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Evaluation of The Proposed Drainage Network for Lowering the Groundwater Levels in Al-Dawadmi Town

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ABSTRACT. Rapid urban extension in Al-Dawadmi Town, near Al Riyadh, has led to a continuous increase of waste water from houses, industry, and commercial buildings. Because the town has no sewerage network, the waste water has been drained and accumulated in the shallow sedimentary layer that has a shallow bed rock. A continuous rise of the groundwater table has been expected to cause health, environmental, and infrastructural problems. To control the problem, installation of a drainage network has been considered and designed. In this paper, a finite element numerical model, MAWF has been employed to check the effectiveness of the network in lowering the rising groundwater table. It has been checked in a period of fifty years. Two scenarios have been considered. The first scenario traces the problem assuming no drainage pipes are installed. The second one checks the effect of the drainage network in lowering the groundwater table. The results of the study are presented in the form of contour maps and hydrographs. The results show that the drainage pipes are effective in lowering the water table by a depth of not less than 2.0m and keeping it steady for fifty years despite the increasing recharge rate.

Introduction

Many cities and towns, all over the world, suffer from the problem of groundwater rise. Reasons of this phenomenon can be classified into two categories. The first category is related to natural settings, such as topography, geology, and precipitation while the second one is connected to the man made environment, such as excess irrigation, leakage from water networks and finally seepage from septic tanks. This high groundwater level has bad consequence on the environment, populations public health, and the infrastructures of the cities. Al-Dawadmi town, in Saudi Arabia, is one of these cities. The effect of the problem has appeared in many places in the town forming surface bonds, soil humidity and salt accumulation. There are cracks in roads and seepage inside basements. The town authorities have started looking for a solution for that problem. Among a number of alternatives, the installation of a network of subsurface gravity drainage pipes has been considered to drain, collect, and lower the groundwater level. The design objective was lowering the water table by at least 1-1.5m. The design (Modern Center, 1997) has included the alignment of the network; the pipes' diameter, spacing and slope; and the filter cover. The diameter of the draining pipes is 0.15m. The design took into consideration the increase of surface recharge due to the population growth and the consumption increase. The design was checked by using a finite element numerical model, MAWF (El-Didy, 1988) simulating the hydrogeological system with and without the proposed drainage pipes. The third type of boundary condition has been used to simulate the pipes network. The numerical model has proven useful in further understanding of the behavior of the groundwater system in terms of providing quantitative results. The model has helped in tracing the history of the problem of groundwater rise and diagnosing the actual reasons behind its progress. On the other hand, the model has helped in testing and evaluating the effectiveness of the proposed plans for controlling and lowering the groundwater table in the future.

Region of Study

Al-Dawadmi town is located about 330 kilometer west of Al Riyadh City at latitude 24°30′ North and a longitude 44°20′ East. It occupies an area of about 20 square kilometers. Topography in the town is gradually descending down from level 996m in the west to 950m in the east along a distance of 10 kilometer. The shallow sedimentary layer changes in thickness between 1.0m and 7.0m (Fig.1) overlying a rock bed whose surface level changes from 994m on the western border of the town to about 948m on the eastern one.

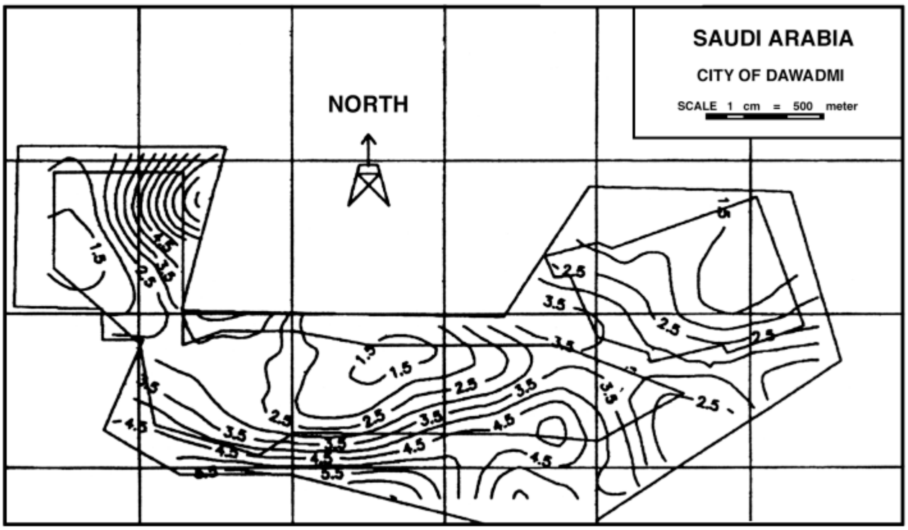

FIG. 1. Depth from ground surface to bed rock.

Monitoring of groundwater levels (Modern Center, 1996) shows that flow takes place from the west to the east along the town between levels 996 and 949m, respectively (Fig. 2). The groundwater is at a depth less than 2.0m below the ground surface and reaches 0.7m in zones A, B, and C (Fig. 3). The town has no sewerage network. As a result, sewerage water is the main source that feeds the shallow phreatic layer. The feeding recharge was calculated from the past records and predicted in the future for 50 years depending on the population and the rate of per consumption capita. Table 1 shows the results during the period 1416 through 1466 after Hegira (A.H.) (Modern Center, 1996) considering irrigation water and leakage losses for the network. The results of sieve analysis show that coarse sand is the main content and represents 30-50%. Percentage of 8-30% is silt and clay. The remaining soil is fine to medium sand with fine gravel.

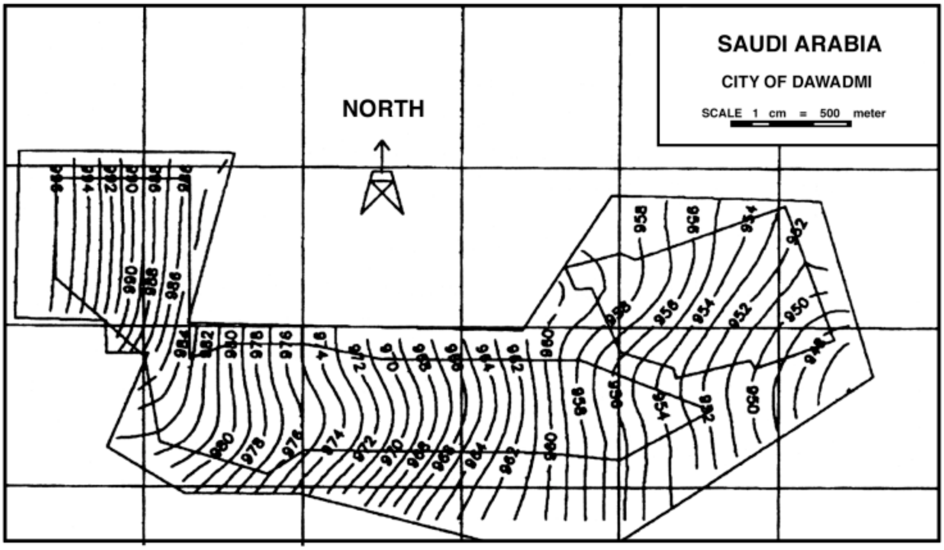

FIG. 2. Measured levels of groundwater.

Methodology

A finite element numerical model, MAWF has been used to check and evaluate the performance and the efficiency of the proposed drainage pipe network. The model was developed by El-Didy and Contractor in the period 1984 through 1986. Then it has been used in many theoretical and practical applications (El-Didy and Cotnractor, 1987 and El-Didy, 1989). The model uses eight nodded isoparametric quadrilateral elements (El-Didy, 1988 and 1992). The drainage pipes have been simulated by utilizing the third type of boundary conditions (Mixed type). The unsteady flow rate of the pipes is calculated by the model depending on the difference in level between the groundwater table and the pipe center line; and permeability, thickness and length of the filter. Two scenarios have been studied. The period of the study was 50 years. The first scenario assumes that no pipes are installed. The second one simulates the conditions in case of installing the drainage pipes. Contour maps are drawn for fifty years. Hydrographs for water table with and without the pipes are drawn at a number of nodes. The smallest and the largest time steps used are 0.005 and 5.0 days, respectively. During each scenario, 4084 steps were used. Each time step includes 2-3 times of iteration because the aquifer is a phreatic one. Contour maps for depth to groundwater were produced for the years 1417, 1418, 1420, 1425, 1430, 1440, 1450, 1460, and 1466A.H. for the two cases of study.

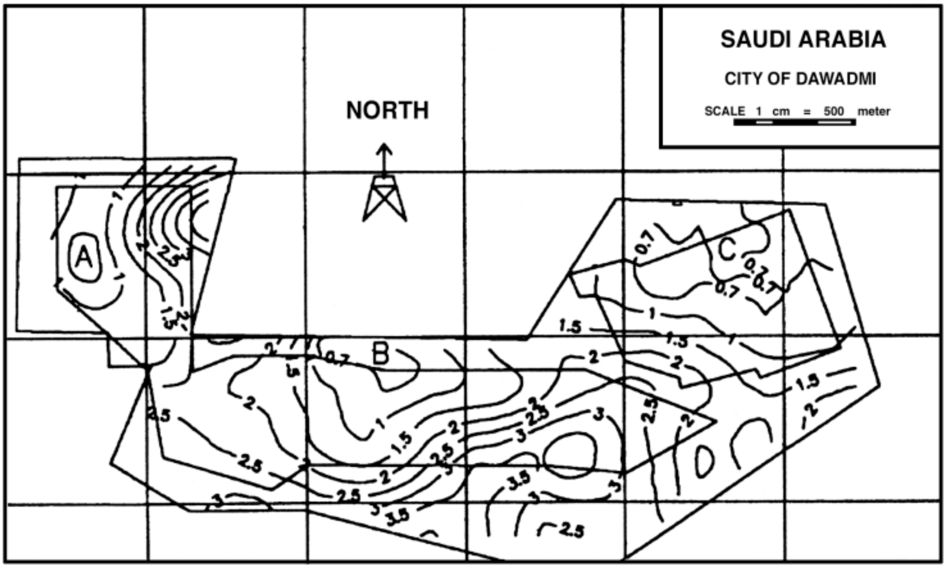

FIG. 3. Measured depth of groundwater table under the ground surface.

TABLE 1. Predicted recharge as equivalent head over the town versus the population (Modern Center, 1996).

Year A.H.	Population	Equivalent recharge head over the entire area of the town (mm/day)*
1417	42,500	0.152
1418	43,650	0.456
1430	59,600	0.637
1435	67,300	0.735
1440	75,850	0.847
1450	95,800	1,090
1466	138,751	1.650

*Consumption, irrigation and leakage losses are included.

The efficiency degradation with time of the proposed system was not considered in the numerical simulation assuming perfect and continuous maintenance for the system. The maintenance will be achieved by cleaning the pipes slots and the filter pores using the flushing pipes system designed to be installed above the drainage ones inside the filter layer.

Numerical Modeling

Equations Simulated by the Model

The partial differential equation of two-dimensional groundwater flow in heterogeneous anisotropic medium with concentrated or distributed sources and sinks is as follows (Bear, 1972, 1979 and Bouwer, 1978):

$$\frac{\partial}{\partial x}\left(K_{xx} B \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y}\left(K_{yy} B \frac{\partial h}{\partial y} \right) + r + Q = S \frac{\partial h}{\partial t} \tag{1}$$

where

∂	=		= partial differential operator,
d	=		= total differential operator,
h	=	$h(x,y,t)$	= average piezometric head over thickness [L],
K_{xx}	=	$K_{xx}(x,y,t)$	= hydraulic conductivity in x direction [L/T],
K_{yy}	=	$K_{yy}(x,y,t)$	= hydraulic conductivity in y direction [L/T],
B	=	$b(x,y,t)$	= thickness of confined aquifer [L],
	=	$[h(x,y,t) - z(x,y)$	= saturated thickness of unconfined aquifer where $z(x,y)$ is the aquifer bed [L],
Q	=	$Q(x,y)$	= concentrated discharge (–ve) or recharge (+ve) [L/T],
r	=	$r(x,y)$	= distributed concentrated discharge (–ve) or recharge (+ve) [L/T],
S	=	$S(x,y)$	= specific yield for the case of unconfined aquifer [L^o],
x,y	=		= Cartesian coordinates (coincides with the directions of principal hydraulic conductivity) [L], and
t	=		= time [T].

To solve the above equation (1) numerically, initial as well as boundary conditions are necessary. When solving unsteady state problems, initial conditions are necessary in the form:

$$h = h_o\,(x,y,t=0) \quad \text{for} \quad x > 0 \text{ and } y > 0 \tag{2}$$

Figure 1 presents the used initial condition in this study:

There are three types of boundary conditions. The first type of boundary conditions requires the specification of head values on some parts of the boundary where it is applicable.

$$h = h_1\,(x,y,t) \quad \text{for} \quad x > 0 \text{ and } y > 0 \tag{3}$$

This type of boundary condition has been used on the eastern and the western boundaries of the region of simulation.

The second type of boundary conditions requires specifying the flux value on other parts of the boundary. No flow boundary is a particular case. The mathematical form of that type of boundary is as follows:

$$(K_{xx} B \frac{\partial h}{\partial x} \underline{n}_x + K_{yy} B \frac{\partial h}{\partial x} \underline{n}_y) = q_s(x,y,t) \tag{4}$$

No flow boundary was assumed on both the northern and the southern limits of the region where the boundaries are perpendicular to the equipotential lines.

The third (mixed) type of boundary conditions relates the flux to the head (Eq. 5).

$$(K_{xx} B \frac{\partial h}{\partial x} \underline{n}_x + K_{yy} B \frac{\partial h}{\partial x} \underline{n}_y) = \frac{K_f W_f}{B_f}(h_f - h) \tag{5}$$

Figure 4 shows properties of the drainage pipes used in the simulation by the third type of boundary conditions (Wilson and Townley, 1981).

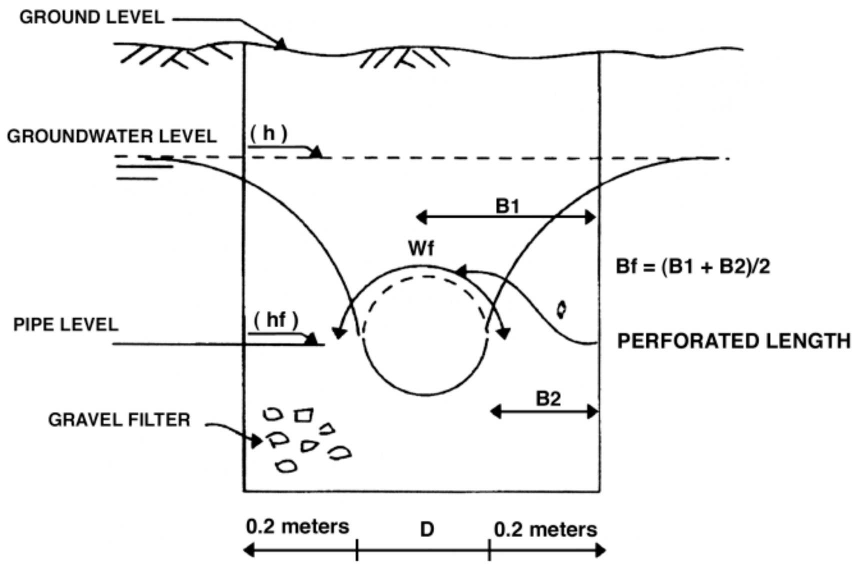

FIG. 4. A pipe cross section.

where

$\underline{n}_x$ and $\underline{n}_y$		are the components of vector $\underline{n}$ perpendicular to the boundary,
K_f	=	hydraulic conductivity in the filter around the pipe,
W_f	=	length of filter layer,
B_f	=	thickness of filter layer, and
h_f	=	level of the pipe center line.

Grid Construction

The area of the numerical simulation chosen was larger than the town area; enough to use specified head condition on some parts of the boundary. The area was divided into a grid of eight nodded isoparametric quadrilateral elements. The sides of the elements were planned to coincide with the drainage pipes. The grid has 534 elements with 1725 nodes (Fig. 5). The smallest element area is $0.01 km^2$ and the largest one is $0.405 km^2$. The resulting properties matrix is a banded one with a band width of 141 elements. Specified head and specified flow conditions are used on the boundary. The specified flow is used to simulate the impermeable boundary that is perpendicular to the equipotential lines. A number of 372 elements have 716 side that coincide with pipes. Hydraulic conductivity of the used filter was taken 300m/day (Bouwer, 1978 and Cedergrem, 1977) for 3/4" gravel (Powers, 1981). The perforated length of 15cm pipes was calculated with 50% perforation. The filter thickness was calculated with an average value 0.2375m. Depth of the pipes was in the range of 2-3m below the ground surface.

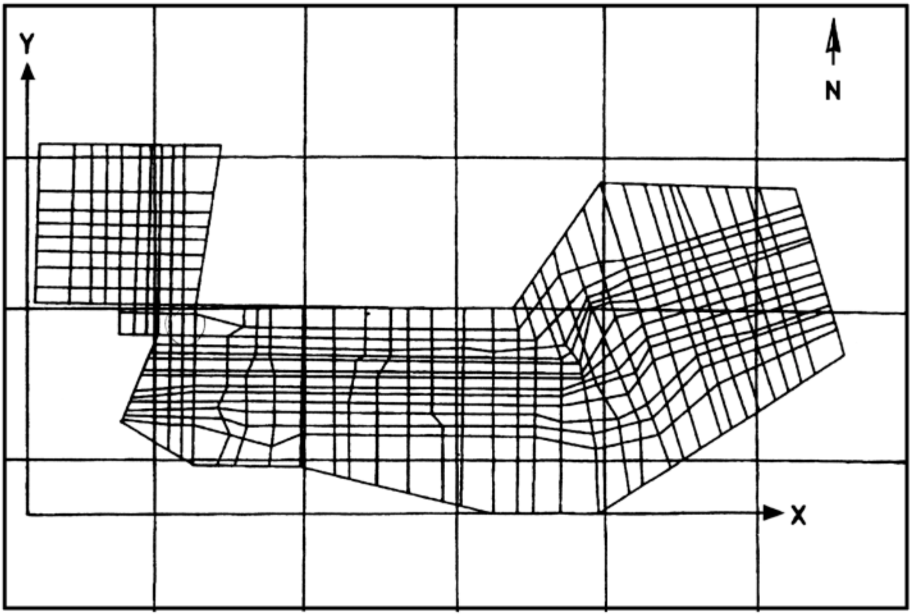

FIG. 5. The finite element grid used in the simulation.

Hydrogeological Properties

Hydraulic conductivity was measured in the field in 18 holes using the Inversed Auger Hole Method. The minimum and the maximum values are 0.0026m/d, and 0.088m/d, respectively (Modern Center, 1996). Specific yield was taken to be 0.23.

Sensitivity Analysis and Calibration

Sensitivity analysis showed that the recharge and the hydraulic conductivity are the most effective factors on results. Steady state calibration was performed by changing the hydraulic conductivity and the recharge then calculating the average error over the entire region. Results in Fig. 6, and 7 show that the minimum error is obtained at 0.1 and 2.4 times the measured hydraulic conductivity and recharge, respectively.

Results and Conclusion

Results of the numerical modeling show that if no drainage pipes are used, the groundwater will keep on rising to reach the ground surface in the year 1425 (Fig. 8). In case of using the pipes, the water level gets down to the design levels in the year 1420 as shown in Fig. 9 and keeps steady at the same levels till the year 1466 as presented in Fig. 10 in spite of the increasing recharge rates. These results prove and ensure the efficiency of the designed pipes' network in lowering the groundwater table in Al-Dawadmi town to be at least 1-1.5m below the ground surface.

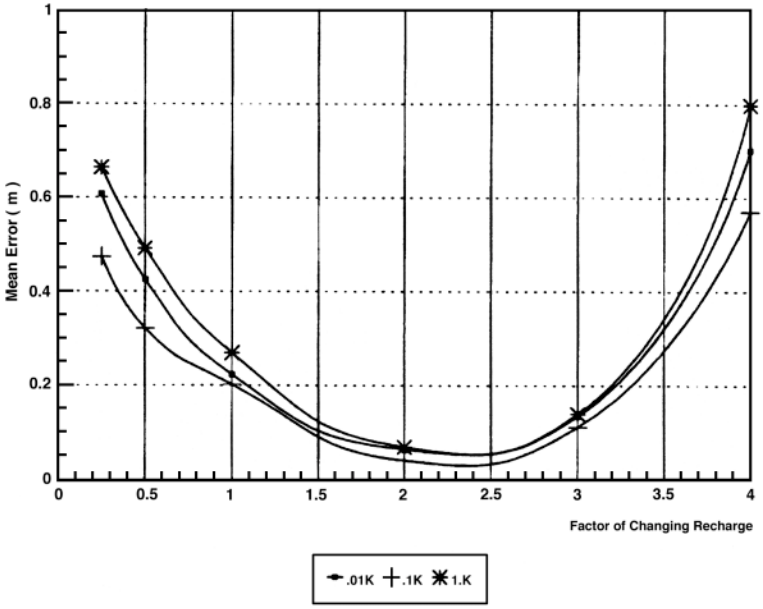

FIG. 6. Error versus factor of changing recharge in the calibration process.

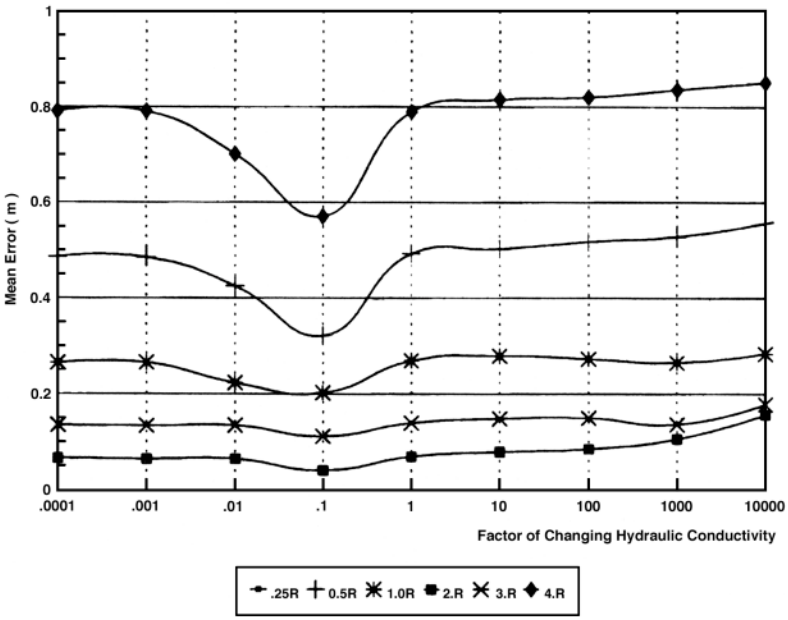

FIG. 7. Error versus factor of changing hydraulic conductivity in the calibration process.

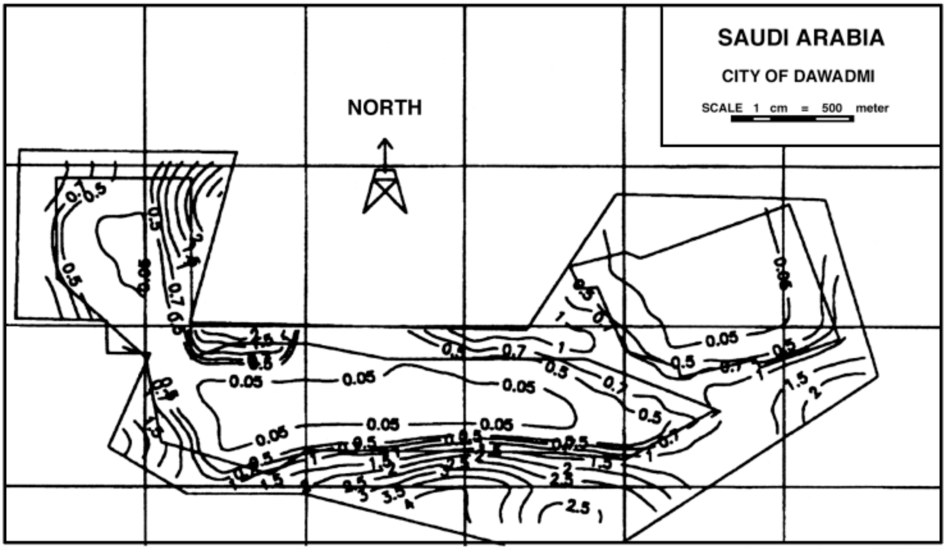

FIG. 8. Calculated depth to groundwater table tends to zero, in the year 1425, in case of not using the drainage pipes.

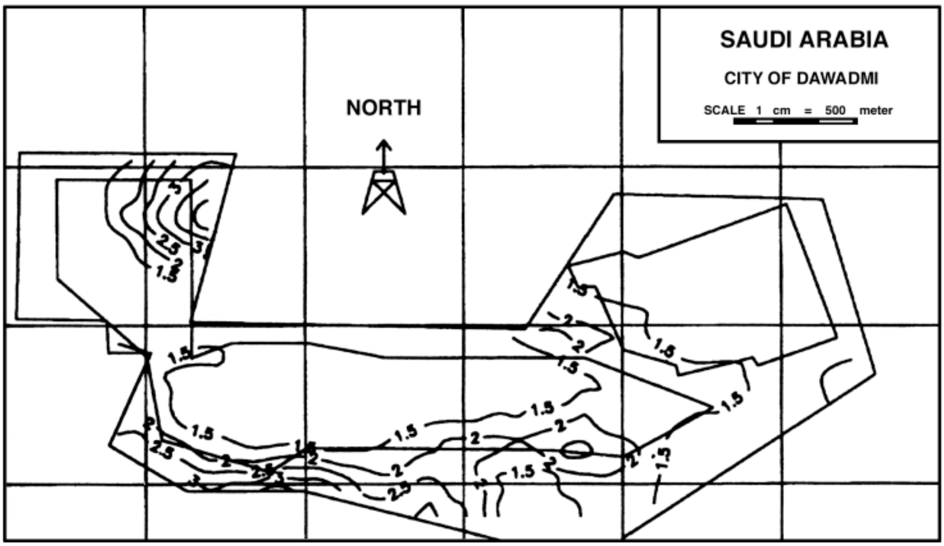

FIG. 9. Calculated depth to groundwater table reaches the design value in the year 1420 in case of using the drainage pipes.

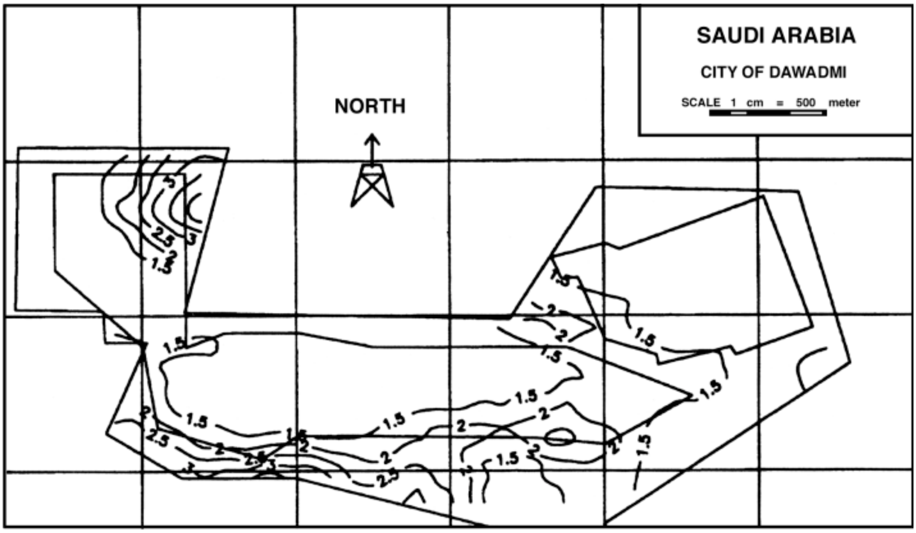

FIG. 10. Calculated depth to groundwater table keeps steady till the year 1466 in case of using the drainage pipes.

Also, hydrographs at different nodes were plotted showing the ground surface, the bed rock, and the groundwater level in the cases of using and not using the drainage pipes. Figures 11, 12, and 13 show the results at nodes 210, 849, and 1581.

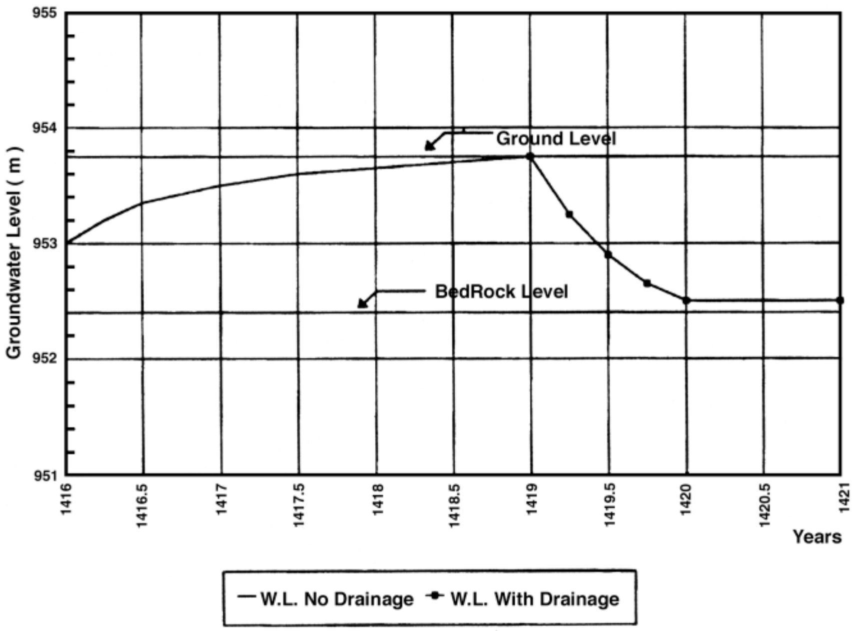

FIG. 11. Hydrograph of the groundwater table with and without using the drainage pipes at node #210 in the town west.

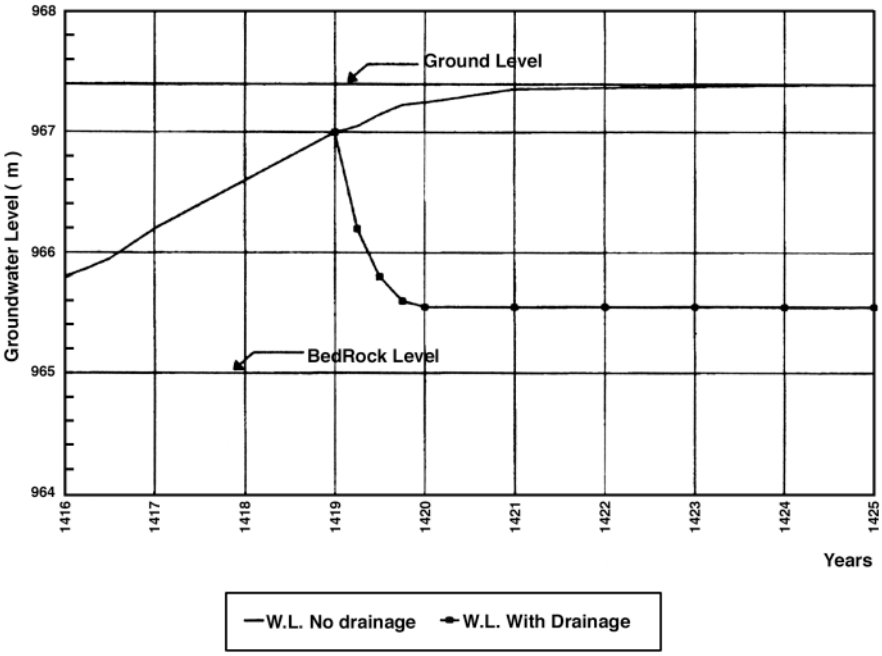

FIG. 12. Hydrograph of the groundwater table with and without using the drainage pipes at node #849 in the town center.

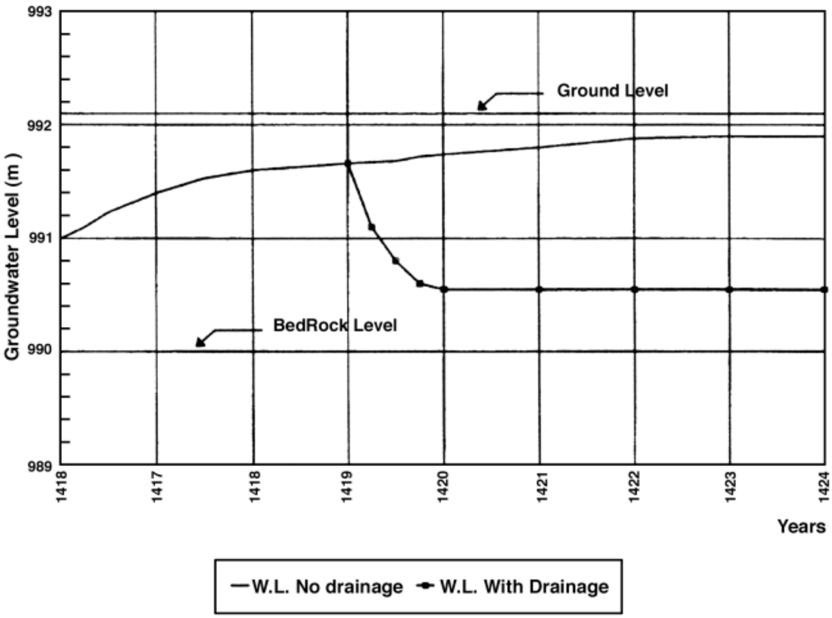

FIG. 13. Hydrograph of the groundwater table with and without using the drainage pipes at node #1581 in the town west.

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تقيـــيم شبـــكة الصـرف المقـترحـــة لتخفيـض مناسيب الميـاه الجوفيـة في مدينـــة الدوادمي

شريف محمد أحمد الديدي

كلية الأرصاد والبيئة وزراعة المناطق الجافـة

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المستخلص . لقد أدى التوسع العمراني السريع في مدينة الدوادمي في السنوات الأخيرة إلى ازدياد كبير في كميات مياه المجاري الناتجة من الأحياء السكنية والمصانع والمؤسسات التجارية ونتيجة لعدم توفر شبكات الصرف الصحي اضطر السكان إلى استخدام البيارات الشعبية والتي من خلالها تتسرب مياه المجاري إلى التربة مسببة ارتفاع منسوب المياه الجوفية ، ويؤدي هذا الارتفاع إلى العديد من المخاطر الصحية والبيئية والفنية مثل طفح مياه البيارات إلي سطح الأرض وانبعاث الروائح الكريهة بالإضافة إلي تشويه وتخريب الطرقات والشوارع وخرسانات قواعد المنشآت وتلوث التربة التي تصبح مرتعا خصبا للميكروبات .

وقد تم في هذا البحث استخدام النموذج العددي (MAWF) بطريقة العناصر المحددة لمراجعة تصميم شبكة الصرف المغطى المقترحة لخفض المياه الجوفية والتأكد من فاعليتها وقد تم التمثيل العددي في حالتين الأولى منهما تفرض عدم وجود شبكة الصرف أما الثانية فإنها تفترض وجود الشبكة مع تتبع الحالتين لمدة خمسين عاما للتنبؤ بالتغيرات المناظرة في مناسيب المياه الجوفية . وقد أظهرت الدراسة أن المياه الجوفية سوف تصل إلى سطح الأرض في كامل المدينة في عام ١٤٢٥هـ وذلك في حالة عدم استخدام شبكة الصرف أما في حالة تركيب الشبكة فإن المياه الجوفية سوف تنخفض خلال فترة عامين من تركيبها وتستمر في حالة استقرار حتى نهاية الدراسة في عام ١٤٦٦هـ وذلك بالرغم من التزايد المستمر في تصرفات مياه الصرف وكميات الرشح خلال التربة .