4.2 Development Potential of Water Sources

Based on the basic investigation conducted in the Study as discussed in Chapter 3, water availability from the four alternative sources is assessed in this Section. This is compared with the future water demand presented in Section 4.1.

4.2.1 West Baray

Water availability from West Baray is calculated based on data provided in Section 3.2. From the potential inflow capacity from the river to the West Baray, the minimum inflow capacity is estimated to be 34.1 million m^3. The average potential inflow capacity is an estimated 45.8 million m^3, which is equivalent to around half the average total Siem Reap River flow of 90.9 million m^3.

The minimum potential capacity is estimated at 40.7 million m^3 (34.1 million m^3: inflow, 6.6 million m^3: storage by rainfall). Considering the maximum use of 36 million m^3, the remainder will still be 4.7 million m^3. The remainder can provide a water supply of around 12,900 m^3/day for the whole year.

It will be most preferable that the surplus water use for water supply be done only by coordinating with MOA based on the integrated reservoir operation. However, according to circumstances, it might be necessary to secure additional reservoir capacity for the dry season (8 months) by the following methods in addition to the integrated operation:

- Excavation in the eastern part of the reservoir (around 4.0 million m^3): or
- Increment of capacity by maximum rise of water level up to EL. 19.9 m (25.3 m, reading) on condition that the West Mebon be preserved (around 4.3 million m^3).

In any alternatives, it is essential to rehabilitate the French Weir, American, and Takav Gates, tree cutting and clearing of inner surface of dikes, rehabilitation of the reservoir ring dyke and bank protection, and gravel metalling of the inspection road on the dikes to facilitate the reservoir work effectively.

4.2.2 Siem Reap River

Water availability from the river is calculated based on the data provided in Section 3.2. The possible amount of intake from the river is estimated by taking into account the methodology widely applied in Japan.

Possible amount of intake=10-year drought flow - Maintenance flow - Vested water use

10-year Drought Flow: The 10-year drought discharge (minimum 355-day flow/10 years) is assumed to be 0.48 m^3/s at the French Weir.

Maintenance Flow: There is no regulation on the maintenance flow of the Siem Reap River. However, it is necessary to maintain a certain amount of water for conservation of the natural river environment. Therefore, the maintenance flow at the weir is assumed here equivalent to the estimated minimum daily flow at 0.36 m^3/s in year 1990.

Vested Water Use: There is no water use right along the Siem Reap River at present. However, the river water has been utilized mainly for the Crocodile Weir Irrigation System. The intake amount is approximately 1.0 m^3/s in average. It is necessary to respect this vested intake amount; the Siem Reap flow can not satisfy this amount in the driest season in reality, though.

Possible Amount of Intake: The amount of possible intake becomes 0 m^3/s and accordingly the Siem Reap River option (with no dam) can be completely discarded from surface water source alternatives.

Possible way to use Siem Reap River water: Some kind of storage is required if Siem Reap River water is used as water supply source. One possibility is to construct a dam in the upstream of the river in the vicinity of Kulen Mountain. Another possibility is to renovate the North Baray to store water in rainy season and to release it in dry season.

4.2.3 Lake Tonle Sap

Lake Tonle Sap is the largest freshwater lake in Southeast Asia and its storage is more than 1,300 million m^3 even at the lowest water level as shown in Supporting Report, Annex 3.2.1. The possible yield from the lake can be considered practically as unlimited.

From a hydrological viewpoint, the intake should be secured even when water level becomes below approximately EL. 0.7 m, which is 20-year return period minimum water level in the dry season. Therefore, the available intake site is recommended to be located at least 4 km offshore from the existing boat house.

The distance from the town center will exceed 19 km. It is naturally impossible to convey the raw water by gravity to the city center.

4.2.4 Groundwater

(1) Groundwater Extraction and Land Settlement

As explained in Section 3.4, the wells of WT4, LTa-2, and LTb-2 installed in alluvial and diluvial sand layer have a good capacity. It was also found that when a screen longer than 12 m is installed in the vicinity of WT4 or near the area of airport, a discharge of more than 444 liters/minute can be expected. At the same time, careful consideration has to be given to the fact that excessive groundwater abstraction will cause land subsidence in and around the well field including Angkor Wat area. The groundwater is expected to be able to meet the increasing demand for water in the Siem Reap Region. However, once the adverse side effects of groundwater development occur, it takes a long time for the aquifer to recover. Therefore, groundwater development and management should consider how much and from where an aquifer can supply water for a long time without causing adverse side effects.

So the maximum yield from the well field near WT4 will be governed by the maximum allowable land subsidence. As explained in Section 3.4, the natural fluctuation of GWL in front of Angkor Wat area was found to be 2.3 m. The land movement in the same period was recorded as 1.3 mm corresponding to the GWL fluctuation and it was found that the land movement is reversible.

In order to find out the maximum groundwater extraction, a computer simulation was conducted.

(2) Objectives, Methodology and Software

The main objective of this simulation is to make a “perennial-yield pumping plan” for the project area. “Perennial Yield” is defined as the maximum quantity of water that can be continuously withdrawn from a groundwater basin without adverse side effects. A “perennial-yield pumping plan” is a specific pattern of spatially distributed pumping that causes the evolution and maintenance of an appropriate potentiometric surface. Thus, a perennial-yield pumping plan assures a certain amount of water to the user for a long time. Such a perennial-yield pumping strategy can be computed using a steady-state combined Simulation and Optimization (S/O) model. The simulation analysis was carried out twice, that is 1st simulation by using the 8 drilling and pumping test result in the study area, and 2nd simulation after the 2 pilot wells with pumping test completed. Both simulations used the software “MODFLOW 96” to make a quasi-three dimensional groundwater flow.

A : 1st Simulation

For the computational purpose, the Project area was divided into 500 m size mesh. First, a basic geological frame, such as a layered system, and boundary conditions was delineated based on the 11 geological profiles and surface drainage conditions obtained from 1:50,000 topographical maps. Next, hydrogeological data was prepared for the model. Those data include recent groundwater distribution, rainfall, evaporation, aquifer thickness, storage coefficient, and hydraulic conductivity obtained from the pumping test in 8 locations. Finally, the model was calibrated under a transient-simulation for a one-year period (February 1998 to February 1999).

After the model was calibrated and the spatial distribution of hydrogeological parameters was determined, a steady-state simulation was made to forecast the groundwater level change if a production well was installed near Siem Reap airport.

Further, an optimal spatial distribution of pumping was determined for sustainable groundwater development by planning a candidate production well field in the south of the West Baray. The combined simulation and optimization (S/O) model was formulated to maximize the perennial-yield (steady) pumping rate from the candidate production well field subject to the physical aquifer system. The same spatial distribution of hydro-geological parameters used for the simulation model is involved in the S/O model. Because of the great concern about land subsidence around Angkor heritage area by pumping, constraints are 1) allowable maximum withdrawal rates from the well field and 2) allowable drawdowns at Angkor heritage area.

(1) Influence by New Groundwater Development

If steady pumping is implemented and maintained, the groundwater level of the aquifer will reach a certain level and, once achieved, will be maintained forever (discounting seasonal and daily changes, and assuming other recharge and boundary conditions remain constant). To evaluate influence on the surrounding area by pumping from a new well, steady-state simulations were made for 1) without the new well (present condition) and 2) with the new well, assuming the last 10 years climatic condition continues for the time being. Both results were then compared.

Location of production wells:	South of West Baray and near Siem Reap Airport (the location was selected from the viewpoint of the most suitable water quality)
Plan of groundwater withdrawal:	Case1 : 5,000 m^3/day
	Case2 : 10,000 m^3/day
	Case3 : 15,000 m^3/day

Figure 4.2.1 shows the simulated groundwater levels for the Case2, a) with pumping, represented by red contour lines, and b) without pumping, represented by black contour lines, respectively.

The groundwater drawdowns caused by new wells are given below. The drawdown means the average values in the 500 m cell.

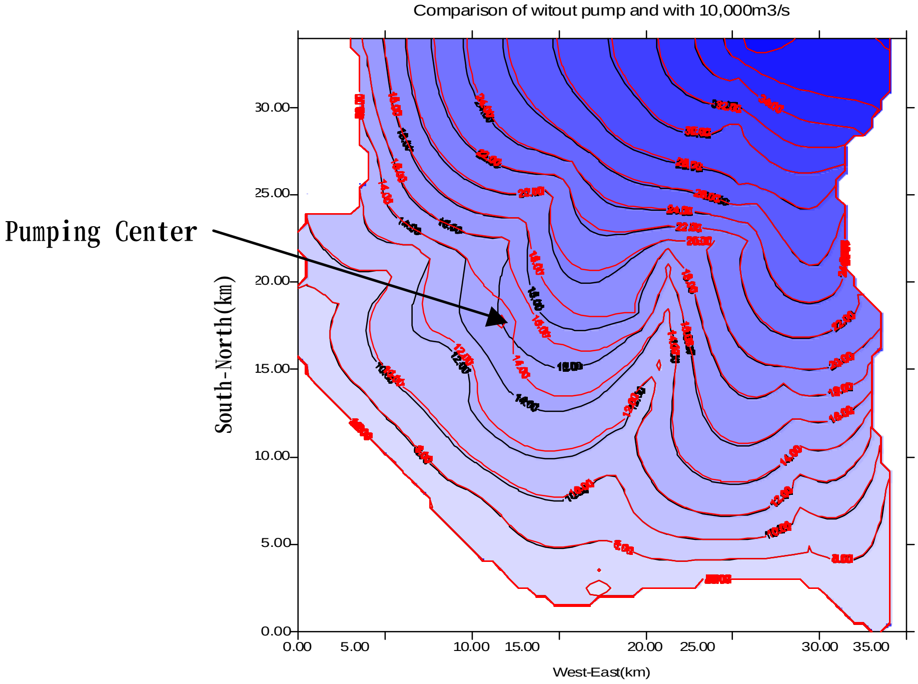
	Drawdown in the production well	Drawdown at Angkor Wat and Angkor Thom area
Case 1	2.3 m	15 - 20 cm
Case 2	5.5 m	20 - 30 cm
Case 3	8.5 m	30 - 40 cm

(2) Perennial-yield Pumping Plan

The combined simulation and optimization (S/O) model was used to optimize the perennial-yield pumping for the Siem Reap Region. The combined model can predict the behavior of a complex aquifer (a multi-layer, unconfined/confined) and optimizes the perennial–yield pumping plan for the specified objectives and constraints. The model was written in the General Algebraic Modeling System (GAMS) language. Optimization was performed with the MINOS LP solver using an advanced simplex method.

The maximum allowable well drawdown in a cell of the candidate well field (total 69 cells) is varied from 1.00 m, 2.00 m, and 3.00 m

The maximum allowable drawdown at the exact pumping center is assumed to be 10 m (around 40% of the thickness of Layer 1). On the other hand, maximum pumping rates are varied from 0.02 m^3/s to 0.01 m^3/s. The drawdown, calculated from the finite difference model, is the average over a 500 m x 500 m size cell. The maximum pumping rate is assumed as 0.02 m^3/s for each pumping cell. This can be pumped up by two wells installed in one cell (500 m x 500 m) if one well can produce 600 liters/minute. Therefore, the maximum allowable pumping rate of Q = 0.01 m^3/s were assumed as a conservative case. Also it is possible to have only one pump in one cell. In that case, the maximum allowable pumping rate is 0.02 m^3/s (around 1,200 liters/minute).



(Red Contour Line: with Pump of 10,000 m3/day)

The Study on Water Supply System for Siem Reap Region in Cambodia	**Figure 4.2.1** **Comparison with Case-2 and the Present Condition**
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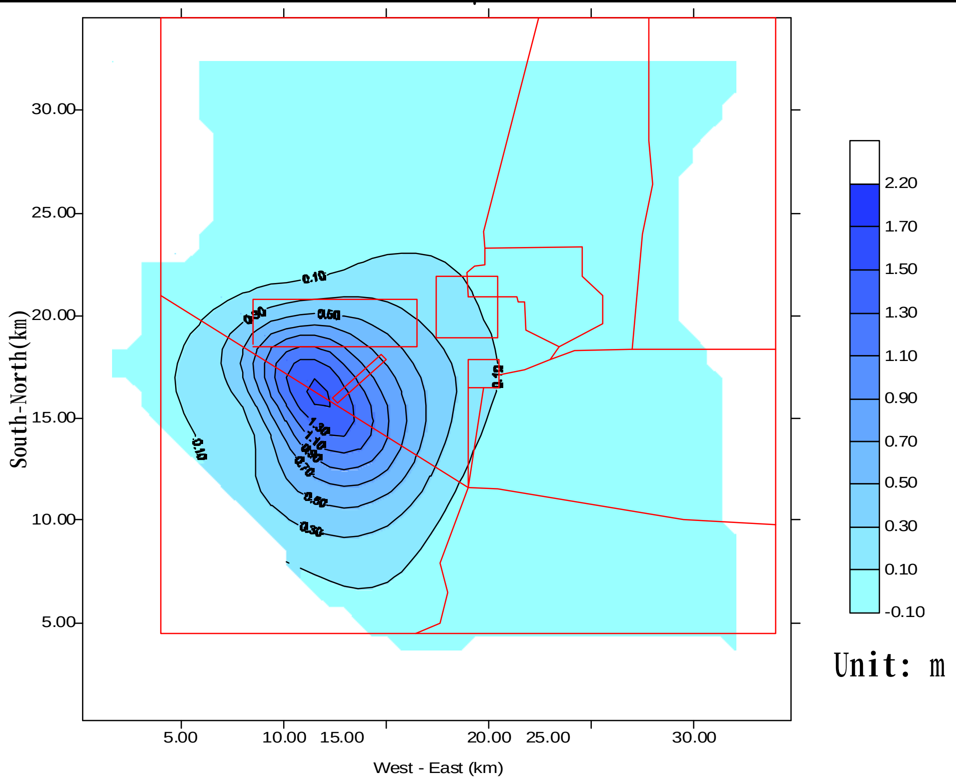


The Study on Water Supply System for Siem Reap Region in Cambodia	**Figure 4.2.2** **Drowdowns for Case-5**
Japan International Cooperation Agency	

The maximum allowable groundwater drawdown in Angkor heritage area was assumed to be 0.30 m. Judging from the result of land subsidence monitoring near Angkor Wat, the drawdown of 0.30 m by new production wells would cause the land subsidence of less than 1.6 mm.

The groundwater development area was limited in the area where the groundwater has low iron content and low clay content, based on the water quality and VLF sounding tests performed in this Study. The best access place along the National Road No.6 was selected for a planned well field from practical viewpoint.

(3) Optimization Results

Seven cases are considered for various upper bounds on head and pumping at the pumping location. This is summarized in the following table.

Optimization Results

	Conditions for Optimization			**Results**	
	GWL drawdown in Angkor area	GWL drawdown in the well field	Upper bound on allowable discharge (m^3/sec/cell)	Daily Maximum allowable discharge in total	Number of pumping cells
Case-0	0.30m	3.00m	0.20	16,330	6
Case-1	0.30m	1.00m	0.01	9,850	31
Case-2	0.30m	2.00m	0.01	14,170	18
Case-3	0.30m	3.00m	0.01	14,342	17
Case-4	0.30m	1.00m	0.02	9,850	31
Case-5	0.30m	2.00m	0.02	14,947	10
Case-6	0.30m	3.00m	0.02	15,638	10

The total of 31 wells in Case-1 and Case-4 is not economically practical because of well construction cost and land acquisition. Due to high drawdown than other cases, Case-3 and Case-6 may cause stopping or less production of the shallow wells. In conclusion, the Case-5 is the best plan at present. However, the maximum allowable pumping rate should be considered around 12,000 m^3/day (80% of the pumping rate obtained from the model) since some planned production wells in the Case-5 have more discharge than that of the WT4 monitoring well. Figure 4.2.2 shows the drawdowns for Case-5.

B: Pilot Well Construction

In order to confirm the possibility of the groundwater development and to check its feasibility regarding the yield and quality of the groundwater, two pilot wells were newly constructed at the planned well field. The locations of the pilot wells are both ends of the conceivable well field along the National Road. The wells are named as PP-99-1 for the western end, and PP-99-2 for the eastern end of the well field. The location of the two pilot wells is shown in Figure 4.2.3.

(1) Drilling Result and Well Design Concept

The geological sequence, electrical logging result and well structure for PP-99-1 and PP-99-2 are shown in Figure 4.2.4 and Figure 4.2.5, respectively.

The design is based on the concept that the allowable drawdown should be within the 5 m to limit the lowering of the groundwater at the Angkor heritage area less than 30 cm, thereby minimizing harmful effect of land subsidence to the heritage foundation.

The Specification of Well:

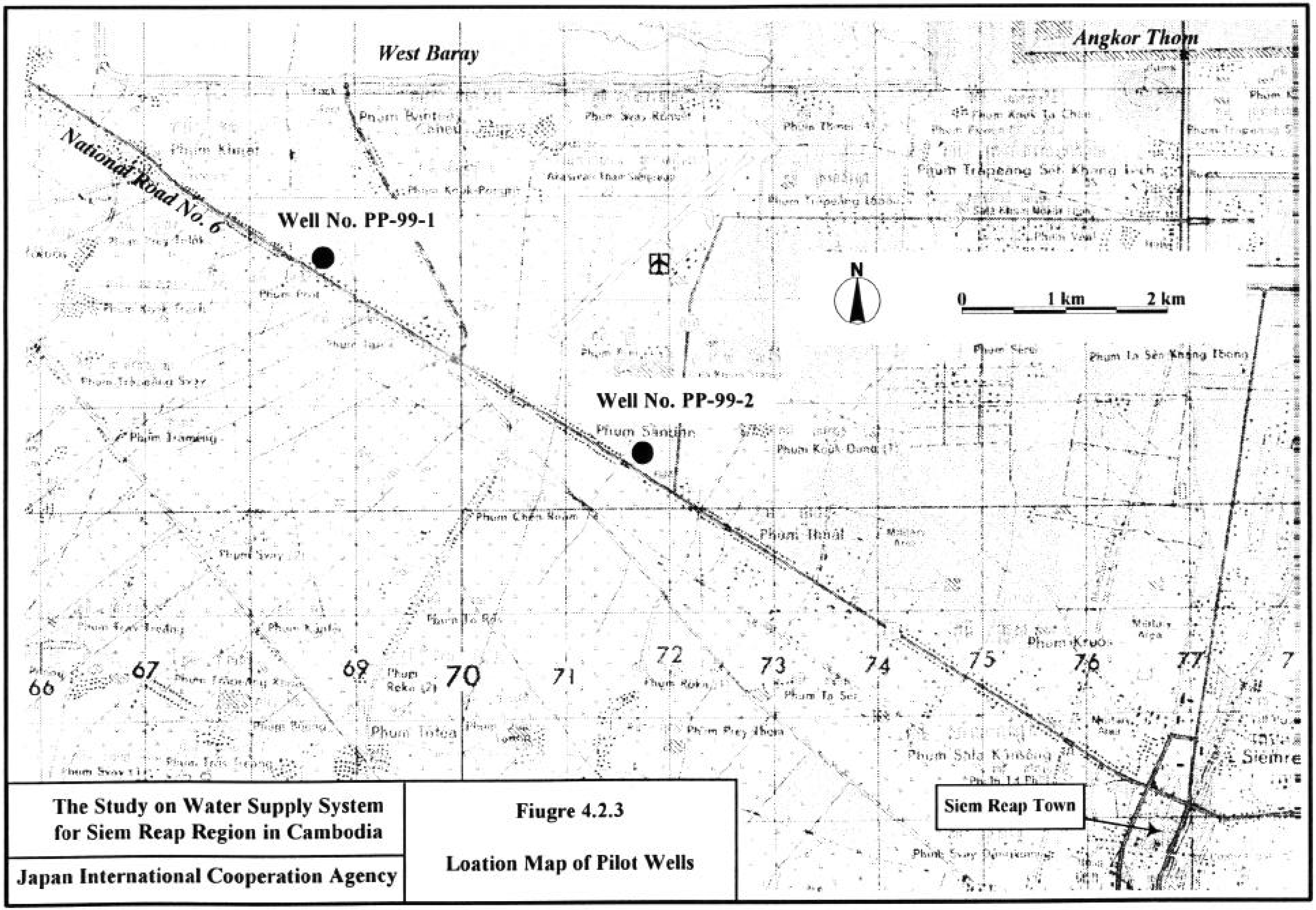
Drilling depth: 50 m (must be below the bottom of alluvial and diluvial deposits)
Opening ratio of screen: 2%
Length and upper most depth of Screen: 12 m (0 – 12 m: pump chamber)
Diameter of drilling hole: 432 mm
Volume of gravel packing: 200 bags (each 50 kg)
Casing pipes: PVC materials
Diameter of casing pipes: OD 216 mm and ID 198 mm
The inner diameter of the casing: enough to rest the pump size for 0.69 m^3/min capacity and/or equivalent one
Centralizer: 6 pieces

(2) Estimated Groundwater Yield of the Production Well

The yields of PP-99-1 and PP-99-2 wells were confirmed as 800 m^3/day and 730 m^3/day, respectively, for the said specification of the 5 m drawdown as shown in Figure 4.2.6 and Figure 4.2.7.

It can be expected that the other production wells of total 8 locations will have an yield in the range of 800 m^3/day from the result of the pilot wells and the hydrogeological condition in the well field.

The constructed two pilot wells in the field will be used as the production well when the Project is started.



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Fiugre 4.2.3

Loation Map of Pilot Wells

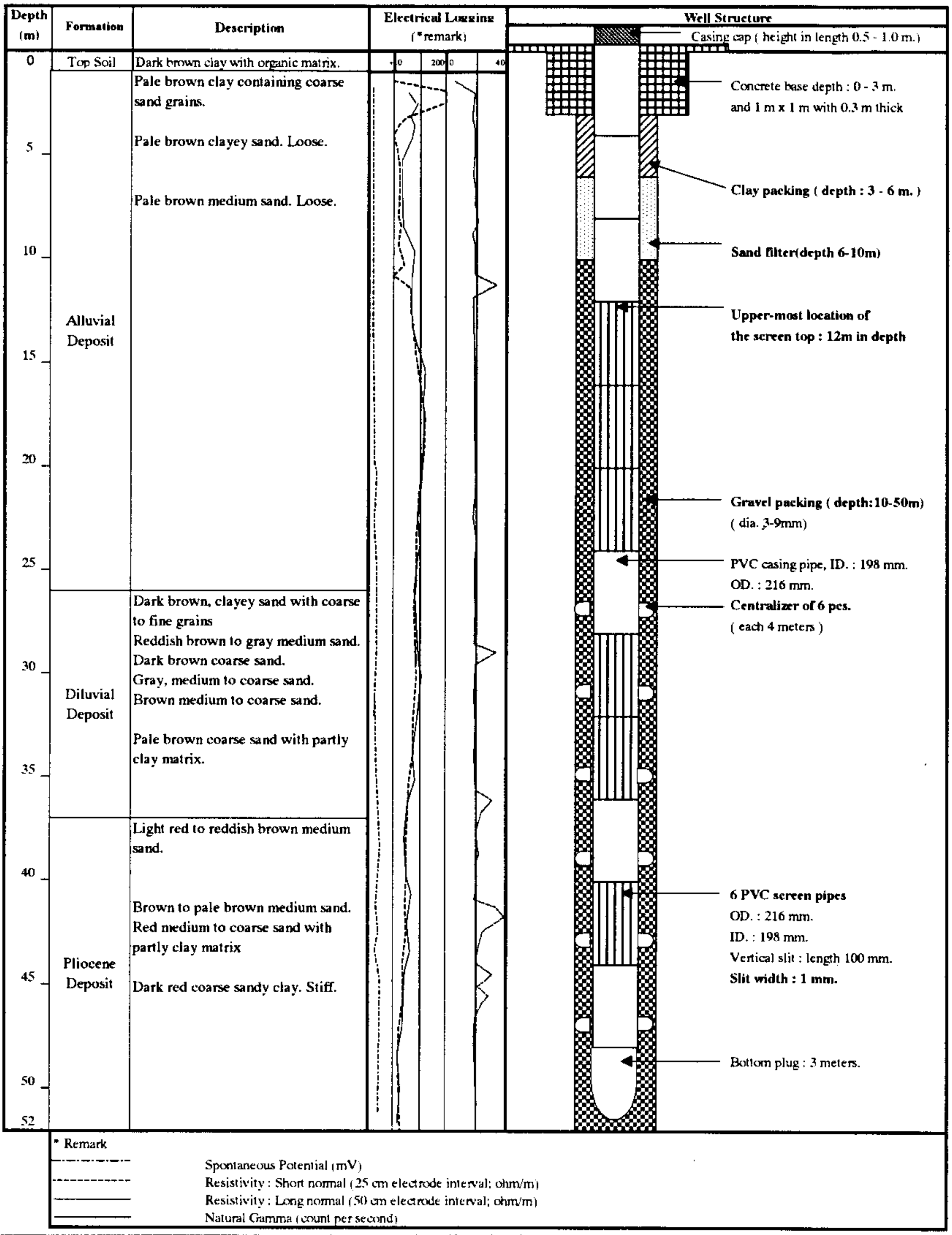

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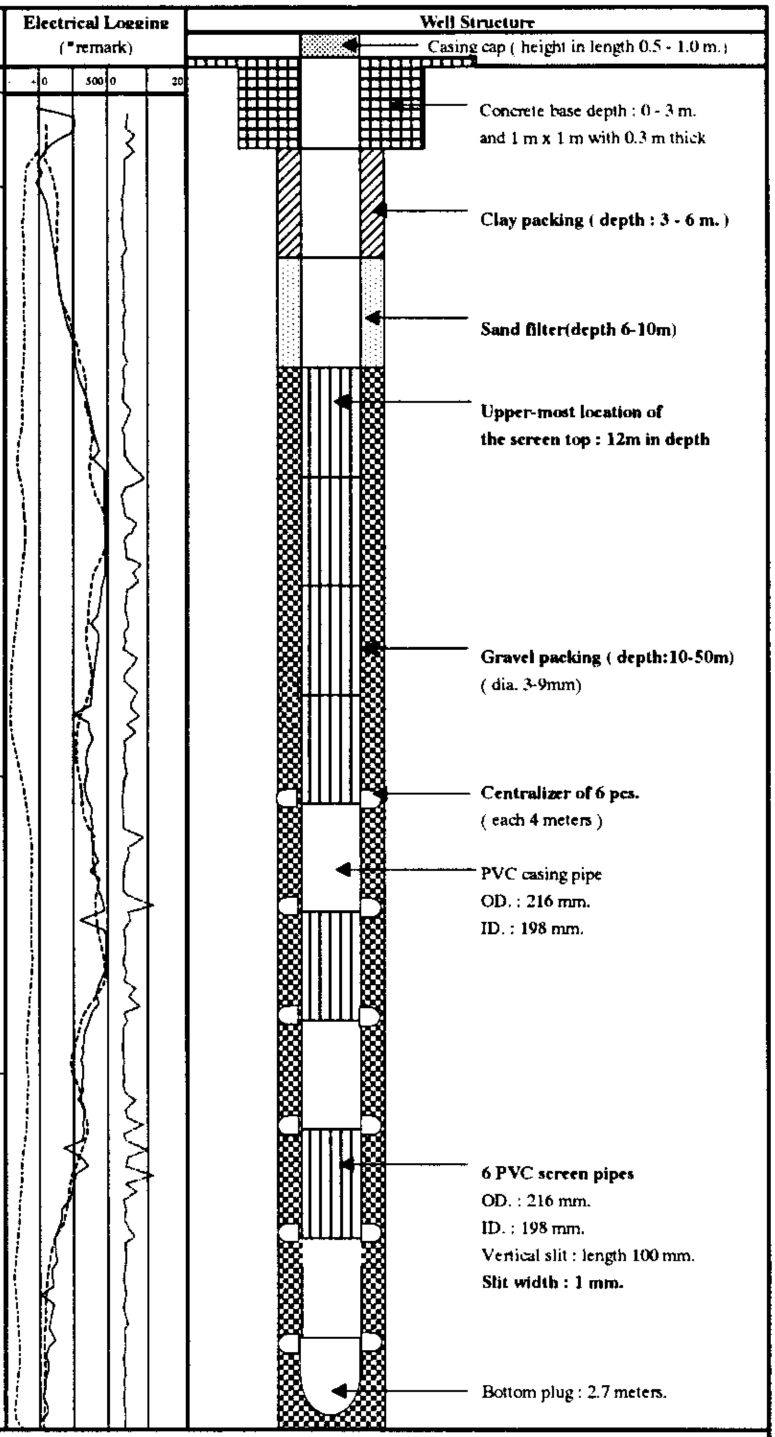
Figure 4.2.4
Drilling Log, Electrical Logging and Well Structure of PP-99-1

Depth (m)	Formation	Description
0	Top Soil	Red laterite soil with gravels (foreign).
	Paddy soil Laterite	Reddish clayey sand with coarse qz grains (1 - 3 mm size). Yellowish red to brown, laterite with organic roots. Clayey and gravelly sand.
5	Alluvial Deposit	Pale brown clayey and gravelly sand with pebbles. Pale brown sandy and gravelly clay. Pale brown clayey fine to medium sand with pebbles.
10		Whitish gray coarse sand with slightly clay matrix. Maximum pebble is 2 cm diameter. Pale brown coarse sand.
15		pale gray to gray gravelly coarse sand. Partly clay matrix at 15 - 16m. Yellowish gray gravelly sand with slightly consolidated.
20		Gray gravelly coarse sand. Maximum grain is 4 mm. Pale brown coarse sand to sand and gravel.
25		Gray medium sand and gravel. Yellowish brown medium sand
	Diluvial Deposit	Gray to pale brown sandy clay. Slightly stiff. 28 - 30m : Gray gravelly sand with laterite chips containing.
30		
35		Gray to brown coarse gravelly sand. Maximum quartz grain is 4 mm diameter.
	Pliocene Deposit	Reddish gray laterite with gravels of 5 mm in diameter.
40		Gray fine sand to silt. Partly medium sand.
45		Reddish brown coarse to medium sand.
50		Red laterite at 47 - 48m. Clay matrix.



* Remark

Spontaneous Potential (mV)
Resistivity : Short normal (25 cm electrode interval; ohm/m)
Resistivity : Long normal (50 cm electrode interval; ohm/m)
Natural Gamma (count per second)

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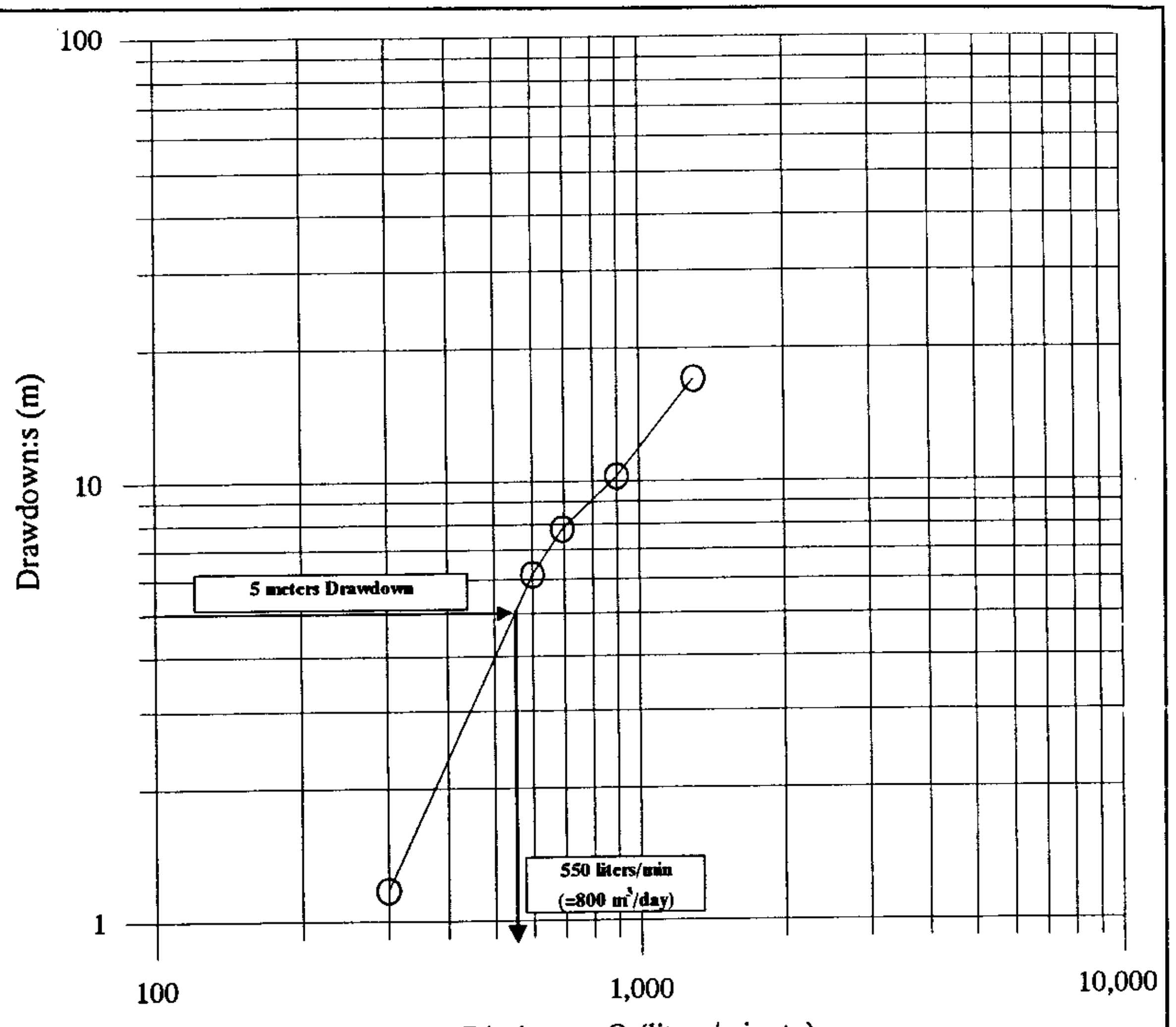
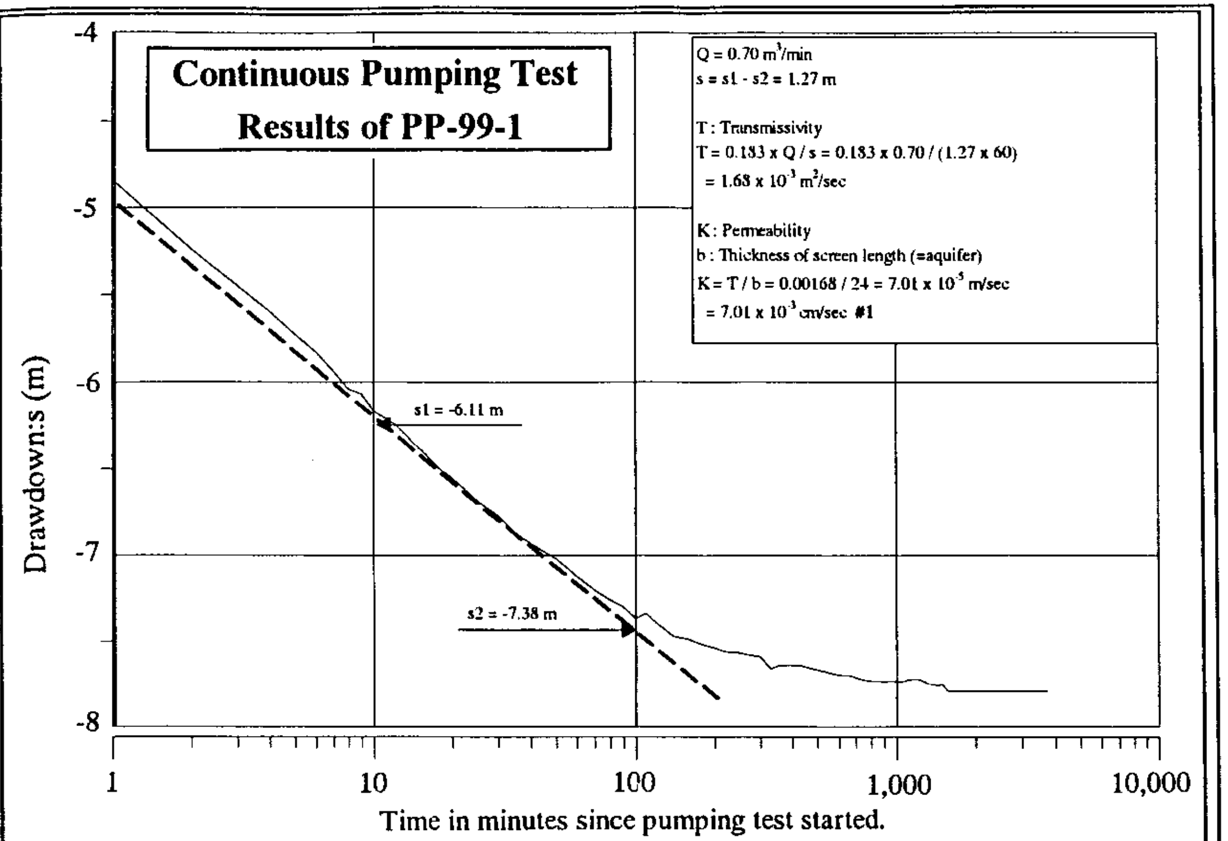
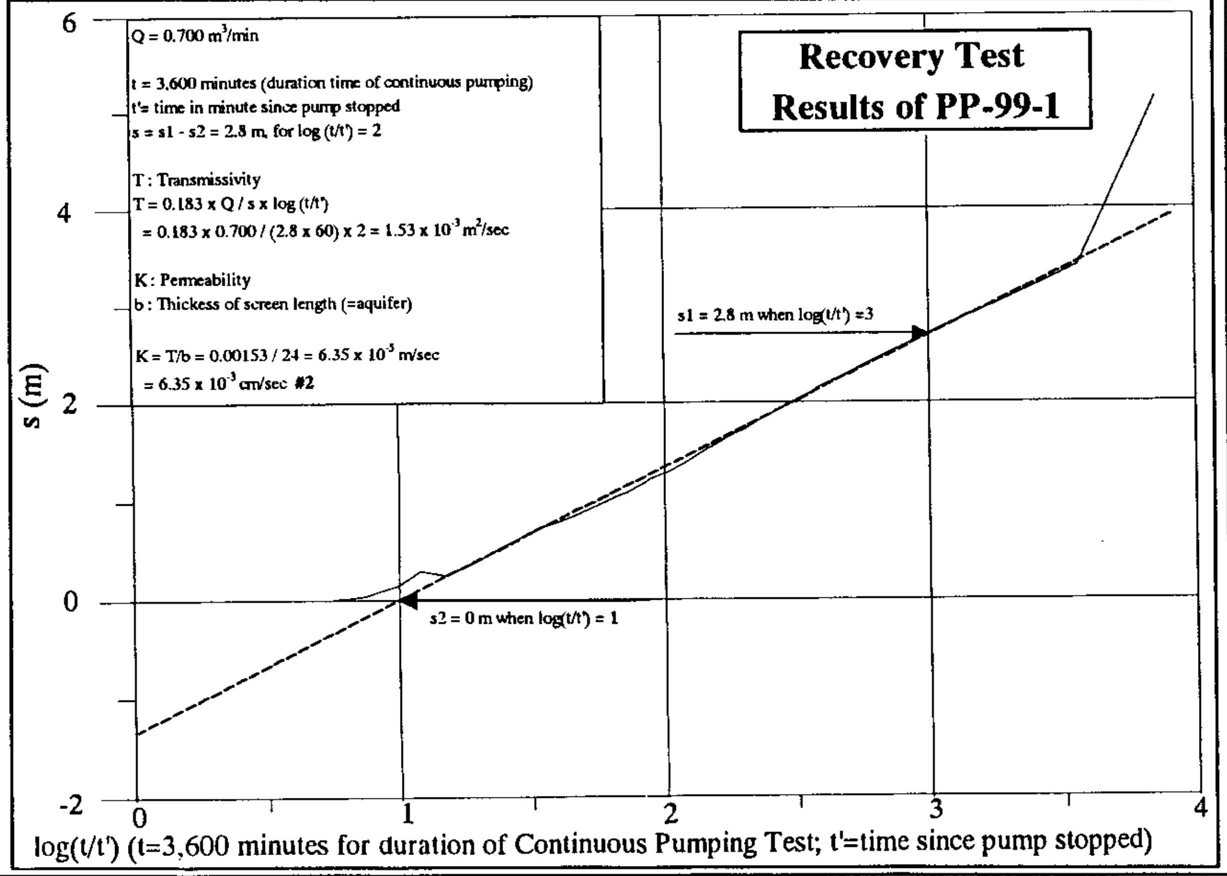
Figure 4.2.5
Drilling Log, Electrical Logging and Well Structure of PP-99-2

Item	Discharge			Drawdown	Remarks
	(liter/sec)	(liter/min)	(m^3/day)	(m)	
1st Step	5.0	300	432	1.17	
2nd Step	10.0	600	864	6.14	
3rd Step	11.6	700	1,000	7.79	Continuous test
4th Step	15.0	900	1,300	10.24	
5th Step	21.7	1,300	1,875	17.00	Preliminary test



Step Drawdown Test Results of PP-99-1





Note

Analyzed by Jacob Method

Permeability Coefficient k = 7 x 10^{-3} cm/sec in average from the above 2 data (**#1** and **#2**)

The data was used for the Groundwater simulation

The Study on Water Supply System for Siem Reap Region in Cambodia	Figure 4.2.6 Pumping Test Results of PP-99-1
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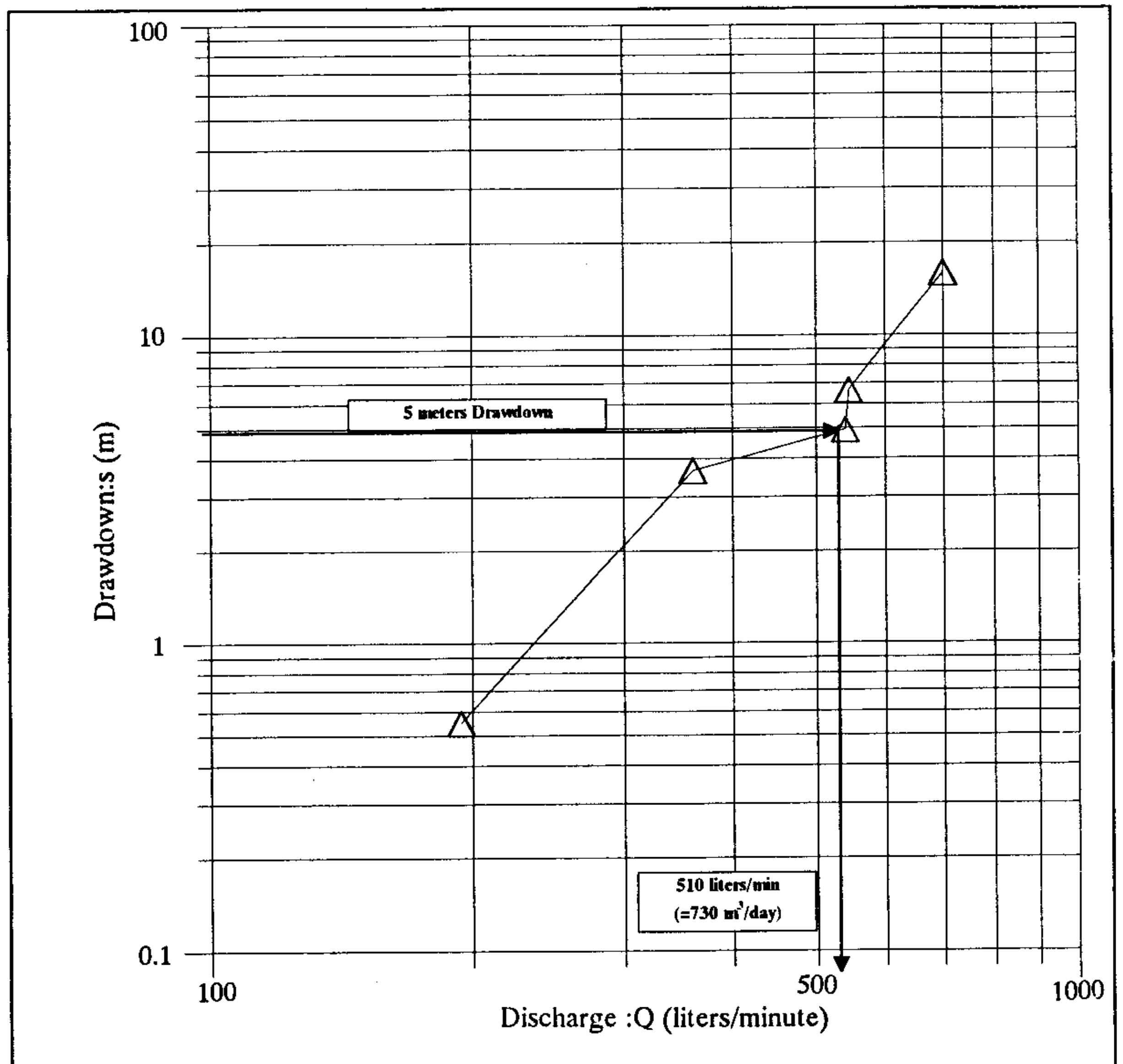
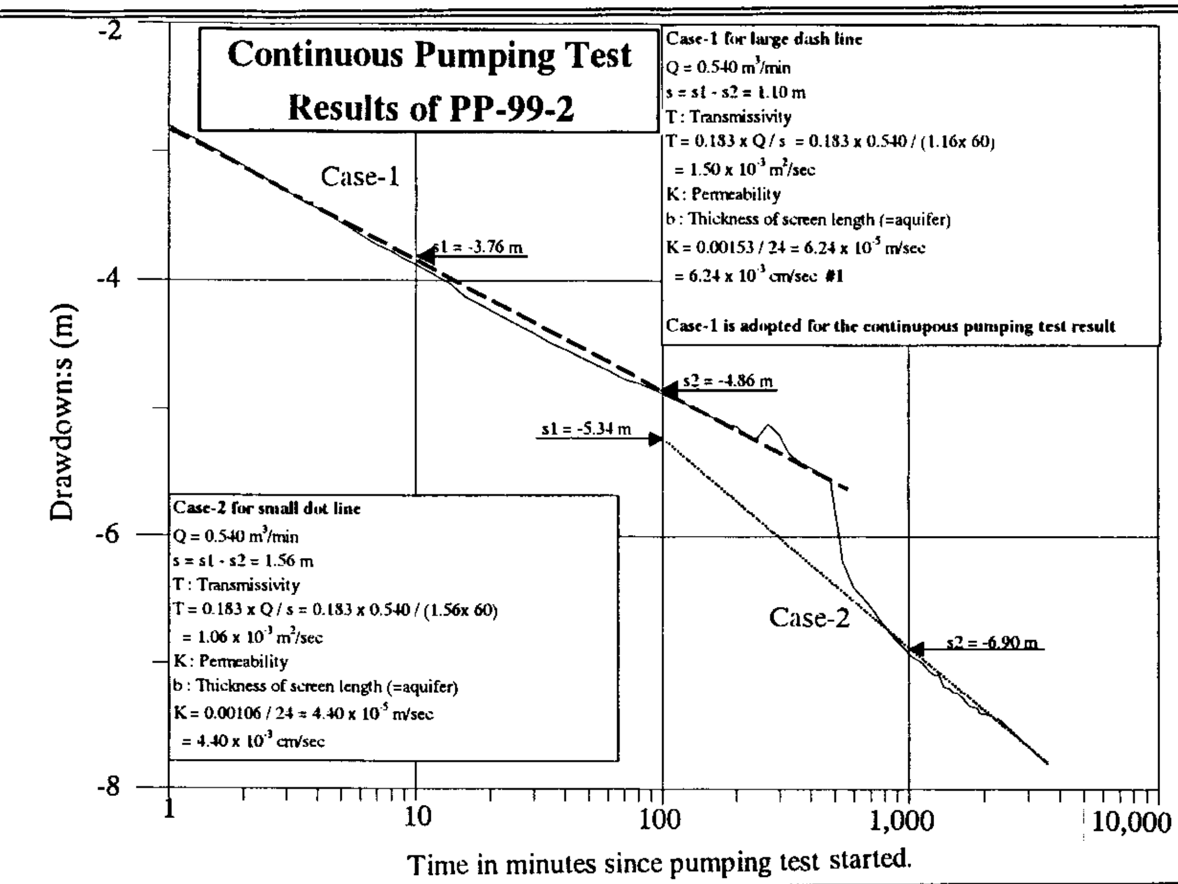
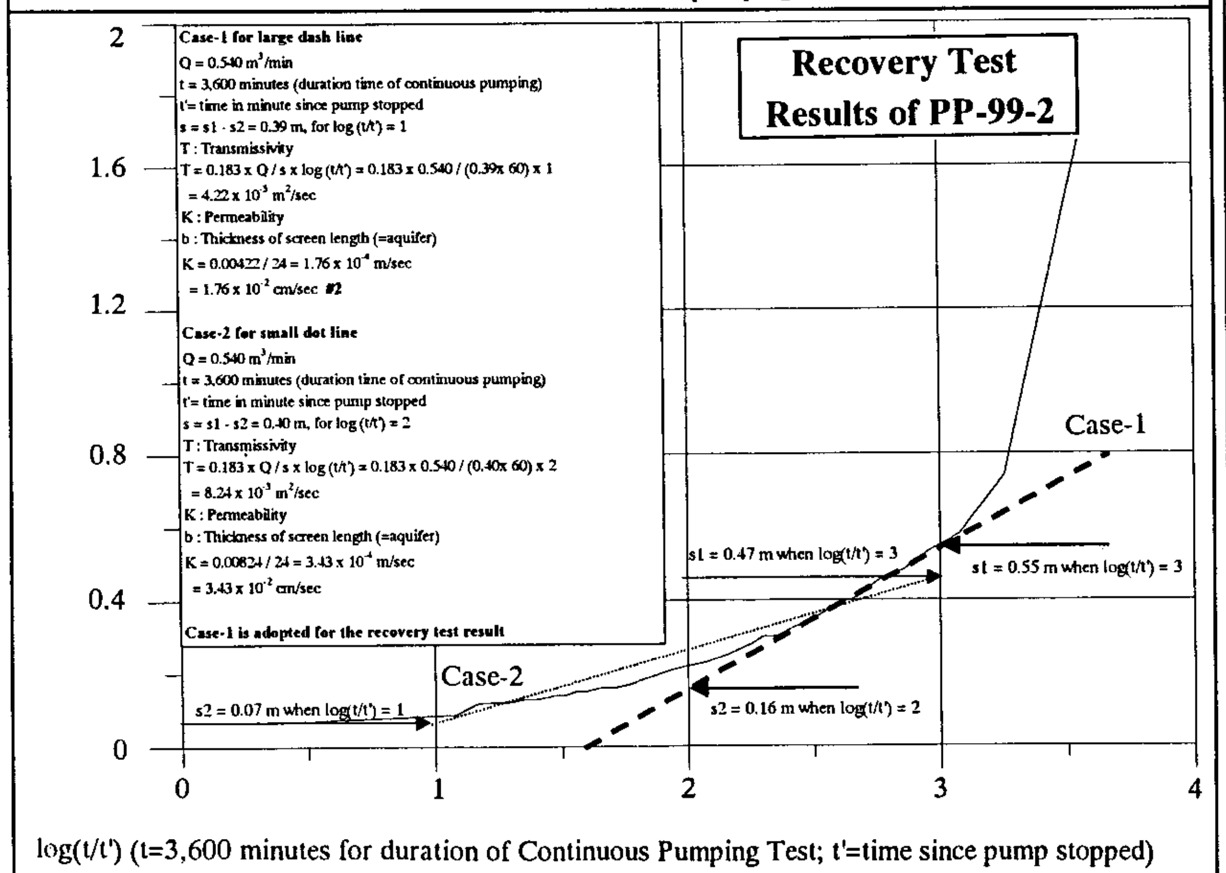
Item	Discharge			Drawdown	Remarks
	(liter/sec)	(liter/min)	(m^3/day)	(m)	
1st Step	3.22	194	280	0.55	
2nd Step	5.98	360	517	3.65	
3rd Step	9.00	540	780	4.94	Continuous test
4th Step	9.10	545	785	6.64	
5th Step	11.70	700	1,011	15.88	Preliminary test



Step Drawdown Test Results of PP-99-2





Note

Analyzed by Jacob Method

Case-1 was adopted for simulation analysis, case-2 was judged that discharge was changed

Permeability Coefficient $k = 1 \times 10^{-2}$ cm/sec in average from the above 2 data (**#1** and **#2**)

The data was used for the Groundwater simulation

The Study on Water Supply System for Siem Reap Region in Cambodia	**Figure 4.2.7 Pumping Test Results of PP-99-2**
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C: 2nd Simulation

(1) Objectives and Methodology

As the first simulation result mentioned in Section A, an optimal spatial distribution of well plan was determined for sustainable groundwater use of the conceivable well field in the south of the West Baray. The pumping test was conducted by the two pilot wells. The monitoring of groundwater level was continuously carried out at eight monitoring wells until November 1999. Then, a plan by 15 production wells of 800 m^3 day/well (total 12,000 m^3/day) was examined by the second simulation. The main purpose of this second simulation is to predict the influence by groundwater extraction on the surrounding area as well as the Angkor heritage. Figure 4.2.8 shows the simulated area and the discretization with 500 m mesh for the surrounding area.

(2) Comparison of Models of first and second Simulation

The second model was newly formulated by modifying the first model. The differences between the first and second models are given below. Other items involved in the second model are same as in the first model. More detail on the procedure is described in Section 4.4 of Progress Report No.3 (March 1999), and Supporting Report Annex 4.2.1.

	First model	Second model
Hydrological Boundary North: Mountainside South: Lake Tonle Sap	No-flow boundary Constant-head boundary	General head boundary Quasi-high conductivity zone
Hydraulic conductivity of layer 1 (upper unconfined aquifer)	2.0×10^{-4} m/s to 8.0×10^{-4} m/s and 5×10^{-4} m/s on average.	Central area: 7.0×10^{-5} m/s Other areas : less than 3.0×10^{-4} m/s
Thickness of the aquifer (Layer 1) in the well field	25 m to 35 m	around 40 m
Size of a finite-difference cell	500 m x 500 m only	500 m x 500 m + 100 m x 100 m in the well field
Well No., interval, and discharge	20 wells total 14,947 m^3/day	15 wells @ 400 m, 800 m^3/day for each well total 12,000 m^3/day
Calibration period	Feb. 1998 to Feb.1999	Feb.1998 to Nov.1999

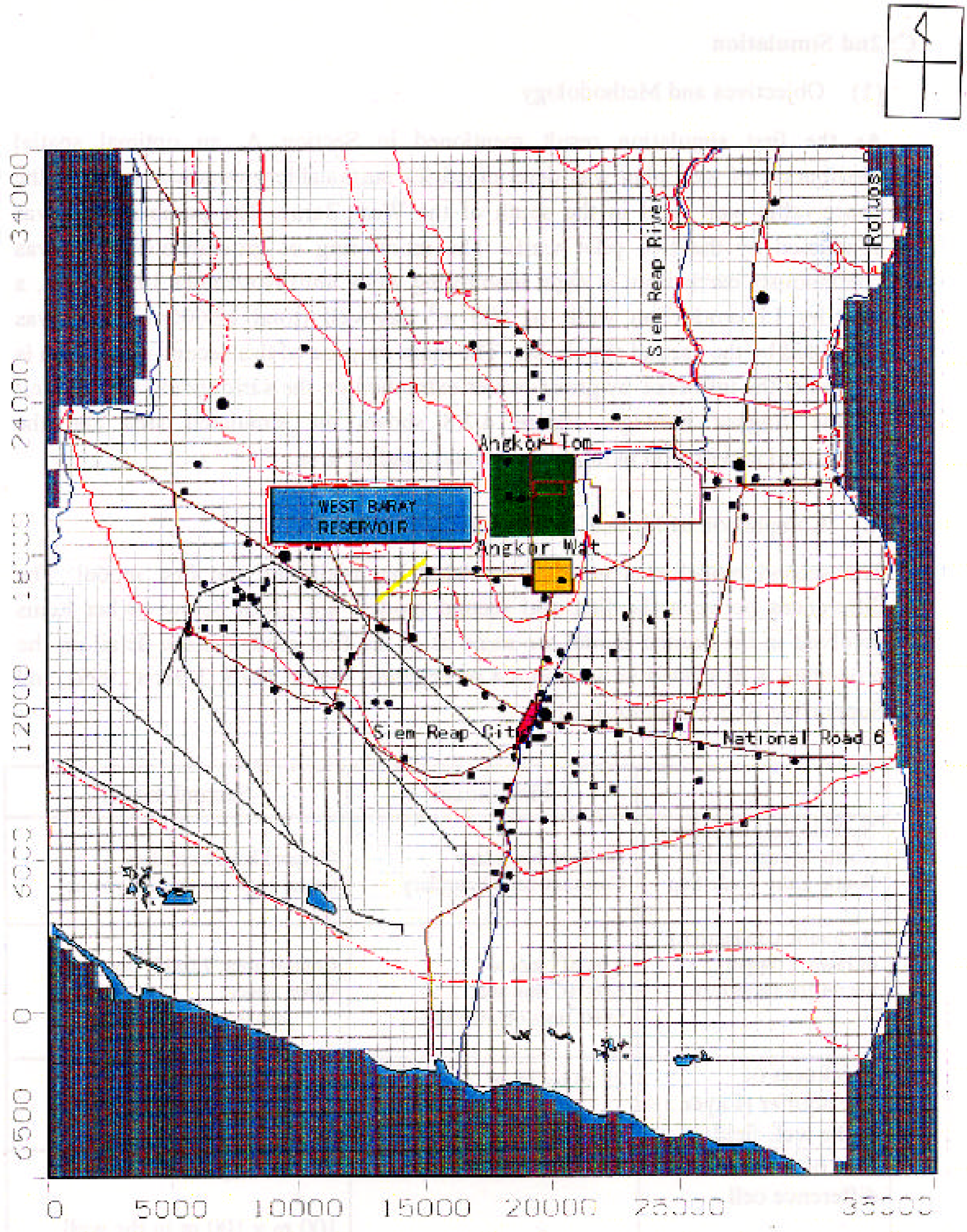

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Figure 4.2.8
The Project Area and Model Discretization

1) Boundary Conditions

On the north side, a no-flow boundary was used in the first model. In the second model, however, a general head boundary is used to allow the inflow from the northern mountainside to the aquifer.

In the first model, a constant head boundary along the shoreline of EL. 6.0 m was used on the south side to permit the inflow to the lake. The head of 6.0 m was assumed constant although the lake water level fluctuates from 1.0 m in the dry season to 9.0 m in the wet season. In the second model, the quasi-zone having high hydraulic conductivity is placed at the shore area between EL. 1.0 m and EL. 9.0 m so that the groundwater level can vary in accordance with the seasonal fluctuation of the lake.

2) Hydro-Geological Parameters

The pumping test was additionally conducted by the two pilot wells in the candidate well field. The hydro-geological parameters, obtained from the pumping test from the two wells, were additionally included in the second model.

The values of hydraulic conductivity at No.PP-99-1 and No.PP-99-2 wells are 7.0×10^{-5} m/s (7.0×10^{-3} cm/s) and 6.2×10^{-5} m/s (6.2×10^{-3} cm/s), respectively. Since the values of hydraulic conductivity are lower than those used in the first model, the spatial distribution of hydraulic conductivity in the upper unconfined layer was newly determined through the calibration. In this second model, the values of hydraulic conductivity are 7.0×10^{-5} m/s in the central area including the well field, and less than 3.0×10^{-4} m/s in the other area.

3) Thickness of Aquifer

Based on the new finding by the drilled result of the two wells, the aquifer thickness of 40 m is input in this second model.

4) Deep Percolation

Deep percolation was estimated from the daily-based water balance estimation from February 1998 to November 1999.

5) Discharge from the Drains

In the first model, discharges coming from the old course of the river and natural drains on the low lands along the lake shore were estimated using the relationship between the groundwater level and the surface drain elevation at each cell. In addition, discharge from the paddy fields was estimated in this second model. The locations of such drain cells are shown in Figure 4.2.9.