Comparative Study between Predicted and Observed Records of Implementation Dewatering Systems at Abu Qir Intake Power Plant, Alexandria

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Abstract: When a deep excavation reaches groundwater, that water must be extracted from under the development to provide a stable foundation during construction. The configuration of the extraction system depends largely on the soil properties and the volume of water that must be removed. This paper presents a case history of Abu Qir thermal plant units since dewatering system required for construction of the foundation of the intake structure was studied using the conventional design equations, then the results of recorded field data after pumping test have been used to calibrate the dewatering system model. Moreover, a comparison study has been carried out between the values of drawdown estimated by equilibrium formulas (closed form solutions), 3D finite difference code (Visual Modflow), and those values obtained from field measurements. Accordingly, the accuracy and predictability of the proposed analytic solution was evaluated.

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1.Introduction:

Dewatering means the separation of water from the soil or perhaps taking the water out of the particular construction site completely. The purpose of construction dewatering is to control the surface and subsurface hydrologic environment in such a way as to permit the structure to be constructed in the dry. In many coastal regions, especially in lower elevations, groundwater is situated very close to the surface.

Subsurface construction activities in these regions require some method of controlling ground water.

Typical applications include sewer and water pipe installation and repair, roadway construction, foundations for power plants, buildings, water and wastewater treatment plants, retention ponds and gas line burial.

Flow in a water table aquifer is greatly complex, since the saturated thickness, and therefore the transmissibility, decreases as we approach the well. Furthermore, because of complex boundary conditions at the phreatic surface, water table problems theoretically are indeterminate. Some other causes of complicity are soil anisotropy and partial penetration of well (Kruseman *et al.,* 1994).

Several methods have been introduced for design dewatering and control ground water systems. These methods can be classified as follows: (1) analytic solutions (i.e., equilibrium formulas); (2) graphical solutions of Laplace's equation (i.e., flow net method); and (3) numerical ground water models (e.g. Modflow software). Design of dewatering systems using the equilibrium formulas (closed form solutions) have been used for decades in the. These formulas were basically developed by Thiem (1906) and Muskat (1937) and have been supplemented by many other investigators (Powers, 1992). A complete review of these formulas was given by Mansur and Kaufman (1962).

Selection of the values to be used in those equations (i.e., Eqs. 1 and 2) requires judgment, and is based on information from many sources, including a field pumping test, the boring logs, data on surface hydrology and groundwater hydrology. However, inexperienced judgment in the selection of the above values has resulted in significant error in analyses of this type. The values of coefficient of permeability (k) can be estimated from pumping test. In most of pumping tests, however, the indicated value is for horizontal permeability (k_h). This value is only appropriate when flow to dewatering system is essentially horizontal. Isotropic estimate of (k) can be can be used in the analysis if both horizontal and vertical flow occurs to the system (Powers, 1992).

For fully penetrated confined aquifer:

$$Q = \frac{kB(H - h_w)}{\ln(R_o / r_w)} \quad (1)$$

For fully penetrated unconfined aquifer:

$$Q = \frac{k(H2-h2)}{\ln(R_o / r_w)} \quad (2)$$

Where: k= permeability coefficient; B= thickness of the aquifer; Ro= radius of influence;

H= initial head in the aquifer; and h_w = final head at the equivalent well (Figure 1).

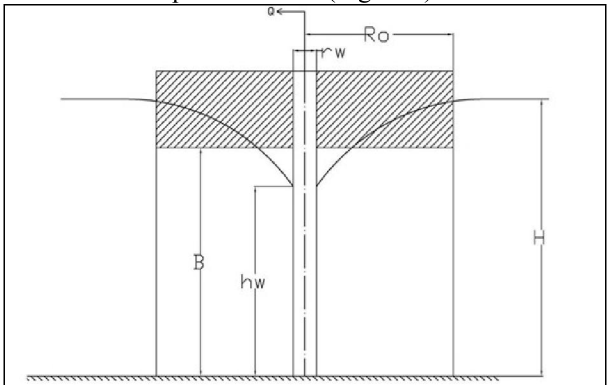

Fig (1) Equilibrium radial flow to a frictionless well in a confined aquifer

The main purpose of this paper is to make a comparison scheme between the results of closed form equations and finite difference code (Visual Modflow) compared to the field results obtained from a case history that is a construction project of intake structure of power plant close to the Mediterranean coast. First, the dewatering system of this project was designed using equilibrium formulas, and then the result of the calculated drawdown was compared with the measured values which were obtained from the observation wells and piezometers installed in the site. Second, Visual Modflow is used to analyze the problem. Furthermore, the results of water drawdown calculated at some specific locations, are validated by comparing its results with the results of the 3D finite difference code (Visual Modflow).

Project Description and Site Conditions

The project site is located on the Mediterranean coast 30 km East of Alexandria.

The cooling water for the power plant (circulating and service water) is withdrawn from the Mediterranean Sea by Intake structure which shall deliver the cooling water through four 2500 mm inside diameter concrete pipes to CW pump house. The cooling water shall be pumped out through a piping system to the power block area. The discharge water shall be connected to a two 3500 mm inside diameter concrete pipes to the Discharge structure located also on the Mediterranean Sea. The project shall include channels diversions. Figure 2 shows the general layout of the project.

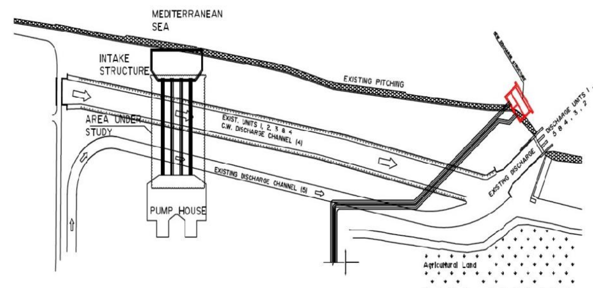

Fig (2): General layout and location of the studied area

The zero project level shall be (+1.50) mean sea level (MSL) and the finished grade level shall be (+1.00) project level (i.e. 2.50 MSL). The structure under study is the Intake structure. The dimension of the intake structure is about 34X74 m, and the excavation level is about (–10.00) (project level) while the ground level shall be considered at (-1.00) (project level) after excavation and grading. The intake structure is located on the shoreline. Table 2.1 represents the geotechnical properties if the main layers in the field.

Table (2.1) Idealized geotechnical parameters

Layer Id.	Top/ bottom level (MSL)	Soil unit weight (kN/m^3)	Case of during construction	
			Shear strength parameters	Deformation parameters
Silty clay	-3.50 to - 4.75 / -11.00	16	c_u = 15 kN/m^2 - Φ_u = 0	E_u = 3.3 x10^3 kN/m^2 ν = 0.49
Layer A: Silty sand	-11.00 / -14.00	17	c_u = 0.0 kN/m^2 - Φ = 36^o	E = 18x10^3 kN/m^2 ν = 0.33

Four boreholes were carried out in the site representing the formation of soil stratum and stratification of these layers is as follows:

Fill layer: This layer is composed of sand, gravel, crushed stone, and silty clay. It appears in all boreholes from the ground level and extends down to level ranging between (1.72) and (-1.80).

Silty clay: This layer is grey silty clay with traces of silty fine sand and broken shells in some depths. This layer follows the fill layer and extends down to level ranging between (-1.78) and (-6.30) with thickness ranging between 4.5m and 6.5m.

Silty sand / sandy silt: This layer is changing from sandy silt to silty fine to medium Sand. This layer extends down to a level ranging between (-12.28) and (-12.80) with thickness ranging between 6.5m and 9.0m.

Sand: This layer is grey medium sand with some silt. It follows extends down to level ranging between (-22.94) and (-26.80) with thickness ranging between 10.5 m and 14.0 m.

Sandstone: This layer is yellow to light grey calcareous sandstone with some voids and coral reef. It extends down to end of boreholes.

Table 2.2 tabulates the geological stratification of the field and the levels and depth of each stratum.

Table 2.2 :Summary of subsurface conditions in the site

Boring No.	Data	Fill layer	Layer A: Silty		Layer B: Silty sand / Sandy silt		Layer C:	Layer D:
IN-1	Top level	2.70	-1.80		-6.30		-12.80	-26.80
	Bottom level	-1.80	-6.30		-12.80		-26.80	-32.30
	Thick. (m)	4.5	4.5		6.5		14.0	5.5
IN-2	Top level	2.54	1.54		-3.46		-12.46	-25.96
	Bottom level	1.54	-3.46		-12.46		-25.96	-32.46
	Thick. (m)	1.0	5.0		9.0		13.5	6.5
IN-3	Top level	2.56	1.56		-4.94		-12.44	-22.94
	Bottom level	1.56	-4.94		-12.44		-22.94	-32.44
	Thick. (m)	1.0	6.5		7.5		10.5	9.5
IN-4	Top level	2.72	1.72	-4.28	-1.78	-6.28	-12.28	-24.78
	Bottom level	1.72	-1.78	-6.28	-4.28	-12.28	-24.78	-32.28
	Thick. (m)	1.0	3.5	2.0	2.5	6.0	12.5	7.5

Dewatering System

General

The geotechnical investigation in the site showed that the type of flow is confined flow, and the ground water level is about (-1.50) as well as the foundation level of the intake structure is at level (-10.00). Therefore, the required draw down is equal to 8.50m to help constructing the intake raft concrete body. Besides, the dimension of the dewatering area is 34x74m. The preliminary design revealed that a set of (20) wells (arranged in the intake cofferdam and around the excavation) with depth of 25m were installed during the intake excavation and construction. Due to the uncertainties included in the dewatering system, set of (4) wells shall be placed inside the intake structure working as standby wells. Figure 3 shows locations and arrangements of the wells in the site.

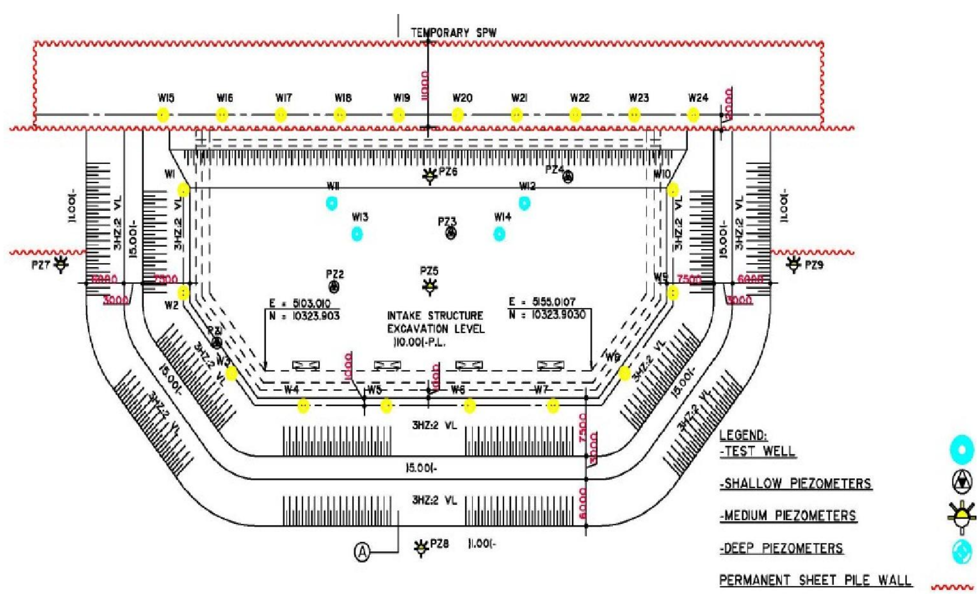

Fig (3): Dewatering and monitoring wells and piezometers arrangement layout for intake structure

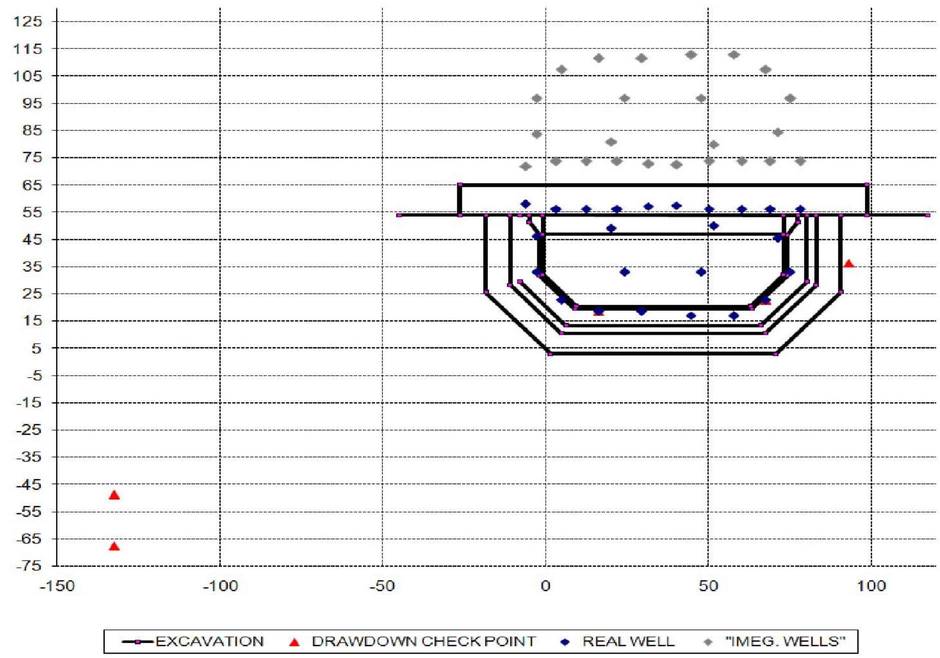

Fig (4): The model of dewatering system executed in the field and calculated using EXCEL spread sheet

A line source at a distance equal to 11.00 (cofferdam width) shall be considered in the analysis of the dewatering system which shall has a major effect in the design. The dewatering wells shall be provided with sand trap. The sand trap shall be used to monitor the sand content. It is recommended to take readings of sand trap every two days. The maximum allowable percent of fines shall be 15 PPM. The model of dewatering system had been calculated using an EXCEL spreadsheet that had been design to calculate drawdown values at some critical point as well as to depict the problem and its boundaries (Figure 5). Figure 6 illustrates a typical configuration of one of the deep wells which were established in the site.

Design procedures

The following steps were performed in designing the dewatering system of this project.

Step 1: Obtain a rough guess of the total quantity of water to be pumped: In a simple aquifer situation, a suitable design approach is to model the excavation area as a single well. With effective diameter equivalent to the well system to estimate the total expected discharge (Q). If the actual excavation is rectangular with a length (a) and width (b), then the equivalent radius (r_w) can be calculated by the relation:

$$r_w = \sqrt{\frac{ab}{\pi}} \qquad (3)$$

Then the total expected discharge (Q) can be estimated using Eq. 5.

$$Q = \frac{kB(H - h_w)}{\ln(R_o / r_w)} G \qquad (4)$$

Where: G = partial penetration coefficient

Without recharge of the barrier boundaries, R_o is function of the transmissibility and storage coefficient of the aquifer and expands with the square root of time (Kozeny, 1953). Approximate value of R_o can be estimated according to the empirical formula proposed by Sichardt (1928):

$$R_0 = C(H - h_w)\sqrt{K} \qquad (5)$$

Where: C is a constant = 3000 for wells and 1500 to 2000 for single line well points. Initial head H can vary seasonally with the perception or the stage of an adjacent river and can be affected by pumping in the vicinity. Final h_w must be lower than final head h that is desired under the excavation. The difference ($h-h_w$) is a function of (kxB), well yield (Q_w) and geometry of the system.

Step 2: Estimate of Individual well capacity: Well Capacity (Q_w) can be evaluated from the results of pumping test. In extrapolating the test results, adjustments must be made for conditions of well that are different from those during the test. A factor of major influence is the saturated length of the aquifer in contact with the well of the well (l_w). Well losses should also be considered.

Poorly constructed wells can have very high well losses. Powres (1990) described a method for estimating the well losses from a step drawdown pumping test. In the absence of a pumping test, Q_W can be estimated from the empirical formula:

$$Q_w = 0.42\, l_w\, r_w \sqrt{K} \qquad (6)$$

Where: l_W = screen length in meters; r_W = screen radius in mm, and k = coefficient of permeability in microns per second.

Step 3: Estimate the number of wells needed: After appropriate estimates of the total discharge and well yield, the required number (N) of wells can be estimated as:

$$N = \frac{Q}{Q_w} \qquad (7)$$

Step 4:

Return to the original excavation: Distribute N wells around the excavation parameter. The equilibrium formulas can be used to calculate the drawdown at any point due to each of the operating well system. Then, the drawdown due to the group of wells can be calculated using the cumulative drawdown method. In this method, the assumption is made that the drawdown at any point in the vicinity of a well array will be the sum of the drawdown that would have been caused by each well operating alone. Hence, checks can be made that the water level at the critical points below the excavation satisfies the design requirements. For example, the water level at the center and near the corners of the excavation should be calculated. If the water level does not satisfy the design requirements, wells can be added, removed, or rearranged until satisfactory array has been achieved.

Observation wells (piezometers)

A set of five (5) new deep observation wells (piezometers) shall be installed to monitor the water level in the sand aquifer during the dewatering process beside the existing piezometers installed for pumping test purpose. These piezometers are denoted as $PZ_5 - PZ_9$.

Fig. 5 shows the typical configurations respectively. A set of seven (7) observation wells (piezometers) installed to monitor the water level in the sand aquifer during the dewatering process beside the existing piezometers installed for pumping test purpose.

Numerical Model

The finite difference program (Visual Modflow), developed by Water Hydrogeologic Software (Guiguer and Franz, 1996), is used to analyze the problem. Fig. 6 shows the developed finite difference mesh. The soil profile (i.e. soil layers) and boundary conditions of the problem are represented in the model. Modflow has a package called "River" which allows the incorporation of surface water boundary conditions in the ground water model. This package is used to represent ware source (Mediterranean sea) in the model. Also, Modflow has a package called "Wall" which allows to simulate thin vertical low permeability barriers. This package is used to simulate the sheet pile wall (cofferdam) existing in the construction area. Constant head boundary condition is also applied at a distance equal to the obtained radius of influence of the dewatering system (165m measured from the center line of the construction area).

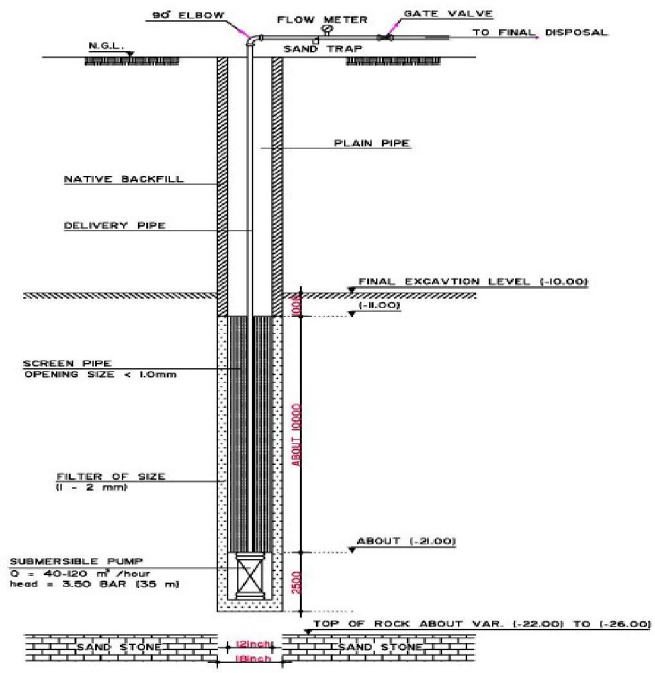

Fig (5):Typical configuration of deep well

Analysis of Results

Pumping test

Kruseman et al. (1994) showed that the principle of a pumping test is that if we pump water from a well and measure the discharge of the well and the drawdown in the well and in the piezometers at known distances from the well, we can substitute these measurements into an appropriate well-flow equation and can calculate the hydraulic characteristics of the aquifer (Figure 7).

Fig. 6 shows the results of pumping test conducted in the field and Table 4.1 gives the hydraulic characteristics of the aquifer. It is obvious that the pumping test in the site gave the permeability coefficient of 0.127 cm/s and the average The Radius of influence had been established as 165 m.

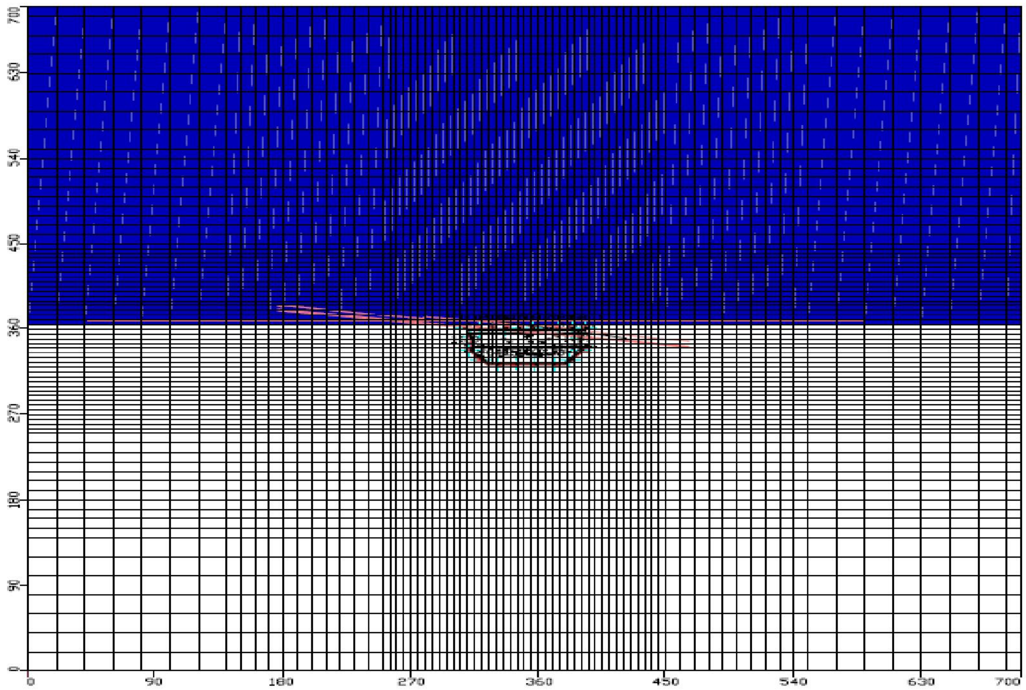

Fig (6): Finite difference mesh, the intake project

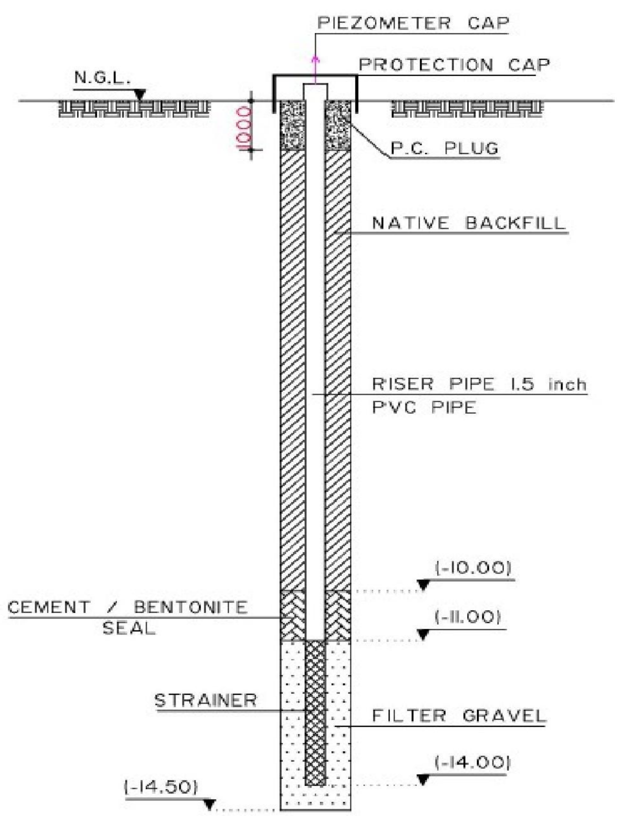

Fig (7): Typical configuration of peizometers

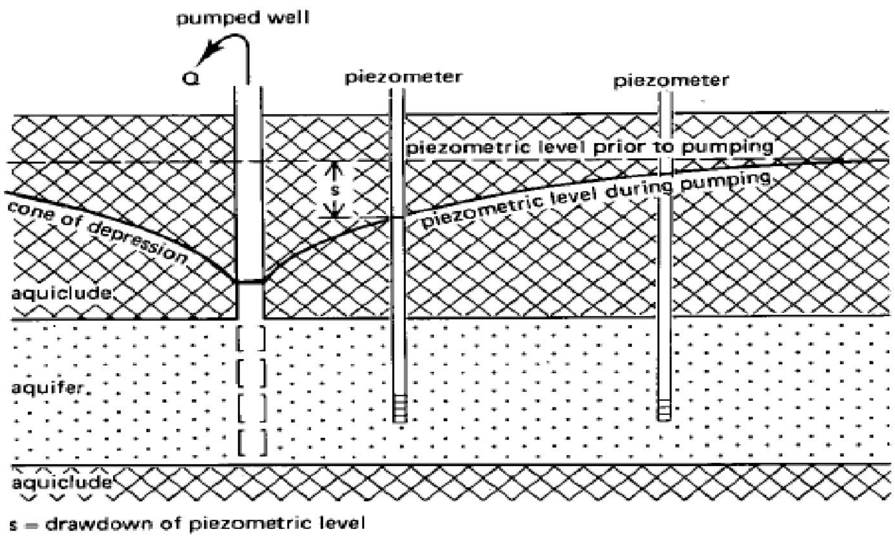

Fig (8): Drawdown in a pumped aquifer

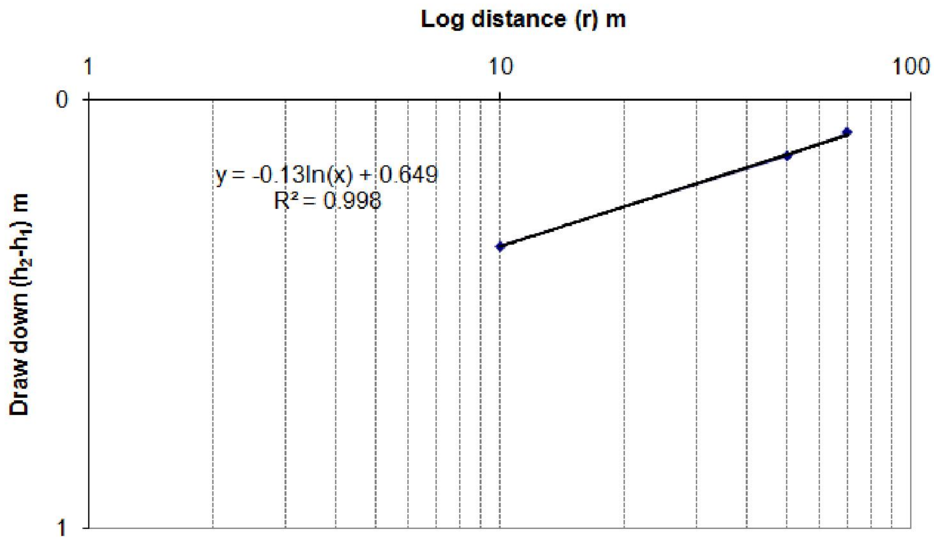

Fig (9): Relationship between the draw-down and the distance at the steady state

Table (4.1) Hydraulic characteristics of the aquifer in the site

G	KG	K cm/s	Piezometer ID
0.282	0.018	0.064	1 & 2
0.282	0.024	0.084	1 & 3
0.282	0.023	0.081	1 & 4
0.282	0.079	0.280	2 & 3
0.282	0.035	0.124	2 & 4
Average	0.036	0.127	

Modflow results

Fig. 10 depicts contours of the obtained drawdown in the site using the dewatering system explained previously. It can be seen that the drawdowns are affected by the recharge boundary north of the intake.

Measured drawdown

The records of five monitoring points installed in the field have been reported delay. Based on the results of pumping test, the model of dewatering (Fig. 3) had been calibrated instantaneously. Table 4.2 lists the recorded data of the drawdown values after steady state reached in the site for both of Modflow results and .

Figure 11 gives the correlation between measured and calculated drawdown in the field. It is worthily to notice that the measured drawdown values were higher than those ones calculated using equilibrium formulas given by Eq. 1 to Eq. 7. Besides, Figure 11 shows that there is an excellent and strong coefficient of correlation (R^2) between the data obtained by equilibrium formulas and those measured from the field at the critical points.

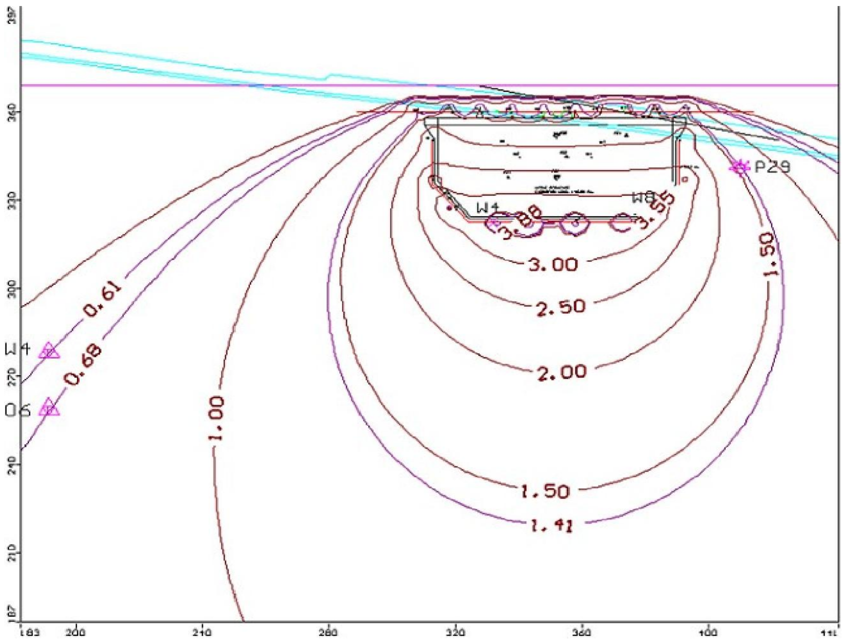

Fig (10) Contours of the drawdowns in the construction area

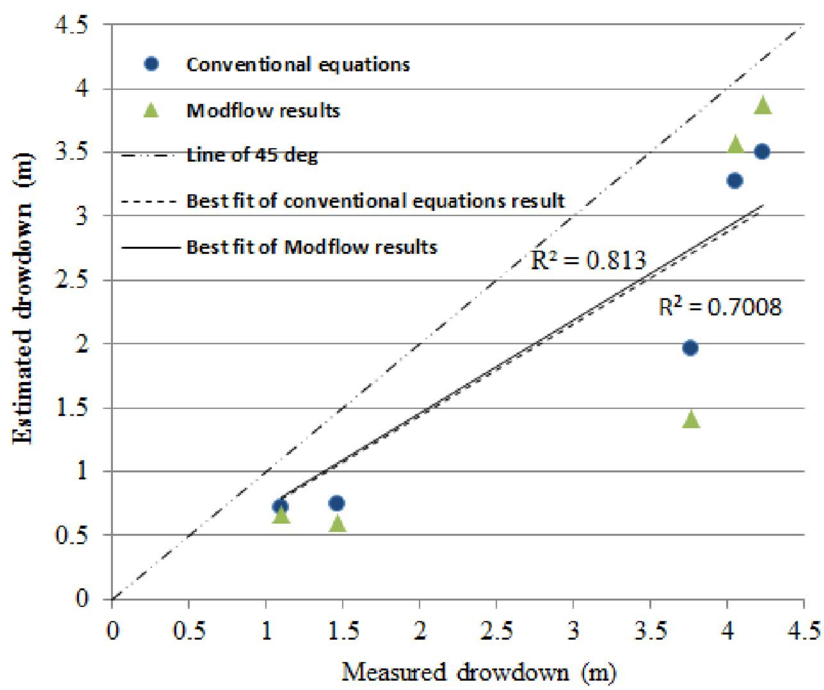

Fig (11) Relationship between measured and calculated drawdown

Table (4.2) Comparison between calculated and measured drawdown

Point ID	Position of point on the model		Drawdown (m)		
			Conventional equations	MUDFLOW	Measured values
	X	y			
W4	18.5	16.1	3.27	3.88	4.050
W8	22.7	67.3	1.97	3.57	3.765
PZ9	36.3	92.9	0.75	1.42	1.47
RW4	-67.5	-132.5	0.72	0.6	1.10
MWO6	-48.8	-132.7	3.50	0.67	4.23

Summary and Conclusion

This study introduces a comparative study between the proposed analytical solution and the results of field measurements for the evaluation of groundwater control (dewatering system), and the predictability and the accuracy of analytical solution was evaluated. The main outcomes of this paper are:

1. Level drawdown values in specific wells placed along the pit edge could differ from the respective level values of wells located in the central part. This difference depends not only on the number of wells but also on the leakage from the lower layer and on the volume of atmospheric precipitation.
2. An excellent and strong correlation (R^2) is obtained between the equilibrium formulas and the field results measured at the critical points. Hence, the proposed analytical solution gave acceptable predictability compared to the results of finite difference analysis since the correlations of both techniques are quite close.

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