CHAPTER 5

CASE STUDY OF GROUNDWATER LOWERING FOR THE EXTENTION OF WADI EL-NILE HOSPITAL, CAIRO

5.1 Project Description

The selected case study concerns the dewatering process during the construction of the extension of Wady El-Nile hospital, El-Kobba, Cairo covering an area of about 12000 m^2. The project site is divided into four parts A, B, C & D as shown in Fig. 5.1. The building consists of basement on the whole area of the building, ground floor, mezzanine and a number varies from one to eleven upper floors. All floors are constructed of reinforced concrete skeleton.

The operation of lowering groundwater table is divided into 4 stages (of the four parts of the entire project site) in order to minimize the effect of the drawdown on the existing near-by buildings. This research is focused on lowering the groundwater table below the foundation level over an area of about 4200 m^2 of this site (Part D).

This chapter presents all available details on the case study concerning the geotechnical conditions, description of the dewatering system and preliminary studies of the dewatering system. Data collected from pumping test were used to evaluate the hydrogeological parameters. This chapter also presents records of daily monitoring of groundwater table using piezometers installed for this purpose. In addition, records of the discharge of each pump during each are included.

5.2 Geotechnical Condition

To select and design a dewatering system, many factors have to be investigated. Geological features of the site, the geotechnical properties of the soil and their variation as well as the groundwater depth and the quantity of discharge are the most important factors. Coefficient of permeability, soil stratification, thickness and depth of the pervious strata to be dewatered, effect of groundwater table lowering on the adjacent buildings (especially in urban areas) are geotechnical parameters to be considered and checked. The time required for dewatering is also an important factor.

Identification of aquifer nature and its chemical characteristics are required to investigate the properties of water to be lowered. Importance of each item varies according to each project problem and its site condition.

5.2.1 Subsurface soil condition

A site investigation was carried out to identify the soil formation for the site and to determine the required parameters. Seven boreholes were drilled at the locations shown in Fig. (5.1) using rotary drilling machine. Three of them were drilled under the low rise buildings, 15 m deep each and four boreholes at the high rise buildings, 25 m deep. Standard penetration tests were carried out at different depths of the sand deposits to determine their relative density. Physical, chemical and mechanical properties of the soil layers were measured using field and/or laboratory tests.

An extensive site investigation was performed by the contractor using regional geological conditions and the representative geotechnical parameters. The ground stratifications at the project site can be summarized as follows:

1. A surfacial layer of stiff to very stiff brown silty clay is present. This layer starts from the ground surface to a depth of 6.50 to 7.50 m. This layer can be considered impermeable in the vertical direction with more significant horizontal permeability due to the presence of thin layers or separators of silty and clayey sand.

2. A second layer of graded to coarse sand with different fine gravel contents and having relative density of dense to very dense is present below the silty clay layer. Its permeability is relatively high and it extends to the end of the borehole.

The piezometric measurements taken during the geotechnical site investigation showed that groundwater table was at depth ranging between 1.3 to 1.7m. The groundwater level was considered in this study to be at a depth of 1.30m corresponding to the highest level at all boreholes.

Representative soil profile at the project site is illustrated in Figure (5.2).

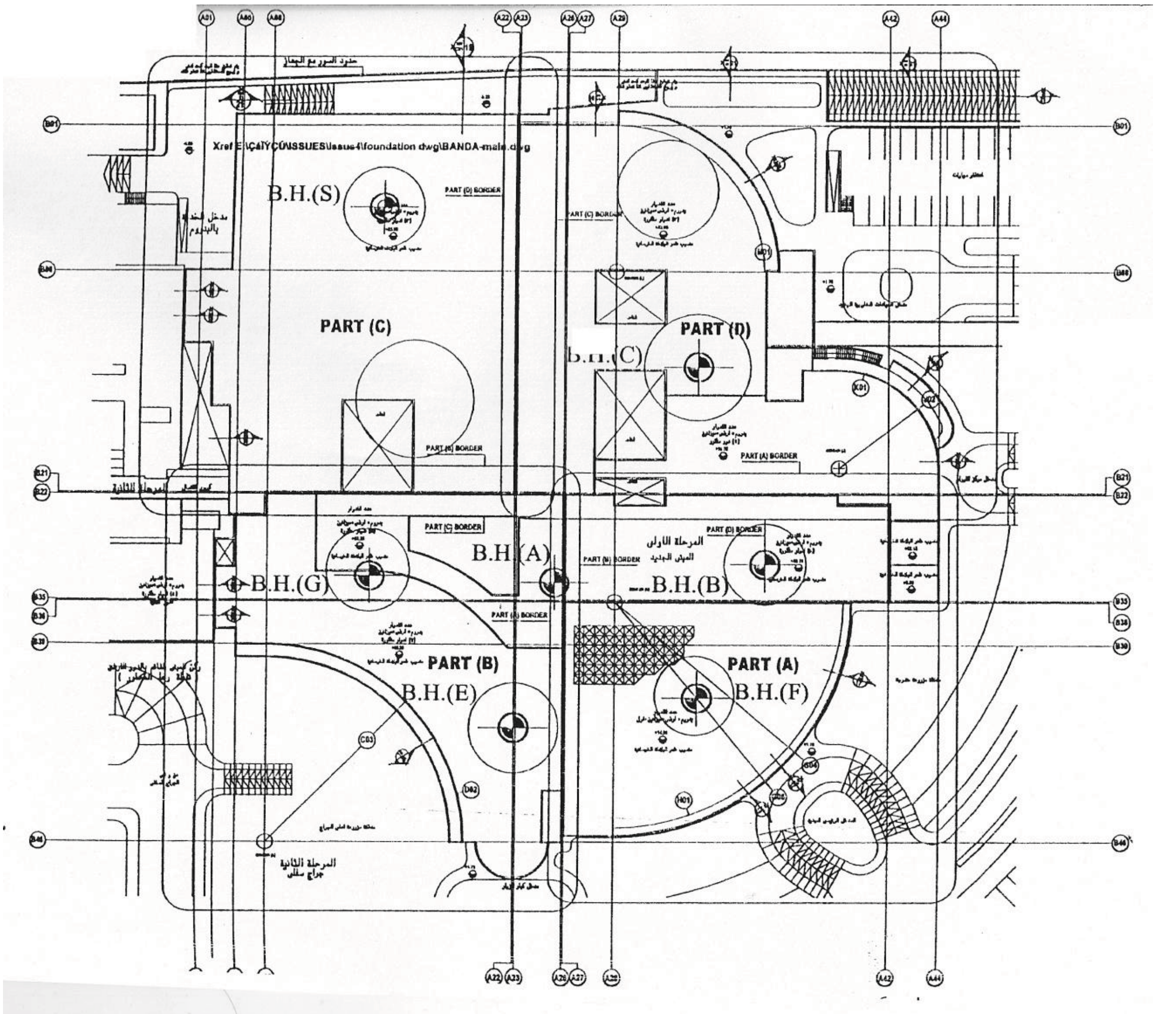


Fig. (5.1) *Boreholes locations*

Depth	End of surface	Soil description	R.Q.D (%)	SPT	qu (kg/cm2)	Gs	Bulk density (kN/cm3)	wc	L.L/P.L	Test type
1.00					3.80					
2.00					2.40					
3.00		Brown silty clay, stiff to very stiff Traces of calcareous bubbles			2.60					
4.00					2.20					
5.00					2.10		19.80	34.50		Uncon.
6.00					2.20					
7.00	7.60									
8.00										
9.00				41						
10.00		Light Brown coarse to medium sand, dense to very dense, some fine gravel, traces of silt and calcareous materials								
11.00				51						
12.00										
13.00	13.00			50/13						
14.00		Light brown medium to graded sand, very dense, some fine gravel, traces of silt and calcareous materials								
15.00				50/12						
16.00		End of boring								
17.00										
18.00										
19.00										
20.00										

Fig. (5.2) *A typical soil profile at the project site*

Some laboratory tests were conducted to estimate the required soil parameters. Complete grain size analysis was carried out on selected representative samples. Figure (5.3) shows the typical grain size distribution curves for the sand layers in the site for depths from 7.5

m to 10 m. Figure (5.4) shows grain size distribution curves for sandy soils depth from 11m to 20m. Table (5.1) summaries some gradation properties of selected samples. The unconfined compressive strength of the cohesive specimens were measured using the pocket penetrometer and the unconfined compression test. The bulk density and natural moisture content of the cohesive soil were determined in the laboratory on undisturbed samples.

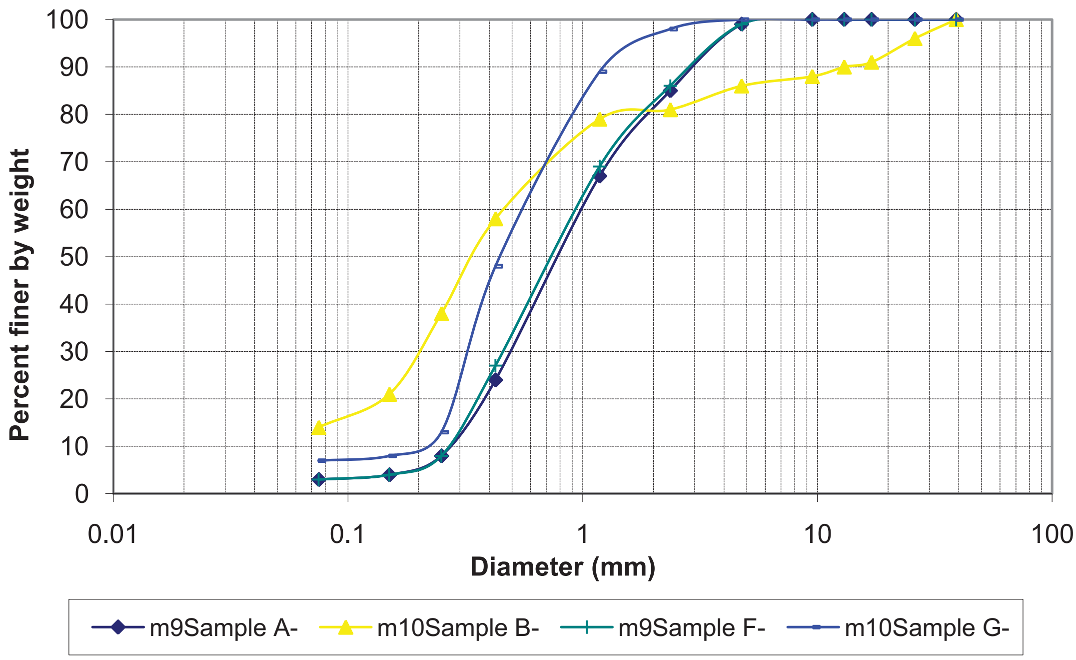


Fig. (5.3) *Grain size distribution curve of the sand samples of depths from 7.5m to 10m*

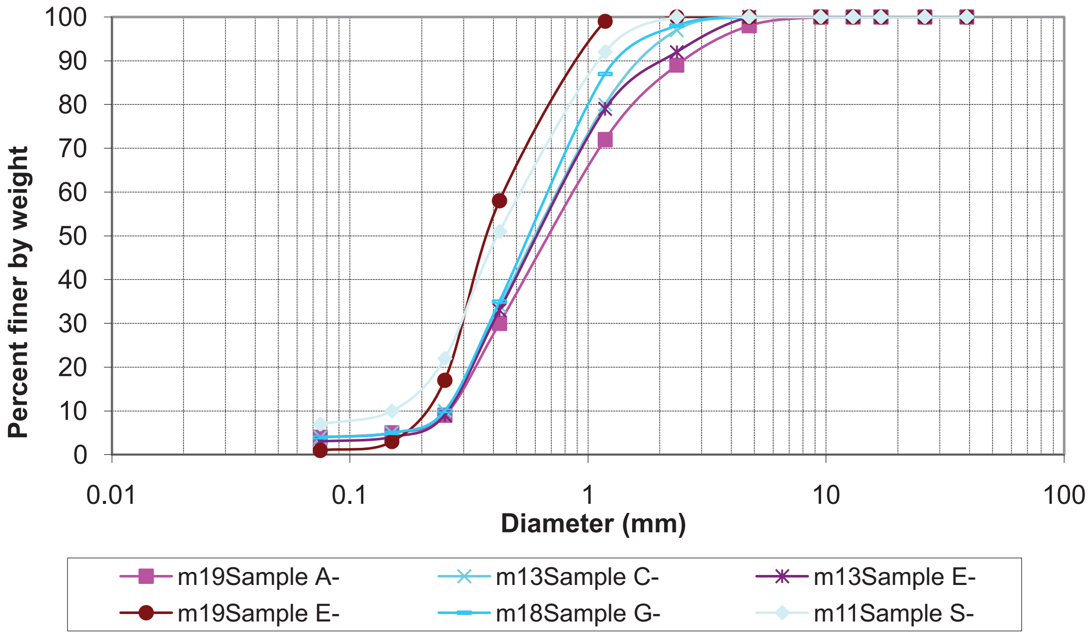


Fig. (5.4) *Grain size distribution curve of the sand samples of depth from 11m to 20m*

Table (5.1) *Gradation properties of sandy soil at different samples*

BH No.	Sample No.	Depth	D_{60} (mm)	D_{30} (mm)	D_{10} (mm)	C_u	C_c
(A)	A-9	9.00	0.8825	0.4837	0.2361	3.7380	1.1232
(B)	B-10	10.00	0.4562	0.1670	#N/A	#N/A	#N/A
(C)	C-13	13.00	0.6941	0.3795	0.2055	3.3785	1.0100
(E)	E-13	13.00	0.7023	0.3878	0.2194	3.2009	0.9760
(F)	F-9	9.00	0.8352	0.4599	0.2318	3.6028	1.0923
(G)	G-10	10.00	0.5520	0.2992	0.1578	3.4970	1.0276
(S)	S-11	11.00	0.5105	0.2564	0.1158	4.4094	1.1127

5.2.2 Chemical analysis of groundwater

There are mineral and biological constituents in the groundwater that can affect the dewatering system by corrosion of metal components or by incrustation with precipitates that clog screens, pumps and piping. Simply, corrosion is the deterioration of a material and incrustation is the precipitation or deposition of material onto surface. Technically, although a corrosion-like process can occur with any material (e.g., decay, decomposition, oxidation, and so forth); it is most associated with metals, while incrustation can and does form on almost any material.

Table (5.2) shows the results of groundwater chemical analysis carried out on a specimen from one of the boreholes.

Table (5.2) *Results of groundwater chemical analysis*

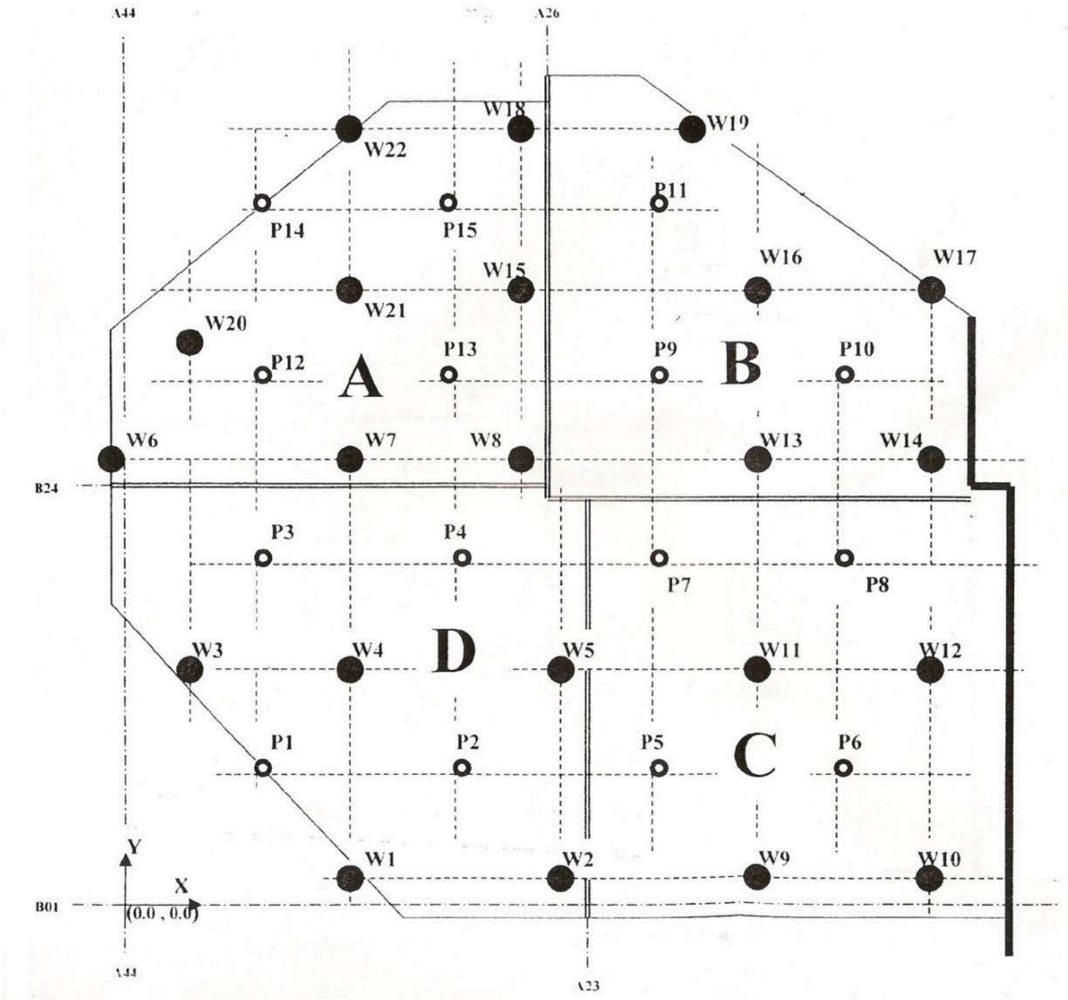
SNO	Chemical compound calculated as part per million or milligram per liter	Molecular Formula and	Results
1	Total mineral soluble salts (at 105°C)	**Ionized salts**	1715
2	Total alkalinity as sodium carbonate (methyl orange indicator method)	**$Na_2\ CO_3$**	165
3	Salinity as sodium chloride (Mohr�= s method in titration)	**Na Cl**	573
4	Sulphate as sulpher trioxide (gravimetric by precipitation the sulphate with Barium chloride)	**SO_3**	567
5	The negative logarithm to the base ten of the conventional hydrogen ion activity by sensor from glass electrode (pH)	**Log 1/(H^+)**	7.2

Visual examination of the groundwater shows that its color after filtration is transparent and colorless. The color of the suspended soil and colloids is brown. The odor of water is nonsulphidic without organic contents.

5.3 Groundwater Lowering in the Construction Site

The selected dewatering system for the project consisted of twenty two deep wells distributed as shown in the layout figure (5.5). It was constructed and operated successfully to reduce the original groundwater level from (1.30 m) to the required level (4.75 m). The system was operated on four stages to reduce the settlement resulting from the dewatering process. The dewatering process of each stage was not stopped until maintaining equilibrium to the uplift.

Generally, it was required to lower the groundwater table by about 3.50 m. It was estimated that the daily quantity of water discharged for every stage will be in the order of 11500 m^3 and for some periods, when operating two stages at the same time, the discharge was about 20160 m^3 using deep well, each one was pumping at a rate of 65 m^3/h under 22 m head.



Wells & Piezometers Coordinates :

Well	X	Y
W1	33.0	4.0
W2	64.0	4.0
W3	12.0	35.0
W4	33.0	35.0
W5	64.0	35.0
W6	-3.0	63.0
W7	33.0	63.0
W8	58.0	63.0
W9	85.0	4.0
W10	106.0	4.0
W11	85.0	35.0
W12	106.0	35.0
W13	85.0	63.0
W14	106.0	63.0
W15	58.0	90.0
W16	85.0	90.0
W17	106.0	90.0
W18	58.0	112.0
W19	79.0	112.0
W20	12.0	80.0
W21	33.0	90.0
W22	33.0	112.0

Piezo.	X	Y
P1	22.0	20.0
P2	49.0	20.0
P3	17.0	49.0
P4	40.0	49.0
P5	75.0	20.0
P6	96.0	20.0
P7	75.0	49.0
P8	96.0	49.0
P9	75.0	76.0
P10	96.0	76.0
P11	75.0	101.0
P12	22.0	76.0
P13	45.0	76.0
P14	22.0	101.0
P15	45.0	101.0

Well

Piezometer

Fig. (5.5) *Layout and distribution of the deep wells and piezometers*

5.4 Construction Procedure of the Dewatering System

The most appropriate method for lowering groundwater is governed by adequate soil surveys, test borings to delineate the soil strata, laboratory testing of soil specimens and the values of the soil-water parameters (especially coefficient of permeability) which are determined from pumping test results

5.4.1 Field pumping test

When the groundwater control has a potentially major impact on the design of a project or its cost, then the pumping test is important. A pumping test is the preferred method for obtaining reliable data on hydraulic conductivity, transmisivity, radius of influence, storage coefficient, recharge, capacity of wells and other factors that will determine the scope and cost of dewatering effort required.

In the Wady El-Nile hospital project, a pumping well and four piezometers were specially installed for the pumping test as shown in Fig. (5.6). It should be noted that this pumping well and the associated piezometers were completely different than those utilized for the dewatering system .The well was pumped at a steady state with an average rate of 65 m^3/h. The pumping period was five hours. The water levels and discharge were observed at small intervals to assure accuracy and the observations. Table (5.3) includes all the compiled records of the pumping test.

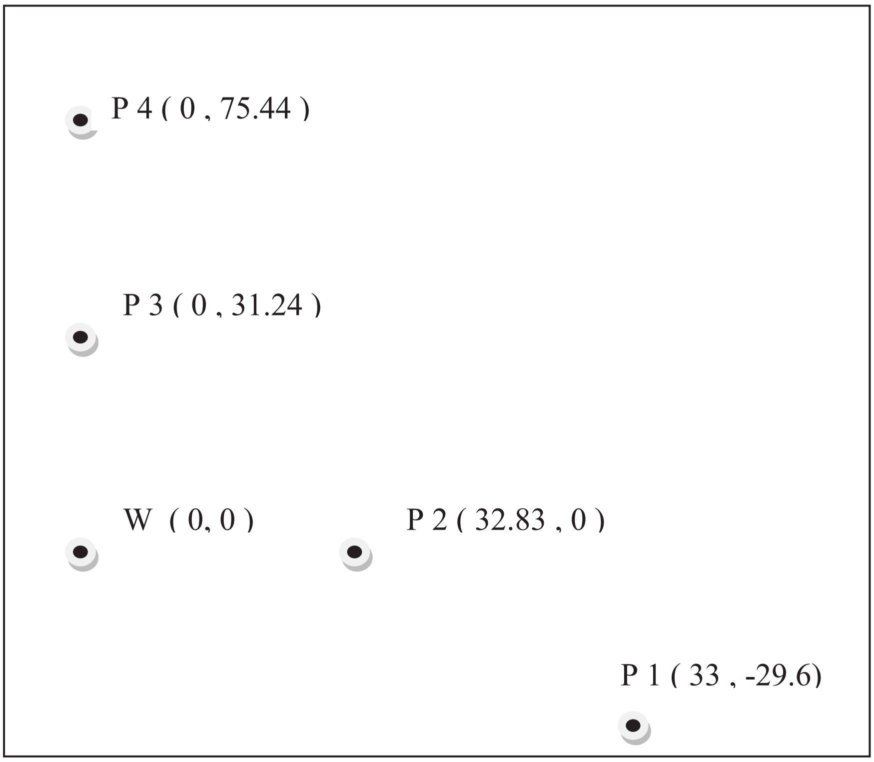


Fig. (5.6) *Well and piezometers configuration of pumping test*

Table (5.3) *Water level in the piezometers during the pump test*

Time (min)	Volume-Meter (m^3)	W (0)*	P3 (31.2m)*	P2 (32.8m)*	P1 (71.9m)*	P4 (75.4m)*
0	26	2.06	2.01	1.95	1.96	1.85
1.0		2.59				
2.0		3.67	2.17			
3.0		3.69		2.16		
4.0		3.71		2.21		
5.0		3.74	2.32			
10.0		3.8	2.36	2.28	2.14	1.99
20.0		3.85	2.45	2.33	2.2	
30.0		3.89	2.45	2.36	2.21	2.04
60.0	91	3.93	2.47	2.39	2.24	2.1
90.0		3.95	2.49	2.43	2.26	2.17
120.0	155	3.96	2.5	2.43	2.27	2.18
150.0		3.97	2.5	2.43	2.27	2.19
180.0	220	3.98	2.5	2.43	2.27	2.19
240.0	289	3.98	2.5	2.43	2.27	2.19
300.0	345	3.98	2.5	2.43	2.27	2.19

**:Radial distance from pumping well to piezometer*

The pumping test results were analyzed to obtain the soil parameters using the following relations:

1. Drawdown-distance from pumping well relationships as shown in Fig. (5.7).
2. Drawdown-time relationships as shown in Fig. (5.8)

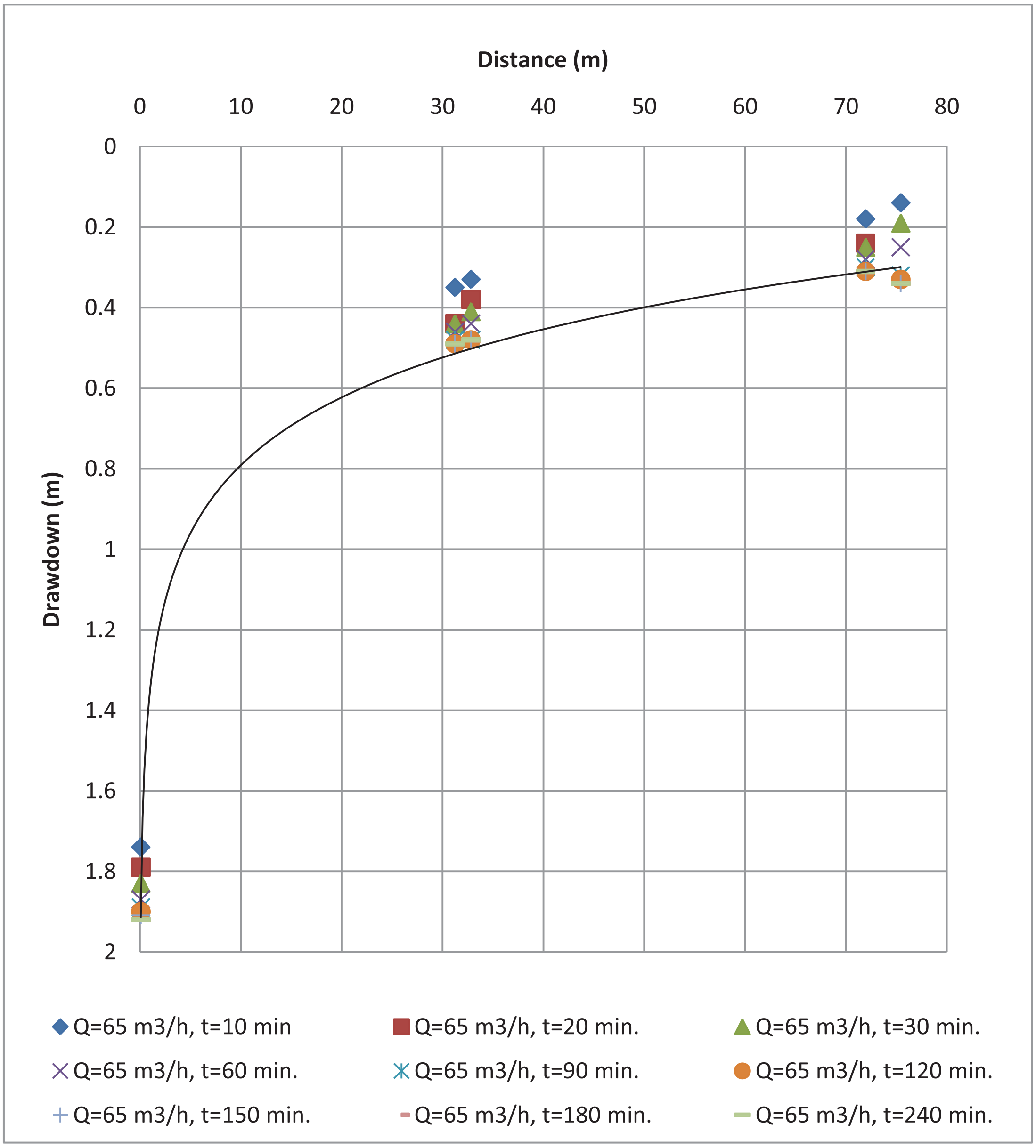


Fig. (5.7) *Drawdown distance relationships for different times*

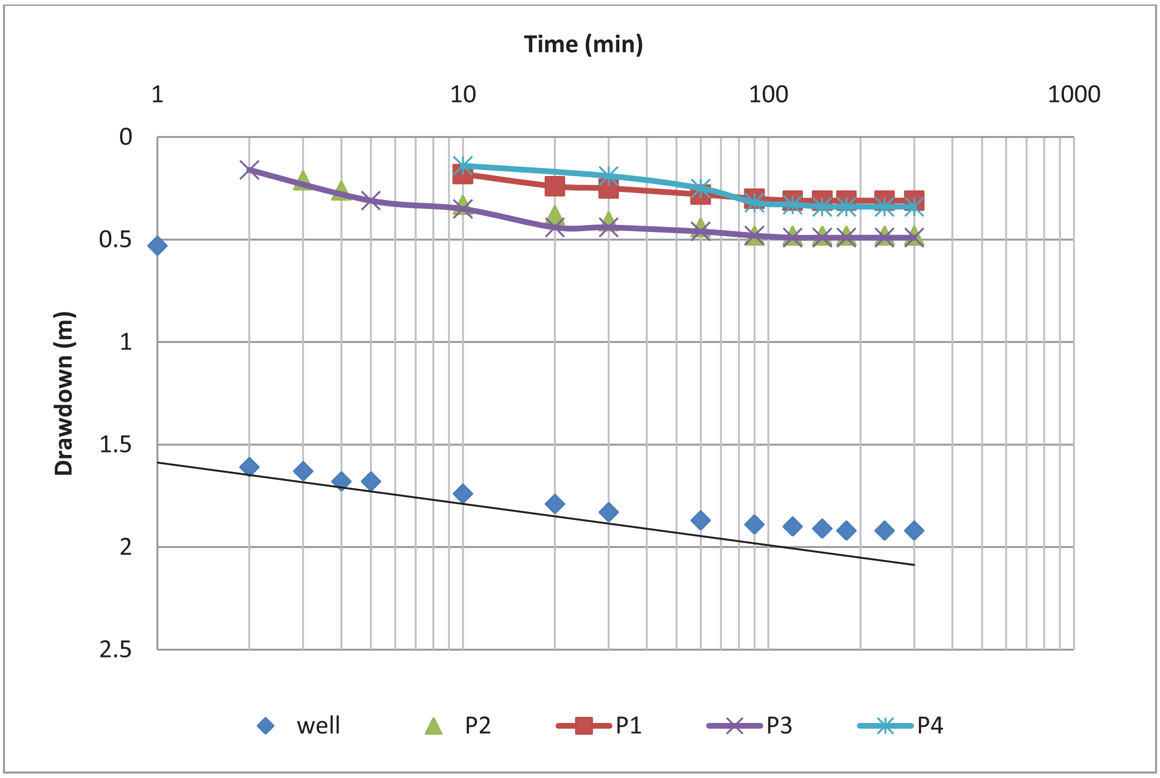


Fig. (5.8) *Drawdown-Time relationships for different piezometers*

5.4.2 Deep wells

The spacing between deep wells varies from site to another. The spacing between the 22 wells in this project ranges between 20 and 30 m. The depth of each well is 22 m. Figure (5.9) shows a schematic section in one of the used deep wells. The construction utilized a 40 cm outer pipe which was removed after construction of the well and utilizing an inner strainer pipe of 10 inch. Filter packs with size between 1 to 20 mm were used outside the 10 inch pipe. The submersible pump of 65 m^3/h discharge rate is attached to a riser pipe of 4-inche diameter.

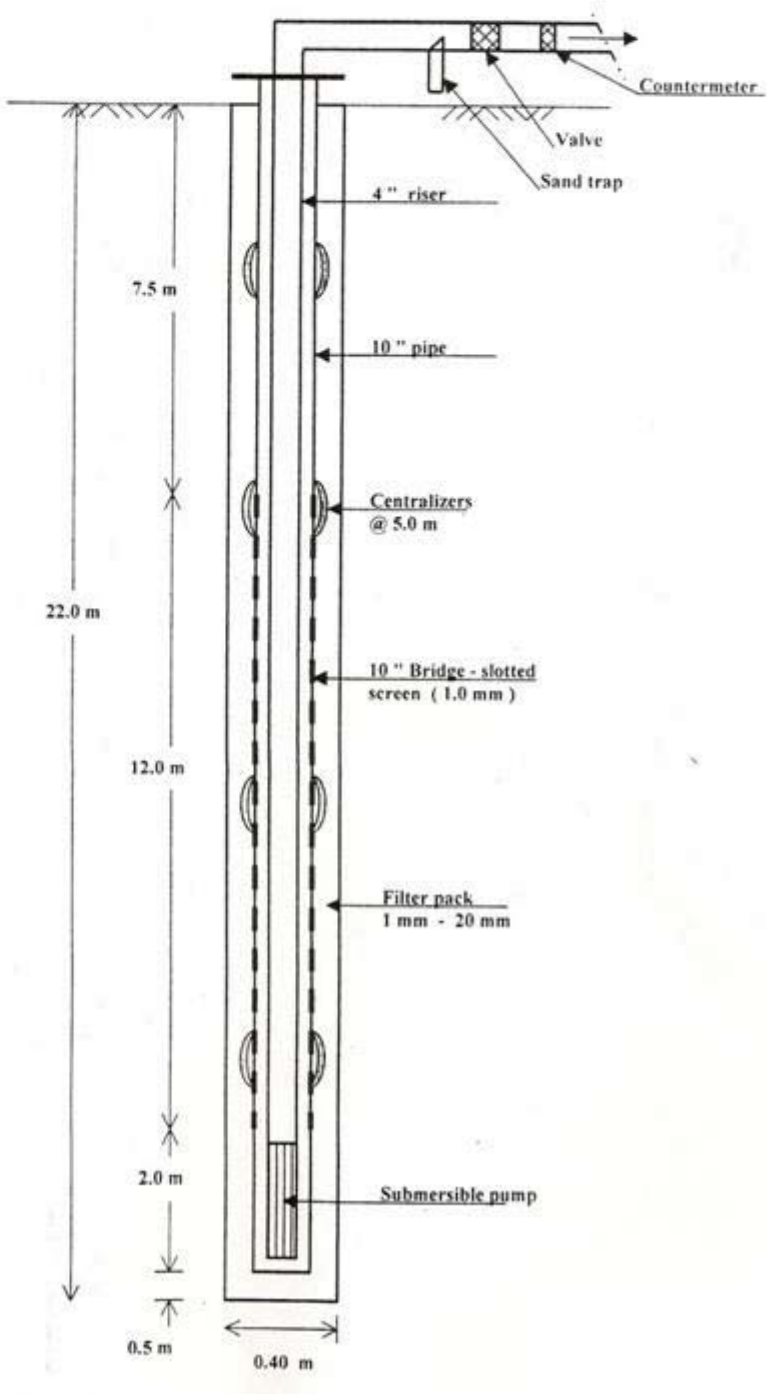


Fig. (5.9) *Deep well detailing*

5.4.3 Piezometers

The piezometer and the observation well are the fundamental tools for measuring the hydraulic head in an aquifer and for evaluating the performance of dewatering systems. While the terms piezometer and observation well are commonly interchanged, the term piezometer is more precisely defined as a device that measures the pressure in a confined isolated zone, while an observation well is a device that measures the water level in unconfined or unisolated zone. However, in common use, they are used interchangeably to describe any device for determining water head. The piezometer seems a simple tool but it can be subtly complex, and misinterpretation of piezometer data can result in serious difficulties with performance of a dewatering system.

In this project, each piezometer consists of 2.0 inch PVC pipe with 1.0 m piezometer tip embedded in filter pack starting 1.0 m before the piezometer tip. The piezometers were installed at depth of 15 m using rotary drilling machine. Figure (5.10) shows a schematic section diagram in one of the piezometers.

The water level in the piezometers was measured two times daily during actual pumping using deep meter of electric probe type with neon lamp and alarm as shown in Fig. (5.11). The water level can be measured by inserting the tip of the electric probe till it touches the water and the electric lamp and alarm will turn on. The piezometers were distributed in the mid distance between the deep wells as shown in figure (5.5).

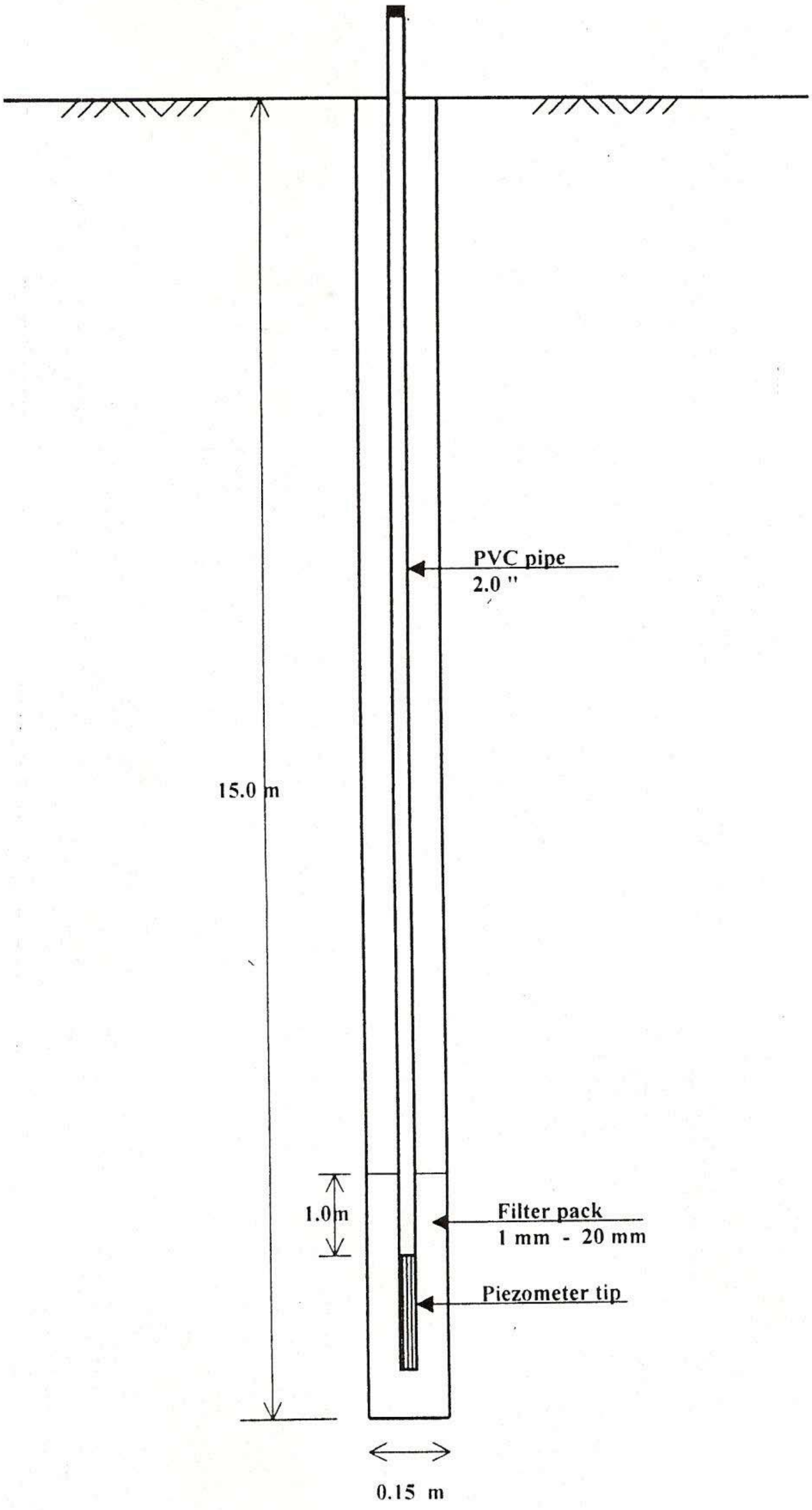


Fig. (5.10) *piezometer detailing*

Fig. (5.11) *Water level indicator*

5.4.4 Dewatering process

The project and hence the dewatering process were divided into four parts (A, B, C and D). Each part used an average number of eight wells as shown in Figure (5.5). The total number of wells for the whole project was twenty-two wells. The construction started with phase D then C, B and A consequently. Figure (5.12) illustrates the position of the wells and the piezometers in stage D. The first stage D operated with eight wells (W1, W2, W3, W4, W5, W6, W7, W8) and four piezometers (P1, P2, P3, P4) to lower the groundwater table to level -4.75 m in the piezometers. After excavation, foundations and one floor were constructed to maintain equilibrium of the uplift.

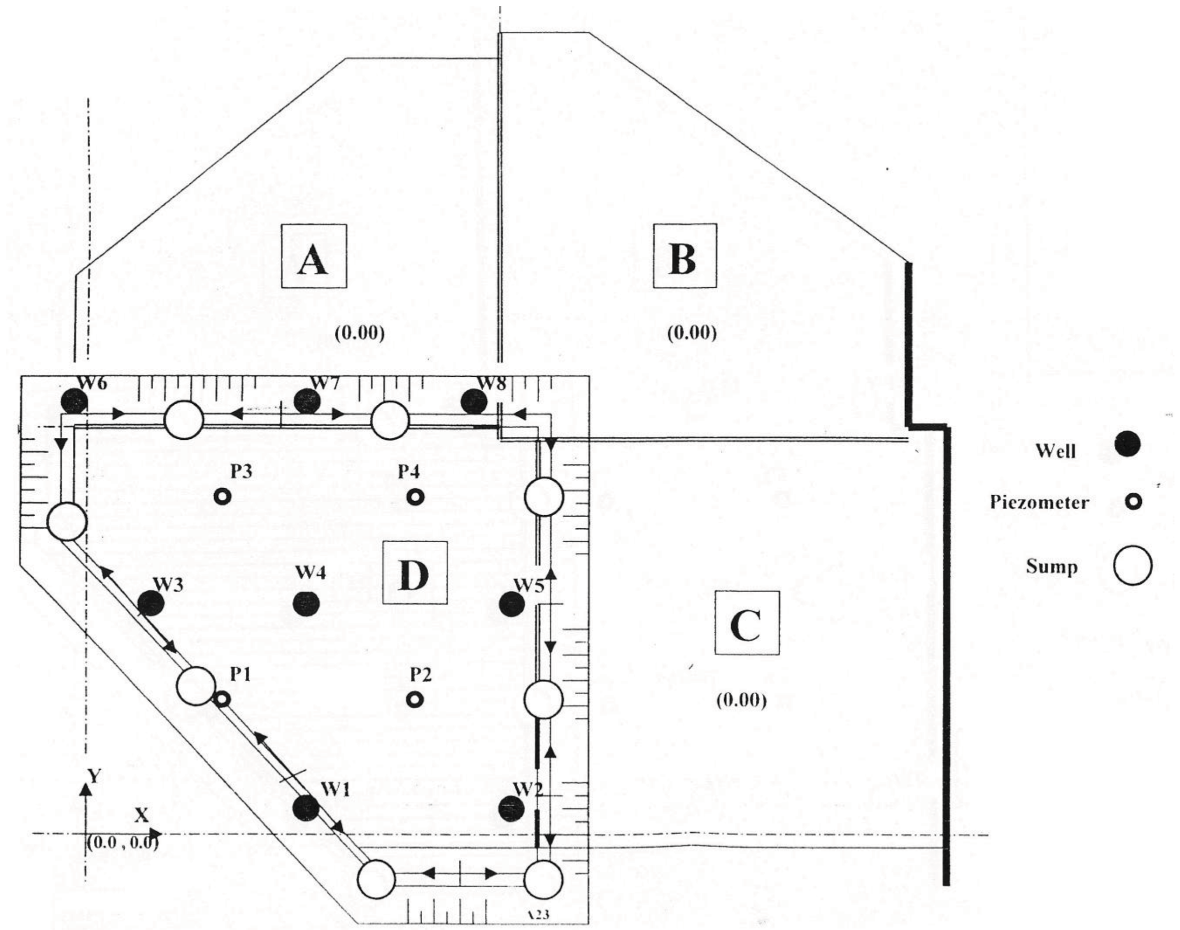


Fig. (5.12) *Wells and Piezometers in part D*

After boring the plain and reinforced concrete for the footings of part D, excavating the six wells (W9, W10, W11, W12, W13, W14) and four piezometers (P5, P6, P7, P8) in stage C started. Figure (5.13) illustrate the wells and piezometers positions. The operation procedure is as follows:

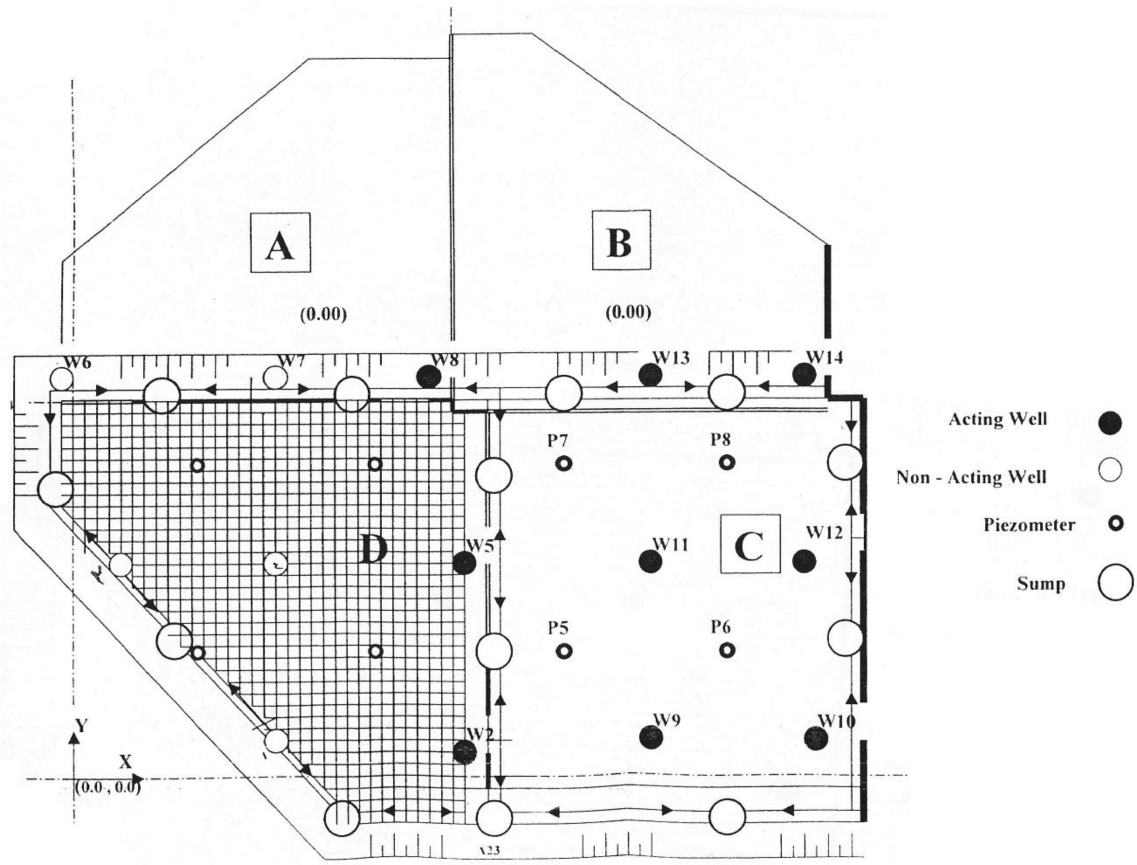


Fig. (5.13) *Wells and Piezometer in part C*

1. Close wells W1 and W4 gradually.
2. Operate wells W9, W11 then close W3.
3. Operate wells W10, W12 then close well W7.
4. Operate W13, then close W6, then operate W14.
5. Adjust the discharge in the wells to reach level (-4.75 m) in piezometers of part C.
6. Start construction in part C and grout wells W1, W3, W4 and piezometers P1, P2, P3, P4 in part D.

After boring plain and reinforced concrete of part C and backfilling between R.C footings, excavate wells W19, W18, W17, W16, W15 and piezometers P9, P10, P11 in part B.

Figure (5.14) illustrate the wells and piezometers in part B, The procedures of operation is as follow:

1. Close wells W2, W9, and W10 gradually and operate wells W15, W16, W17.
2. Close wells W5, W11, W12 and operate W18, W19.
3. Adjust the discharge in the wells to reduce the groundwater table to level (-4.75m) in piezometers of part B.
4. Grout the wells W5, W11, W12, W2, W9 and W10 and the piezometers P5, P6, P7 and P8 in part C.

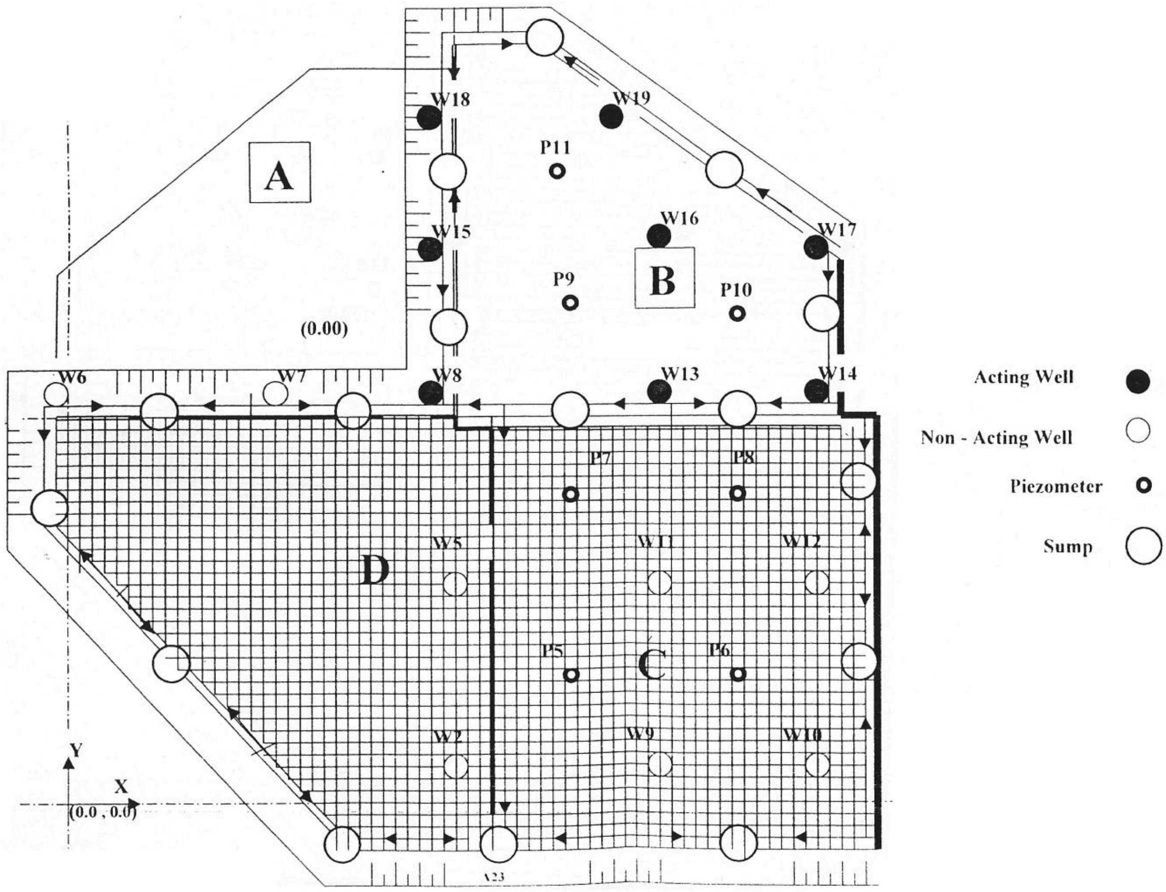


Fig. (5.14) *Wells and Piezometer in part B*

After finishing P.C and R.C footing in part B, excavate wells W20, W21, W22 and piezometers P12, P13, P14 and P15, Figure (5.15) illustrate the wells and piezometers in part A, then the operation procedures will be as follow:

1. Close wells W17 and W14 gradually and re-operate wells W6 and W7.
2. Close wells W13, W16 and W19 and operate wells W20, W21 and W22
3. Adjust the discharge in the wells to reduce the groundwater table to level (-4.75 m) in the piezometer of part A.
4. Grout the wells and piezometers of part B.

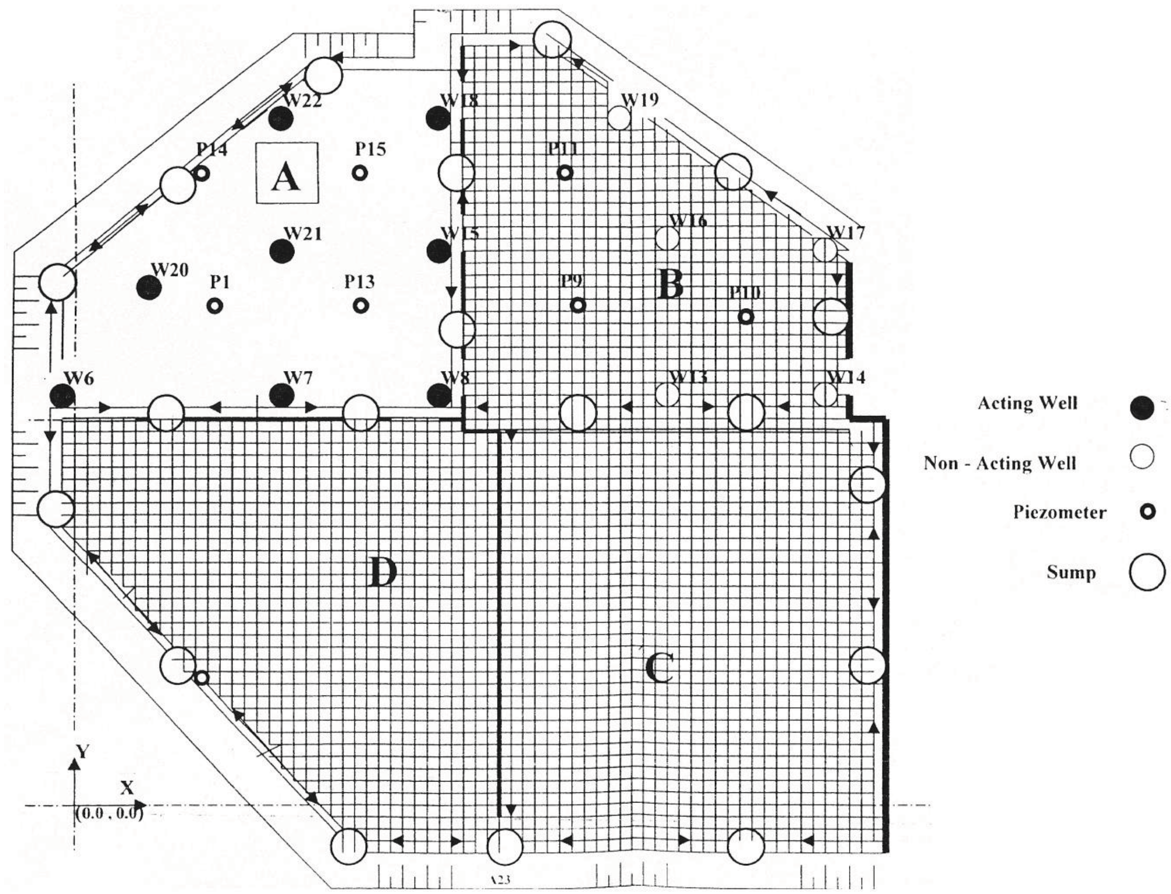


Fig. (5.15) *Wells and Piezometer in part A*

5.5 Dewatering Design

This research will focus on dewatering of part D whose monitoring data was the most reliable.

Structure information

Area of part D that needs Dewatering		
= 65 * 65	=	4225 m^2
Required drawdown	=	4.75m

Well information

Radius of well r_w	=	200 mm
Length of blank part of the well L_1	=	6 m
Length of the screen L_2	=	12 m
Length of sand trap L_3	=	2.5 m
Depth of sand layer H	=	50 m
Depth of upper clay layer under water	=	6.0 m

Aquifer information

Coefficient of permeability k	=	0.031 cm/sec
Coefficient of permeability k		
= 0.031 * 60 * 60 / 100	=	1.11 m/hr
Coefficient of permeability k		
= 0.031 * 10000	=	307 mic/sec

Flow regime ……….**Confined flow**

Bottom depth of the well		
= $L_1 + L_2 + L_3$		
= 5.5 + 12.5 + 2.5	=	20.5 m
Depth of well under clay		
= Bottom depth of well - Depth of clay layer under water		
= 20.5 – 6	=	14.5 m

Well penetration/layer thickness (α)

= Depth of the well / Depth of sand layer H

= 15 / 50 = 0.29

Partial penetration factor G (Eqn. 2.16)

$$= \frac{H1}{H}\left[1 + 7\sqrt{\left(\frac{r_w}{2H1}\right)} \times cos\left(\frac{\pi H1}{2H}\right)\right]$$

$$= 0.3\left[1 + 7\sqrt{\frac{0.2}{2\times15}} \times cos\left(\frac{\pi\times15}{2\times50}\right)\right] = 0.441$$

Difference between initial & final levels (h_2-h_1) = 4.75 m

Dewatering calculations with deep wells using Dupuit Equation (Eqn. 2.14)

$$Q = \frac{2 \times \pi \times k \times D \times G \times (\mathrm{h}_2 - \mathrm{h}_1)}{ln\left(\frac{r_2}{r_1}\right)}$$

Calculation of approximate number of wells

Equivalent wells radius $r_e \equiv r_1$

= $(\text{area} / \pi)^{0.5}$

= $(4225/3.14)^{0.5}$ = 36.67 m

Radius of influence $R \equiv r_2$

= (1:3) (h_2-h_1) $k^{0.5}$

= 1.5 * 4.75 * ($307^{0.5}$) = 245 m

≈ 250 m

Total expected discharge Q

= (2 * 3.14 * 1.11 * 50 * 0.441 * 4.75) / Ln (250 / 36.67)

= 383 m^3/hr

Optimum well discharge According to Sichardt & Kyrieleis (1930)

= 0.0247 * L_2 * r_w * $k^{0.5}$

= 24.7 * 12 * 0.20 * ($1.11^{0.5}$) = 62.3 m^3/hr

Chosen pump capacity Q per well = 65 m^3/hr

Approximate number of wells

= Total expected discharge / Chosen pump capacity

= 383 / 65 $\approx$ 6 wells

Constant C_1

= (Q / 2 * π * k * D * G)

= 65 / 2 * 3.14 * 1.11 * 50 * 0.441 = 0.42

Check by accumulated drawdown at specific points

REAL WELL			Well number
X	Y	Type	
33	4	1	W1
64	4	1	W2
12	35	1	W3
33	35	1	W4
64	35	1	W5
-3	63	1	W6
33	63	1	W7
58	63	1	W8

LEGEND	
TYPE 1	Active well
TYPE 0	Idle well

Check point	Coordinates		D.D. (m)	Piezometer
	X	Y		
1	22.00	20.00	6.81	P1
2	49.00	20.00	6.92	P2
3	17.00	49.00	6.98	P3
4	40.00	49.00	7.30	P4
5	75.00	20.00	5.97	P5
6	96.00	20.00	4.72	P6
7	75.00	49.00	5.98	P7
8	96.00	49.00	4.72	P8

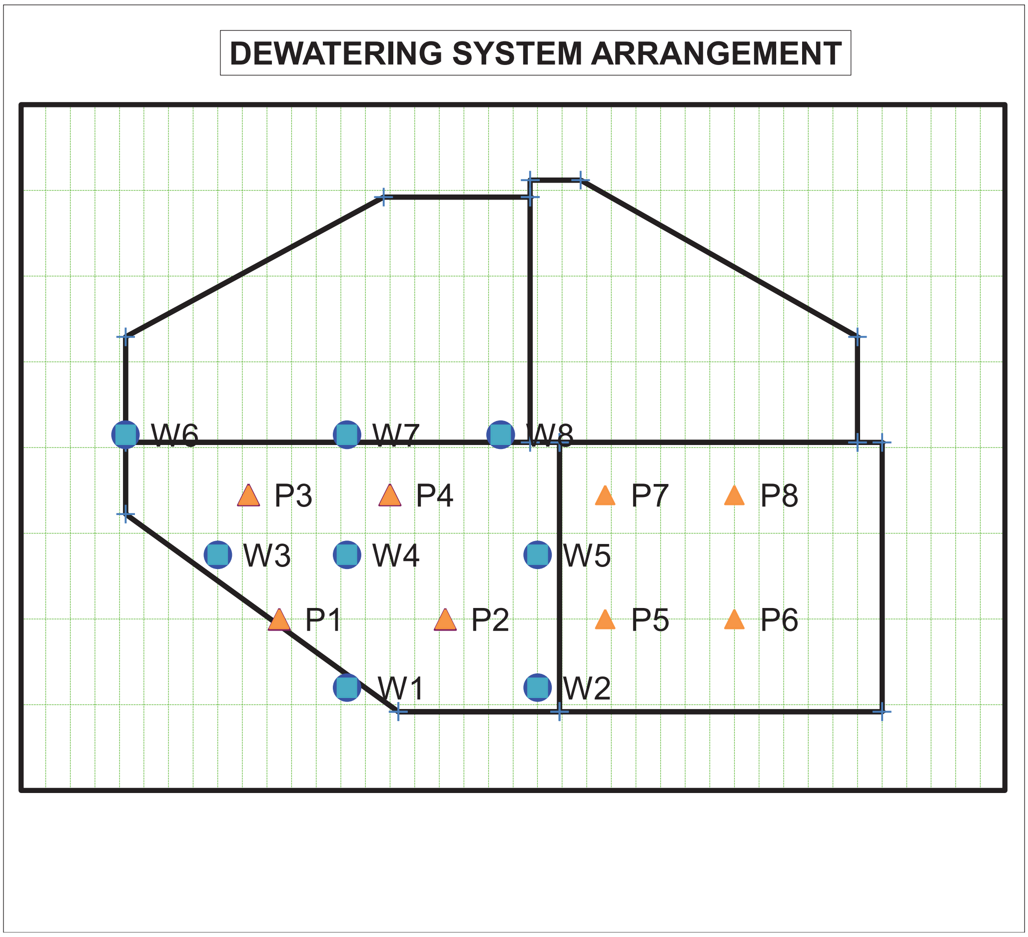


Fig (5.16) *Plan view illustrates wells and piezometer*

B.3 CALCULATION AT PIZOMETER (1)				
P 1		Well No.	r (m)	Ln(R/r)
x=	22.0			
y=	20.0	1	19.416	2.535
		2	44.944	1.696
		3	18.028	2.609
		4	18.601	2.578
		5	44.598	1.704
		6	49.739	1.594
		7	44.385	1.708
		8	56.080	1.474
			sum	15.90

Required minimum drawdown = 4.75 m

Accumulative drawdown

$= C_1 * \Sigma \text{Ln}(R/r)$

$= 0.42 * 15.90$ = 6.81 m *safe*

B.3 CALCULATION AT PIZOMETER (2)

P 2		Well No.	r (m)	Ln(R/r)
x=	49.0			
y=	20.0	1	22.627	2.382
		2	21.932	2.413
		3	39.925	1.814
		4	21.932	2.413
		5	21.213	2.447
		6	67.476	1.289
		7	45.880	1.675
		8	43.932	1.719
			sum	16.15

Required minimum drawdown = 4.75 m
Accumulative drawdown = 6.92 m *safe*

B.3 CALCULATION AT PIZOMETER (3)

P 3		Well No.	r (m)	Ln(R/r)
x=	17.0			
y=	49.0	1	47.760	1.635
		2	65.069	1.326
		3	14.866	2.802
		4	21.260	2.444
		5	49.041	1.609
		6	24.413	2.306
		7	21.260	2.444
		8	43.324	1.733
			sum	16.30

Required minimum drawdown = 4.75 m
Accumulative drawdown = 6.98 m *safe*

B.3 CALCULATION AT PIZOMETER (4)

P 4		Well No.	r (m)	Ln(R/r)
x=	40.0			
y=	49.0	1	45.541	1.683
		2	51.000	1.569
		3	31.305	2.057
		4	15.652	2.751
		5	27.785	2.177
		6	45.222	1.690
		7	15.652	2.751
		8	22.804	2.374
			sum	17.05

Required minimum drawdown = 4.75 m
Accumulative drawdown = 7.30 m *safe*

B.3 CALCULATION AT PIZOMETER (5)

P 5		Well No.	r (m)	Ln(R/r)
x=	75.0			
y=	20.0	1	44.944	1.696
		2	19.416	2.535
		3	64.761	1.331
		4	44.598	1.704
		5	18.601	2.578
		6	89.067	1.012
		7	60.108	1.405
		8	46.239	1.667
			sum	13.93

Required minimum drawdown = 4.75 m
Accumulative drawdown = 5.97 m *safe*

B.3 CALCULATION AT PIZOMETER (6)

P 6		Well No.	r (m)	Ln(R/r)
x=	96.0			
y=	20.0	1	65.000	1.327
		2	35.777	1.924
		3	85.329	1.055
		4	64.761	1.331
		5	35.341	1.936
		6	107.935	0.820
		7	76.276	1.167
		8	57.385	1.451
			sum	11.01

Required minimum drawdown = 4.75 m
Accumulative drawdown = 4.72 m *safe*

B.3 CALCULATION AT PIZOMETER (7)

P 7		Well No.	r (m)	Ln(R/r)
x=	75.0			
y=	49.0	1	61.555	1.381
		2	46.325	1.666
		3	64.537	1.334
		4	44.272	1.711
		5	17.804	2.622
		6	79.246	1.129
		7	44.272	1.711
		8	22.023	2.409
			sum	13.96

Required minimum drawdown = 4.75 m
Accumulative drawdown = 5.98 m *safe*

B.3 CALCULATION AT PIZOMETER (8)

P 8		Well No.	r (m)	Ln(R/r)
x=	96.0			
y=	49.0	1	77.421	1.152
		2	55.218	1.490
		3	85.159	1.057
		4	64.537	1.334
		5	34.928	1.948
		6	99.985	0.896
		7	64.537	1.334
		8	40.497	1.800

sum 11.01

Required minimum drawdown = 4.75 m
Accumulative drawdown = 4.72 m *safe*

B.3 CALCULATION AT POINT - WELL LOCATIONS

DD = 3.31 m

WELL No	**W4**	Well No.	r (m)	Ln(R/r)
x=	33.00			
y=	35.00	1	31.000	2.067
		2	43.841	1.721
		3	21.000	2.457
		4	0.000	0.000
		5	31.000	2.067
		6	45.607	1.681
		7	28.000	2.169
		8	37.537	1.876

sum 14.04

Final drawdown at well = 493.99 m
Accumulative drawdown = 9.32 m *safe*
Immersed screen length = 8.18 m

Chapter 6

APPLICATION OF FINITE DIFFERENCE MODEL ON THE CASE STUDY

6.1 Introduction

Numerical models are currently considered as powerful tools used in solving many civil engineering problems. Their practical potential was recognized in the mid-1960s but their complexity limited their use. In the recent decades, powerful personal computers were developed and the use of such numerical models became easier and more common. The associated computer programs need to be extensively tested to be used with trust in actual studies. There are several types of numerical methods and in this study we will use the finite difference approach will be used to carry out the numerical analysis of the dewatering system that is under consideration.

This chapter discusses the application of the numerical modeling software called MODFLOW to analyze the case study and compare the numerical results with those of the analytical design method and the in-situ measured drawdown values.

6.2 The Finite Difference Method

Finite difference models are generally easier to use and utilize for practical projects. To use this method for analyzing the case study, the MODFLOW software was chosen. MODFLOW was developed by the U.S. Geological Survey (2000), and is available with the public domain. Versatile user friendly, pre- and post- processing programs that stream line data entry, model construction and analysis are also commercially available.

Although groundwater modeling allows the analyst to solve ever more complex problems in dewatering, it is not without limitations. These limitations are:

- A model is an approximation to the real groundwater status.
- The model will not be reliable until it is calibrated with appropriate field data.
- A calibrated model is only a solution of many ones to a given field data.
- The model is not a substitute for the practical experience and the judgment of the dewatering system and its operation in the field.
- The reliability of the model will increase as the quality of the subsurface data increase.

Hence, it is evident that the results of such numerical technique with actual field measurements from case studies is highly needed. In fact, this is the main aim of this research.

6.2.1 Case study application

To start in this part, the site exploration data, pump test results and the dewatering system details and monitoring data were collected. All these information are summarized in Chapter 5 of this thesis. The first step will be modeling the pump test on the MODFLOW program to calibrate the model.

The coefficient of permeability is the most contradicting and effective parameter in the dewatering process. The ways to evaluate this parameter were summarized in Chapter 2 of this thesis.

6.2.1.1 Pump test analysis

The field pump test performed in the site as illustrated in Section 5.4.1 was examined by the Soil Mechanics and Foundation Engineering Unit, Faculty of Engineering, Ain Shams University. The analysis to get the coefficient of permeability was performed using three methods; analytical analysis of Ain Shams Geotechnical Unit, analytical analysis of this thesis and numerical analysis using the MODFLOW software.

1- The analysis of Ain Shams Geotechnical Unit is illustrated below:

The number of production wells:	1 well
The number of piezometers:	4
Outer diameter of well:	40-45 cm
Diameter of well pipe:	25 cm
Diameter of piezometer:	10 cm
Total depth of piezometers:	15 m

Coefficient of permeability is determined from different cases of well and piezometers setting after reaching the steady state condition. It was considered that the steady condition was reached after about 2 hours and 30 minutes from the testing start. Therefore, it was estimated that the average value *"$k_{av.}$"* = 0.024272 cm/sec.

2- The analytical analysis performed for this research using equation that Ain Shams Geotechnical Unit used but the average coefficient of permeability was estimated with an average value *"$k_{av.}$"* = 0.0307194 cm/sec.

3- The calibration of the coefficient of permeability was performed by MODFLOW program. To create the model for this program the mesh and boundary conditions have to be identified. Therefore, after studying the site conditions a mesh was designed to work for both the pump-test and the model of the project in order to reduce the error induced from changing the mesh. Figure (6.1) illustrates plan view of the mesh of the pump test model.

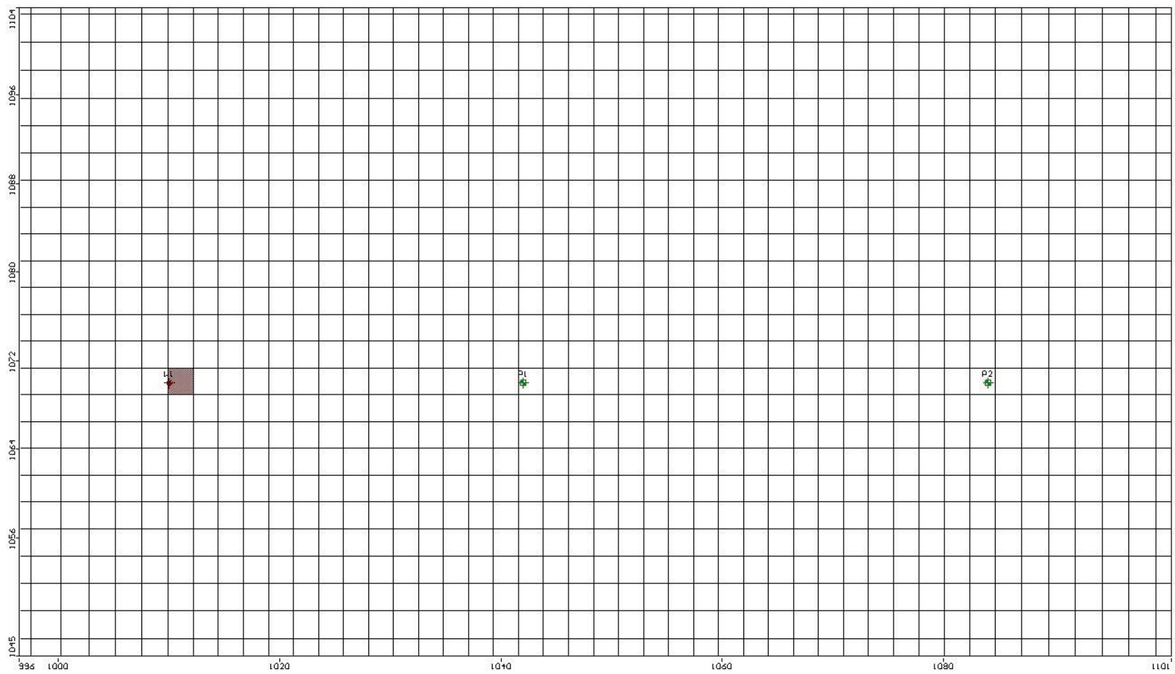


Fig. (6.1) *Illustrate a close plan view of the mesh and the location of the well (red point) and the piezometers (green points)*

The discharge from the test well continued for five hours and has an average rate of 65 m^3/h. Table (5.3) shows the readings compiled during the pump test. The well was 22 m in depth and perforated after seven and half meters from the ground surface (the thickness of the clay layer).

The pump-test applied in the field used four piezometers for monitoring the groundwater table. Figure (5.6) shows the well and piezometers arrangement. However, it was observed that the radial distance of the nearest two piezometers (P2 and P3) were similar also their drawdown readings were almost the same. The same observation was noted for the other two piezometers (P1 and P4). Hence, when modeling the pumping test the mean radial distance and drawdown of the nearest pair were calculated and modeled as one piezometer, and the same thing was considered for the other pair.

The calibration involved many trials due to some mistakes and program errors. At first, the mesh used in the pumping test calibration was different from the mesh of the whole project. The second trial was performed using the same mesh of the pumping test calibration and for the whole case study to reduce the error from changing the mesh. Some of these errors were also because the mesh was too fine. Therefore, after coarsing the mesh to appropriate degree the errors reduced significantly. A trial model was also made to check if the pump place away from the mesh center will affect the drawdown. It was concluded that this factor will make no significant difference on the calculated drawdown.

The result of the calibration (coefficient of permeability) was found to have a wide range depending on the ratio between horizontal and vertical permeability. When the ratio of $k_h/k_v=1$ (isotropic condition) the range of k was between 0.016 and 0.024 cm/sec. This range of k gives drawdown results from 80 to 120% of the real readings recorded during the pumping test. Figure (6.2) shows the relation between the permeability when $k_h/k_v=1$ and average percentage of drawdown. From this analysis using isotropic condition, the best result was when k = 0.016 cm/sec.

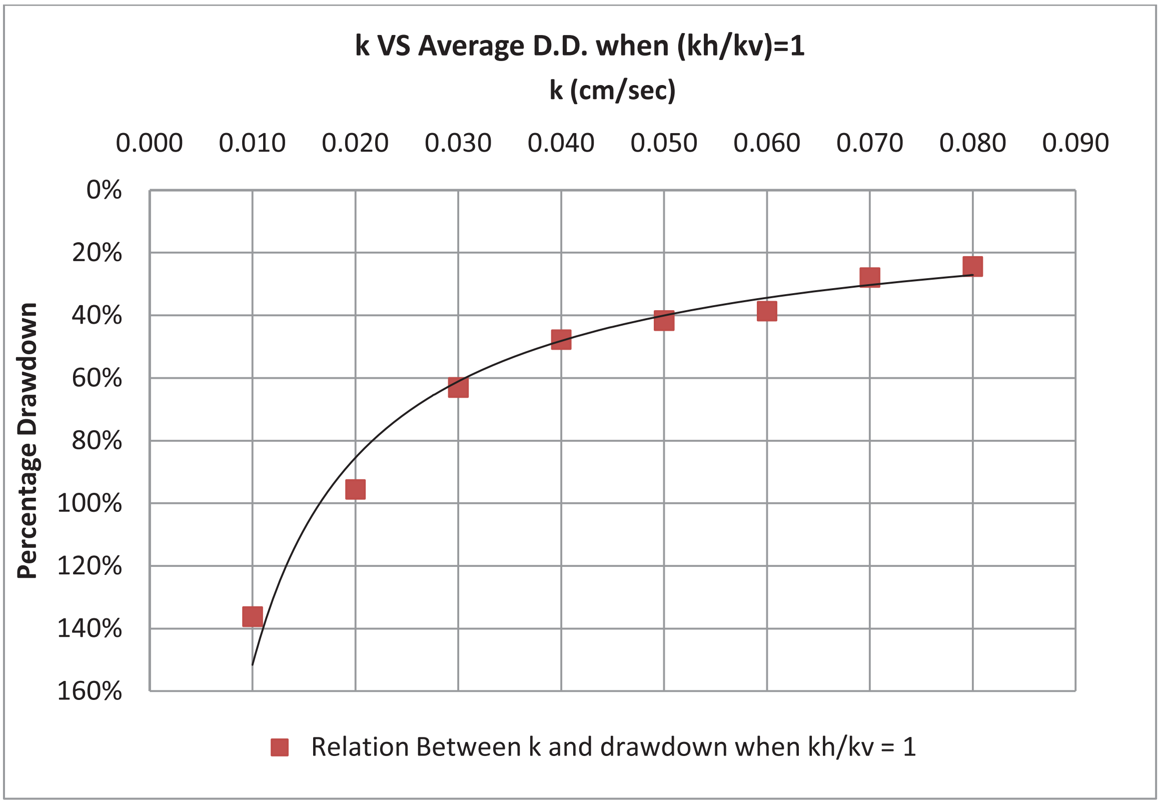


Fig. (6.2) *Relation between k and percentage of the real drawdown when $k_h/k_v=1$*

The k_h to k_v ratio that achieved the required drawdown ranged from 0.4 to 2 leading to so many values of k_v and k_h. The best value was k_h = 0.02 cm/sec and k_v = 0.01 cm/sec with ratio equals to 2.

6.2.2 Application on case study using MODFLOW

After calibration on the pump test model and as illustrated in the previous paragraphs, several values of coefficient of permeability were obtained. These k values have to be applied on the main project using the MODFLOW program and compare its results with the real monitored drawdown. To achieve such a target, a numerical model has to be established for the project. After studying the available monitoring data, it was realized that the project was executed on four stages starting with stage D then C, B and at last A. The available data for the first part D was the most accurate one because many parameters in the dewatering system changed during construction and operation of the other stages as discussed in Section (5.4.4).

Stage D consisted of eight pumps and eight piezometers (four of them are located outside the boundaries of area D). The wells specifications were all the same and as discussed in Section (5.4.2) and illustrated in Figure (5.9). The quantity of flow was monitored during a whole month and the quantity of flow for each pump was considered the average during one month. The pumps power varies from 60 to 90 m^3/hr.

The utilized observation wells (or piezometers) were 15 m deep and their specifications are given in Section (5.4.3) and the schematic diagram illustrating their details is shown in Figure (5.10).

The model was generated and the coefficients of permeability from the previous section were applied in addition to the coefficient of permeability calculated from Hazen formula, Eq. (2.13):

$$k_{hazen} = \frac{0.01 * 0.2382}{6} = 0.000397\, m/sec$$

The results of this analysis are shown in Table (6.1). The calculated drawdown in this table was taken as the average drawdown at the location of the eight piezometers as a percentage from the real readings from the site.

Table (6.1) *Results of modeling the case study using MODFLOW*

Trial	K_h (cm/sec)	K_v (cm/sec)	K_h/K_v	Average D.D	Notes
1	0.02100	0.02100	1.00	103%	k from MODFLOW analysis of pump test ratio=1
2	0.02000	0.01000	2.00	113%	k from MODFLOW analysis of pump test ratio=2
3	0.03970	0.03970	1.00	68%	k from Hazen empirical equation
4	0.03072	0.03072	1.00	87%	k from my pump test analytical analysis
5	0.02427	0.02427	1.00	91%	k from pump test analysis of Ain-Shams

6.2.3 Application on case study using Dupuit empirical method

Preliminary analysis of stage D using the empirical method was illustrated in Section (5.5). In this part of the chapter, Dupuit empirical method, utilizing the different values of k, was applied to observe the difference between drawdown in the two techniques. Table (6.2) shows the results of the analysis by this method. Knowing that the effect of coefficient of permeability on the radius of influence was taken into consideration to assure the accuracy. That is because the radius of influence increases by increasing the coefficient of permeability, theoretically.

The way to estimate drawdown in the piezometers was the superposition method, by applying it to estimate every well effect on the piezometer then adding the results to reach the cumulative drawdown at the required point. This method has some disadvantages such as it assumes isotropic conditions and idealized boundary conditions.

Table (6.2) *Results of modeling the Case Study using the empirical method*

Trial	K_h (cm/sec)	K_v (cm/sec)	K_h/K_v	Average D.D	Notes
1	0.02100	0.02100	1.00	122%	from MODFLOW analysis ratio=1
2	0.02000	0.01000	2.00	N.A	from MODFLOW analysis ratio=2
3	0.03970	0.03970	1.00	73%	k from Hazen empirical equation
4	0.03072	0.03072	1.00	97%	k from my pump test analytical analysis
5	0.02427	0.02427	1.00	127%	k from pump test analysis of Ain Shams

6.3 Discussion

Coefficient of permeability is the main variable that makes a remarkable difference on the calculated drawdown. To investigate its effect, both of the horizontal and vertical coefficients have to be considered. The question is which is more effective on the drawdown and how does it affects the overall performance. To answer this question, many numerical analyses were performed for this study. One of the permeability coefficients was kept constant and the other was changed to observe its effect. The range of the horizontal and vertical permeabilies ranged from 0.01 to 0.05 cm/sec. The analysis was conducted in steps by fixing the vertical permeability and changing the horizontal one. Table (6.3) shows the results. Figure (6.3) summarizes the main results from these analyses. The figure exhibits that the drawdown increases with increase of the degree of anisotropy, k_h/k_v-values. The figure exhibits also that for a k_h/k_v greater than 2, the variation of the drawdown is insignificant, for the site considered in the current study.

It was observed that the horizontal permeability has small effect when compared with the effect of the vertical permeability as shown in figure (6.3). It affects the drawdown in the farthest piezometers more than the nearest one but the effect remains small. Also it was apparent that the relation between the drawdown and the coefficient of permeability in isotropic condition ($k_h/k_v = 1$) is not linear (for k values between 0.01and 0.08 cm/sec) and the effect is more significant for low values of the coefficient of permeability.

Table (6.3) *Numerical analysis of the pump test*

Trial	K_h (cm/sec)	K_v (cm/sec)	K_h/K_v	Percentage Drawdown from the real reading in piezometer 7	Percentage Drawdown from the real reading in piezometer 8	Average Drawdown
1	0.01000	0.01000	1.00	169%	103%	136%
2	0.02000	0.01000	2.00	133%	73%	103%
3	0.02000	0.02000	1.00	118%	73%	96%
4	0.03000	0.01000	3.00	98%	52%	75%
5	0.03000	0.02000	1.50	80%	48%	64%
6	0.03000	0.03000	1.00	78%	48%	63%
7	0.04000	0.01000	4.00	78%	42%	60%
8	0.04000	0.02000	2.00	67%	33%	50%
9	0.04000	0.03000	1.33	61%	36%	49%
10	0.04000	0.04000	1.00	59%	36%	48%
11	0.05000	0.01000	5.00	67%	33%	50%
12	0.05000	0.02000	2.50	55%	30%	43%
13	0.05000	0.03000	1.67	51%	30%	41%
14	0.05000	0.04000	1.25	49%	30%	40%
15	0.05000	0.05000	1.00	53%	30%	42%
16	0.06000	0.06000	1.00	47%	30%	39%
17	0.07000	0.07000	1.00	35%	21%	28%
18	0.08000	0.08000	1.00	31%	18%	24%

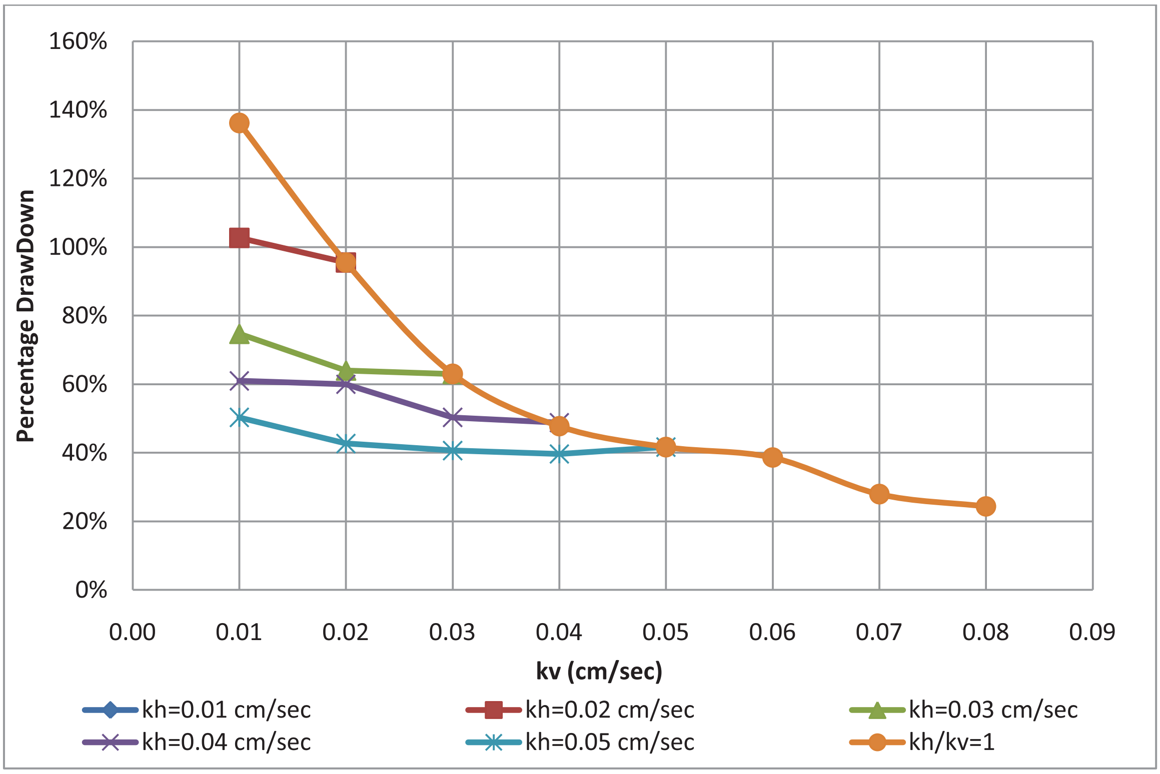


Fig. (6.3) *Relation between k_v and drawdown at constant k_h values*

Figure (6.4) sums up the results of all these analyses where five values of the coefficient of permeability were selected as discussed in Sections 6.2.1 and 6.2.2. The first one 0.02427 cm/sec that is the result value from the analysis of the pump test of Ain Shams Soil Mechanics and Foundation Engineering Unit and when it was applied on the pump test model made on MODFLOW the results were about 79% of the real drawdown of the pump test. By applying it on the project model on MODFLOW its results were the most reliable (91% of the real drawdown) and the best from all other k values despite of it is under estimating the drawdown. Applying this value on Dupuit equation does not give the best result but it was acceptable over estimating the drawdown (127% of the real drawdown).

The second k value was from the analysis of this thesis equals to 0.03072 cm/sec. Its result on the pump test model is (61%) but it gives good values for the draw down in the project model (87%) and also in Dupuit model (97%).

The third value obtained from Hazen formula (0.0397 cm/sec) results drawdown in the pump test model (53%) of the real pump test drawdown and values of 68% and 73% for the project model and Dupuit model, respectively.

The fourth and fifth values of the coefficient of permeability (k_h = 0.02 at k_v = 0.01 and k = 0.021 cm/sec, respectively) were the permeability values that gives the best results during calibration of the pump test model on MODFLOW. The fourth value was of anisotropic condition with ratio of k_h/k_v equals to 2 and the fifth was for isotropic condition with ratio k_h/k_v equals to 1. The drawdown percentage of the pump test varies from 90 to 110%. The drawdown results for the project numerical model ranges from 103 to 113% from the real drawdown. For Dupuit model the average drawdown was equal to 127% of the real drawdown. But, applying the anisotropic condition on Dupuit model was not applicable.

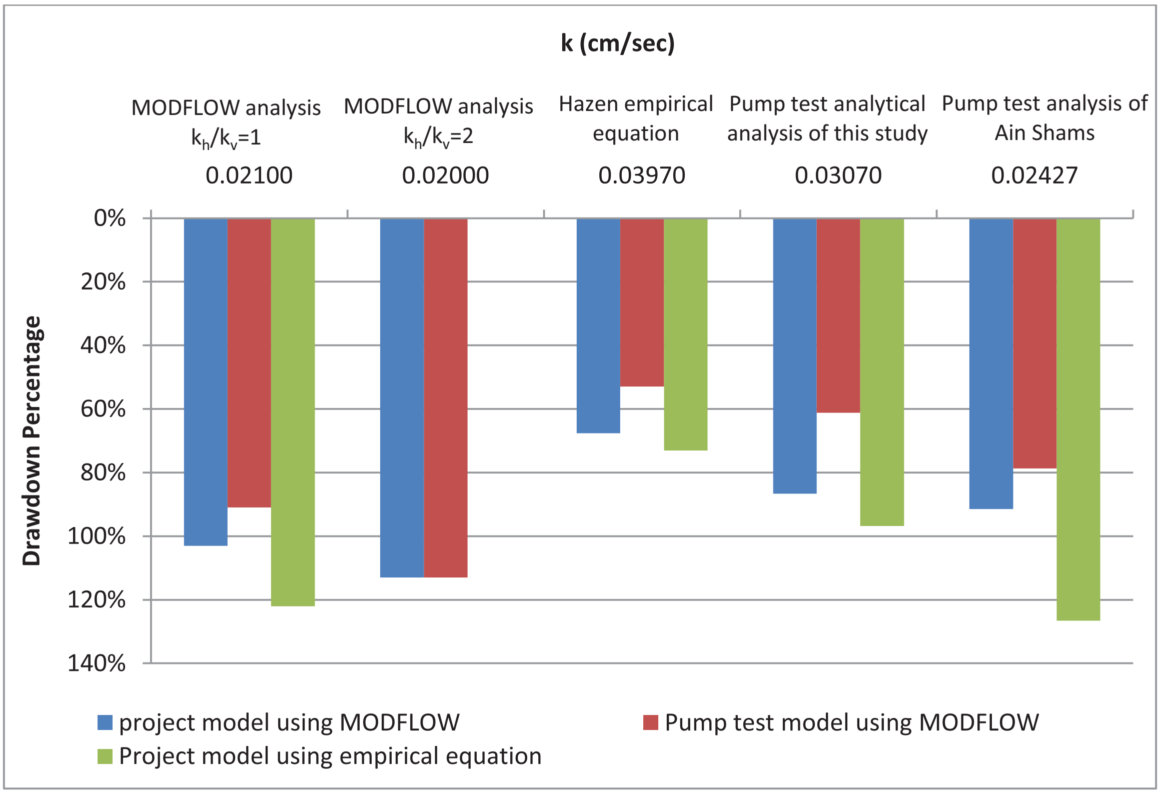


Fig. (6.4) *Application of selected k values on the models in concern*

Figure (6.4) exhibits that the drawdown calculated whether by MODFLOW or Dupuit model for the whole project is close to the calculated from pump test model by MODFLOW.

The best results of k with respect to the whole project not the pump test only is that calculated from the pump test calibration (the isotropic condition), i.e. k = 0.021 cm/sec, using the MODFLOW software.

A comparison between the water profile monitored on site and the results of the various analysis approaches is shown in Fig. (6.5). The Figure exhibits that the general trend of the calculated water table profiles are similar to the monitored water table which validate these methods for usage. In this analysis the coefficient of permeability used for the application was equal to 0.021 cm/sec. which is the most reliable value as discussed in the previous paragraph.

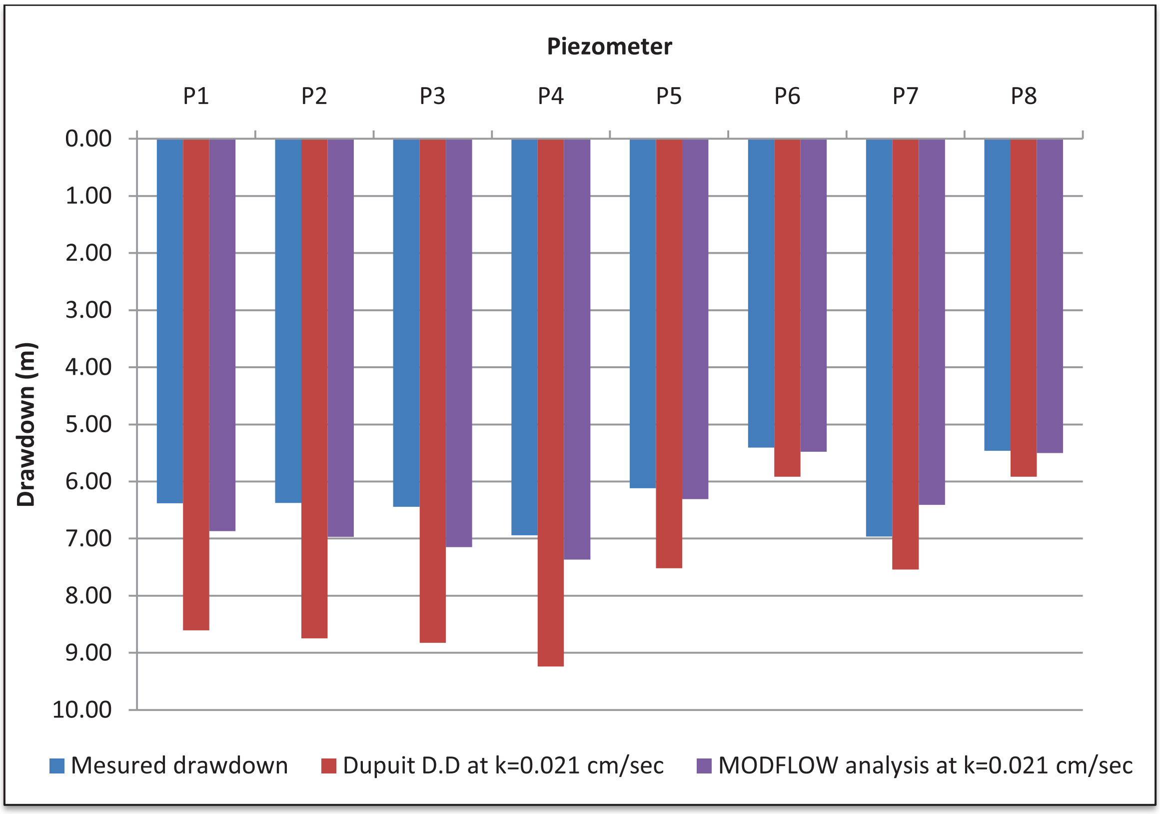


Fig. (6.5) *Comparison of water table profile from various methods*

Chapter 7

CONCLUSIONS AND

RECOMMENDATIONS FOR FURTHER STUDIES

7.1 Conclusions

7.1.1 Parametric studies

Analytical and numerical parametric studies on the behavior of a single partially penetrated pumping test through a confined aquifer were conducted for this thesis. The results of these two studies are presented and discussed in Chapters 3 and 4, respectively. The analytical parametric study was based on the modified Dupuit equation of radial flow of partially penetrating deep well in confined aquifer. On the other hand, the numerical parametric study utilized the finite difference method using the MODFLOW program. From the compiled relationships between the various soil-water-well parameters and the calculated drawdown at different distances from a single pumping well, the following main points were concluded for the investigated case of partially penetrated well through a confined aquifer:

1. Increasing the thickness of the confined aquifer tends to decrease the drawdown for a range of up to 30 m. Beyond this value, no significant influence was noticed. Therefore, aquifer thickness of 50 m was considered an adequate value to represent the very thick sand layers particularly for the parametric numerical study.
2. It is evident that the drawdown tends to decrease when the well penetration into the confined aquifer increases. The associated drawdown profile tends to be non-linear for distances from the pumping well and up to about 20 m; then tends to be linear at larger distances. It is also noticed that the calculated drawdown at a distances more than 50 to 80 m is not significantly affected by the well penetration into the confined aquifer, for the considered site conditions.
3. Both the analytical and numerical parametric studies confirmed that the drawdown is highly affected by the rate of pumping. The drawdown increases by increasing the well discharge and this effect extends to a radial distance of about 80 m from the pumping well for the investigated partially penetrated well through the confined aquifer of this study.
4. As expected, the drawdown tends to increase by increasing the radius of influence. Comparing the calculated drawdown profiles with the results of the pumping test, it was estimated that the representative radius of influence should be in the order of

about 150 m for the investigated partially penetrated pumping well through the confined aquifer and site condition of this study.

5. The effect of varying the well radius was investigated in the analytical parametric study. The results indicated that increasing well radius leads to a slight decrease of drawdown.
6. Both the analytical and numerical parametric studies confirmed that the drawdown is highly affected by the value of the coefficient of permeability. The drawdown increases for lower values of this coefficient in isotropic condition.
7. The anisotropic condition of the coefficient of permeability ($k_h \neq k_v$) was investigate in the numerical parametric study using the finite difference method and the results showed that the drawdown is more affected by the value of horizontal coefficient of permeability.

7.1.2 Case study

This thesis presents the results of detailed back-analyses of a dewatering system that was used to control the groundwater level at a site northeast of Cairo-Egypt using the finite difference modeling approach. The employed dewatering system composed of twenty two deep wells and was monitored using fifteen piezometers. Dewatering for the deep excavation of the project was implemented in four stages to reduce the effect of groundwater drawdown on the surrounding buildings. From the results of the back-analyses of the first stage of this case study, the following were concluded:

1. It is evident that drawdown prediction of this case study is highly dependent on the value of the coefficient of permeability of the existing confined aquifer. Furthermore, for anisotropic condition the effect of k_h is more pronounced than that of k_v.
2. Five probable values of the coefficient of permeability of the case study were investigated. Four of them were based on assessment of the pumping test results using both the analytical and numerical techniques while the fifth was based on empirical evaluation relating the coefficient of permeability to the soil gradation. These values ranged between about 0.02 to 0.04 cm/sec for isotropic condition and k_h= 0.02 cm/sec and k_v= 0.01 cm/sec for the anisotropic condition.

3. Results of the detailed back-analyses of the case-study confirmed that the best matching of the measured drawdown at the piezometers were for the case of isotropic coefficient of permeability of 0.02 cm/sec. The associated calculated drawdown profile using the MODFLOW program was similar in shape and the closest to the measured profile.

7.2 Recommendations for Further Studies

Based on the conducted research of this thesis, the following items are recommended for further studies in order to advance the present status of this topic:

1- It is apparent that any tool of numerical modeling of dewatering systems will not be reliable until it is calibrated with appropriate field data from large number of case-studies. Therefore, back-analyses of more case-studies are needed particularly those with different boundary conditions such as; retaining walls, more complex soil strata, discharge and recharge sources.

2- Cases of preconstruction prediction could contribute significantly to closing the present gap between the design and construction of dewatering systems. Therefore, coupling type-A prediction, using conventional as well as more sophisticated analyses, with detailed pumping tests and full scale in-situ monitoring should be attempted on as many projects.

3- Relationship between the drawdown and the associated settlement of adjacent buildings and other structures should be investigated particularly for relatively weak subsurface conditions. Such studies will be most useful if both numerical finite element modeling and field geotechnical monitoring were utilized.

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الشقين الحجقن للنزح الوحف ىف ىالقا هرة

)دارصت عوليت(

فىخصرل لرصلت

ا ّحأيش لي َ ِل جفيت عه ُ ل ثّريهئ حح ج ا بضرتيت عظيي جذاف ً ٍان ً ك ٍ ا وُجث ش ع ه ُحصرَيى ان َش ش ُ ع،طشيت افخيُز أ خلكفعت. جزبملكيشي ٍ اَش بلك لمان ُخجيش لثب ُافخيُز لك ٍ ب ّسب بي شلك مان يَب ِرلجفيتف ٍ جذبل نيؤ دمل ُ اعادة ُحصرَيى ان َش ش ُ ع ُ فبُعض لالبي ب ُ انخخ ه ُ ع ُ ٌ.

ان ٓذف ي ٍ رُ ان حثمحشيلي فج ٍة ان خبي ٍ انحصرَي ى ُ افخيُز يفش ُ بسيغ انُزح انجف ُ رنكخمي مرابخ ى شي ٍ ان ٍ لج أنَ بل ف صُ ب تاخيي ذ. رنك ع ٍ ط شيكي شافبيت ليَ ب ث انخض ش يف ي ٍ الابار ان ُحعذدة ُ يسيخبث أفيب ٍ رلجفيت ثب ءانخ ُ يريم ُبس خ ب يُخبئج انخي ل حصل عليب ي ٍ ان ُ عادلات خ ج ش يىيت ُ انخحمي مان عذد ٍ.

ف ُ رشاس بتيخ ى انحث ع ي ُ ش ُ عي خ ش ف ُ ا دانيُ م انالبت ف ُ شَبلشفان مب ُش ة ي مغ ان ٍل غ ف ُي ُطت بعليت نب فنبل ى ب ويت. ان َش شُ ع خ ى غ ُ بز ع ه ُ اسب غ ُ ي ش اح م رُنك خ ُ س ي ّ فع ُ ي ز ع ُ يت ان ُزح ا ن جف ُ حمي نش ُ ية ش ع ه ُ ان َ ب ب ُ أن َ حي ُط ت بيخ ك ٍ ظ بوان ُ زح انجف ُ ن ُ ش ش ُ ع ك م ي ٍ ب ن ن ا و عشر و ب ُ ئ ش ا ُ خ ُ سيت عشر ب وي خ ُ ش.

انخحي مان عذ د ي ط شيت Finite difference بس خخ ذ ي ش َبيج MODFLOW. رُ بانش َبيج يخ ص ص ف ُ سلرت حشكت انيَب ِرلجفيت ُ يسيخبخ ب رُ ان حث د ُ سخ نك ذ ي ٍ ا ع ب ُ ي ت رُ بانش َبيج ف ُحصرَيى مشروعات انُزح انجف ُ.

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