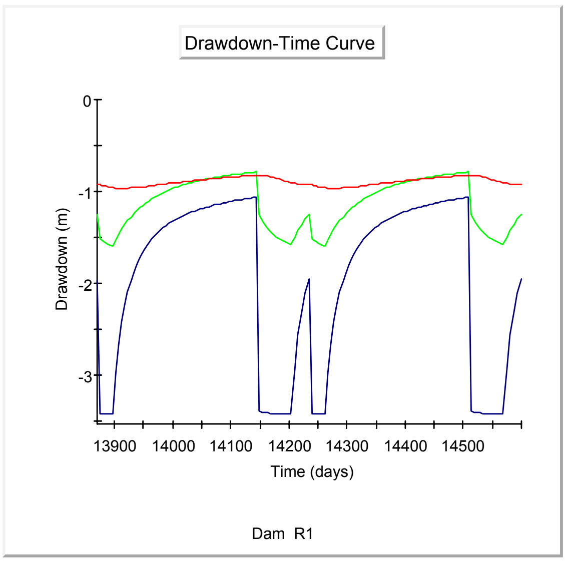

Figure 3.7. Drawdown-time curve for one dam model, dam case, Scenario 1

Concerning to the variation in heads in the river length direction, the water table was raised not only behind the dam but also downstream of the dam because of the water flow in the riverbanks towards downstream. This can be seen in Figure 3.9 where a plan view of the drawdowns when comparing the end of the longest dry season in the situation with dam and without dam are shown for *Scenario 1*, *3* and *4*. Considering as affected by the dam the area where the rises were larger than 10 cm, the influence area extended approximately 350 m towards downstream and upstream (see Figure 3.8 *Scenario 1*). Modflow displays the drawdowns only for those cells which were already wet at the beginning of the simulation (not dry in the initial condition), therefore Figure 3.9 allowed estimating the influence area extension in the river length direction as these cells in the riverbed and vicinity were wet before the construction of the dam (initial condition). However, the influence area extension in the direction perpendicular to the river length (towards the riverbanks) can not be determined in this graph because Modflow didn’t display the drawdown (or rise) for those cells in the riverbank which were dry before the dam was built (initial condition).

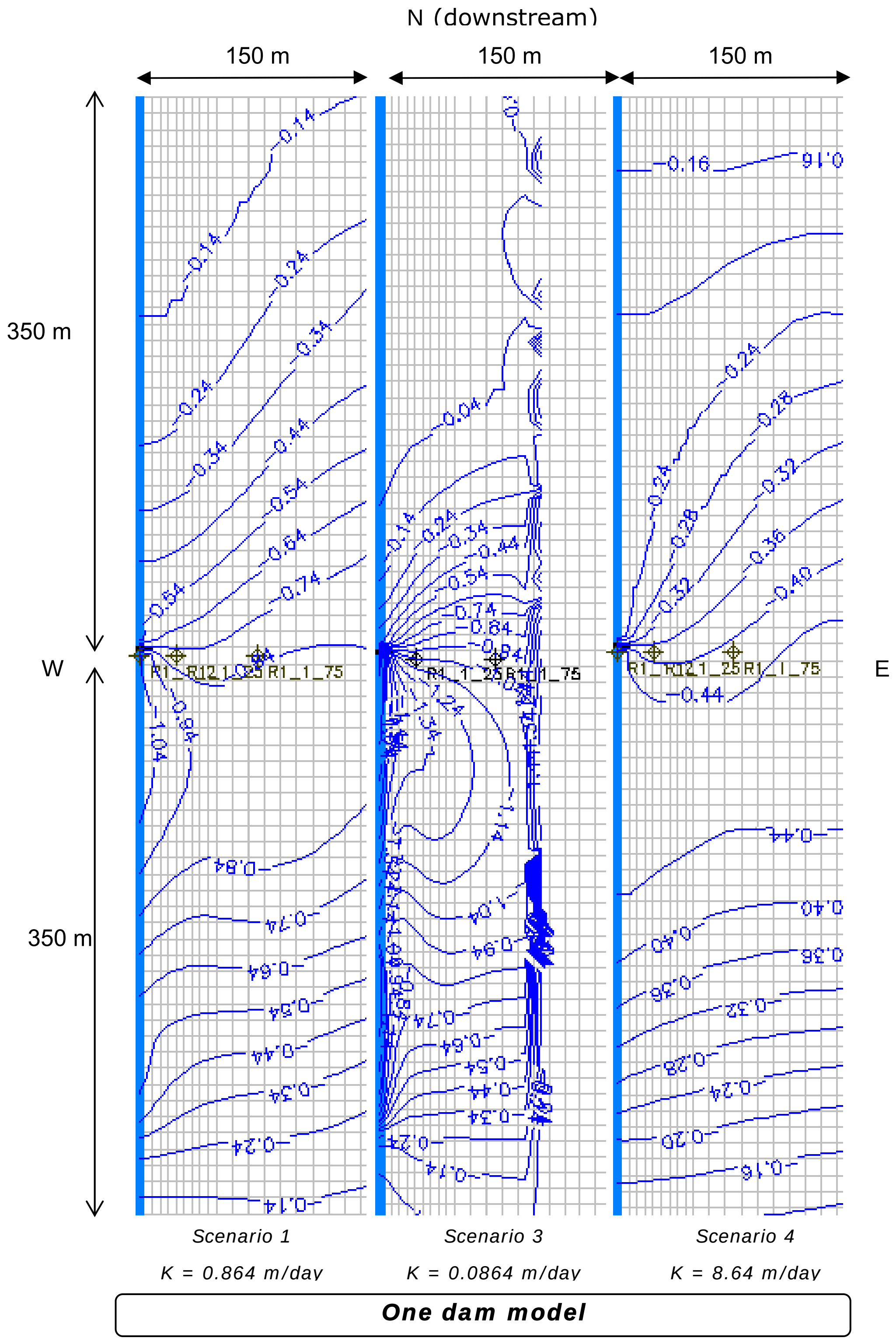

Figure 3.8. Drawdowns due to the dam at the end of the longest dry season in One dam model, dam case, Scenario 1, 3 and 4

An interesting tool to evaluate the gain on water volume in the system due to the construction of the dam was to compare the water volume in the model domain at certain times of the year between the situations without and with dam. This comparison could be made by using the information Modflow displayed for each time step. This information included a volumetric budget for the entire model at the end of each time step for each stress period. In this model each season was considered to be one independent stress period. Time steps were approximately 5 days long. An example of this can be seen in Figure 3.9.

In the water budget, the "*in storage*" water volume was the sum of the decreases in water storage, for each time step and for each cell. An example of water volume *in storage* appears highlighted in Figure 3.9. On the other hand, the "out *storage*" water volume was the sum of the increases in water storage. An example of volume of water *out storage* appears highlighted in Figure 3.9.

Thus, the difference between "*out storage*" and "*in storage*" water volumes for a certain time step indicated the incremental water volume in the system in this time step when compared to the initial situation.

Curve in Figure 3.10 presents this incremental volume at the end of the longest dry season every year during a fifty year simulation. From the first year after the dam was built the gain on water volume (compared with the situation with no dam) increased every year. After 40 years the gain in water volume remained the same every year. This final asymptotic value represented the extra water volume in the model domain on the long term at the end of the longest dry season due to the dam. See Figure 3.10. This graph shows that not only the dams were useful to store water from the wet to the dry season but also to store water between years. Even though in this analysis all the years were equal in terms of rainfall, it can be seen that in case a dry year occurred, an extra water volume from the previous years would be available.

VOLUMETRIC BUDGET FOR ENTIRE MODEL AT END OF TIME STEP 49 IN STRESS PERIOD138

CUMULATIVE VOLUMES	L^3	RATES FOR THIS TIME STEP	L^3/T
IN:		IN:	
STORAGE =	209978.7500	STORAGE =	14.4743
CONSTANT HEAD =	1159.3218	CONSTANT HEAD =	0.2447
DRAINS =	0.0000	DRAINS =	0.0000
ET =	0.0000	ET =	0.0000
RECHARGE =	9009329.0000	RECHARGE =	0.0000
TOTAL IN =	9220467.0000	TOTAL IN =	14.7190
OUT:		OUT:	
STORAGE =	230341.1560	STORAGE =	0.1665
CONSTANT HEAD =	35001.3672	CONSTANT HEAD =	1.7895
DRAINS =	8767875.0000	DRAINS =	0.3101
ET =	198647.1410	ET =	12.5083
RECHARGE =	0.0000	RECHARGE =	0.0000
TOTAL OUT =	9231865.0000	TOTAL OUT =	14.7744
IN - OUT =	-11398.0000	IN - OUT =	-5.5359E-02
PERCENT DISCREPANCY =	-0.12	PERCENT DISCREPANCY =	-0.38

SUMMARY AT END OF TIME STEP 49 IN STRESS PERIOD138

	SECONDS	MINUTES	HOURS	DAYS	YEARS
TIME STEP LENGTH	4.32000E+05	7200.0	120.00	5.0000	1.36893E-02
STRESS PERIOD TIME	2.11680E+07	3.52800E+05	5880.0	245.00	0.67077
TOTAL TIME	1.09581E+09	1.82635E+07	3.04392E+05	12683.	34.724

Figure 3.9. Volumetric budget displayed by Modflow for each time step

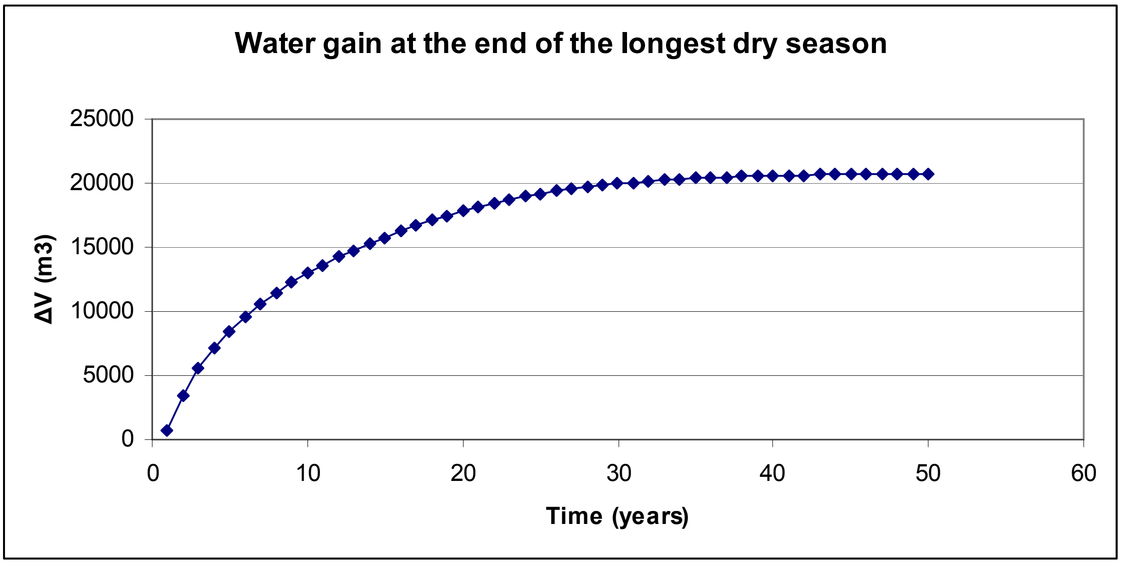

Figure 3.10.The evolution over time of the gain on water volume in the system at the end of the longest dry season due to the construction of the dam

Scenario 2

Increasing the water thickness in the aquifer by building a dam, would allow a larger pumping rate in the well before the well got dry, as shown in Figure 3.11. When a sand storage dam is built, the rises in the water levels due to the dam depend on several factors as the hydraulic conductivity, the hard rock geometry and the dam's height. According to this graph, when deciding a new sand storage dam location the previous factors should be chosen carefully in order to maximize the allowed pumping rates.

Figure 3.11 presents the drawdowns with the distance to the well for different pumping rates values. To obtain these curves a simple model was developed in Modflow. This simple model had the following features:

- Grid: 2000 x 2000 m^2
- 100 rows and 100 columns
- 1 layer 10 m thick
- Hydraulic conductivity value: 0.864 m/day
- Pumping well in the centre of the grid
- Initial heads: saturated
- Boundary conditions: constant heads in the four boundaries
- Simulation time: 5000 days

The following pumping rate values were tested: 1.05 m^3/day, 2 m^3/day, 3 m^3/day and 5 m^3/day, and 8 m^3/day, and 10 m^3/day, and 15 m^3/day and and 20 m^3/day.

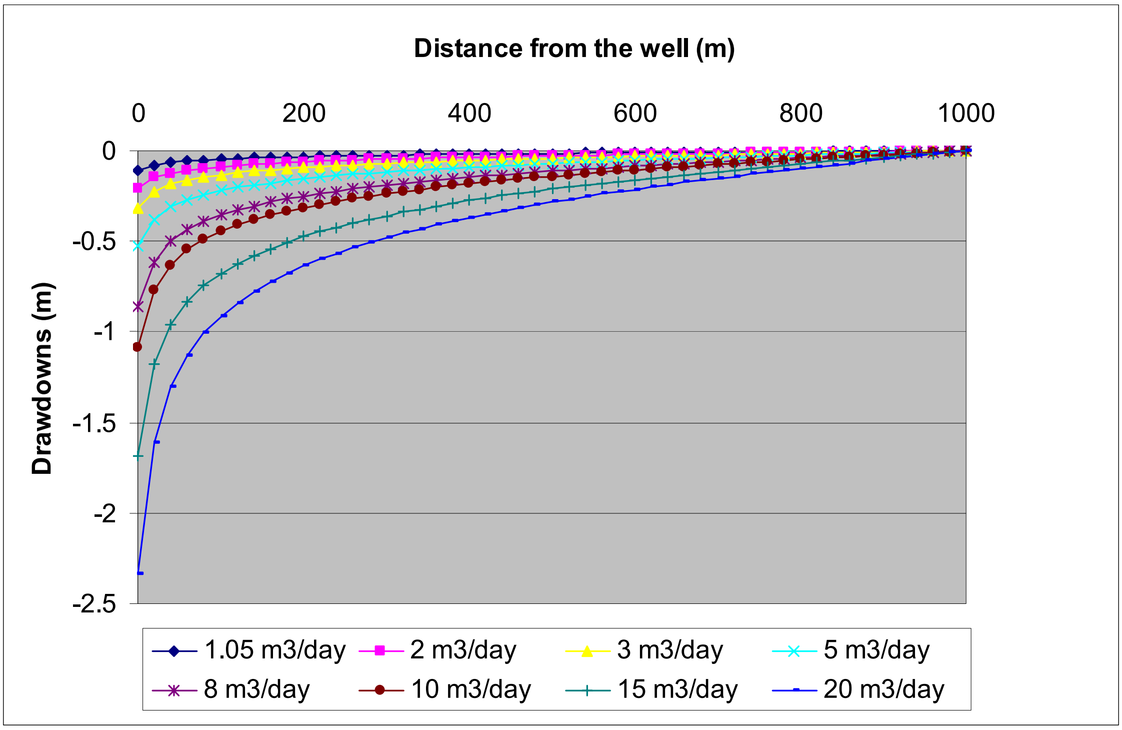

Figure 3.11. Drawdowns with the distance to the well for different pumping rates values

To quantify (approximately) the benefit of building the dams, wells can be sited in the model. Keeping in mind that a family in Kitui uses around 100 l/day, it could be established how many families could be provided when pumping the extra water stored due to the dams.

The optimum location for the well could be determined having a look at the rises plan view (see Figure 3.8). A well was sited 75 m upstream of the dam and 15 m towards the riverbank (refer to Figure 3.13).

The well was assumed to be pumping during the 245 days of one longest dry season (from March to November). The head in the well after these 245 days simulation could be compared with the head in the same location at the end of the dry season in the no dam model (comparison value), which was 6.31 m. When these values were the same, "the extra water due to the dam in the well location had been pumped"; the drawdown caused by the well was equal to the rise caused by the dam in the well location. For pumping rate $Q_{p1} = 2.35\ m^3/day$, the head in the well after pumping during the 245 days (dry season) was equal to the comparison value. This would supply water for 23 families. Considering the assumed symmetry, that means 46 families. In the Table 3.1 the difference between the observed heads in the well location and the comparison value after pumping different pumping rates during the 245 days of the dry season are shown. Figure 3.12 presents the same in a curve.

	Q_p (m^3/day)					
	2	2.35	2.4	2.5	3	4
H_{well} (m)	6.48	6.3	6.28	6.22	5.89	5.25
Difference (m)	0.17	0	-0.03	-0.08	-0.42	-1.06

Table 3.1 Difference between head in the well and comparison value for different pumping rates

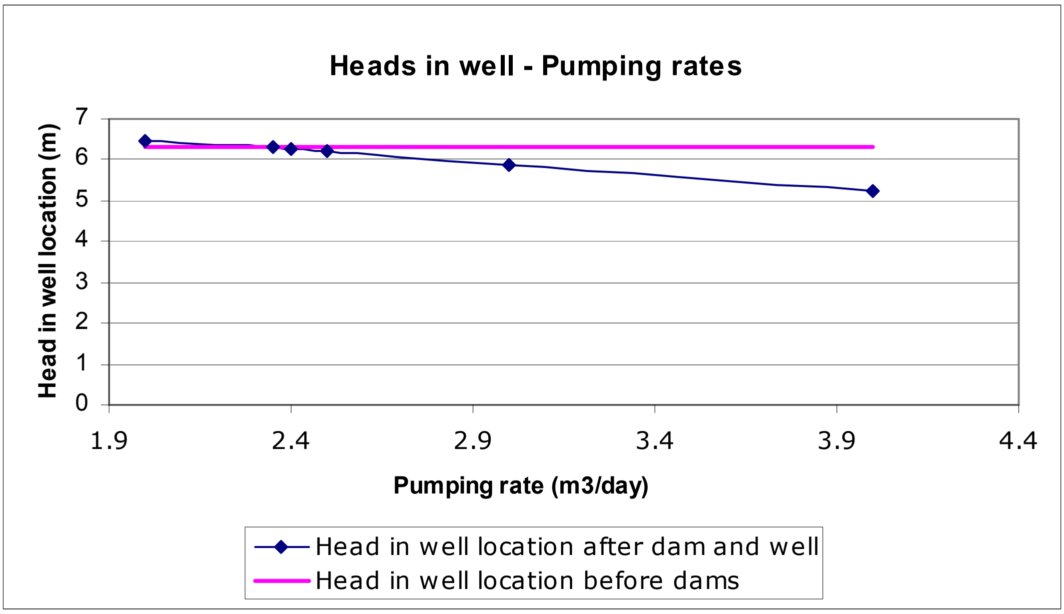

Figure 3.12. Heads in the well location – Pumping rates

When pumping Q_{p1} during 20 years, the head in the well location was 20 cm below the comparison value. When $Q_{p2} = 2.1\ m^3/day$ (21x2 families) was pumped during 20 years, the head in the well at the end of the longest dry season was equal to the comparison value. Figure 3.13 shows the drawdowns compared to the situation without dam and without well for three cases: pumping Q_{p1} during the 245 days of the longest dry season (March to November), pumping Q_{p1} during 20 years, pumping Q_{p2} during 20 years. The values shown in this figure represent the rises (negative drawdowns) caused by the combination of the well and the dam.

Comparing to *Scenario 1* in Figure 3.8, it can be seen how the drawdowns caused by the pumping well were not very substantial in the surroundings of the well, the effect was localized. The rises (negative drawdowns) caused by the dam and the well were almost the same as before setting the well. Consequently there was still a large extra water stock to be used. Several additional wells could be sited providing household water for much more people or even allowing some irrigation in the area.

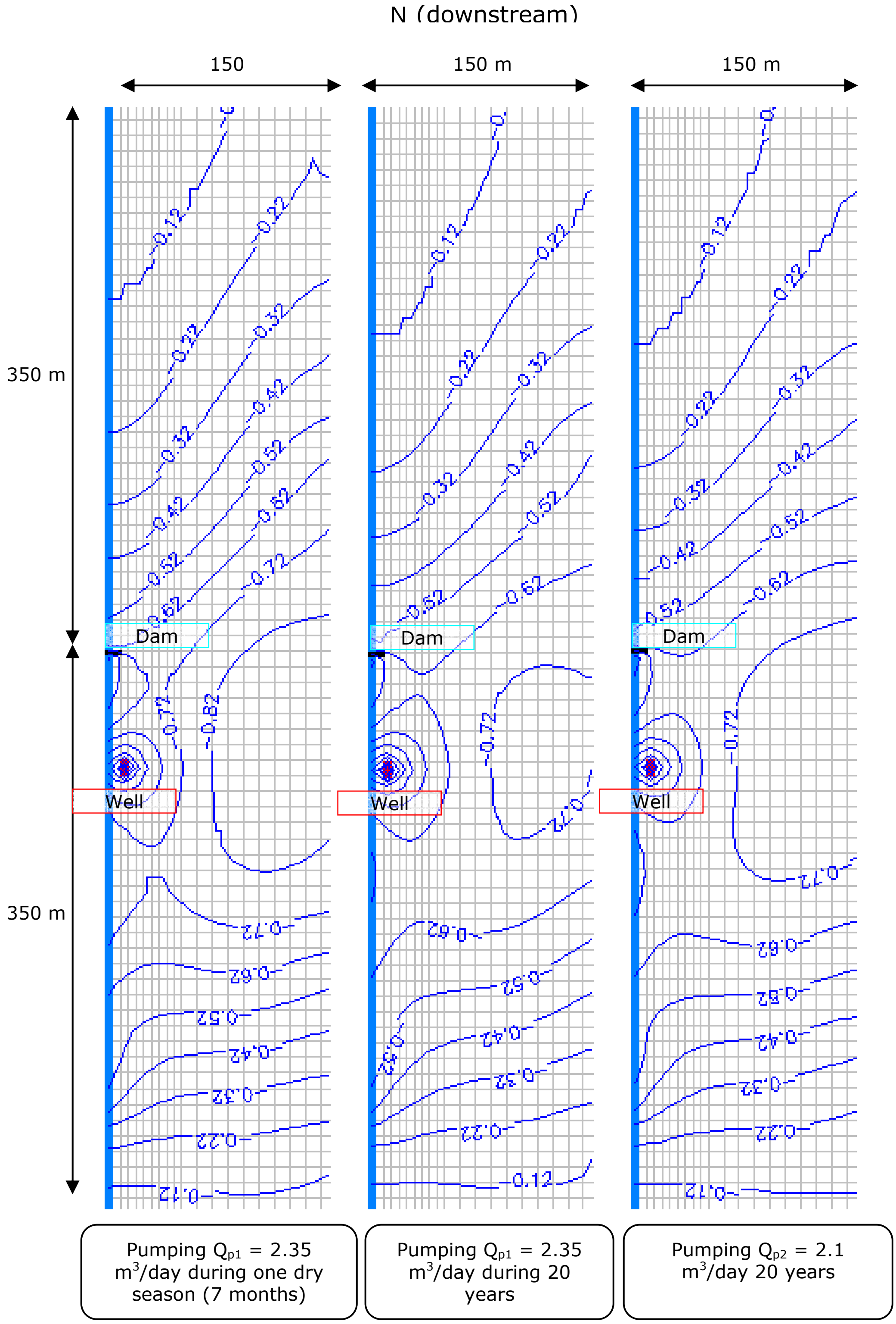

Figure 3.13. Plan view of drawdowns due to the dam and the well at the end of the longest dry season for several pumping rates

A second case was simulated adding two more wells to the model. The chosen locations for three wells can be seen in *Figure 3.14*.

The maximum pumping rates allowed before the water levels in the wells location dropped below the comparison value (value at the end of the longest dry season previously to the construction of the dams) were determined for two situations:

A. Pumping during one longest dry season (March to November)

B. Pumping during 20 years

When pumping during 245 days of the longest dry season, the maximum allowed pumping rates in each of the three wells were respectively:

1. $Q^{w1}_{pA} = 1.45\ m^3/day$
2. $Q^{w2}_{pA} = 2.3\ m^3/day$
3. $Q^{w2}_{pA} = 1.4\ m^3/day$

When pumping during 20 years, the maximum allowed pumping rates in each of the three wells were respectively:

1. $Q^{w1}_{pB} = 0.65\ m^3/day$
2. $Q^{w2}_{pB} = 2\ m^3/day$
3. $Q^{w2}_{pB} = 0.7\ m^3/day$

Figure 3.14 presents the drawdowns compared to the situation without dam and without wells for the two simulated situation. Again, the values shown in this figure were the rises (negative drawdowns) caused by the combination of the wells and the dam. Comparing to *Scenario 1* in Figure 3.8, it can be seen how the drawdowns caused by the wells had still a localized effect, there was a large extra water stock to be used.

To better quantify the available pumping water in the system, additional wells or either higher pumping rates could still be considered. In such case, even if the heads in the wells location were below the situation without dams and wells, larger drawdowns in the aquifer would occur profiting from the water stock caused by the dam.

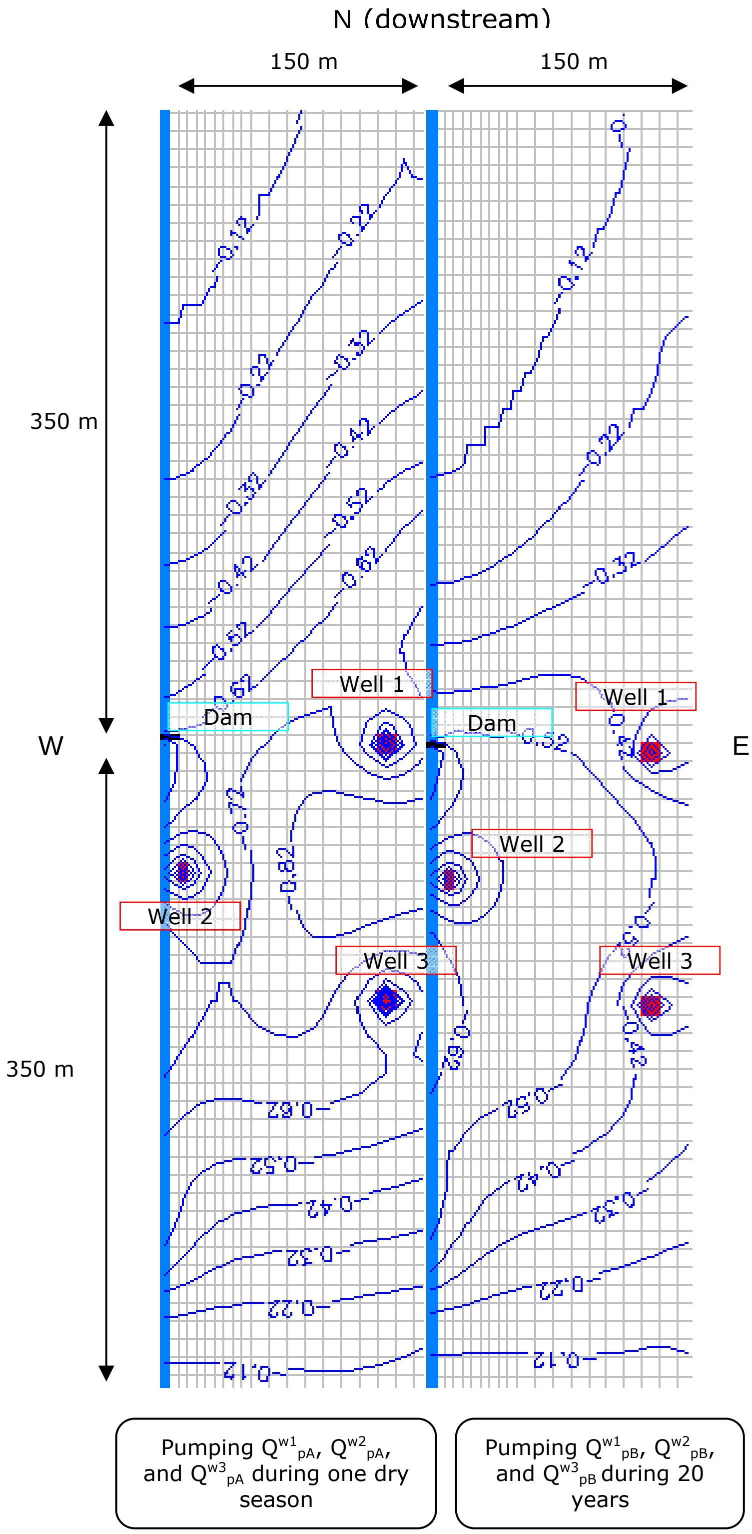

Figure 3.14. Plan view of drawdowns due to the dam and the wells at the end of the longest dry season for several pumping rates in three wells

Scenario 3

As pointed before when studying this scenario for the case without dam, the lowest the hydraulic conductivity was, the slower the water flowed, the inertia of the system was larger. Consequently a longer time was needed to reach the equilibrium in the general trend of the head levels.

At the end of the longest dry season, due to the dam, the rises in the heads in the three boreholes location were (compare the situation with and without dam for this scenario, Figure 3.3 and Figure 3.15):

- Borehole in the riverbed: 2 m
- Borehole 25 m towards the riverbank: 1.1 m
- Borehole 75 m towards the riverbank: 0.8 m

So, on the long term, the rises just upstream of the dam, where the maximum values were observed, were larger than for *Scenario 1*. The low hydraulic conductivity value made the flow of the water retained behind the dam more difficult. On the other hand, this difficulty for water to flow made the influence area to be smaller; the maximum distance from the dam to which rises were observed was minor. Considering as affected by the dam the area where the rises were larger than 10 cm, the influence area extended approximately 100 m towards downstream and 300 m towards upstream (see Figure 3.8 *Scenario 3*).

For certain time step, the head variation in the space was sharper than for more permeable scenarios, consequently in Figure 3.8 the level lines representing the rises (negative drawdowns) with an equidistance of 10 cm for *Scenario 3* are closer to each other than for *Scenario 1*.

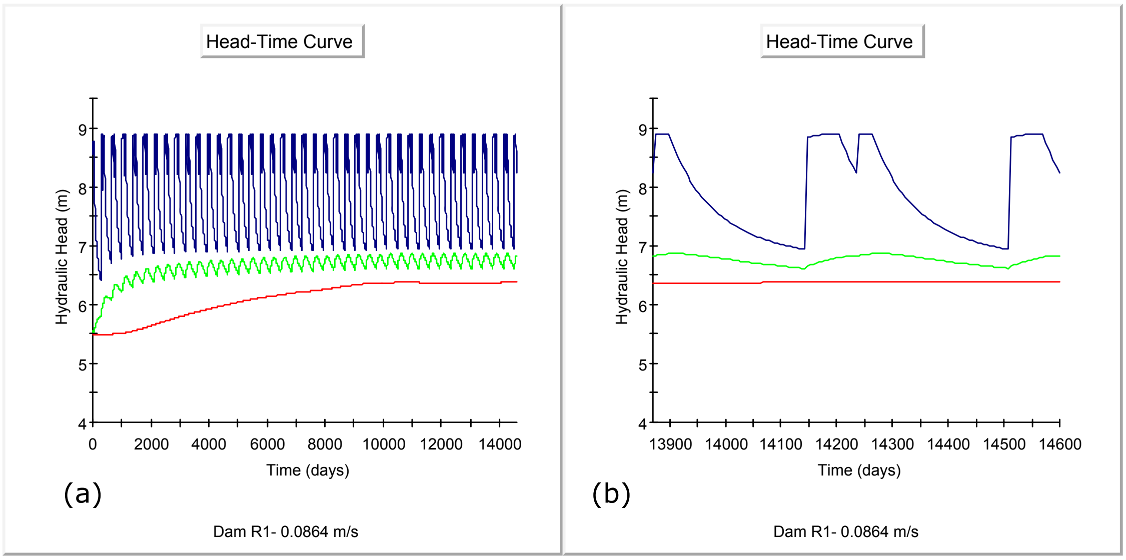

Figure 3.15. Head-time curves for One dam case, Scenario 3

Scenario 4

Higher hydraulic conductivity made the response of the system to changes faster (see Figure 3.16). A shorter time was needed to reach the equilibrium in the general trend of the head levels.

During the wet seasons a larger volume of that water retained behind the dam infiltrated from the riverbed towards the riverbank. Due to the ease of water to flow the heads were higher than in the previous scenarios and the influence of the dam reached further. Considering as affected by the dam the area where the rise were larger than 10 cm, the influence area expended approximately 450 m towards downstream and 350 m towards upstream (see Figure 3.8 *Scenario 4*). On the other hand the infiltrated water also flowed easily downstream in the dry seasons.

The rises of the heads levels due to the construction of the dam at the end of the longest dry season were about 0.5 m in the three boreholes (compare the situation with and without dam for this scenario, Figure 3.4 and Figure 3.16).

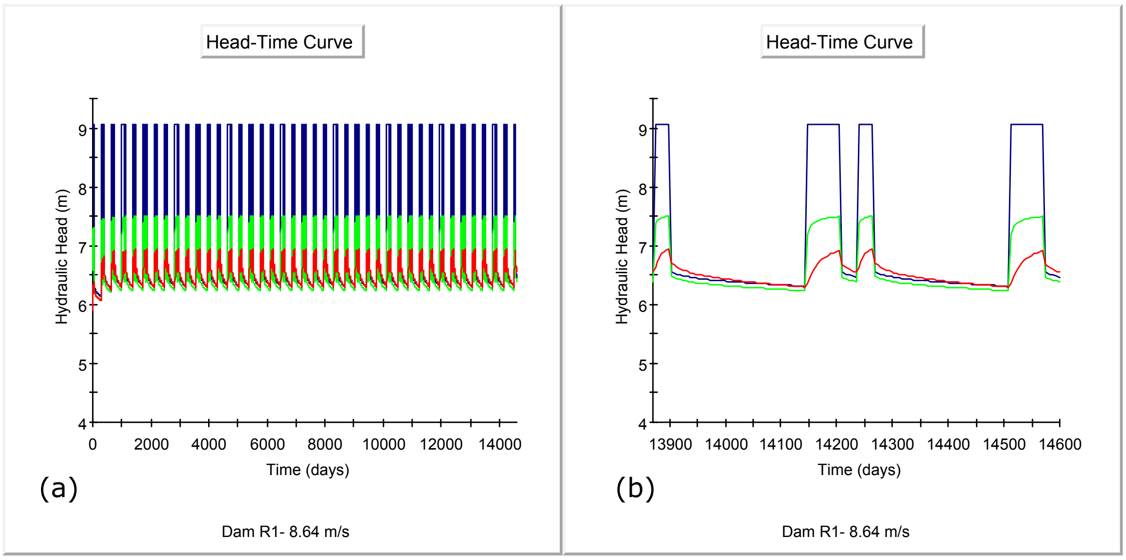

Figure 3.16. Head-time curves for One dam case, Scenario 4

When the permeability was high the changes in the heads over time for certain location were sharper than for lower *k* values (compare Figure 3.16 with Figure 3.15) due to the ease of water to flow. On the other hand, for certain time step, the variation in the heads over the space was gentle. The effects of the dam could be observed further to the dam but the magnitude of these effects was lower (refer to Figure 3.8).

Figure 3.17 presents the rises in the riverbed for *scenarios 1*, *3* and *4* in the shallow system, which is for sand hydraulic conductivities of 0.864 m/day, 0.0864 m/day and 8.64 m/day respectively.

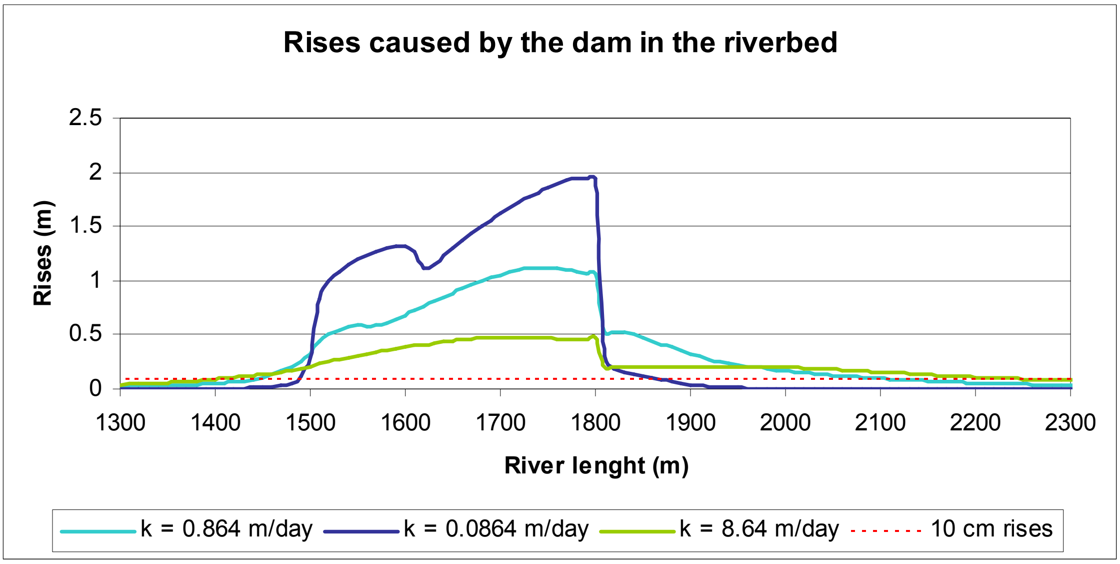

Figure 3.17. Rises in the river bed due to the dam for scenarios 1, 3 and 4 in the shallow system

The effects of the hydraulic conductivity in the rises caused by the dam can be seen in Figure 3.8 and Figure 3.17. In Figure 3.17 the rises caused by the dam in the riverbed for *Scenarios 1, 3* and *4* are shown. Lower *k* values produce higher rises immediately upstream of the dam but the influence area is smaller. This can also be seen in Figure 3.8; the lower *k*, the higher the rises close to the dam and the minor the distance between rise level lines.

Figure 3.18 presents a conceptual summary of the influence of the permeability in the damming effect (considering the end of the dry season). The optimum *k* value to store the larger water volume in the aquifer would be that which made maximum the area below the curves rise-distance to the dam. The water volume stored in the system due to the dam was measured for the three studied *k* values at the end of the longest dry season 40 years after the dam was built. In this case, the maximum water volume was stored for the lowest k value (see Table 3.2)

Water gain in the model due to the dam		
Scenario	k (m/day)	ΔV (m^3)
1	0.864	21000
3	0.0864	43000
4	8.64	12000

Table 3.2. Water volume stored in the aquifer due to the dam

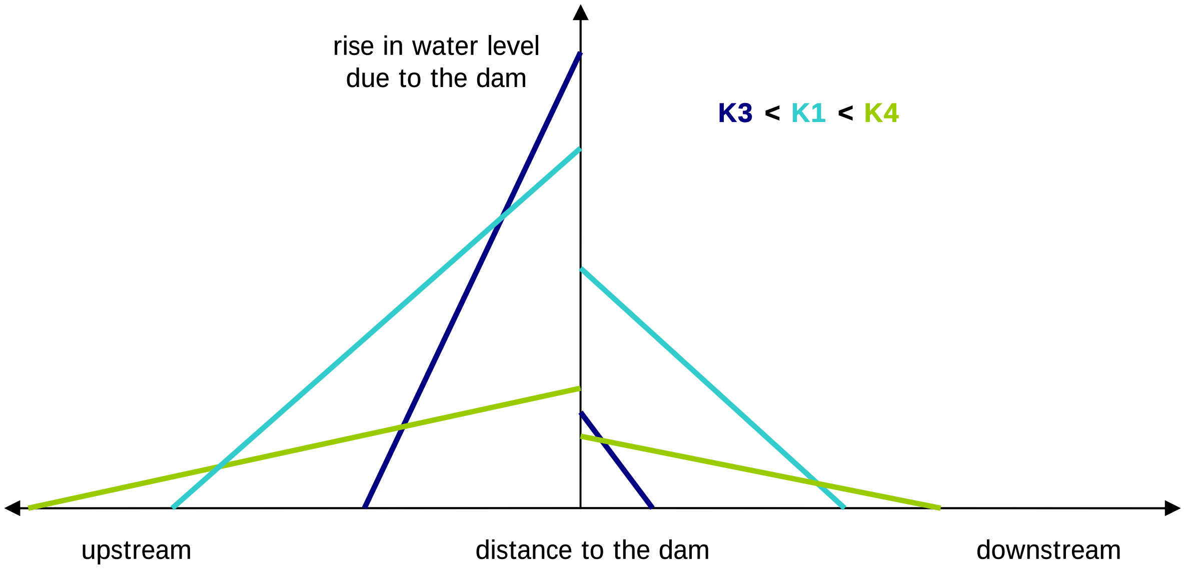

Figure 3.18. Scheme of the influence of permeability in the damming effects

Scenario 5

When the slope in the river length direction was steeper, the wedge of sediments carried by the river and deposited behind the dam was smaller, which means that the artificial aquifer created by the sand storage dam was minor. Furthermore, the flow downstream was easier.

As a result, the distance to which the influence of the dam could be noticed were shorter and the rises in water levels caused by the dam were also lower (see Figure 3.19 and Figure 3.20).

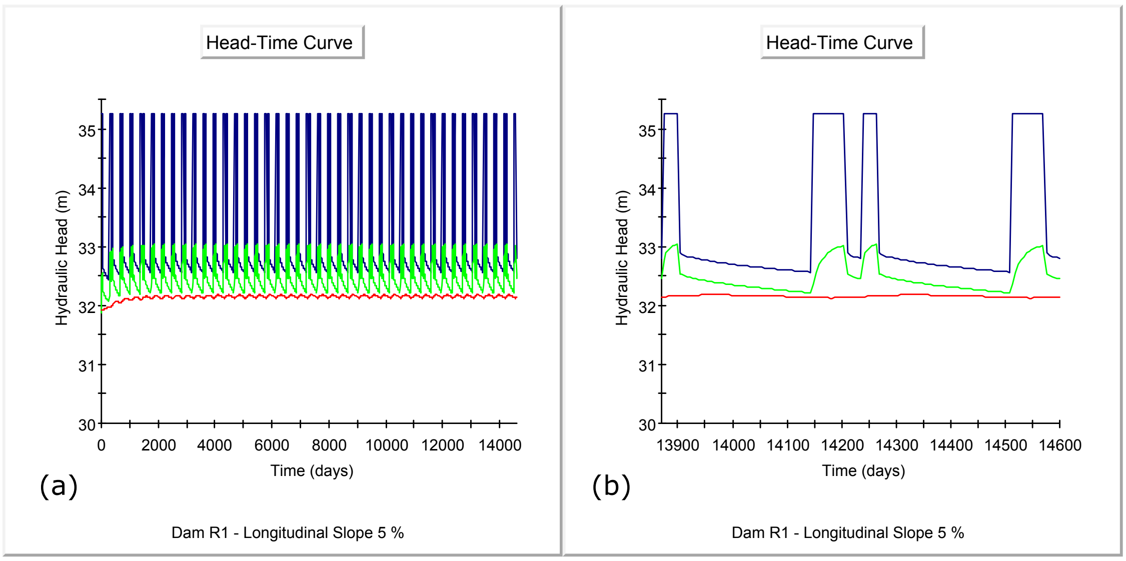

Figure 3.19. Head-time curves for One dam case, Scenario 5

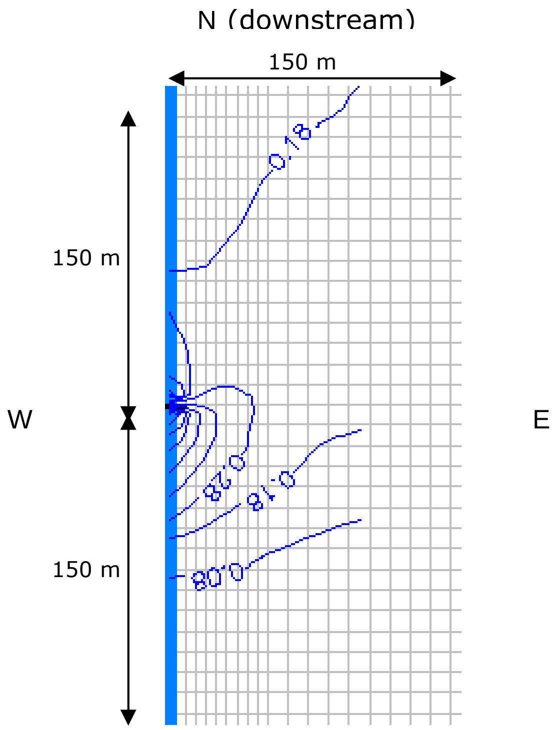

Figure 3.20. Plan view of drawdowns due to the dam at the end of the longest dry season in One dam case, Scenario 5

3.1.3. Three dams model

Scenario 1

The purpose of the *Three dams model* was to analyze the effects of a networks of dams in the water levels. When analyzing the *One dam model*, the influence area (considered as that in which the rises in the heads due to the dam were larger than 10 cm) was found to extent around 350 m towards downstream and upstream of the dam. In Figure 3.21 a plan view of the rises (negative drawdowns) at the end of the longest dry season caused by the dam is shown for the single dam and for each dam of the *Three dams model*. When studying the dams' network, two questions could be raised: was there any overlap between the influence areas of the dams? In such case, did synergic effects take place or the result was three times the case of the single dam?

In the present case (shallow system) as the distance between dams was 500 m and the influence area for a single dam was 350 m towards downstream upstream of the dam, there was overlap of influence areas. In Figure 3.22 a scheme of these overlaps is presented. Certain synergic effect was expected due to the fact that the water stored in the downstream rises of one dam, instead of flowing freely downstream, was obstructed by the next dam.

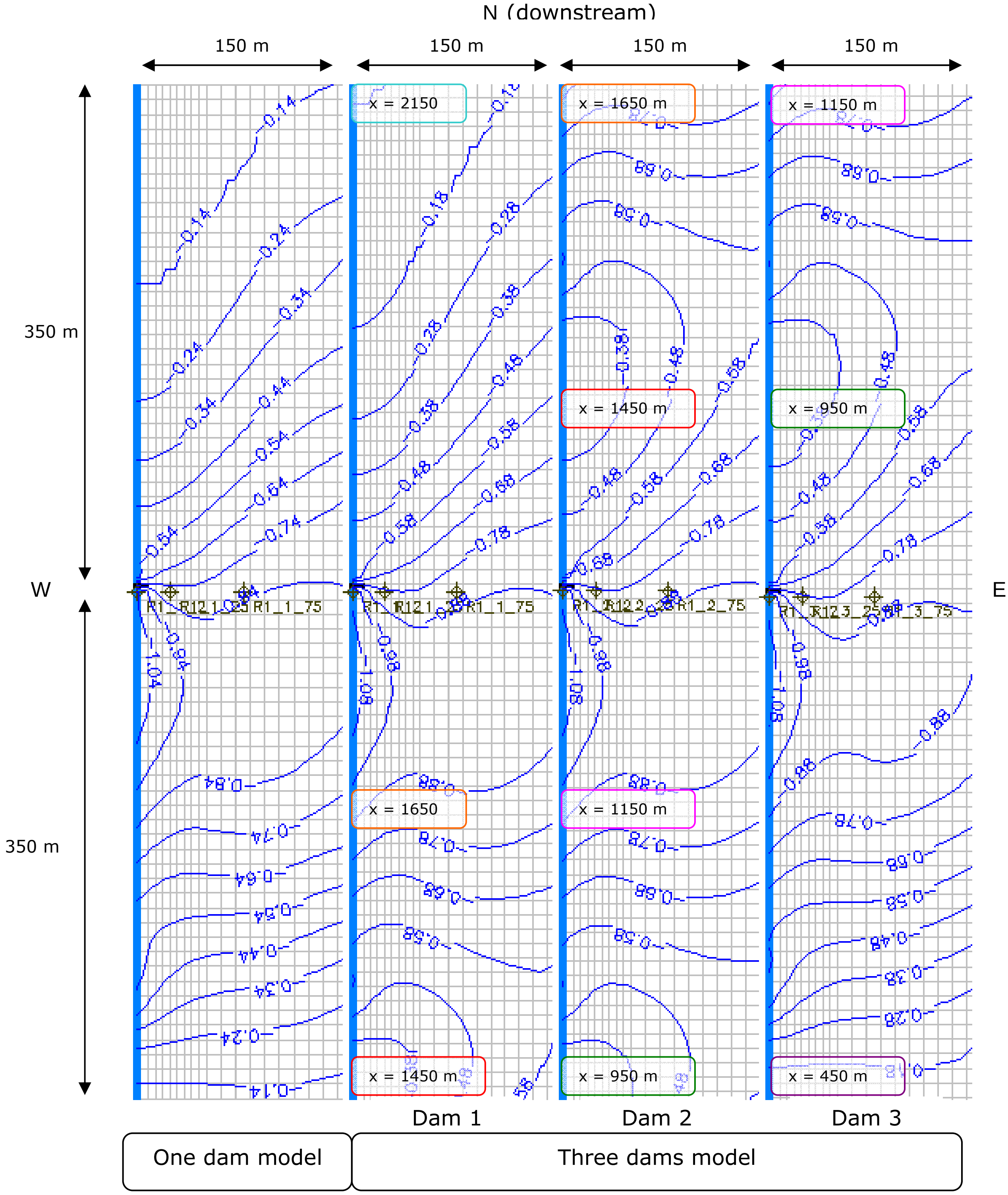

Figure 3.21. Plan view of the drawdowns due to the dam at the end of the longest dry season for the One dam and Three dams cases in Scenario 1

The observations in the boreholes rows *R1_1* (upstream of *Dam 1*), *R1_2* (upstream of *Dam 2*) and *R1_3* (upstream of *Dam 3)* showed that, the water thickness levels in the vicinity of the three dams were the same (see Figure 3.23 where the observations in the boreholes rows *R1* (upstream of the dams) are shown). At the same time the observed water levels behind the dam coincided with the observations of the *One dam model*. It was in this location, immediately upstream the dam, where the rises were maximum. However, in the influence area of each dam the rises were higher than for the *One dam model* not only in the overlapping area but also in that area belonging only to the influence area of one dam.

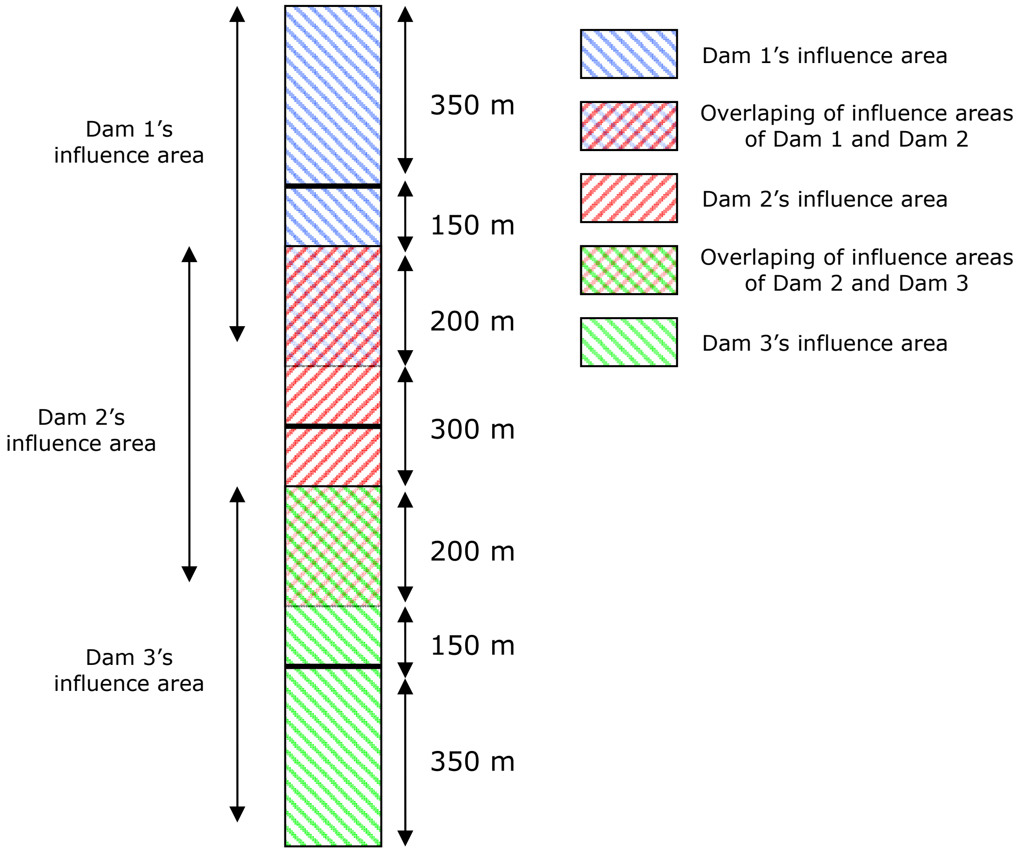

Figure 3.22. Influence areas of each dam in the Three dams model

Thus, whereas behind each dam, where the maximum rises were observed, the rises were the same as for the single dam, in the model domain, in general, the water levels were higher. In Figure 3.21 it can be seen how upstream of *Dam1* and *Dam 2* the separation between level lines were larger than in the single dam case, which means gentler variations in the drawdowns.

Furthermore, downstream of *Dam 1* and upstream of *Dam 3* the effects of the dams were more significant than in the single dam case; the rises at the same distance to the dams were higher and the influence extension was larger.

As it can be seen in Figure 3.21 the heads in the areas between *Dam 1* and *2* and between *Dam 2* and *3* were the same. If the dams' network was formed by more than three dams, the expected results would be the same. Thus by analyzing a system with three dams, the case of *n* dams in a row with this conditions separated 500 m has been studied. The number of dams which must be modelled in order to study the performance of a network of dams in a certain environment depends on the overlapping areas, that is, if the influence areas of *m* dams overlap, modelling results for *m* dams are reasonably comparable to results for *n* dams ($m<n$).

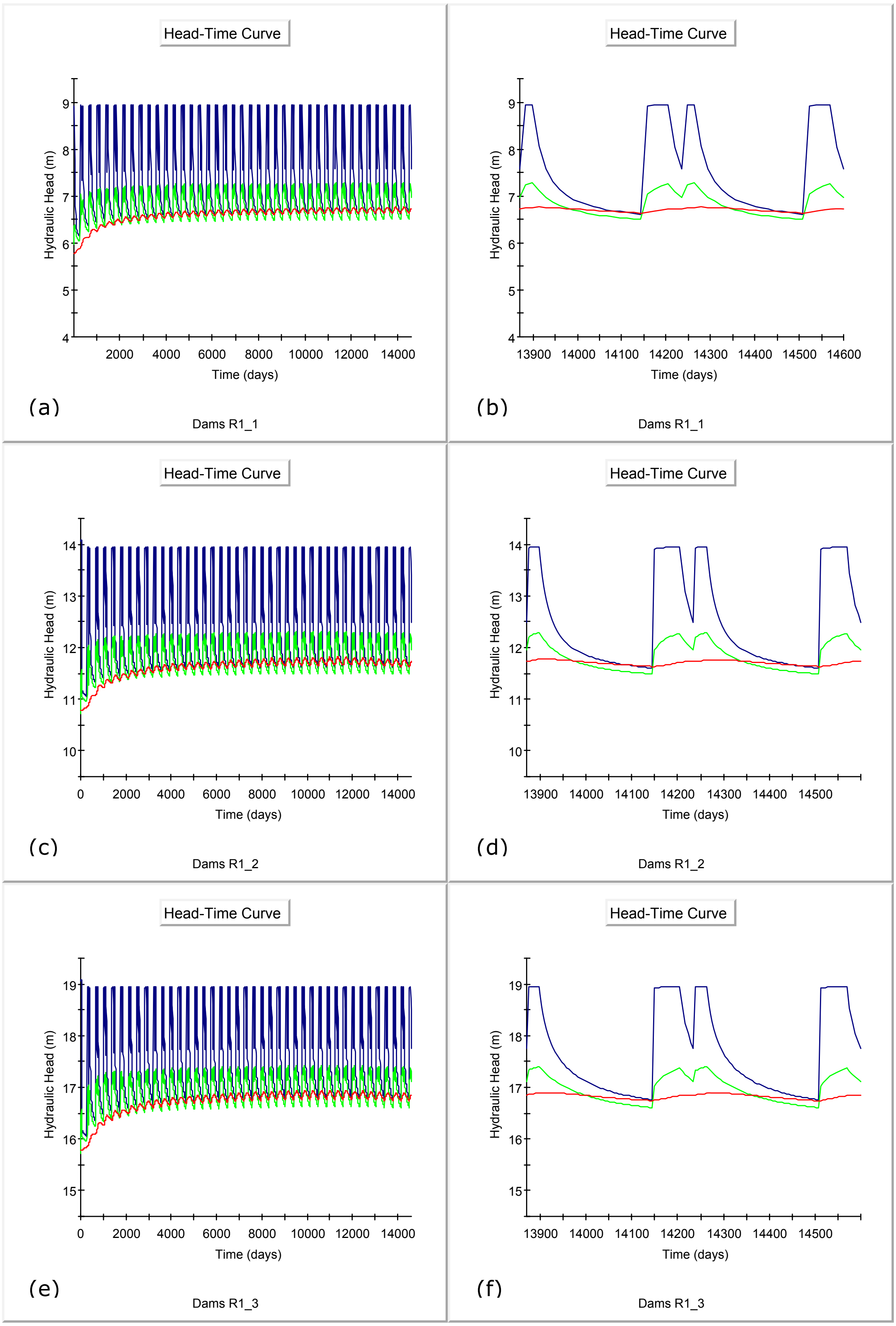

Figure 3.23. Head-time curves for observations in boreholes row R1_1 (upstream of the Dam 1) in the Dams case for Scenario 1, Scenario 3 and Scenario 4

One of the goals of this research was to determine under which conditions several dams behave as a connected system rather than as individual structures. Following, a qualitative analysis of the influence of the distance between dams and the hydraulic conductivity is made.

Given certain dam located in certain environment, two lengths can be established, one in the downstream direction ($L_{d\ max}$) and another one in the upstream direction ($L_{u\ max}$), such that for greater lengths there are no rises in the water levels due to the dam. This means that $L_{d\ max}$ and $L_{u\ max}$ are the limits of the influence area in the river length direction. Curves determining the rises-distance from the dam could be built; an example of a hypothetic case is shown in Figure 3.24. When the distance between dams is less than $L_{d\ max} + L_{u\ max}$, an overlapping between the influence area of two successive dams takes place and the dams behave as a connected system (see Figure 3.25). When the distance between successive dams is longer than $L_{d\ max} + L_{u\ max}$, no interaction is expected and the dams behave as individual structures causing localized effects.

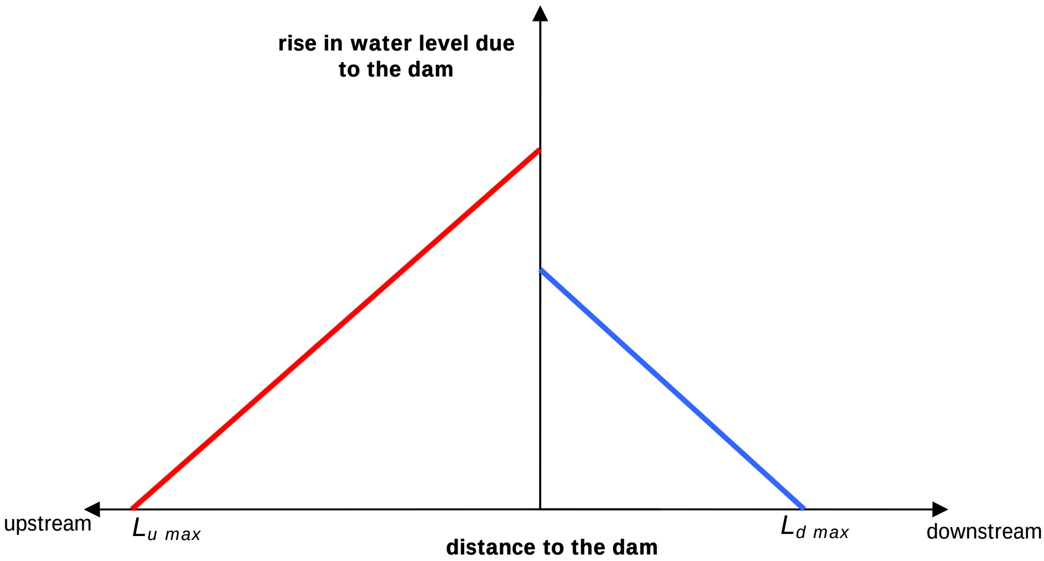

Figure 3.24. Head rise due to the dam - distance to the dam

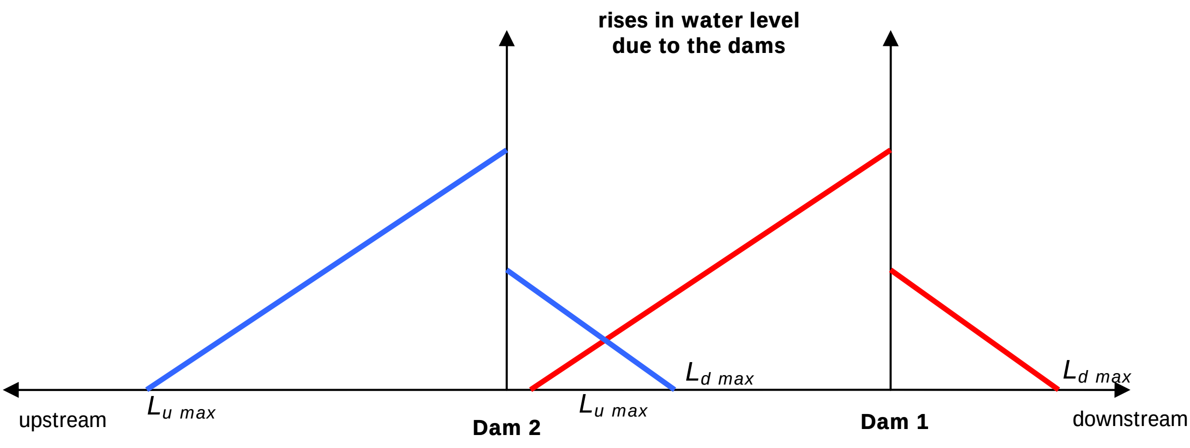

Figure 3.25. Overlapping of influence areas in two successive dams

The influence of the hydraulic conductivity can be explained by a combination of graphs (Figure 3.18 and Figure 3.24). The hydraulic conductivity is one of the factors which would determine the shape of the *rises – distance to the dam* curves, and thus would determine $L_{d\ max}$ and $L_{u\ max}$. As said before, the lower the hydraulic conductivity, the higher origin ordinate and the steeper the curve, which would give shorter $L_{d\ max}$ and $L_{u\ max}$. Shorter distances between dams would be required to assure the interaction between influence areas; however, in those cases in which the interaction occurred, the rises for points close to the dam would be higher (see Figure 3.24).

Others factors affecting the curves shape would be the dam's height, the permeability of the bedrock and the slope in the river length direction. The influence of dam's height and rockbed permeability are discussed later in the text. Concerning to the slope, as mentioned before when studying *scenario 5*, the steeper the slope was, the lower the rises and the shorter the influence area.

Scenario 3

In the current scenario the sand hydraulic conductivity was changed to 0.0864 m/day, which is ten times lower than the assumed value in *Scenario 1*.

In this case, as expected, the influence area for the single dam was lower than in *Scenario 1*. $L_{d\ max}$ could be established as 100 m and $L_{u\ max}$ as 300 m (see Figure 3.26, *One dam model*). Since $L_{d\ max}$ + $L_{u\ max}$ is lower than the distance between dams (500 m), accordingly to what was said, before no interaction was expected. The obtained results confirmed these thoughts as it can be observed in Figure 3.26. That is, the dams in the network behaved exactly the same as the single dam. It could be said that the *Three dams model* was three times the *One dam model*.

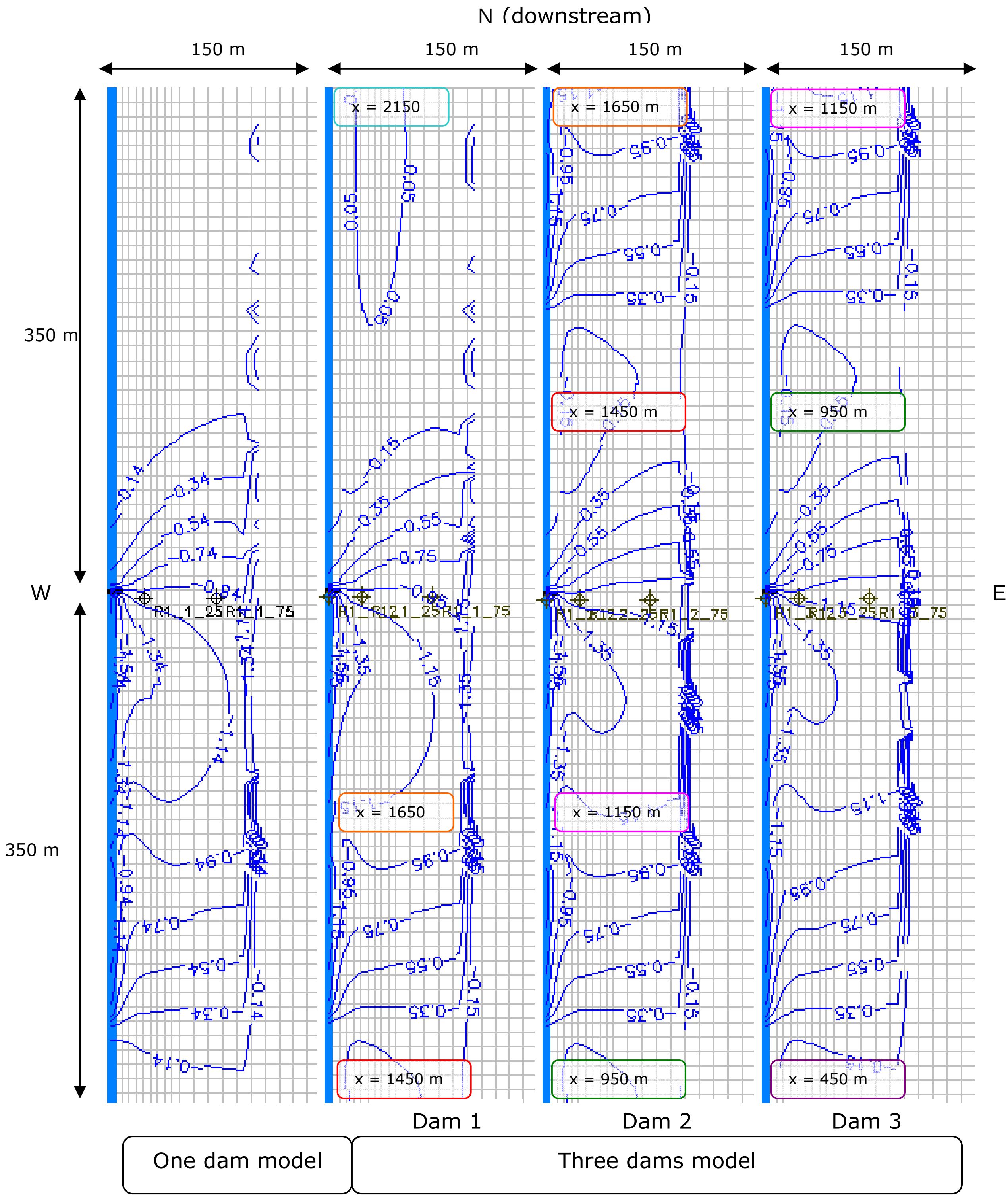

Figure 3.26. Plan view of the drawdowns (due to the dams) at the end of the longest dry season for the One dam and Three dams case in Scenario 3

Scenario 4

In the current scenario the sand hydraulic conductivity was changed to 8.64 m/day, which is ten times more permeable than the assumed value in *Scenario 1*. In this case, the influence area of three successive dams overlap.

When comparing with *Scenario 1* it was observed (see Figure 3.21 and Figure 3.27):

- In the areas close to the dams, the difference in heads between the one dam model and the three dams model was larger. This is because of larger values of $L_{d\ max}$ and $L_{u\ max.}$
- In the areas *"more equidistant"* to both successive dams, the difference between observed levels in both models were minor. This is because in this areas overlapping took place for both *k* values (0.864 m/day and 8.64 m/day), whereas the rises caused by a single dam were larger.

These observations confirmed what was concluded before; the higher the hydraulic conductivity, the lower the rises and the longer the influence area.

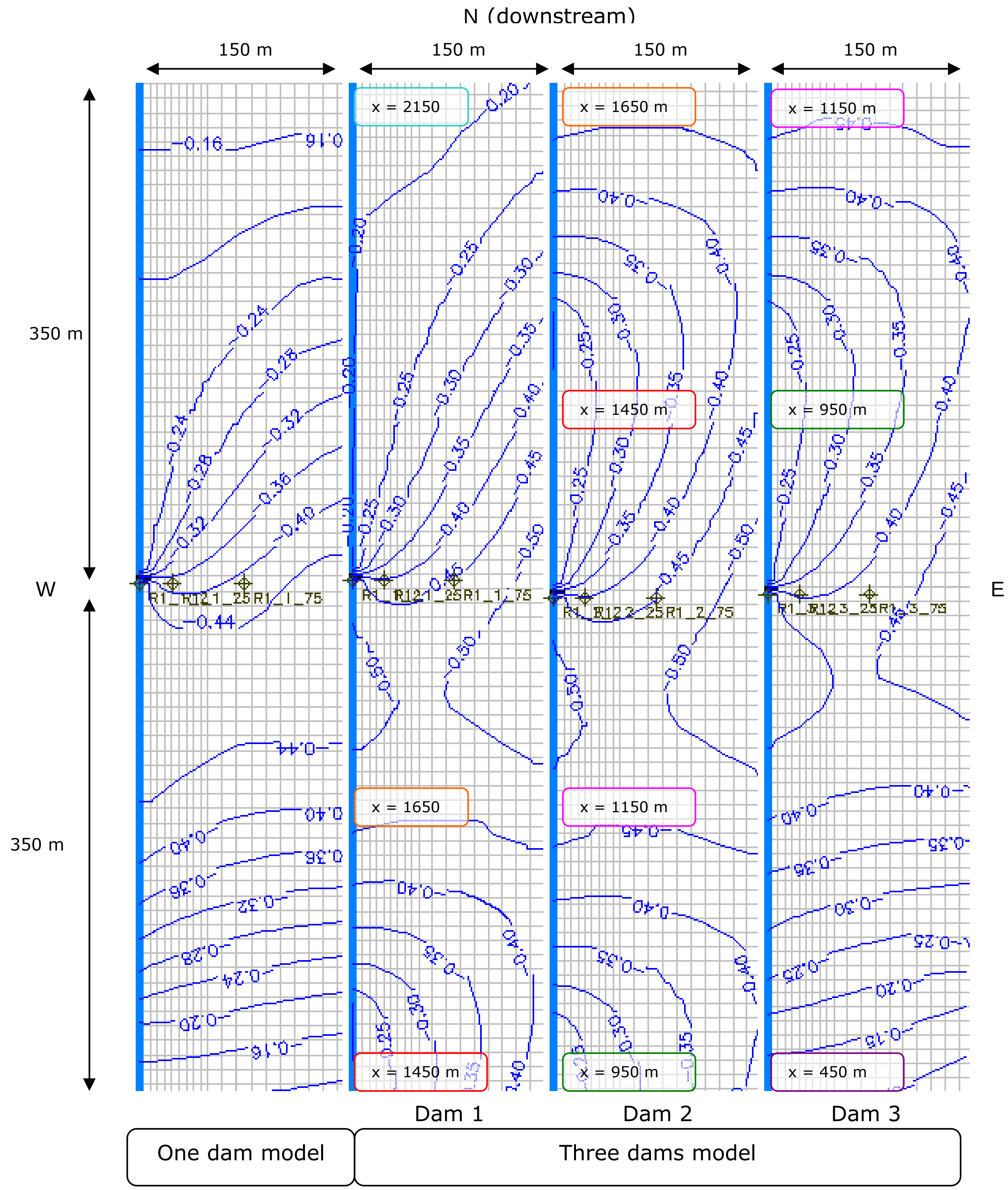

Figure 3.27. Plan view of the drawdowns due to the dams at the end of the longest dry season for the One dam and Three dams cases in Scenario 4

3.2. Deep system

For the analysis of the *deep system* all the boreholes were assumed to have a depth of 4 m, that is, they were located in Layer 2.

3.2.1. No dams case

Scenario 1

When the aquifer thickness was enlarged to 52 m instead of the 2 m assumed before, the way water from the riverbed discharged in the riverbanks changed. The segment of the head-time curves which goes from the time 14268 days to 14508 days for the borehole located in the riverbed (see Figure 3.6 (b) and Figure 3.29 (b) and Figure 3.28) represents this discharge in the longest dry season, that is, from March to October. In the *shallow system* water in the riverbed (in *Layer 2*) discharged in the riverbanks. However in the *deep system*, water in the riverbed in *Layer 2* discharged in the riverbanks (filtration) but also in the deeper *Layer 3* (percolation). As a result, the discharge speed (curve slope) at the beginning of the dry seasons was higher in the *deep system*. Although, after certain time, due to the discharge, the heads in *Layer 2* were lower than in *Layer 3*, thus the flow between both layers changed its sense, and started to flow from *Layer 3* to *Layer 2*. From that moment on, the discharge speed in the riverbed in *Layer 2* decreased because the drawdowns were only due to the filtrations on the riverbanks whereas the vertical flow was filling *Layer 2*.

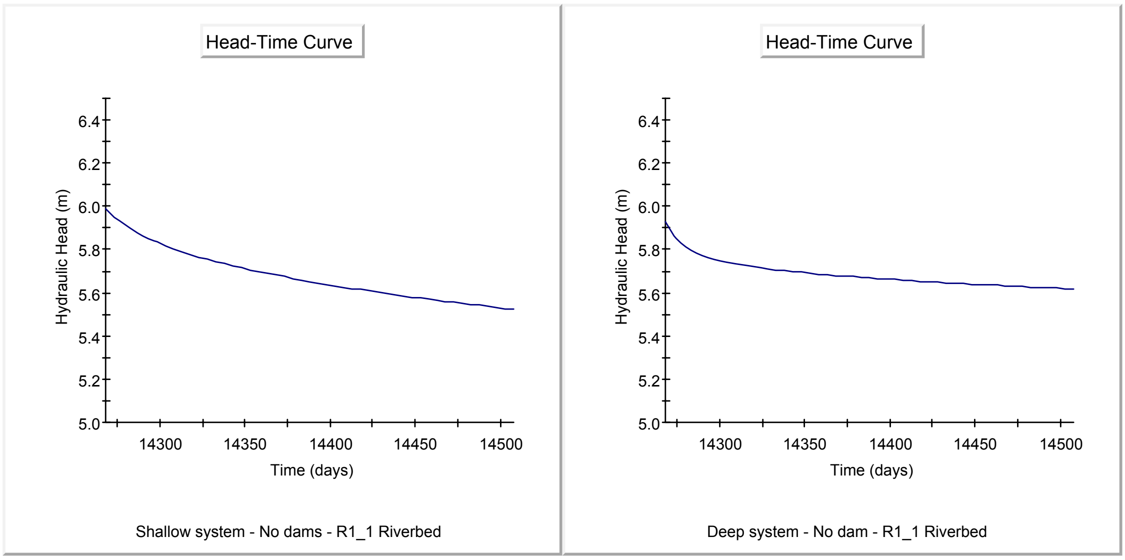

Figure 3.28. Head-time curves observed during the longest dry season in the riverbed in the No dam case for the shallow and deep system.

The water volume infiltrated and stored in the aquifer during the rainy season was higher than in the previous system. That was because there was water infiltrating from the riverbed towards the riverbanks but also towards the deeper layer. It means that for certain rainfall event, the thicker the aquifer is, the more water will infiltrate and store in the aquifer and the less surface-water will flow downstream in the river. As a result the heads during the dry seasons were higher in the *deep system* than in the *shallow system*.

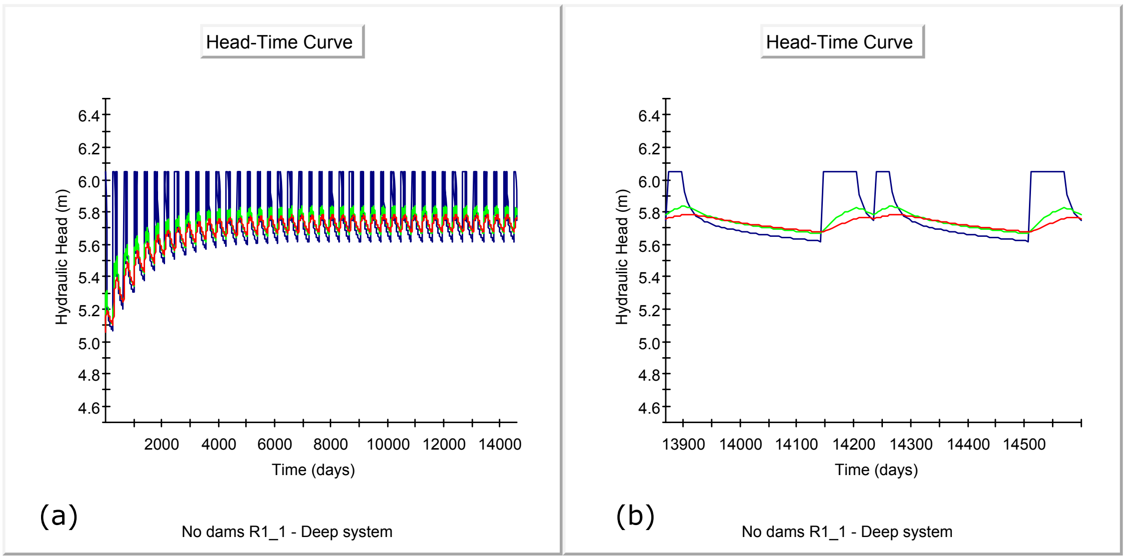

Figure 3.29. Head-time curves for observations in boreholes row R1_1 in the No dams case for Scenario 1

As it was said for the shallow system, in the *no dam case*, head-time curves behaviour was the same for all the cross sections due to the fact that there wasn't any alteration in the river length direction. Thus, for certain time step, the water thickness didn't change throughout any column in the model domain, except in the vicinity of the Northern and Southern boundaries.

Scenario 2

The assumption of a low permeability (0.0864 m/day) bar in *Layer 3* had extraordinary damming effects in the aquifer (see Figure 3.30). The heads were higher than in *Scenario 1* in the entire model domain for every time in the year as shown in Figure 3.31.

Upstream of the low permeability bar there was a large area where the seasonal oscillations couldn't be noticed. In this area the heads in the riverbanks were higher than in the riverbed, so groundwater flowed from the banks towards the riverbed. There was surface-water flow in the river during the whole year, even during the dry seasons (the drains were working in every time step of the year).

From a certain distance upstream of the low permeability bar onwards the seasonal oscillation could be noticed. In this area the heads in the riverbanks were lower than in the riverbed, consequently water from the riverbed infiltrated towards the riverbanks dropping the heads in the river in the dry seasons.

The rises in the heads caused by the low permeability bar could be noticed more than 1000 m further than the bar location. This can be seen when comparing the heads in the borehole row *R1_3* in this scenario with *Scenario 1* (refer to Figure 3.29 and Figure 3.31).

A qualitative comparison was made between the damming effects of the low *k* bar and the studied case of a sand storage dam in the *shallow system*. In the low permeability bar case, the rises in the heads and the distance towards upstream to which the damming effects could be noticed were much larger. That illustrated how when all the other variables remain the same, the thicker the aquifer blocked by the dam, the larger the effects (higher rises and longer influence areas). Thus, given certain distance between dams, when the aquifer thickness is over certain value, there is interaction between the effects of two successive dams.

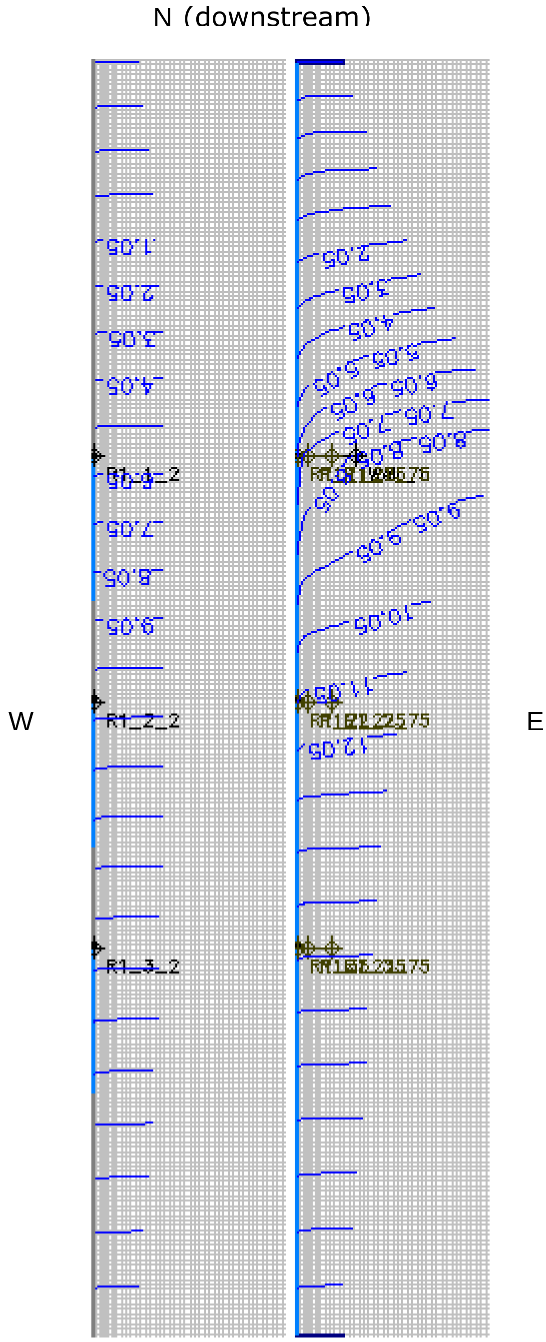

Figure 3.30. Heads at the end of the longest dry season in Layer 2 for the deep system, no dams case, Scenarios 1 and 2 (initial heads for de dams cases)

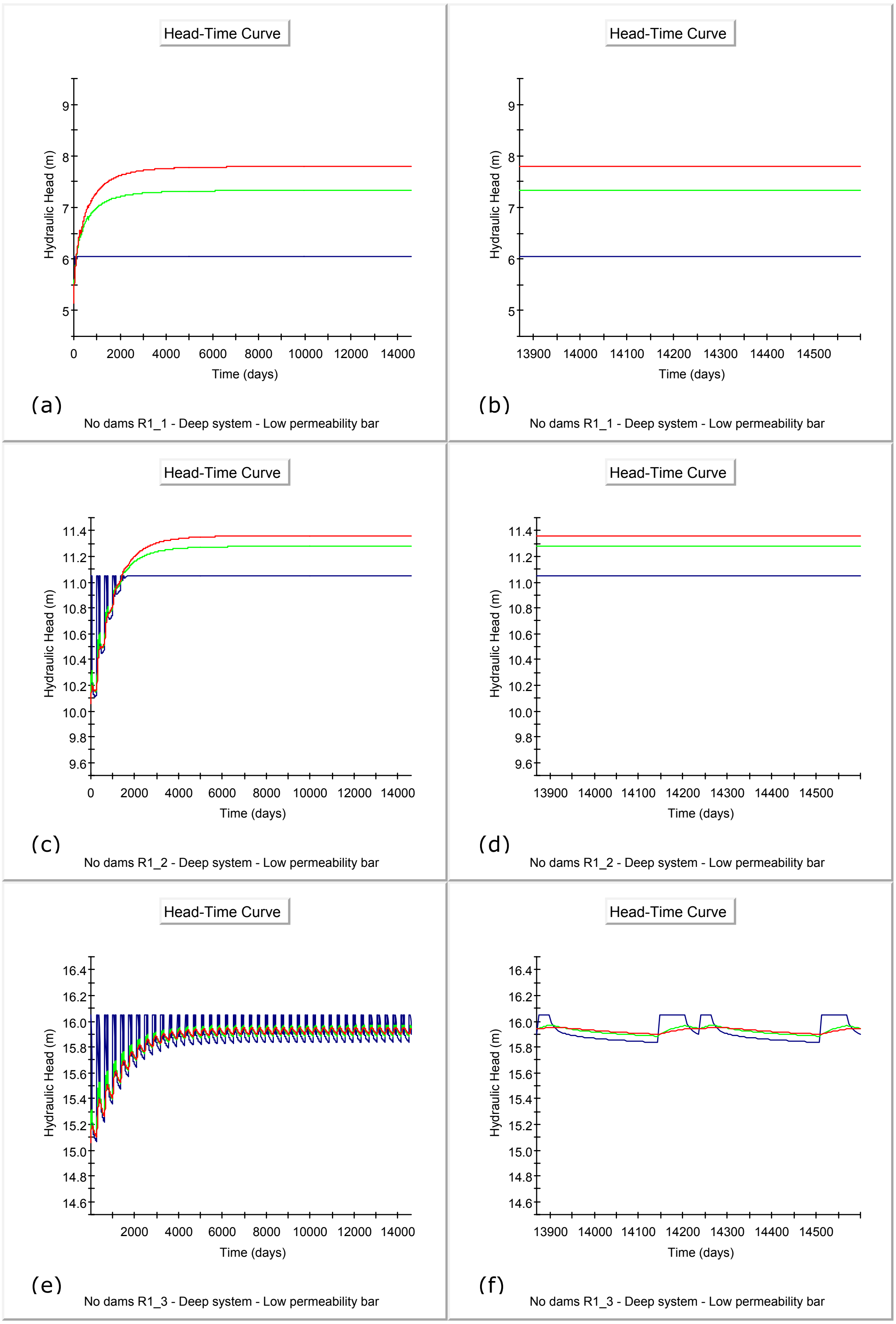

Figure 3.31. Head-time curves for observations in boreholes rows R1_1, R1_2 and R1_3 in the No dams case for Scenario 2

3.2.2. Dams case

Scenario 1

When the aquifer was deep, the rises caused by sand storage dams were lower. In the *shallow system* the dam blocked completely the groundwater flow in the riverbed towards downstream of the dam, thus the groundwater hold out behind the dam could only flow towards the riverbanks. In the *deep system* the dam only blocked the flow in the five upper metres of the aquifer (*Layer 1* and *Layer 2*) remaining 50 m between the dam's base and the rockbed; consequently the water flowed through the permeable layer below the dam towards downstream. As a result the observed rises in the heads due to the dams were lower than in the *shallow system*.

To assure the success of a groundwater dam, the layer on which the dam is built must be as impermeable as possible to minimize the leakage of groundwater below the dam. The optimum situation would be to build the dam over hard impermeable rock, which was the studied case in the *shallow system*. The lower the hydraulic conductivity of the soil below the dam is, the higher the rises caused by the dam will be.

Considering as affected by the dam the area where the rises were larger than 10 cm, the influence area extended approximately 490 m towards downstream and 820 m towards upstream (see Figure 3.32 and Figure 3.33).

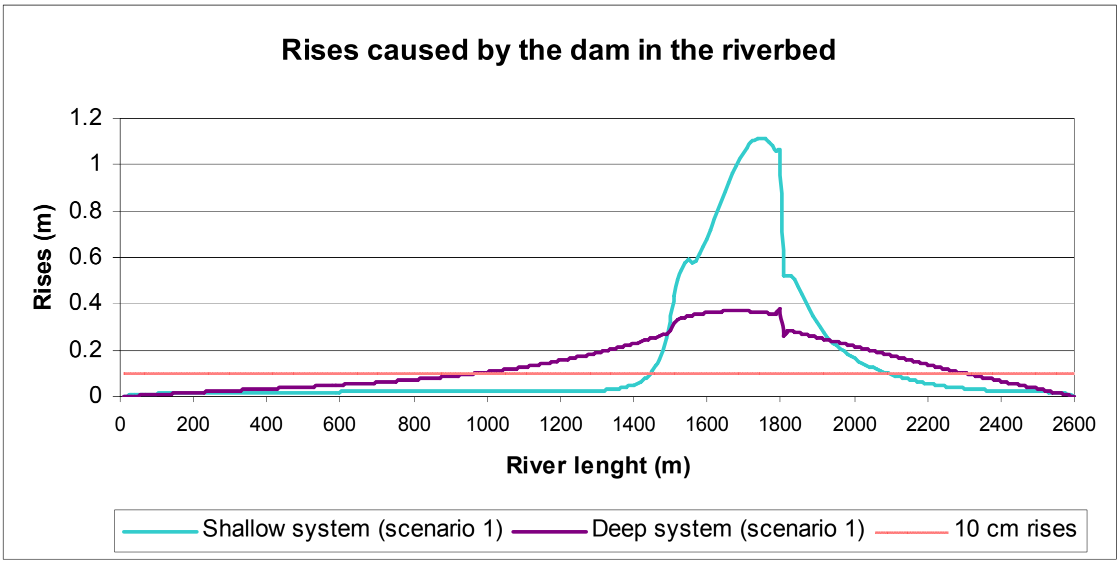

Figure 3.32. Rises in the river bed due to the dam for the shallow and deep systems

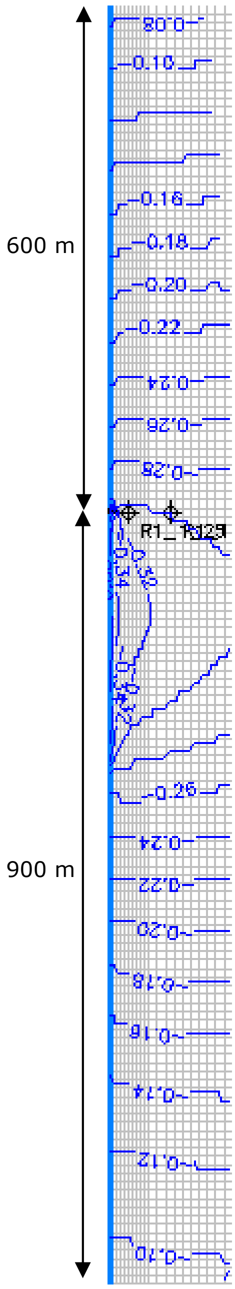

Figure 3.33. Influence area of a single dam in the deep system.

Figure 3.34 presents a scheme of the overlaps which took place when analyzing the deep system. It can be seen how the maximum number of influence area which overlapped were three, which means that modelling results for three dams were reasonably comparable to results for *n* dams. Nevertheless, since the influence areas were larger than in the previous cases, it would be a good idea to model a case with more dams to obtain better results.

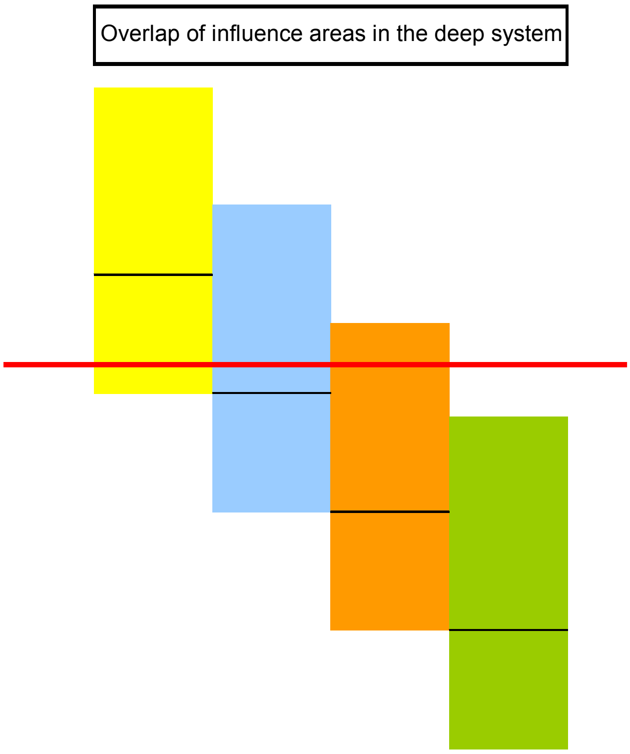

Figure 3.34. Overlap in influence areas in the deep system.

In

Figure 3.35, rises (negative drawdowns) at the end of the longest dry season caused by the dams were compared for the *one dam* and each dam of the *three dams casesl.* It can be observed how the interaction between dams extended longer though the rises were lower than in the shallow system.

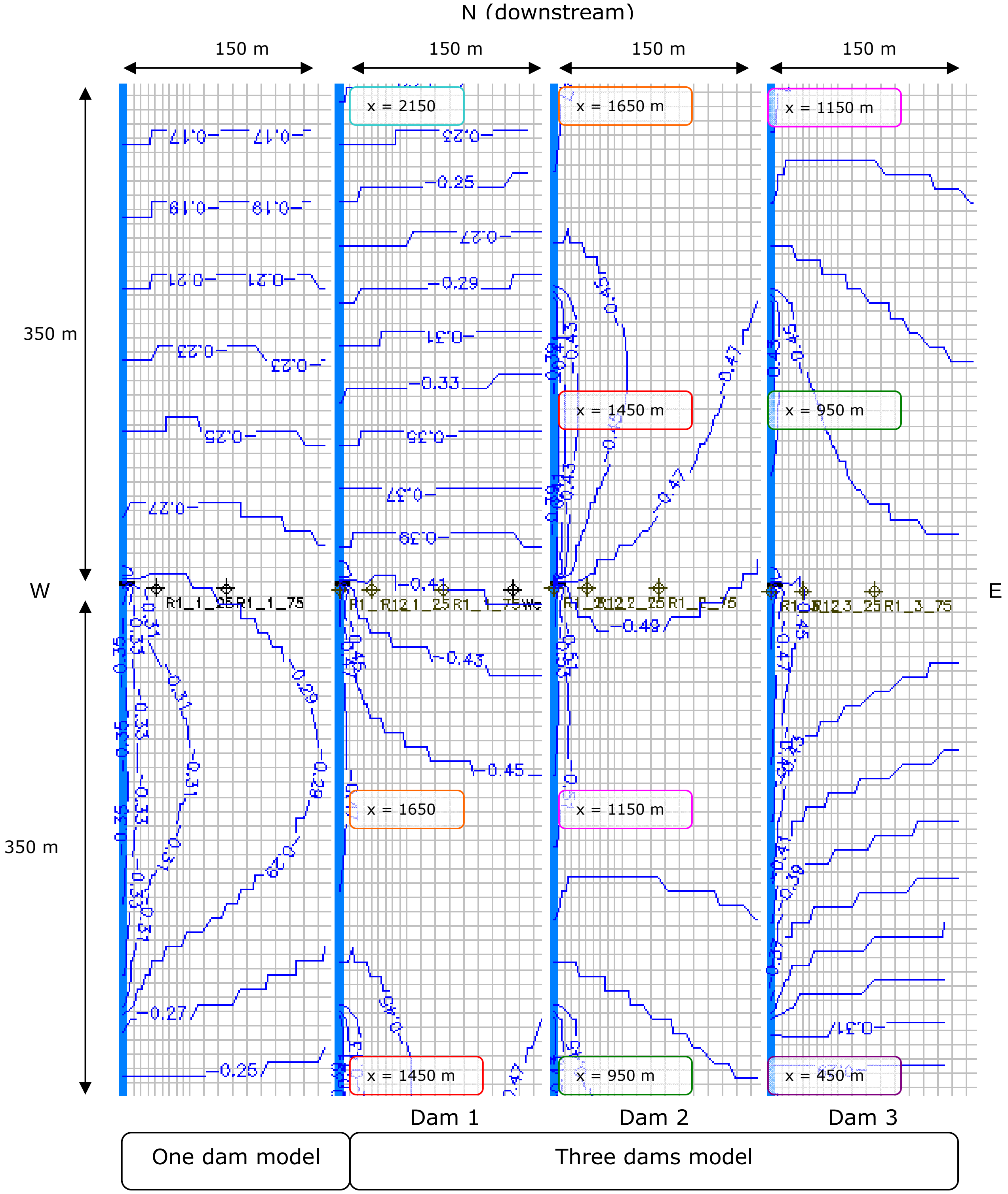

Figure 3.35. Plan view of drawdowns due to the dams at the end of the longest dry season in the deep system for One dam and Three dams in Scenario 1

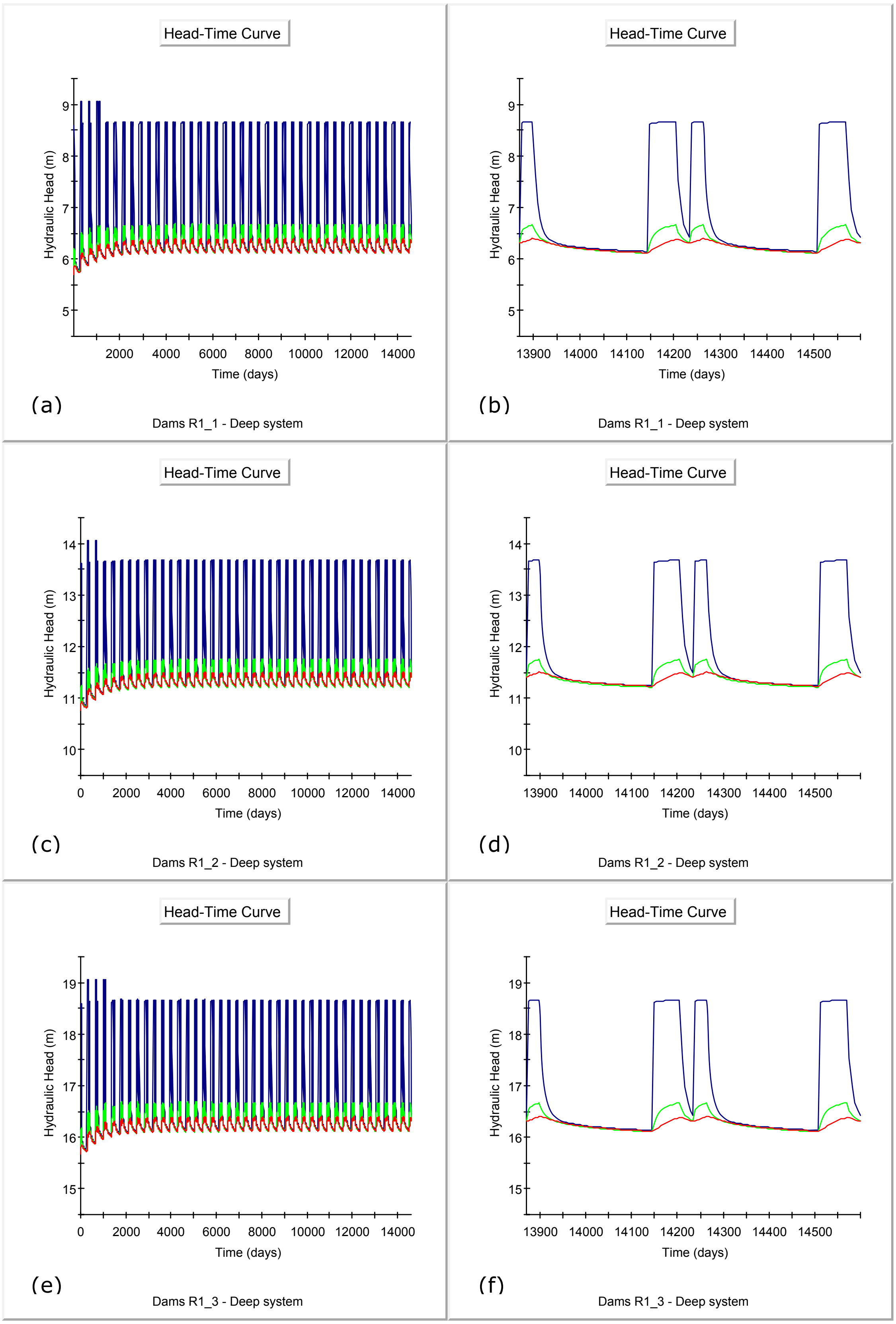

Figure 3.36. Head-time curves for observations in boreholes rows R1_1, R1_2 and R1_3 in the Dams case for Scenario 1

Scenario 2

In this scenario *Dam 1* was assumed to have been built just upstream of the low hydraulic conductivity bar taking advantage of it.

At the end of the longest dry season (end of October), the rises in the heads due to the dams are shown in Figure 3.34. The rises immediately upstream of *Dam 1* were much higher than in *Scenario 1*. Before building the dams, surface-water flowed in the river during all the year in certain segment upstream of the low *k* bar. The effect of *Dam 1* was to block part of this flow and store water below the surface. After the construction of the dams, the river carried water only during the rainy seasons, whereas during the dry seasons it was dry (the heads were below the top of the riverbed). On the other hand, the heads in the aquifer were higher, that means that the groundwater stock to be used was larger.

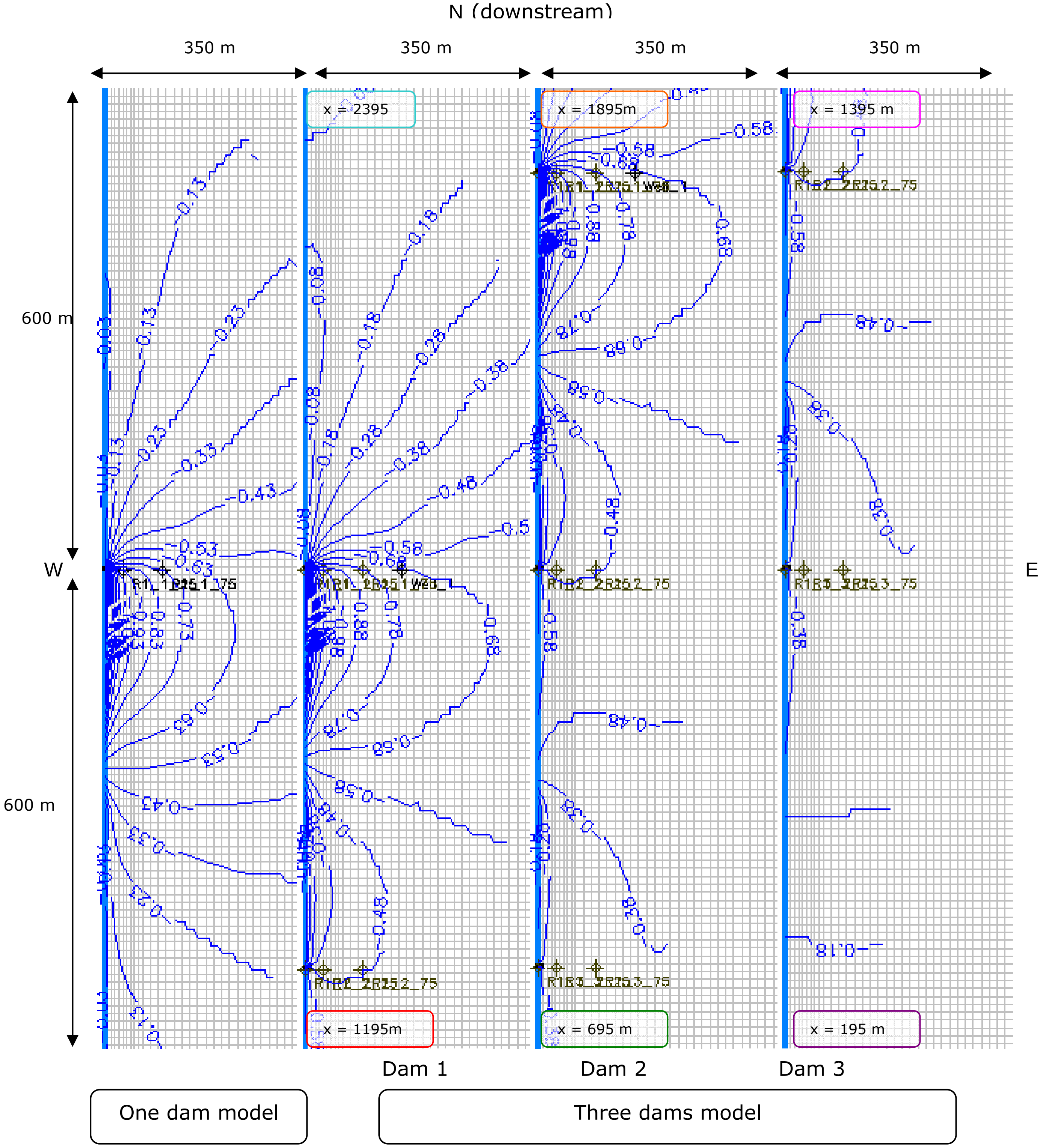

Figure 3.37. Plan view of drawdowns due to the dams at the end of the longest dry season in the deep system for One dam and Three dams in Scenario 2

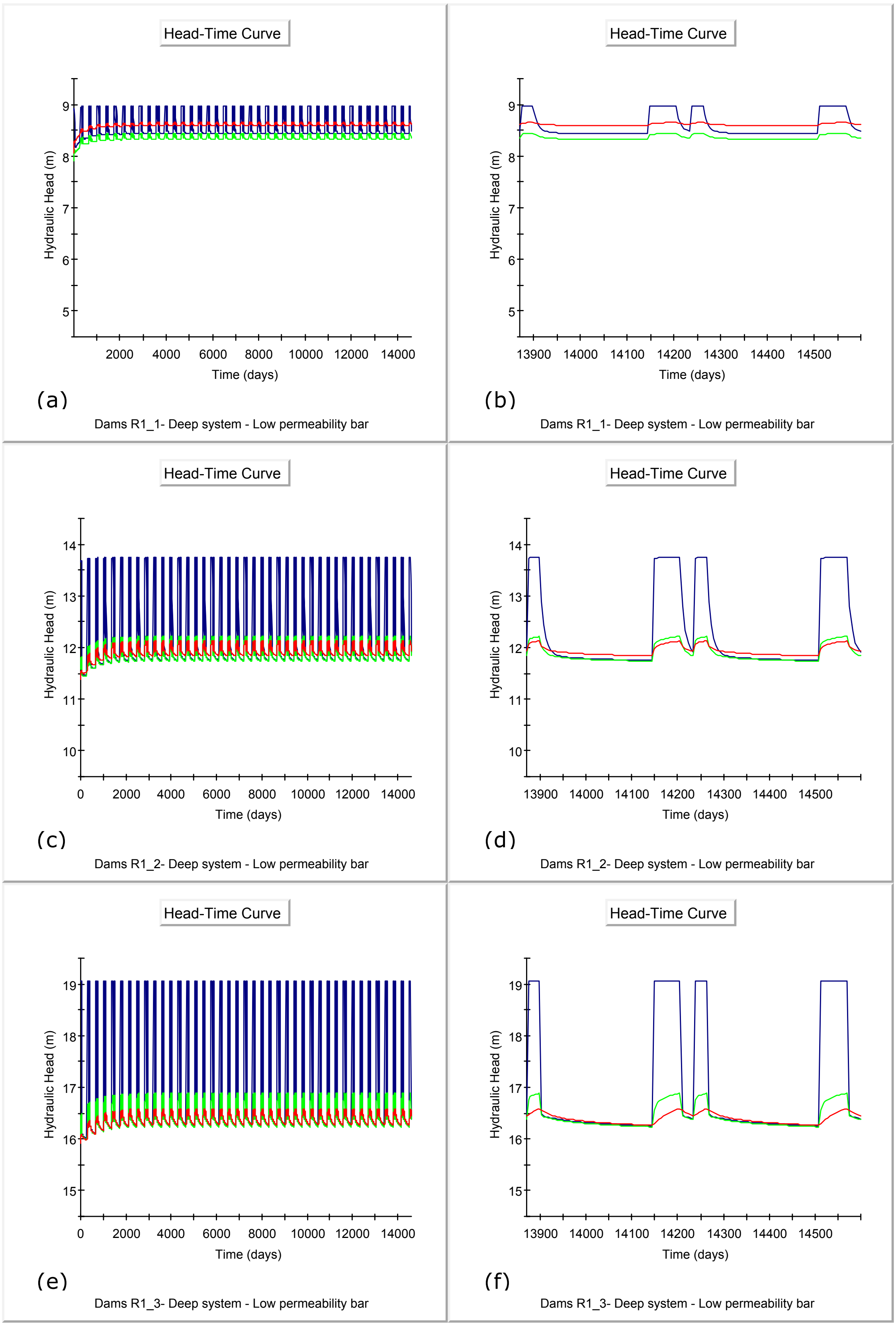

Figure 3.38. Head-time curves for observations in boreholes rows R1_1, R1_2 and R1_3 in the Dams case for Scenario 2

4. Further research

- To continue with the measurements of field data in the Kiindu catchment in Kitui and extend the research to other regions where sand storage dams could be successful.
- To include in the model rainfall and evapotranspiration data and the possibility of dry years.
- To include in the model the external flow coming from the riverbanks.
- To model a more realistic geometry including the variations in the rockbed, the river means, heterogeneous soils, etc.
- To model a case considering the contamination of the aquifer, which often occurs due to the use of water by cattle.

5. Conclusions

The effect of a sand storage dam was to raise the water levels in the surrounding area as desired. The heads were raised not only behind the dam but also downstream of the dam because of the water flow in the riverbanks in the downstream direction. The magnitude and distance to which the effects of the dam took place varied with each studied scenario.

It was observed that from the first year after the sand storage dam was built the gain on water volume (compared with the situation with no dam) increased every year. After 40 years, the gain in water volume remained the same every year. Thus, not only the dams were useful to store water from the wet to the dry season but also to store water between years.

For low hydraulic conductivity values (*k*) the inertia of the system was higher, meaning that the system responded slower to changes. During rainy seasons a lower water volume infiltrated in the riverbanks, whereas during dry seasons the water flow from the banks towards the riverbed was also lower. Lower *k* values produced higher rises close to the dam but the influence area was smaller.

On the other hand, when the hydraulic conductivity was higher, the changes in the heads over time for a certain location were sharper than for lower *k* values (faster response to changes) whereas for a certain time step, the variation in the heads over the space was gentler. During the rainy seasons a larger water volume infiltrated towards the riverbanks. Accordingly, during dry seasons water from the bank flowed easily towards the riverbed. A low *k* produced lower rises due to the dam but the influence area was longer.

When the slope in the river length direction was steeper, the wedge of sediments carried by the river and deposited behind the dam was smaller, meaning that the artificial aquifer created by the sand storage dam was smaller. Furthermore, the flow downstream was easier. As a result, the distance to which the influence of the dam could be noticed was shorter and the rises in water levels caused by the dam were lower.

Concerning the water use, it was observed that the optimal location for placing a well (maximum rises caused by the dam) was not immediately upstream of the dam, but a few meters further towards upstream. The drawdowns caused by the wells pumping water to 80 families had a localized effect, not changing the majority of the water stock caused by the dam.

When the aquifer was thicker the water volume infiltrated and stored in the aquifer during the rainy season was higher and the inertia of the system was larger. The influence area of the dam was longer. When the blocked depth of the aquifer extended until the rock bed the damming effects were much more significant than for the shallow system: larger rises caused by the dam and longer influence area. On the other hand, the highest the sand storage dam is, the larger effects could be expected. However, when the dam is blocking only the flow in the most upper meters of the aquifer, the effect of the dam is lower due to the groundwater flow below the dam. Also, the more impermeable the soil in which the dam base is constructed, the larger effects of the dam could be expected.

As said before the effects of building a dam varied depending on some factors as the slope in the river length direction, the *k* values, the dam height or the permeability of the soil below the dam base. In each case the influence area of the dam (area in which the effects of the dam can be observed) varied. The performance of a dams’ network in certain environment depended on the influence area of each individual dam and the distance between dams. If the distance between dams was such that no overlaps of influence areas took place, the dams behaved as individual structures. On the other hand, when influence areas of successive dams overlapped, the system behaved as a connected network. Then a general rise in the water levels in the area was observed. These rises were not only the result of adding the effects of the successive dams since certain synergic effect occurred due to the fact that the water stored in the downstream rises of one dam, instead of flowing freely downstream, was obstructed by the next dam.

The number of dams which must be modelled in order to study the performance of a network of *n* dams in a certain environment depends on the overlapping areas, that is, if the influence area of each dam is such that two areas overlap, then modelling two dams is enough to explain the performance of *n* dams in the network.

6. References

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