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I

UNITED NATIONS

GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN IN PUNJAB, HARYANA AND RAJASTHAN

Final technical report, volume III
(of three volumes)

(Artificial Recharge Studies)

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GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN IN PUNJAB, HARYANA AND RAJASTHAN

INDIA

Final technical report, volume III
(of three volumes)

Artificial Recharge Studies

Prepared for the Government of India
by the United Nations
acting as executing agency for the
United Nations Development Programme

New York, 1983

NOTES

This final technical report was published in three volumes: Volume I covers surface and ground-water resources; Volume II describes the mathematical modelling studies; and Volume III deals with the artificial recharge studies.

References to dollars are to United States dollars. The monetary unit in India is the rupee (Rs); during the period covered by this report the value of the rupee in relation to the dollar stood at $1 = Rs 8.2.

DP/UN/IND-74-009/2

Abstract

Special preliminary studies on ground-water artificial recharge were carried out within the work programme of the IND/74/009 project; ground-water studies in the Ghaggar river basin.

The studies included actual trials of injecting canal water from the Bakra system into confined aquifers; examining the possibility of using Ghaggar flood water for recharge through infiltration ponds in the Kandi area; investigating the possibility of induced infiltration along the Ghaggar and Markanda rivers and their tributaries and preparation of proposals for improving ground-water supply to the city of Chandigarh by artificial recharge of imported surface water.

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INTRODUCTION

The project "Ground-Water Studies in the Ghaggar River Basin" (IND/74/009) covered an area of about 42,200 km^2 comprising parts of the states of Himachal Pradesh (560 km^2), Haryana (13,400 km^2), Punjab (14,800 km^2) and Rajasthan (13,300 km^2) and the whole of the Union Territory of Chandigarh (114 km^2).

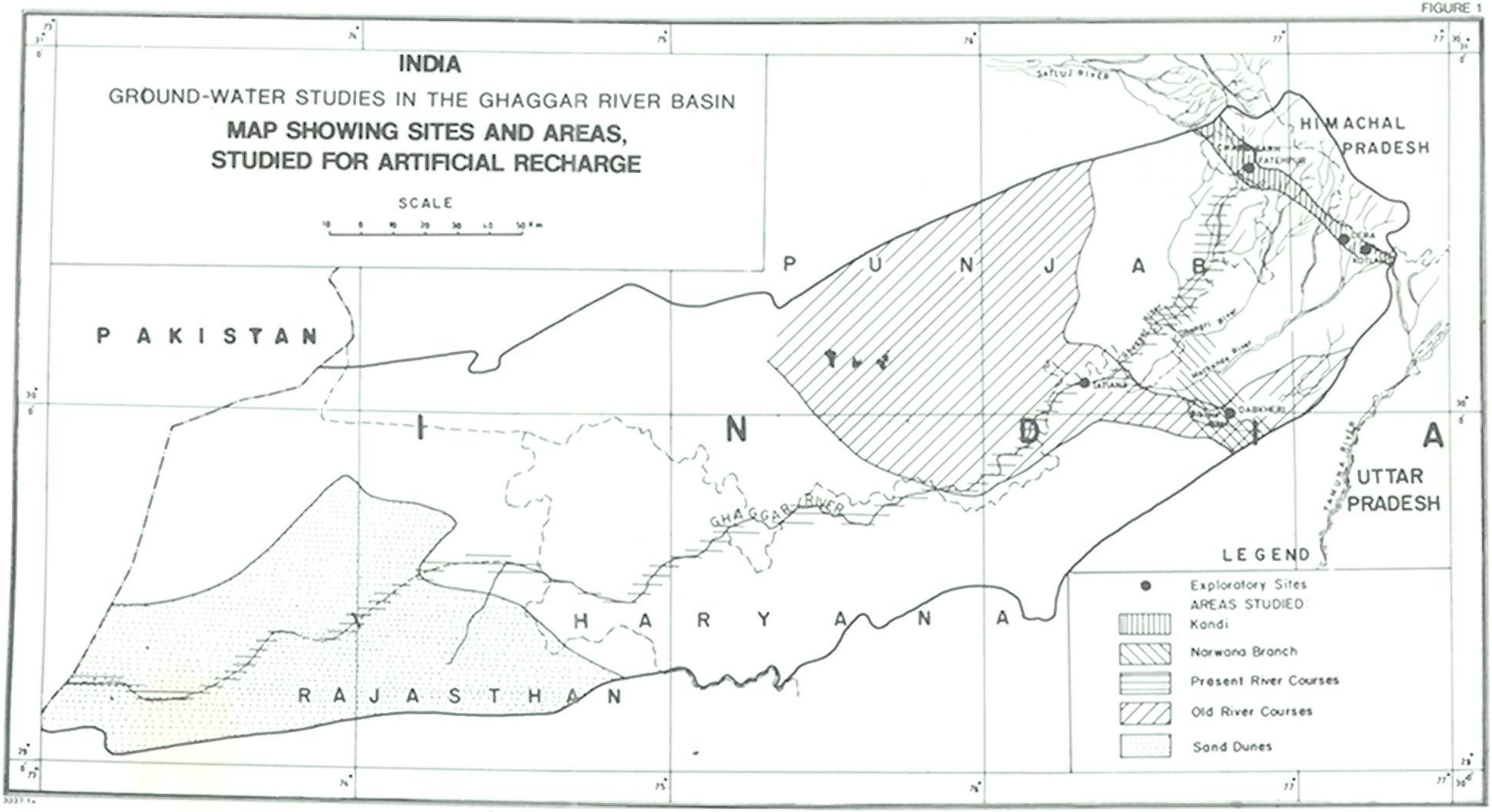
One of the important objectives of the Project was to investigate the feasibility of artificial recharge, using the flood-waters in the Project area, with emphasis on the Kandi area. However, after the Project had begun, other areas, in which overdraft of ground water was reported, were also studied. Thus, the original scope of the artificial recharge studies was enlarged considerably.

Three localities--the Kandi belt in the north-eastern part of the Project area, and the Dabkheri and Tatiana regions in the district of Kurukshetra in the south-eastern part of the Project area--were selected for the artificial recharge studies, each by a different method (figure 1). The selection of these areas for study was based on available hydrogeological data as supplemented by geophysical surveys. The availability of surface water for recharge was also considered in the selection of the areas and the determination of the method of recharge.

The studies carried out in the Kandi belt were in the nature of preliminary explorations and consisted in the determination of subsurface geological and hydrogeological conditions, determination of the hydrogeological parameters of the aquifers and the aeration zone, calculation of exploitable ground-water resources and estimation of the feasibility of artificial recharge by various methods. As Chandigarh City was already reported to be an overdraft area, special attention was given to means of improving the water supply there.

Along the Narwana Branch canal in the Kurukshetra district of Haryana, there are a number of tubewells for augmenting the canal waters, and the area in the vicinity of this canal has been earlier identified as an area of ground-water overdraft. It was therefore decided to study the feasibility of artificial recharge at Dabkheri, close to the Narwana Branch canal, by injecting water, both under gravity and under pressure, into a confined aquifer through a tubewell constructed for the purpose.

FIGURE 1


INDIA
GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN
MAP SHOWING SITES AND AREAS, STUDIED FOR ARTIFICIAL RECHARGE
SCALE
10 0 10 20 30 40 50 Km
PAKISTAN
PUNJAB
INDIA
HARYANA
RAJASTHAN
HIMACHAL PRADESH
UTTAR PRADESH
SATLUJ RIVER
GHAGGAR RIVER
YAMUNA RIVER
FATEHPUR
DABKHERI
TATIANA
LEGEND
Exploratory Sites
AREAS STUDIED
Kondi
Narwana Branch
Present River Courses
Old River Courses
Sand Dunes

The studies conducted at Tatiana, on the bank of the Ghaggar River, were designed to investigate the possibility of induced recharge. On the basis of the Project's work, exploitable groundwater resources were calculated for the above areas and site. Recommendations were made as to the type of method favoured for each area, as well as to the construction of wells and related works for carrying out a programme of artificial recharges; an economic assessment has also been made of the various recharge methods described.

I. PHYSIOGRAPHY

A. Relief

The north-eastern part of the Project area is occupied by the Siwalik Hills range, with an elevation ranging from 400 to 1,000 m above mean sea level. To the south-west of the Siwalik Hills there ia a 2- to 10-km-wide area of overlapping alluvial cones, called the Kandi belt or Kandi area. South of the Kandi belt, an almost flat plain extends towards the western and south-western parts of the Project area, merging into an undulating topography that results from the formation of sand dunes in that arid region.

B. Climate

Rain falls in the Project area mainly during the monsoon season, from June to September. The highest rainfall (600-700 mm) occurs in the Siwalik Hills and the Kandi area (table 1). There is a gradual decline in rainfall as one proceeds towards west and south-west. The minimum rainfall (80 mm) is thus recorded in the sand-dune area.

The rains generally take the form of heavy showers, resulting in flash floods in the numerous rivers and streams and thus escaping as surface run-off.

The annual variation in maximum air temperature is from 18.8° C in January to 41° C in June; average minimum air temperature ranges from 4.6° C in January to 26.9° C in July (these figures are based on observations recorded at the meteorological stations at Chandigarh, Sirsa and Hanumangarh).

C. Hydrography

The drainage system in the Project area consists of the Ghaggar River and its tributaries. The Ghaggar rises in the Siwalik Hills and flows across the Project area from east to west. It is a perennial river, but with a variable discharge over different periods of the year, being rain-fed during the monsoon seasons but carrying mainly base-flow during the dry season.

During the dry period of the year the water in the Ghaggar River is clear, whereas during the monsoons it carries from

Table 1. Project area rainfall data

(mm)

Station	Period of date considered	95% confidence rainfall year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
Naraingarh	1901-1950	1929	89.7	0.0	0.0	7.6	4.6	53.1	79.8	274.8	27.4	38.1	0.0	92.2	667.3
Chandigarh	1958-1976	1972	5.2	62.6	32.2	4.1	0.0	0.0	291.2	201.9	62.8	15.0	31.0	7.0	713.0
Thanesar	1901-1950	1920	9.4	10.7	5.1	0.0	0.0	58.2	178.3	89.4	12.7	0.0	0.0	0.0	363.8
Tohana	1901-1950	1938	4.6	0.0	0.0	0.0	0.0	48.3	46.2	67.6	0.0	7.6	0.0	0.0	174.3
Suratgarh	1906-1950	1946	0.0	0.0	0.0	0.0	0.0	21.3	0.0	52.1	0.0	10.9	0.0	0.0	84.3

3,600 to 5,239 mg/l of suspended solids (as determined for samples collected at the hydrometric station at Panchkula, 14 December 1977). The river has a well-defined course, with its width varying from 50-100 to 500-700 m.

The main tributary of the Ghaggar is the Markanda River, which also rises in the Siwalik Hills. The Markanda is perennial in its course through these hills to the Kandi region. Further down-stream, the river becomes ephemeral and carries water only during the monsoon season. The water is muddy during this period, with a concentration of suspended solids ranging from 1,271 to 2,144 mg/l even at the end of the monsoon (based on analyses of six samples collected from the Kalamb hydrometric station, 2 October 1977).

The other tributaries of the Ghaggar are generally dry except during the monsoon period, when the water is highly charged with mud. Suspended-solid concentration varies from 602-839 mg/l in the Roon River (3 October 1977) to 2,062-3,879 mg/l in the Begna River (same date).

The results of measurements of discharge of the various rivers are shown in table 2. As the table shows, perennial flow is present only in the Ghaggar and Markanda Rivers; moreover, the distribution of flow is irregular over the year. Thus, measurements at the Kala Amb hydrometric stations show about 95.2 per cent of the total annual flow of the Markanda to occur during the monsoon season, July to October. Similarly, for the Ghaggar River, measurements at the Panchkula hydrometric station show about 96.8 per cent of the river's total annual flow to occur during the same period.

Table 2. Monthly discharge of Project area rivers
(m^3/sec)

Hydrometric station and period of observations	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
Ghaggar-Panchkula (1) (1976-1977)	0.24	0.21	0.15	0.25	0.19	0.16	21.5	55.74	9.32	1.81	0.96	0.72
Patiali Wali Rao (2) (1977)	-	-	-	-	-	0.00014	0.57	0.92	2.48	-	-	-
Dangri Nadi (54) (1970-1973)	-	-	-	-	-	3.03	21.62	32.41	20.98	0.77	-	-
Baliali Nadi (55) (1971-1973)	-	-	-	-	-	2.42	7.0	22.53	7.18	0.017	-	-
Tangri Nadi (60) (1970-1975)	-	-	-	-	-	0.07	0.08	0.08	0.05	0.01	-	-
Mohaliwala (59) (1970-1975)	-	-	-	-	-	0.04	0.1	0.07	0.04	0.003	-	-
Begga Nadi (63) (1971-1975)	-	-	-	-	-	1.83	6.06	9.08	1.61	0.16	-	-
Laha Nadi (64) (1972-1975)	-	-	-	-	-	0.04	0.08	0.07	0.05	0.003	-	-
Roon Nadi (65) (1971-1975)	-	-	-	-	-	2.15	9.72	9.85	7.59	1.41	-	-
Markanda-Kala Amb (3) (1976-1977)	0.28	0.52	0.2	0.17	0.17	0.1	14.8	44.66	4.8	4.2	0.3	0.28
Ghaggar-Tatiana (1976-1977)	0.09	0.08	0.06	0.05	0.05	1.13	82.7	343.6	37.2	2.1	0.46	0.25
Ghaggar RDo of GDC (1970-1973)	-	-	-	-	-	15.1	77.2	11.4	29.0	11.1	-	-

II. HYDROGEOLOGICAL CONDITIONS

The alluvial aquifer in the Project area consists of sand and gravel, with layers and lenses of clay, loam and sandy loam. In the Kandi area the aquifer consists mainly of sand with gravel, pebbles, cobbles and boulders. The thickness of the aquifer varies within wide limits, from about 15-20 m in the intercone spaces to about 120-140 m in the central parts of the alluvial cones and even to 250 m near the Markanda River at Dera.

In the middle and south-western parts of the Ghaggar basin the alluvial aquifer is made up predominantly of finer-grained sediments, its thickness varying from 20-30 m to about 100-200 m. In this area the aquifer has a number of intercalations of clay or loamy beds.

The alluvial aquifer is unconfined in its upper parts; confined and semi-confined conditions occur locally in areas where clay beds or large clay lenses are intercalated within the coarser alluvial deposits.

Depth to water level varies within a wide range. In the Kani area, near the Siwalik Hills, depth to water level reaches about 40-50 m bgl; in the central part of the Project area depth to water level is about 2-10 m bgl; and in the sand-dune areas to the west depth to water level again increases, to about 30-40 m bgl.

The filtration properties (permeability) of the deposits range from 50 to about 8,000 m^2/d in different parts of the basin.

Well discharge is also highly variable from one place to another within the Project area. The average rate of discharge is about 20-30 l/sec in areas with moderate transmissivity values. The rate decreases to about 5-10 l/sec with correspondingly high drawdowns in areas with low transmissivity values and increases to about 70-80 l/sec with about 5-6 m of drawdown in areas with high-transmissivity aquifers.

Mineralization of ground water in the recharge zone (the Kandi area) is about 0.2-0.5 g/l but increases towards the (lower) southern and south-western parts of the basin. Thus, mineralization rises to about 1 g/l in the transitional area (the central portion) and to about 3-5 g/l and even more in the western parts of the Project area.

Direct natural recharge occurs in areas that are not covered by impermeable deposits. The main sources of direct recharge in these areas are rainfall, the rivers, the unlined canals and return flow from irrigation waters.

The alluvial aquifer in all these areas is extensively exploited for water supplies and for irrigation by means of tubewells and dug wells.

III. METHODOLOGY

A. Hydrogeological investigations

A systematic hydrogeological survey was carried out in the Kandi region, covering an area of about 500 km^2. The survey included, besides hydrogeological mapping, periodic monitoring of water levels and water quality in a network of observation wells. Lithological logs and other relevant information were collected in order to define the geometry of the aquifers. Representative profiles were chosen in different areas to substantiate, or to fill gaps in, the existing subsurface hydrogeological information.

Similarly, 21 representative sites were selected for infiltration tests to determine the parameters of the zone of aeration.

B. Geophysical investigations

Project geophysical investigations included resistivity surveys and geophysical logging of exploratory boreholes. Eighteen profiles were completed for resistivity surveys in the Kandi area and one profile, at Kaithal-Tatiana, was completed in the south-eastern part of the Project area.

The information thus acquired was used in the selection of the various exploratory holes required for determination of the various exploratory holes required for determination of the aquifer parameters and also in the selection of a suitable method of artificial recharge.

C. Exploratory drilling and pumping tests

Exploratory holes were drilled at three different sites in the Kandi area, at Fatehpur, Dera and Kotla. Of these, the Fatehpur site represented the Ghaggar River alluvial fan (or alluvial cone), the Dera site represented the fan of the Markanda River and the Kotla site represented the intercone space between the alluvial fans of the Markanda and the Jamuna. The boreholes at these sites were later converted into test wells. Observation wells, one at each site, were also drilled, and a long-duration pumping test carried out at Fatehpur to determine the aquifer parameters representative of the alluvial cone aquifers and the intercone-space aquifers. Additional drilling was carried out for artificial recharge experiments by the injection method at Dabkheri and for induced recharge at Tatiana.

D. Infiltration experiments

1. Kandi belt

Infiltration experiments were conducted in the Kandi region for the determination of (K), the co-efficient of (vertical) permeability, using the method of A. K. Boldyrev.

Since it was felt that the most effective and economical method of artificial recharge for the Kandi region would be the "river-bed method", most of the experiments were conducted at sites chosen within river beds.

Pits were dug of two sizes, 1 x 1 x 1 m and 1 x 1 x 0.5 m, and clean water was let into them with a constant head of 10 cm. The recharge experiments at a 10-cm head continued for one to two hours after the infiltration rates were stabilized.

The coefficient of vertical permeability was determined by using the formula

$$K = \frac{Q}{w},$$

where Q = rate of stabilized infiltration (m^3/d) and w = area of the bottom of the pit (m^2). Specific yield (μ) (indicated as S in the English literature) was determined from the coefficient of vertical permeability thus found, using the formula of P. A. Betsynskyi:

$$\mu = 0.117 \sqrt[7]{K}$$

In all, 21 experiments were conducted using this procedure.

2. Dabkheri

A recharge test was conducted at the Dabkheri site by injecting water under pressure through a test well into a confined aquifer and observing the rise in water levels in specially drilled observation wells.

The test had three purposes: to establish experimentally the possibility of artificially recharging a confined aquifer through wells by injection of water under pressure; to determine for how long water could be injected into the well without causing clogging; and to explore the possibility of using unfiltered canal water for this purpose.

The test well constructed at Dabkheri was first put to a long-duration pumping test for the determination of aquifer parameters. It was then fitted with a pressure gauge and injection of water into the well began on 19 November 1977.

Two mud pumps were used, each with a discharge of about 21.9 l/sec, for a total discharge of about 43.8 l/sec. A pressure of about 1.6 atm was recorded 10 minutes after the start of the injection process; the pressure rose to about 1.95 atm within the next 30 minutes and remained stabilized at that level for about another four hours.

Some leakage of water through the ground around the housing of the test well was observed during this period, being about 0.14 l/sec. Approximately seven hours after the start of the injection experiment, the pressure suddenly dropped as a result of four seepages within a radius af about 17 m from the test well. The total discharge from these seepages began at about 2 l/sec and gradually increased thereafter; the experiment was therefore stopped.

During this experiment, water-levels were monitored in observation wells at various distances from the test well and at different time intervals; all the observation wells responded immediately after injection began.

In order to achieve the full benefit of the injection experiment it was decided to construct another test well for injection, to be located at a point between observation wells Nos. 3 and 4, and with its first screen length at 40 m bgl rather than (as in the original injection well) at 16 m bgl. In addition, special measures were taken to ensure the efficacy of the cement seal above the gravel packing. Meanwhile, the first test well was used to study quantitatively the possibilities of injecting water into the same by simple gravity-feed. With this end in view, an experiment lasting about 100 hours was conducted at Dabkheri.

E. Water-level measurements

A set of 63 observation wells was monitored twice: the first time between 26 May and 28 May 1977 and the second between 10 October and 12 October 1977. Data were thus obtained for both pre- and post-monsoon periods. In 15 representative observation wells in the Kandi belt, water level fluctuations were recorded every 10 days, as a means of observing changes in the ground-water regime and in order to calculate the natural recharge to the alluvial aquifer. Hydrographs showing water-level fluctuations were constructed and correlated, in some cases, with the figures for rainfall and stream discharge.

F. Laboratory studies

Laboratory studies carried out by the Project were of two kinds: chemical analyses of waters from the dug wells and drilled wells in the Project area, and analyses of sediment load in the canal waters used for the injection tests.

IV. PROJECT RESULTS: THE KANDI BELT

A. General

The Kandi belt (figure 2) is situated along the south-western edge of the Siwalik Hills. It is a somewhat undulating plain with a steep slope near the hills, which gradually grades off towards the plain. Genetically, the Kandi belt is a system of coalescent alluvial cones formed by the rivers' and streams' debouching abruptly onto the plains from the hills.

In the Project area, the Kandi belt is roughly 75 miles long, extending from the divide between the Sutlej and Ghaggar Rivers to that between the Ghaggar and the Jamuna. The area from Sukhna Choa in the north-west to Markanda on the south-east, a stretch of about 45-48 km, is occupied by the main cone deposited by the Ghaggar River and its tributaries. On either side of this cone the intercone spaces, with their distinctive deposits, separate the Sutlej basin from the Ghaggar basin and the latter from the Jámuna basin. The apex of this cone is the relatively high ground in the centre of the area. The width of the Kandi belt varies here from about 2 to 10 km.

Numerous streams of various magnitudes flow through the area occupied by the main alluvial cone, but only the Ghaggar and the Markanda are perennial; the other streams carry water only during the monsoon.

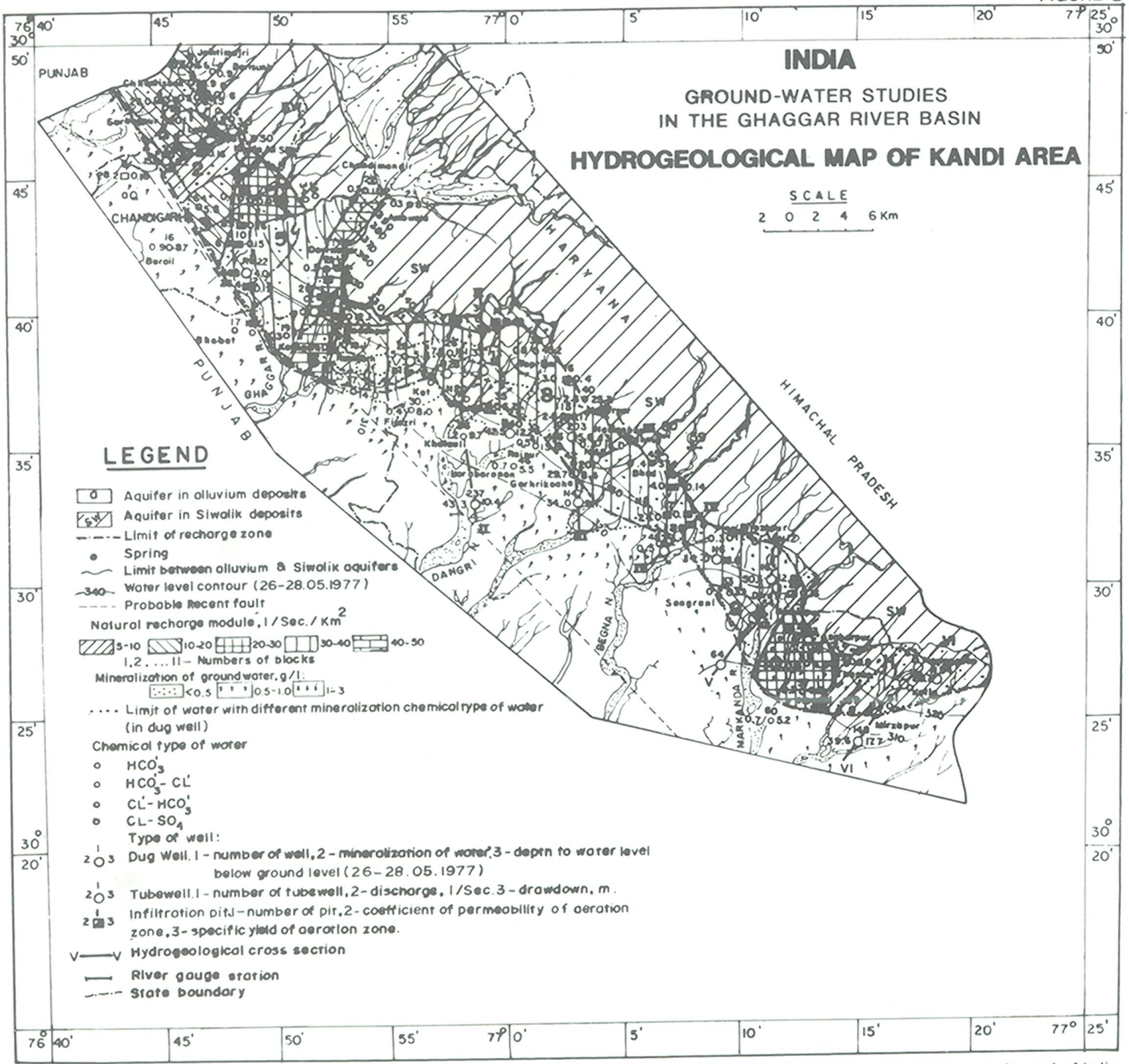
B. Hydrogeological conditions

There are two aquifers in the Project area - one formed by the Siwaliks and the other formed by the alluvium. The Siwalik aquifer, which consists of fine sand, sandy loam and similar materials, has negligible water resources and is therefore a secondary source of recharge to the alluvial aquifer.

The alluvial aquifer extends, by contrast, over hundreds of square kilometres west and south-wast of the hills. Only that part of it, however, that falls in the Kandi area has been investigated in the present context.

Within the cone area, the alluvial aquifer consists of boulders, pebbles, gravel and sand, sometimes with lenses and layers of clay or loam. The material, in the intercone spaces

FIGURE 2



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is finer-grained, with more frequent intercalations of clay. It is also surmised that the thickness of the alluvium as a whole is smaller here than in the cone areas. At places the aquifer is overlain by a layer of sandy loam or loam about 2-5 m thick. The thickness of the aquifer itself is highly variable, ranging from about 120-140 m in the central parts of the cone to about 25-35 m at the sides, and about 15-20 m in the intercone spaces. The aquifer is underlain by a clay bed. At the Dera site the aquifer is about 250 m thick.

At distances between about 2 and 10 km from the hills, the thickness of the aquifer drops sharply and clay sediments become predominant over the boulders and cobbles. As a result of the thinning of the aquifer and the interception of the water table by the topographic profile, as well as a sudden change in the hydraulic gradient on account of the appearance of clay strata, a number of springs issue out at this point to form the south-western boundary of the Kandi belt, call the "discharge zone".

To the north-west of the village of Sandran, the Kandi belt is also delimited by a recent (?) major fault (figure 2), which is very clearly seen on satellite image: the Kandi belt, to the north, seems to be downthrust with respect to the southern plain. Indirect evidence of this fault is seen in a number of sudden deflections and knickpoints in the courses of the various streams that cross this area.

The aquifer in the Kandi belt occurs under phreatic or unconfined conditions. The depth to water level ranges from 40-50 m near the Siwalik Hills to as little as 0-1.5-2.0 m in the zone of discharge (figures 3 through 7).

The filtration properties of the deposits, especially within the limits of the alluvial cones, are very good, the coefficient of permeability ranging from 20 to 171 m/d. In the intercone spaces these values are generally smaller.

The filtration parameters of the deposits in the aeration zone according to the Boldyrev method are summarized in table 3.

The permeability values found in the granular sediments of the zone of aeration are almost the same as those found by pumping tests for the aquifer in the Kandi belt.

There are a number of tubewells in the Kandi area, which were drilled by the Haryana State Minor Irrigation and Tubewells Corporation (HMITC) and the Haryana Department of Public Health Engineering. The discharge figures for these tubewells range from 8 to 56.8 l/sec, with drawdowns ranging between 3.6 and 22 m. No information regarding aquifer parameters was available for these wells. Exploratory test wells were drilled at three

FIGURE 3

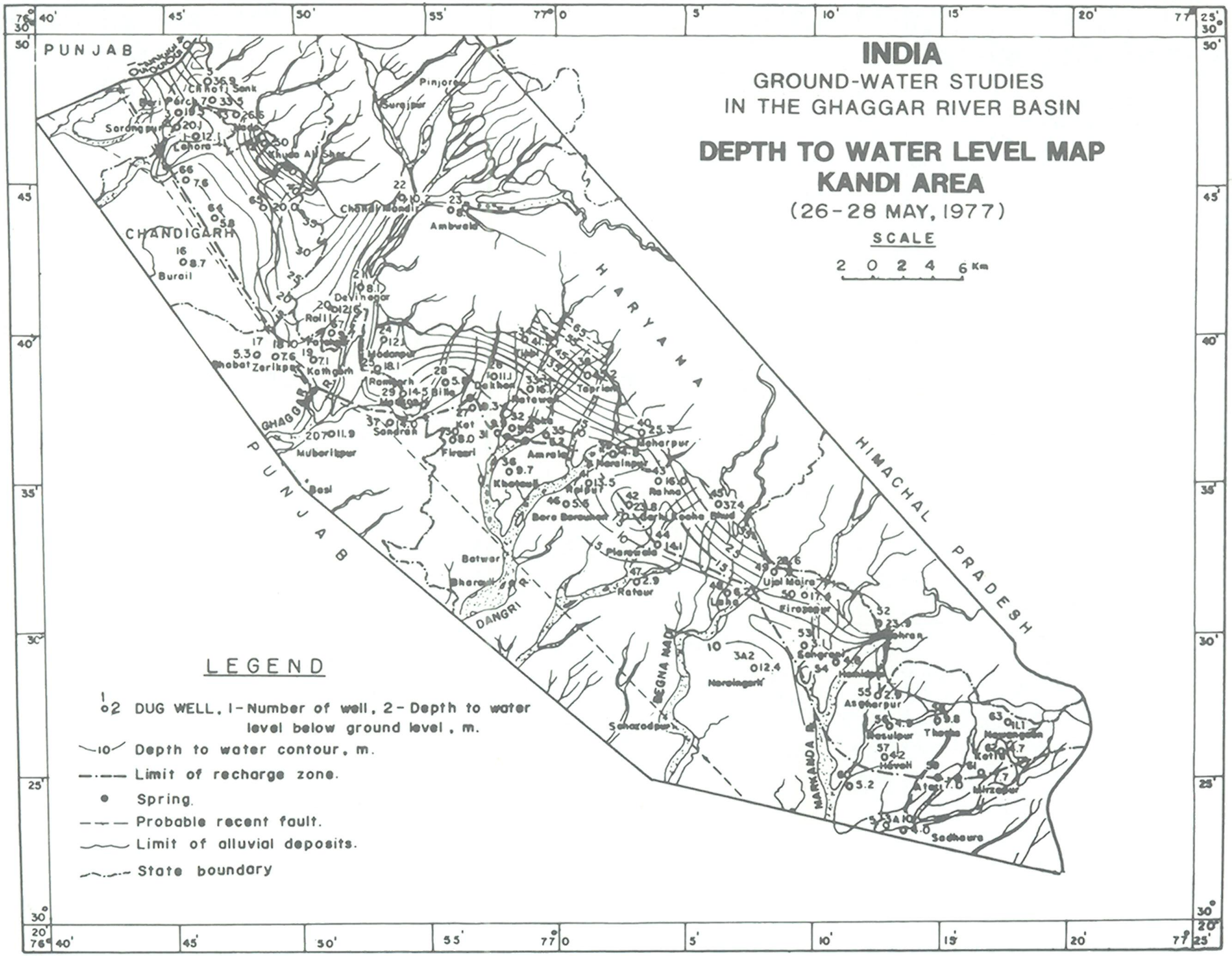

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FIGURE 5

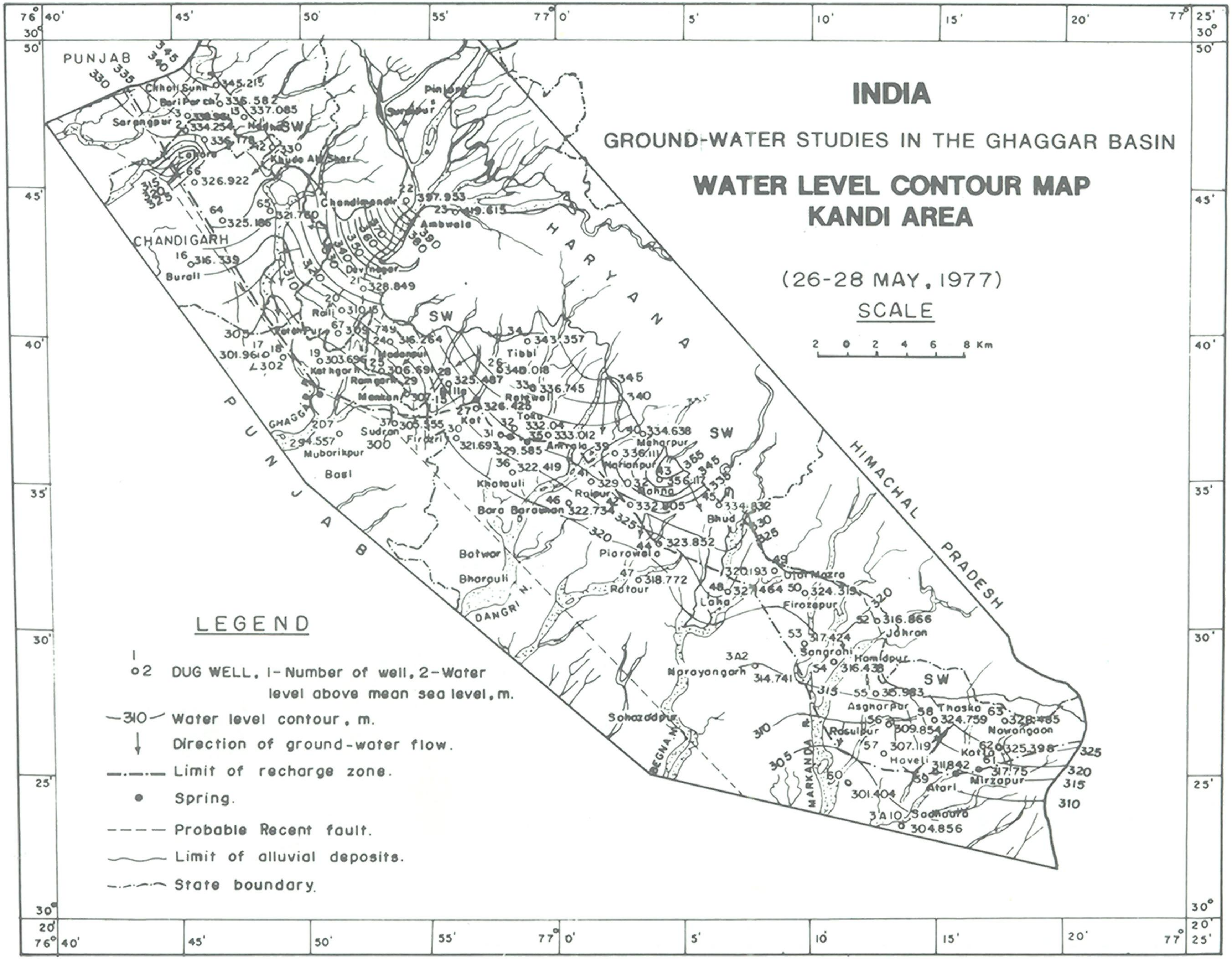

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Table 3. Kandi belt: filtration properties of aquifer deposits

Pit No.[a]	Location	Permeability (m/d)	Specific yield (μ)
1	Ghaggar riverbed	25.9	0.19
2	Ghaggar riverbed	7.2	0.16
3	Ghaggar riverbed	57.6	0.21
4	Jainta Devi Rao riverbed	23.0	0.18
5	Patiali Rao riverbed	9.6	0.16
6	Jainta-Patiali Rao intercone space	1.5	0.11
7	Jainta-Patiali Rao intercone space	0.6	0.11
8	Patiali Rao flood terrace	28.2	0.18
9	Patiali Rao I terrace	9.8	0.16
10	Sukhna Choa I terrace	6.9	0.15
11	Sukhna Choa I terrace	8.5	0.16
12	Sukhna Choa riverbed	26.4	0.19
13	Dangri riverbed	19.3	0.18
14	Small stream bed near Tibbi Village	24.4	0.19
15	Dangri riverbed	12.6	0.17
16	Tributary of Dangri riverbed	3.0	0.14
17	Begna riverbed	24.0	0.18
18	Begna riverbed	4.0	0.14
19	Begna riverbed	23.5	0.18
20	Markanda riverbed	9.7	0.16
21	Markanda riverbed	13.7	0.17

a/ For locations, see figure 2.

sites in the Kandi belt, at Fatehpur, Dera and Kotla, the first two being drilled within the limits of alluvial cones and the third being located in the Ghaggar-Jamuna intercone.

At Fatehpur, two test wells were sunk within 6 m of each other, tapping independently an unconfined aquifer (in the shallow test well) and a confined aquifer (in the deep well).

The results of pumping tests at this site are summarized in table 4.

A pumping test was conducted at Fatehpur by simultaneously pumping both the shallow and the deep test wells at the same rate of discharge (50 l/sec) to exclude the influence of possible leakage from one aquifer to the other during pumping. The test showed that about 100 l/sec can be pumped at this location.

The ground waters in the Kandi belt are mainly fresh, with a total mineralization of less than 0.5 g/l. There are some anomalies, such as the open wells (Nos. 26, 28 and 29), but their higher mineralization is probably connected to contamination of the upper part of the aquifer. Similar anomalous values in wells 40 and 58 are probably due to the effect of recharge into them of higher-mineralization water from the Siwalik Hills.

There are three main types of ground-water regime that have been recognized in the Kandi belt (figure 8). In the first, the ground water is well-connected hydraulically with the surface water bodies. This type is represented by such open wells as Nos. 21, 24, 25, 44, 54, 55, 56, 57, 64 and 67. As seen from the hydrographs for these wells, the amplitude of water-level fluctuations is more than 6 m. It is seen that the response and rise of water level in the wells falling under this regime is related to their distance from the surface-water body, i.e. from the rivers. The nearer a well is to a river, the quicker its response to the fluctuations of water level in the rivers. Wells Nos. 21 and 67 belong to this regime; they are situated, respectively, about 350 and 1,300 metres from the Ghaggar.

The rise in water level in well No. 21 started towards the end of April 1977, after the rain. The rise continued up to the middle of May 1977. Water levels then declined until the middle of June, rose again until 10 August, receded again and then rose until 20 September. The peak water level was observed around 20 September 1977. During and after this period the behaviour of the ground-water level was fully in consonance with the rainfall in the area and with the variations over time of the water or flood-levels in the Ghaggar River. Similarly, the rise in water level in well No. 67 started only at the beginning of July and continued until 20 August, when it started

Table 4. Kandi belt: results of pumping tests at Fatehpur

	Depth to tube-well (m)	Thickness of aquifer (m)	Depth to water level (m, bgl)	Discharge (l/sec)	Discharge (m^3/d)	Drawdown (m)	Permea-bility (m/d)	Transmis-sivity (m^2/d)	Water-level conduc-tivity a/ (m^2/d)
Deep Tubewell	93	65	13.0	50	4 320	6.1	67.6	4 392	3.44×10^5
Shallow tubewell	34	21	7.6	50	4 320	3.2	171.0	3 593	1.3×10^4

a/ "Water-level conductivity," corresponds to the English term "diffusivity" and is defined as T/S.

falling. The water level rose again until 20 September, when the peak was reached. Decline in the water level started again only around 10 October.

The difference in response time to the rise of water level/discharge in the Ghaggar River, as expressed by the rise in water levels of the two wells (separated from each other by a distance of about 1,000 m) is about 18 days. This gives a rate of about 55 m/d (1,000/18) as the rate of travel of the wave of pressure through the aquifer sediments in this area. This is, therefore, roughly the radius of the pressure-wave front as it advances per day. This value more or less confirms the results of the determination of water-level conductivity on the basis of the pumping tests (1.3×10^4 m^2/d), which gives the radius of the circle thus affected as about 64 m/d (the area of the circle being $\frac{\pi D^2}{4} = 1.3 \times 10^4$; D = 128 m/d).

The two wells mentioned above are the type examples for the other wells under the same regime, whose response is essentially similar to that of wells 21 and 67. Wells Nos. 24 and 25, however, are exceptions, the water-level fluctuations in them being somewhat irregular and more frequent. These wells have a rather high position in relief vis-a-vis the Ghaggar riverbed, which may have something to do with the subsurface strata.

The second type of ground-water regime prevails in the areas between the two rivers, and is probably characteristic of the intercone areas. This type is represented by wells Nos. 2, 5, 7 and 66. The hydraulic connexion between the ground-water and the surface-water bodies does not seem to be good. The rise in water levels, as a response to recharge after rains or as a result of increase in the discharge of the bounding rivers, is therefore slow, and the fluctuations are correspondingly smaller in amplitude. The rise in water level starts sometime between July and September and continues until sometime between January and April. This indicates a gradual spreading of the "recharge wave" within the limits of the area.

The third type of ground-water regime, termed the "broken regime", is essentially under the influence of exploitation. Well No. 65 is cited as the type example. The ground-water body is hydraulically well-connected with the surface water. Recharge is fast and copious, but the rise in water level is attenuated as a result of heavy draft. Consequently, the hydrograph curve acquires a slightly wavy form.

Analyses of the geological framework and the hydrogeological and hydrochemical condition of the ground-water regime suggest that the main source of recharge to the aquifer is flood-flow of the rivers through their beds and, to a lesser extent, water that percolates down as direct infiltration from the rainfall in areas where there is no covering clay layer over the aquifer. Flow from the Siwalik Hills aquifers may also contribute to a minor degree.

C. Calculation of ground-water resources

1. Natural ground-water resources (natural recharge)

Natural water resources have been calculated on the basis of the measurements of water-level fluctuations in 15 representative wells, which were monitored every 10 days between March 1977 and March 1978. Water-level measurements were also carried out twice that year in another 48 wells, on 26-28 May 1977 and 10-12 October 1977.

The module of natural recharge (Mr) was determined on the basis of the data collected by applying N. N. Bindeman's formula (1):[1/]

$$Mr = 31.7 \mu \Sigma (\Delta h + \Delta z) \qquad \text{Equation 4.1}$$

where

Mr = module of natural recharge ($l/sec/km^2$),

μ = specific yield, and

$\Sigma(\Delta h = \Delta z)$ = sum of the amplitude of water-level fluctuations (in m) as shown in figure 8.

The average specific yield for the alluvial cones has been assumed as 0.2 and that for the intercone spaces as 0.15.

Modules of natural recharge for areas around the representative wells have been calculated and are summarized in table 5.

On the basis of analysis of the water-level fluctuation data, from the 15 representative wells as well as those selected for semi-annual measurements, and taking hydrogeological conditions into account, the Kandi area was further subdivided into 11 blocks (figure 2), each having a different module of recharge.

1/ Numbers in parentheses refer to items in the numbered reference list that follows this report.

Table 5. Kandi belt: natural-recharge values for areas near representative wells

Well No.	Specific yield (μ)	Σ (Δh +Δz)	Mr 1/sec/km^2
2	0.15	4.1	19.5
5	0.15	2.5	11.9
7	0.15	1.4	6.7
21	0.20	7.2	45.6
24	0.20	2.6	16.5
25	0.20	6.3	39.9
44	0.20	6.8	43.1
54	0.20	3.0	19.0
55	0.20	3.75	23.8
56	0.20	5.30	33.6
57	0.20	3.60	22.8
64	0.15	2.20	10.5
65	0.20	4.40	27.9
66	0.15	1.90	9.0
67	0.20	6.30	39.9

Natural-recharge values were calculated for each block separately and for the Kandi belt as a whole (table 6).

Table 6. Kandi belt: natural-recharge values, by block

Block	Area (km^2)	Mr (1/sec/km^2) Range	Mr (1/sec/km^2) Average	Natural water resources (1/sec)	Natural water resources (m^3/d)
1	30.0	10-20	15	450	38 800
2	16.7	5-10	7.5	125	10 800
3	11.5	10-20	15	173	14 947
4	13.3	20-30	25	332	28 685
5	50.4	30-40	35	1 764	152 410
6	24.5	40-50	45	1 102	95 213
7	14.0	10-20	15	210	18 144
8	167.4	30-40	35	5 859	506 218
9	7.2	10-20	15	108	9 331
10	24.0	20-30	25	600	51 840
11	44.0	5-10	7.5	330	28 512
Totals	403.0			11 053	954 979

For calculation of resources the average value for (Mr) was taken into consideration. The total natural ground-water resources in the Kandi area thus work out to about 11 m^3/sec, or 350 million cubic metres a year.

2. <u>Natural ground-water storage (static water resources)</u>

The static water resources of the Kandi belt have been calculated by the following formula:

$$Vn\ St = \mu\, Vo \qquad \text{Equation 4.2}$$

where

$Vn\ St$ = natural ground-water storage (static water resources, in m^3),

μ = specific yield, and

Vo = volume of the aquifer (m^3).

The values for specific yield are the same as those assumed above, i.e. 0.2 for the alluvial cone deposits and 0.15 for the intercone deposits.

On the basis of the hydrogeological cross-sections of the Kandi area (figure 7), the average thickness of the aquifer over some 312.3 km^2 of cone area works out to about 84 m. The intercone spaces occupy an area of about 46.7 km^2 between the Ghaggar and the Sutlej Rivers, where the average thickness of the aquifer is about 19 m, and about 44 km^2 between the Ghaggar and the Jamuna Rivers, where the average thickness of the aquifer is about 50 m.

The value for static ground-water storage in the Kandi belt as a whole is therefore equal to

$$\begin{aligned} Vn\ St &= 0.2 \times 312.3 \times 10^6 \times 84 \\ &\quad + 0.15 \times 46.7 \times 10^6 \times 19 + 0.15 \times 44 \times 10^6 \times 50 \\ &= 5709.7 \times 10^6\ m^3 \end{aligned}$$

The average thickness of the zone of aeration within the 312.3-km^2 area occupied by alluvial cone deposits is about 18 m. The volume of the aeration zone, therefore, works out to

$$312.3 \times 10^6 \times 18 = 5621.4 \times 10^6\ m^3$$

If a layer of this zone about 10 m thick could be saturated with ground water by artificial recharge, an additional storage, on the basis of $\mu = 0.2$, amounting to $312.3 \times 10^6 \times 10 \times 0.2 = 624.6 \times 10^6\ m^3$ could be produced.

3. Determination of hydrogeological parameters

Four test wells, each with one observation borehole, were constructed at three different sites in the Kandi area. Pumping tests were performed in three of the wells for the determination of the hydraulic parameters of the alluvial aquifer.

Two of the test wells were drilled at Fatehpur, one tapping the unconfined and the other the confined aquifer. One test well was constructed at Dera and the fourth at Kotla (figure 9).

Since enough data were already available regarding the discharge and drawdown values from the tubewells in the north-western part of the Kandi belt (including the Chandigarh well-fields), no exploratory holes or test wells were drilled there. Aquifer parameters such as thickness, lateral extent, composition and the like were obtained from the lithologs of the existing tubewells. Parameters such as transmissivity were estimated from the discharge/drawdown relations in the wells, using mathematical formulas that defined these relations approximately.

The Fatehpur and Dera sites were chosen as representative of the aquifer in the alluvial cones of the Ghaggar and the Markanda Rivers, respectively. Since there were, at Fatehpur, one unconfined and one confined aquifer, with no leakage, two test wells (with their two corresponding observation wells) were constructed there.

Exploratory drilling and construction of a test well were similarly taken up at Kotla with a view to determining aquifer parameters for the intercone alluvial aquifers between the Ghaggar and the Jamuna Rivers.

(a) Fatehpur:

As was noted above, two test wells have been constructed at the Fatehpur site. The shallow test well taps the unconfined aquifer and the deep test well taps the deeper, confined aquifer.

The static water levels in the shallow and the deep test wells were at depths of 7.6 and 13 m bgl respectively, indicating that the two aquifers were not interconnected, at least at the present site.

As a further precaution (to exclude the influence of any possible leakage from one well to the other during prolonged pumping) the two test wells were pumped simultaneously at the same rate of discharge: 50 l/sec or 4,320 m^3/d.

The data were processed by the C. E. Jacob method, plotting drawdown (S) against the log of the time since pumping started

Figure 10. Fatehpur observation wells: pumping test (S/log t)

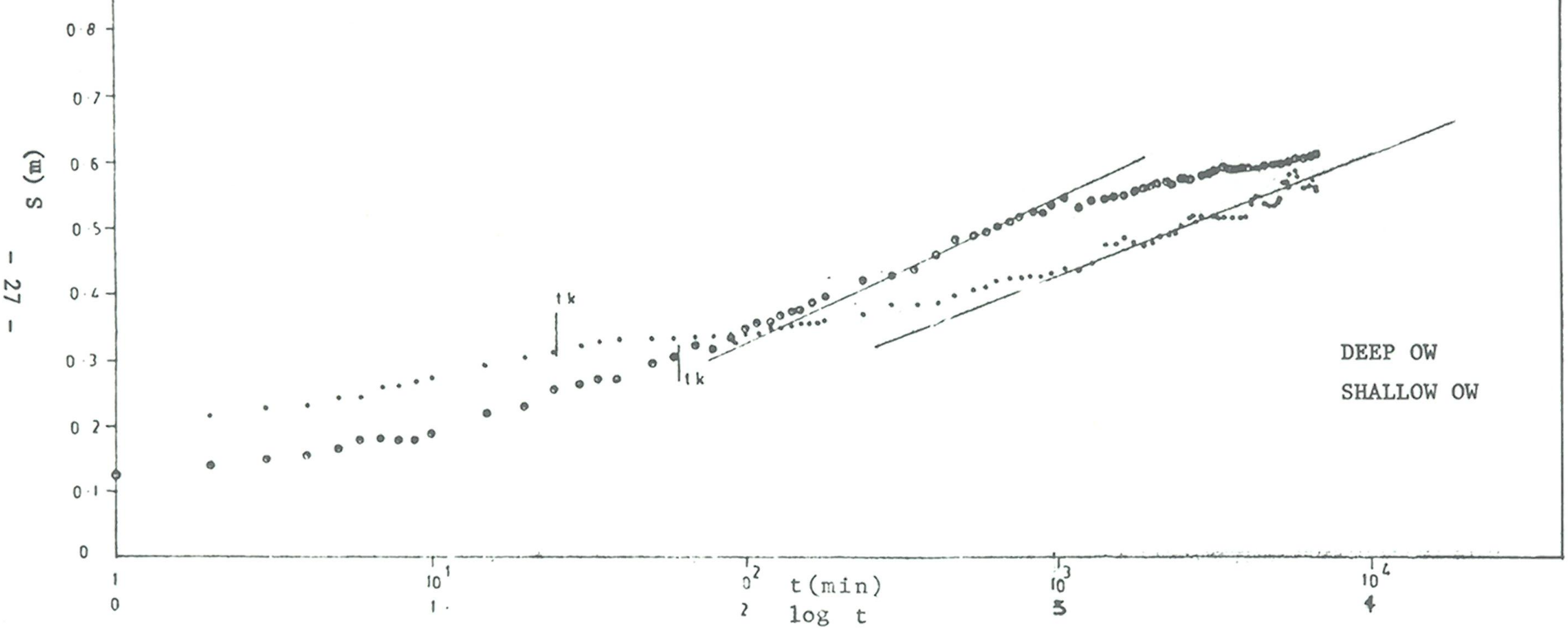


Figure 11. Fatehpur test wells: recovery $(S^*/\log \frac{t}{t+T})$

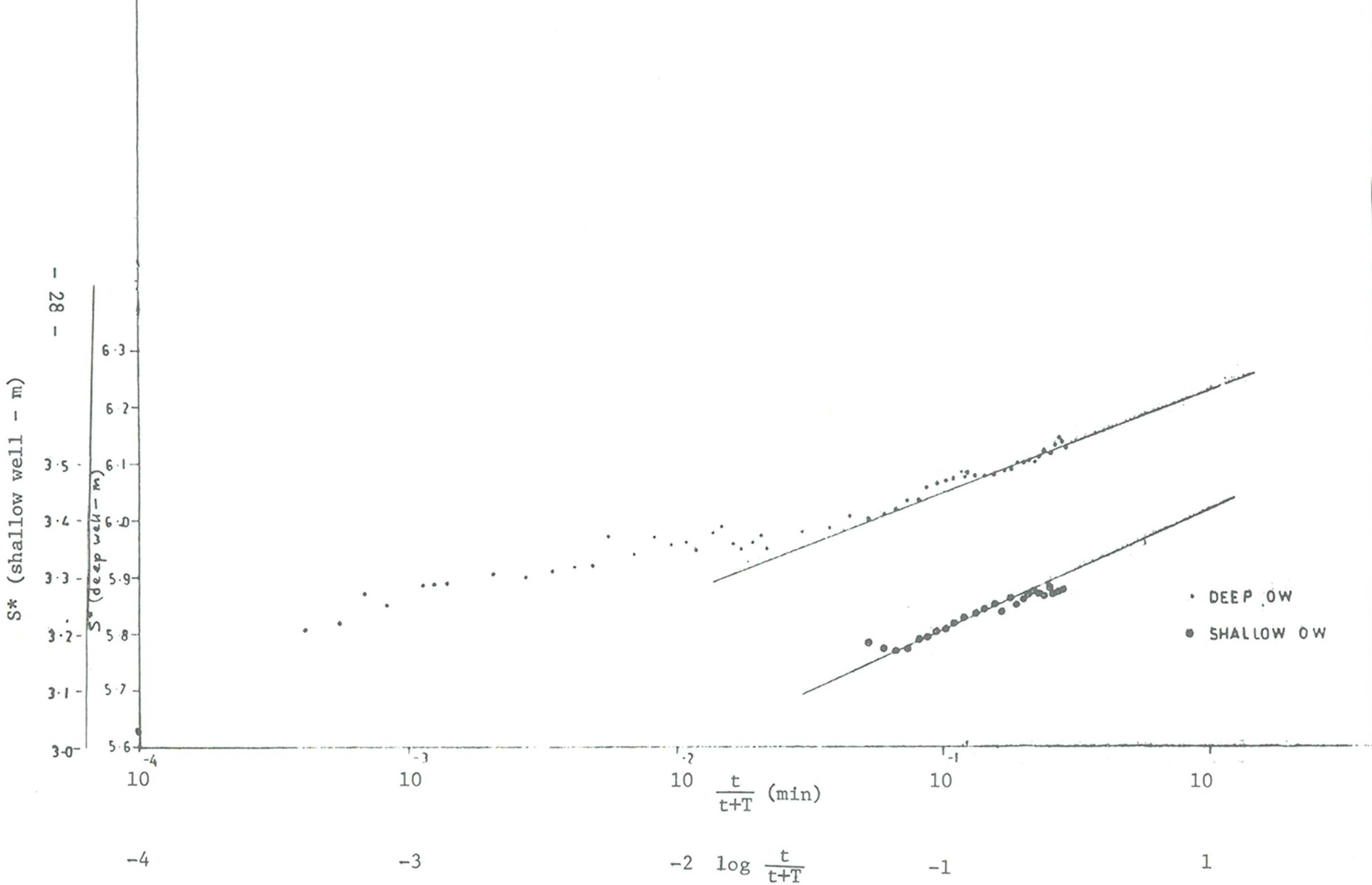
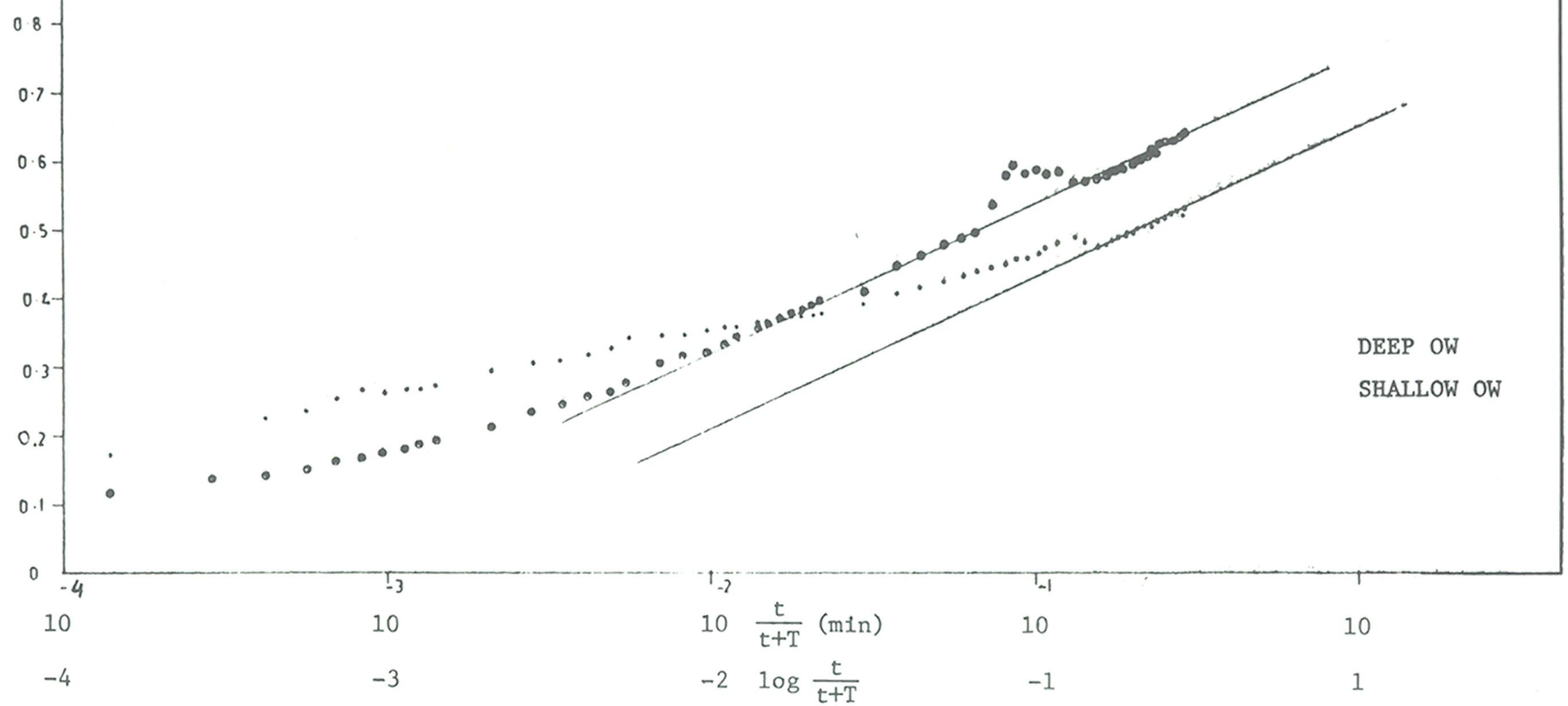


Figure 12. Fatehpur observation wells: recovery ($S^*/\log \frac{t}{t+T}$)



(t) and by plotting recover (S^*) against log t/ t + T. (See figures 10 through 12 for graphs obtained from the observation wells; for complete formulas, see chapter VI, section C.

(i) Pumping test analysis
(S versus log t):

The control time (in minutes) for the test and observation wells to reach the quasi-stationary state of filtration, as per the Jacob formula ($t_k = r^2/0.4a$) works out as follows:

Shallow tubewell	Shallow OW	Deep tubewell	Deep OW
0.006	61	0.0002	26

As the drawdowns in the test wells are affected by well losses (including the "skin effect"), they do not give representative values for the aquifer parameters. Therefore, only those graphs obtained by plotting drawdowns in the observation wells have been used for the determination of these parameters.

As seen in figure 10, the representative part of the curve for the shallow observation well starts almost immediately after t_k has been reached and continues through approximately 1,000 minutes. The slope of the curve then decreases, indicating the influence of some positive recharge boundary. The values for Km (= 3,593 m^2/d) and a (= 4.6 x 10^4 m^2/d) are therefore determined from this (representative) part of the graph.

The value thus determined for (a) shows that the aquifer is not homogeneous or uniform, but rather that it functions as a semi-confined aquifer at the beginning of the pumping test. This phenomenon is probably a result of the process of redistribution of pressure within the aquifer. The value for (a) is therefore not reliable and cannot be used in the determination of water resources.

As is seen in figure 10, the representative part of the graph for deep observation wells begins after approximately 1,000 minutes of pumping and continues until the end of the test. The graph for the period between (t_k) (26 minutes) and 1,000 minutes of pumping shows the effect of leakage within the aquifer, and as such it cannot be used for the determination of parameters.

The value determined for (km) from the steepest part of the graph is about 4,398 m^2/d; that for (a) is about 3.44 x 10^5 m^2/d. These values correspond, in general, to the average filtration constants for such aquifers.

The results of the aquifer parameters as determined above are as follows:

Well	Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
Shallow OW	4 320	0.22	-0.11	3 593	4.6×10^4
Deep OW	4 320	0.18	-0.12	4 392	3.44×10^5

(ii) Pumping test analysis:
recovery (S* versus log t/t+T)

As the recovery time for water level (t) is more than 10 per cent of the total pumping time (T) for both test wells ($t > 0.1$ T), and as the maximum drawdown effected during the pumping test is less than 20 per cent of the thickness of the aquifer (S max < 0.2 H or m), S* has been plotted against log t/t+T as observed, with no modification, for both test wells and for the two observation wells (figures 11 and 12). For the shallow test well and the observation well, the representative parts of the graph are rather long but for the deep test well and the observation well only the final parts of the graphs have been considered. The parameters determined by this method are summarized in table 7.

Table 7. Fatehpur: hydrogeological parameters as determined from recovery data ($S^*/\log \frac{t}{t+T}$)

Well	Q (m^3/d)	C	S_{max} (m)	Km (m^2/d)	a (m^2/d)
Shallow test well	4 320	0.22	3.21	3 593	7.9×10^{11}
Shallow OW	-	0.22	0.62	3 593	1.33×10^4
Deep test well	4 320	0.18	6.1	4 392	1.6×10^{11}
Deep OW	-	0.22	0.59	3 593	1.08×10^5

The values for (Km) and (a) as determined by the same method are summarized in table 8.

Thus, for further calculation the following values for the hydrogeological parameters will be used:

Shallow aquifer: (H) = 21 m; (Km) = 3,593 m^2/d; (K) = 171 m/d; and (a) = 1.3×10^4 m/d;

Deep aquifer: (m) = 65 m; (Km) = 4,392 m^2/d; (K) - 67.6 m/d; and (a) = 3.44×10^5 m^2/d.

(b) Ratewali:

The water-level recovery data recorded by HMITC, which conducted a test in the tubewell at the Ratewali site, were used for the determination of aquifer transmissivity. As can be seen in figure 13, an almost straight-line graph has been obtained by plotting 5* against log t/t+T. The well was pumped at a constant rate of 50 l/s (4,320 m^3/d) and the maximum drawdown (5 max) of 5.4 m was reached before the pumping was terminated and water-level recovery commenced. The thickness of the unconfined aquifer tapped (H) is 135 m.

The transmissivity (Km), as calculated from the above plot, is 2,635 m^2/d, and (K) works out at 19.5 m/d.

The specific yield (storage) has been calculated from the value determined above for (K) by the Betsinskyi formula:

$$\mu = 0.117 \sqrt[7]{K} = 0.117 \sqrt[7]{19.5} = 0.18$$

Water-level conductivity $a = \frac{Km}{\mu}$ will now be given as $\frac{2635}{0.18} = 1.46 \times 10^4 \ m^2/d$

The values for HMITC tubewell No. 201 were: (Km) = 3,360 m^2/d; (K) = 40.5 m/d; (μ) = $0.117\sqrt[7]{40.5} = 0.2$; and (a) = $\frac{Km}{\mu} = \frac{3360}{0.2} = 1.68 \times 10^4 \ m^2/d$.

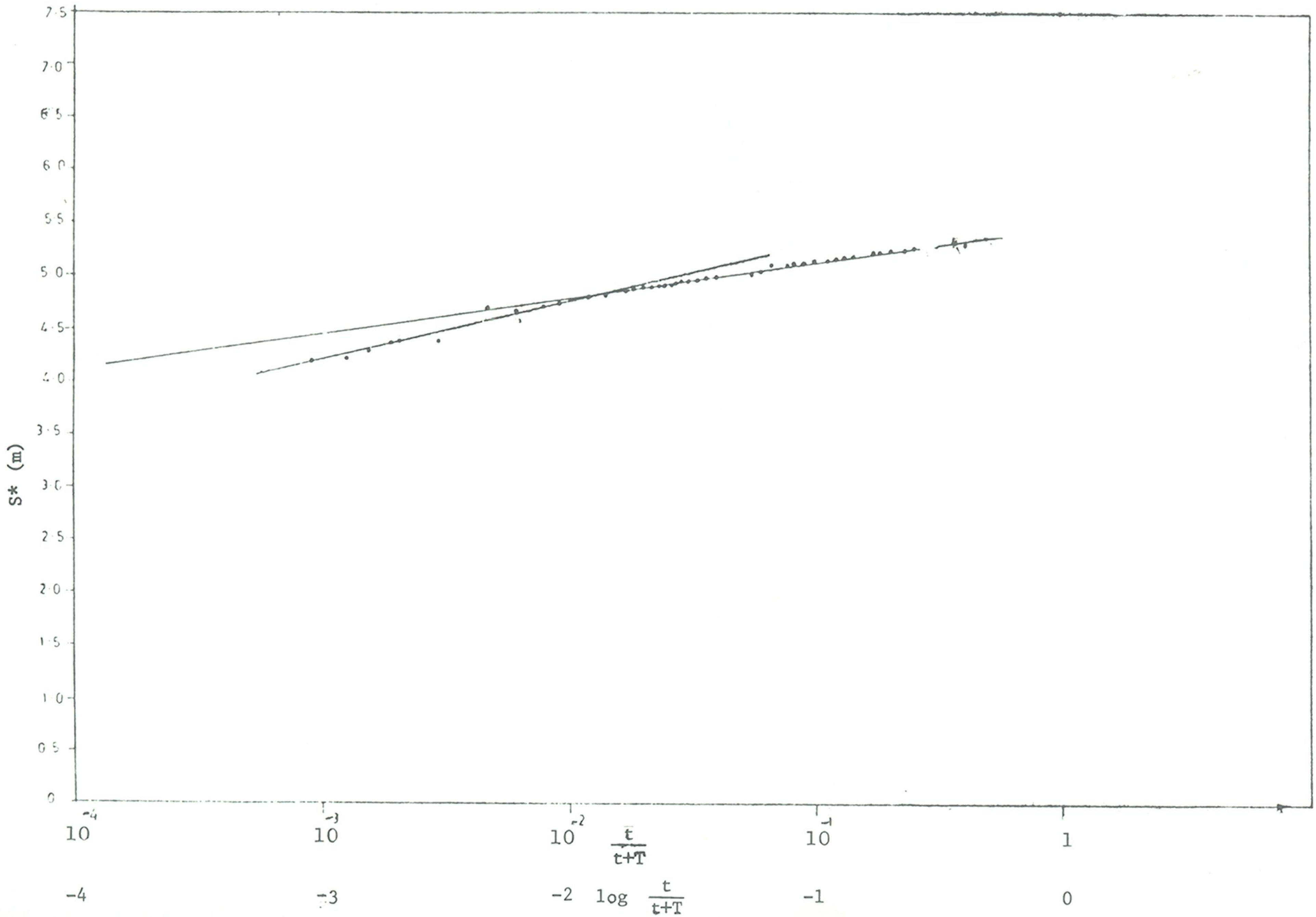
D. Calculation of exploitable ground-water resources

The calculation of exploitable ground-water resources in the Kandi area has been carried out for two proposed well-fields, one of which could be developed along the Ghaggar River, parallel to the riverbank, and the other along the part of the Kandi belt that lies between the Ghaggar and the Markanda Rivers. For the Ghaggar River well-field, the calculations of exploitable ground-water resources have been restricted, for the time being,

Table 8. Fatehpur: results of determinations of hydrogeological parameters (m^2/d)

Well	Hydrogeological Parameters					
	Transmissivity (Km)			Water-level conductivity (a)		
	Method of determination $S/\log t$	$s^*/\log \frac{t}{t+T}$	Accepted value	$S/\log t$	$s^*/\log \frac{t}{t+T}$	Accepted value
Shallow TW	-	3 593	3 593	-	7.9×10^{11}	1.3×10^{4}
Shallow OW	3 593	3 593	-	4.6×10^{4}	1.3×10^{4}	-
Deep TW	-	4 392	4 392	-	1.6×10^{11}	3.44×10^{5}
Deep OW	4 392	3 593	-	3.44×10^{5}	1.08×10^{5}	-

Figure 13. Ratewali test well: recovery ($S^*/\log\frac{t}{t+T}$)



to a short (approximately 1-km) section along the riverbank. This has been done for two reasons: First, the aquifer parameters along this river are based on the analysis of pumping test data at only one site, and may therefore hold good only over a limited area. Secondly, the exact layout of a well-field and the total drawdown to be expected depend upon the amount of water that will be pumped out of the field. For the Kandi area as a whole, however, the calculation of the exploitable ground-water resources has been carried out, using F.M. Bochever's methodology (4).

1. Proposed well-field along the Ghaggar River

The basis for calculations are the parameters as obtained at the Fatehpur site.

It is known that there are two aquifers at the proposed site of the Ghaggar River well-field, one of them shallow, the other deep, the aquifers being separated from one another by a layer of clay about 10 m thick. This clay layer creates confining conditions in the deep aquifer, which causes a piezometric head of 27 m. Under existing natural conditions there is no hydraulic connexion between the two aquifers, as is also inferred from the difference between the static water level and the piezometric level (7.6 m bgl and 13.0 m bgl, respectively). Even during the brief time of the pumping tests there was no evidence of any leakage from the shallow into the deep aquifer, calculations of the ground-water resources have been done separately for each of the two aquifers.

One, a very promising possibility, is exploitation of ground-water resources from the shallow aquifer by developing a well-field based on induced recharge. This proposal is attractive because the shallow aquifer is hydraulically very well connected with the surface water flowing in the Ghaggar River. At the same time it would also be most economical to exploit the deep aquifer at the same location.

The well-field could be constructed along the riverbank, say at a distance of about 200 m from the riverbed. At each site two wells, one tapping the shallow and the other tapping the deep aquifer, could be constructed within a few metres of each other.

Base flow in the Ghaggar River during about six months of the year is less than 300 l/sec. It would be safest to assume, for the purpose of these calculations, that the well-field would operate with no recharge during this period.

During the other six months of the year, however, when the river flow improves as a result of the monsoon rains, the same aquifer will have one positive boundary (the river) to fully replenish the ground-water draft by induced recharge.

The following parameters (already determined by exploratory work or proposed for the design of the well-field) should be taken into account for the calculation of ground-water resources from the shallow aquifer:

Thickness of aquifer (H)	21 m
Coefficient of permeability (K)	171 m/d
Water-level conductivity (a)	1.3×10^4 m^2/d
Distance between any two adjacent wells (λ)	200 m
Discharge per well (Q)	60 l/sec (= 5,184 m^3/d)
Radius of well intake section (r_o)	0.2 m
Maximum permissible drawdown (S_{mp})	$\approx$ 14.7 m

The actual drawdown (S) in the centre of the well-field, at the above-mentioned rate of discharge, is expressed by the Musket formula for unconfined aquifer without any hydraulic boundries. The formula reads

$$S = H - \sqrt{H^2 - \frac{Q}{\pi K}\left(\ln\frac{\lambda}{2\pi r_o} + \frac{3.55\sqrt{at}}{\lambda}\right)} \qquad \underline{\text{Equation 4.3}}$$

Where (t) is the time (in days) during which the well-field is pumped continuously without recharge (183 days in this case).

$$S = 21 - \sqrt{21^2 - \frac{5,184}{3.14 \times 171}\left(\ln\frac{200}{2 \times 3.14 \times 0.2} + \frac{3.55\sqrt{1.3 \times 10^4 \times 183}}{200}\right)} = 9.7 \text{ m}$$

Thus it is clear that a well field exploiting only the shallow aquifer, and using five tubewells, each separated from the next by 200 m and constructed parallel to the river, can give a discharge of about 300 l/sec with a drawdown of 9.6 m in the centre of the field (i.e. at well No. 3, counting from either side) at the end of the dry season. This drawdown is well within the permissible limits (14.7 m for the central well) and even this draft would be fully replenished during the following six-month recharge period.

For the determination of the water resources of the deep aquifer the following parameters have been used:

Thickness of aquifer (m)	65 m
Transmissivity (km)	4,392 m^2/d
Water-level conductivity (a)	3.44×10^5 m^2/d
Distance between adjacent wells (λ)	200 m
Discharge per well (Q)	100 l/sec = 8,640 m^3/d
Total discharge from well-field (5 Q)	500 l/sec = 43,200 m^3/d

In this case, the total time in days (t) over which pumping is continued without recharge (i.e. the period of operation of the well-field without recharge) is 10,000 days, or about 27 years, and the maximum permissible drawdown (S_{mp}) is equal to 0.7 (m) plus the pressure head above the top of the aquifer or (0.7 x 65) + 27 = 72.5 m.

The calculation for drawdown in the centre of the well-field has now been carried out by means of the Bochever formula for a confined aquifer without hydraulic boundaries (4):

$$S = \frac{Q\,\Sigma}{4\pi Km} \quad \frac{2.25\,a\,t}{R^2_{bw}}$$ Equation 4.4

Where Q = summary discharge of well-field (m^3/d)

R_{bw} = radius of one "big well", which will represent the entire well-field and which is given by the formula $R_{bw} = 0.1\,L$ Equation 4.5

and where L = length of the well-field (1,000 m in the present case).

Therefore $R_{bw} = 0.1 \times 1,000 = 100$ m

and

$$S \doteq \frac{43,200}{4 \times 3.14 \times 4,392} \ln \frac{2.25 \times 3.44 \times 10^5 \times 10,000}{100 \times 100} = 10.6 \text{ m}$$

which is, again, much less than the maximum permissible drawdown (72.5 m). Thus, it is possible to get 800 l/sec (69,120 m^3/d) of ground water from these two aquifers, by constructing five pairs of tubewells with a distance of 200 m between wells. This discharge will be available without implementing any artificial recharge measures.

It is even possible to recommend an area along the Ghaggar for exploitation of both these aquifers, as one of the variant schemes for supplying water to Chandigarh and Panchkula. Further detailed hydrogeological work will be required for the determination of the exact location and the optimum discharge to be obtained for the Chandigarh scheme.

Equation 4.3 can be used for the calculation of exploitable ground water resources for a well-field along the Markanda River.

2. Proposed well-field in the Kandi belt

As already stated, none of the rivers in the Kandi belt has perennial flow except for the Ghaggar and the Markanda. Thus, recharge to the aquifer takes place mostly during the rainy season, when the rivers are full. Under these conditions, the most favourable layout of the well-field would be more or less parallel to the recharge zone, i.e. within the Kandi belt, between the Chaggar and Markanda rivers. Such a disposition of the well-field would decrease the free outflow of water from the discharge zone (springline) south of the Kandi belt and correspondingly would increase the monsoon-season recharge through the riverbeds. It would then be possible to draw large quantities of water from the ground-water aquifer in the Kandi belt during the dry period of the year for both irrigation and water supply.

Although a number of tubewells already exist in the Kandi belt, data regarding the parameters of the aquifer are available only for HMITC tubewell No. 201. Reliance has therefore been placed on the values obtained at the Fatehpur site and on evaluation of the pumping test data (recovery data) for the HMITC tubewell at Ratéwali. For the calculation of ground-water resources, these values are likely to be only approximately representative; the results so far arrived at should be regarded as a first approximation only. For more accurate calculations, more detailed hydrogeological work is needed.

The following parameters have been used in the present context:

Average aquifer thicknes (H)	84 m
Coefficient of permeability (K)2/	42.5 m/d
Average water-level conductivity (a)	1.48×10^4 m^2/d
Discharge per well (Q)	100 l/sec = 8,640 m^3/d
Distance between wells (λ)	500 m
Length of well-field (L)	35,000 m
Number of wells	70
Radius of intake section of each well (ro)	0.2 m
Period of well-field operation (t)	10,000 days

Musket's formula (equation 4.3) for unconfined aquifers without any hydraulic boundaries has been applied for calculation of the drawdown in the central wall of the entire well-field.

$$S = 84 - \sqrt{84^2 - \frac{8{,}640}{3.14 \times 42.5}\left\{\ln \frac{500}{2 \times 3.14 \times 0.2} + \frac{3.55\sqrt{1.48 \times 10^4 \times 10{,}000}}{500}\right\}}$$

$$= 51.2$$

that is, less than the maximum permissible drawdown

(S_{mp} = 0.7 x 84 = 59 m).

Thus a well-field within the Kandi belt, between the Ghaggar and the Markanda rivers, could yield 8,640 x 70 = 604,800 m^3/d or 7 m^3/sec of ground water. The combined discharge of all possible well-fields in the Kandi area could amount, at the minimum, to between roughly 1.2 and 1.3×10^6 m^3/d, or 14-15 m^3/sec.

The water draft at present is about 0.2×10^6 m^3/d), including the water supply for Chandigarh (0.13×10^6 m^3/d). In other words, only about 17 per cent of the potential ground-water resources is being used.

2/ Average value, calculated from Fatehpur deep tubewell, Ratewali tubewell and HMITC tubewell No. 201.

E. Conclusions and recommendations

1. It has been found that a great potential of exploitable ground-water resources exists in the Kandi area, which can be effectively used for water supply to the towns of Chandigarh and Panchkula and the nearby villages, as well as for irrigation.

2. Almost all the rivers of the Kandi area lack perennial surface flow. They carry water only during the rainy season and for short periods thereafter. Only the Ghaggar and the Markanda rivers flow continuously, and both are considerably reduced during the dry period. Moreover, the other (emphemeral) streams show wide variations in discharge even during the rainy season, the flow in a given stream sometimes changing from zero to several m^3/sec within a day. It is recommended that the courses of the rivers near and upstream from the proposed well-fields may be levelled to spread the flow over a larger area. The clay film that may be formed on the bed of the channel should be scraped off. In order to increase the stability and life of the levelled areas, transverse-bottomed, ferro-concrete check-dams should be constructed, about 100-300 m apart and with a height of about 10-25 cm above the riverbed. These measures will increase recharge to the aquifer, and the water draft could be increased correspondingly.

3. Artificial recharge measures can be undertaken in the Ghaggar and the Markanda rivers simultaneously with the construction of well-fields parallel to these rivers. The quantity of water that will percolate downwards from such an artificially "improved" riverbed will be proportional to the area improved (F) and the rate of infiltration (V). Thus, if a 1,000-m length of the river (50 m average width) is artificially improved and the rate of infiltration is 2/d, the quantity of water (Q) that will be recharged is given as

$$Q = F \times V = 1{,}000 \times 50 \times 2 = 100{,}000 \ m^3/d = 1.2 \ m^3/sec.$$

4. It is also possible in the Kandi belt to use the other methods of artificial recharge, e.g. through basins and by filling the natural depressions with flood waters.

The basin method is the most effective method of artificial recharge. It does, however, require clean water with sediment load not higher than 20 mg/l. In the Kandi belt, water of this quality is available only in the Ghaggar and the Markanda rivers, and then only during the dry season, when their discharges are very low. Cleansing of flood waters for artificial recharge is a very costly operation, which is undertaken only when there is no alternative; this method should therefore be used only as one of the alternatives for the improvement of drinking-water supplies, as for example at Chandigarh.

The flooding of natural depressions can be undertaken in some of the south-eastern and central parts of the Kandi area in order to augment the local recharge to the aquifers; such depressions do exist in the area, as do a number of ravines that would be ideal for this purpose. The outlets of these depressions and ravines, wherever they are not underlain by clay beds, could be closed by the construction of small dams. Infiltration is likely to be slow but is certain to improve the supply of water from open dug wells.

Other artificial recharge methods, (e.g. the trench method and the injection method, using large diameter wells) are not economical and are therefore not recommended.

5. As discussed above, induced recharge could be carried out in the Ghaggar and Markanda riverbeds by constructing well-fields along their banks at a distance of about 150-200 m. The recommended discharge from these wells (60 - 100 l/sec, or 5,184-8,640 m^3/d will increase the natural rates of infiltration; however, since the flood waters of these rivers are heavily charged with suspended sediments, they may, at such high infiltration rates, clog the aquifer.

With the distance between the well-fields and the rivers established at about 150-200 m, infiltration rates would be kept in check and the clogging of the aquifer thus avoided. Tube-wells must be so constructed as to fully penetrate the aquifer, and their design should conform to their anticipated operational efficiency. Thus, the specifications for the pump installations should take the maximum drawdown values into account: the diameters of the assembly and the intake section of the wells should suit the pumps to be used and the discharge to be obtained, and the open area of the filters should not be less than 20-25 per cent, in order to minimize well losses.

6. Detailed hydrogeological investigations should be carried out prior to the construction of each well-field. The detailed work should include the estimation of the total exploitable ground-water resources, the pumping rates and durations, the anticipated drawdown and so on. The quality of the water should also be studied in detail, including the long-range effects of exploitation. The planning of the well-field will be based on this study; the investigations should therefore involve well-drilling, pumping tests and study of the water-level fluctuations on a more dense network of stations. Any test wells constructed for the purpose of such hydrogeological investigations should be such that they can later be used as production wells; some of the test wells should also have observation wells, which can be used for water-level monitoring after the investigations are over in order to study the long-term behaviour of the aquifer under exploitation.

V. PROJECT RESULTS: UNION TERRITORY OF CHANDIGARH

A. General

The area comprising the Union Territory of Chandigarh (about 114 km^2 between the Sukhna Choa and Patiali Rao rivers) is part of the Kandi belt. Because of ground-water overdraft, the water levels in the water-supply tubewells have declined by about 11.0 m during the 10 years between 1967 and 1976.

The ground-water drafts from the Sukhna Choa and Patiali Rao well-fields for 1976 were 73,627 and 58,565 m^3/d respectively. It is also estimated that during the 15 years from 1977 to 1991 the Sukhna Choa and Patiali Rao well-fields can be exploited at the respective rates of 110,000 and 52,000 m^3/d, which will create the maximum permissible drawdown (24.0 and 19.0 m respectively) in the centres of the fields. However, while a total of 162,000 m^3/d can be obtained from these well-fields, the projected demand for 1991 is 408,000 m^3/d at a time when the human population of Chandigarh is expected to be 625,000, or nearly three times the 1971 population of 218,000 (35). Thus, the existing reserves are inadequate to meet the future water-supply needs of Chandigarh.

However, favourable conditions exist for artificial recharge in this area. Recharge carried out by the basin method can create a yield of about 420,000 m^3/d for approximately 10,000 days (27 years) from the Sukhna Choa well-field alone.

B. Hydrogeological conditions

The water supply to the town of Chandigarh is provided by means of about 110 tubewells constructed in well-fields along the Sukhna Choa and Patiali Rao rivers and a few located between the two streams (figure 14). The distance between the two well-fields is about 8,650 m. The Sukhna Choa well-field has an area of about 20 km^2, with 54 operating tubewells ranging in depth from 50 to 120 m. The Patiali Rao well-field covers about 22 km^2, with 50 operating tubewells ranging in depth from 90 to 140 m.

The aquifer for both well-fields is of the same alluvial deposits, consisting of boulders, pebbles and gravels with intercalated beds of sand (of various grades) and clay. The aquifer is unconfined for the most part, although semi-confined or confined conditions do occur locally (figure 15).

1. Sukhna Choa well-field

The basic data of the Sukhna Choa well-field are described by M. C. Jindal (33) as follows: thickness of aquifer, 22.9 to 58.5 m (average 36.0 m); coefficient of transmissivity, 448 to 4,383 m^2/d (average 1,374 m^2/d; coefficient of permeability, 10.5 to 82 m/d (38.2 m/d average); specific yield, 0.16. The initial water levels at the time the tubewells were drilled ranged from 5.0 to 20.0 m bgl, while the present water levels in these tubewells stand between 9.0 and 24.0 m bgl. The discharge of the tubewells at the time of the pumping tests was 70-76 l/sec, with between 5.0 and 12.2 m drawdown. The water-level rise after the monsoon is between 0.9 and 6.1 m.

2. Patiali Rao well-field

The basic data of the Patiali Rao well-field are described (33) as follows: aquifer thickness, 13.0 to 36.4 m (26.8 m average); coefficient of transmissivity, 146 to 1,154 m^3/d (average 605 m^2/d; coefficient of permeability, 11.6 to 41.5 m/d (average 22.7 m/d); specific yield, 0.10. The initial water levels in the tubewells at the time of their construction were between 7.0 and 25.0 m bgl and the discharge of the tubewells during the pumping tests was from 34 to 38 l/sec, with a drawdown of 9.8 to 11.3 m. The post-monsoon rise in water levels was from 0.61 to 2.44 m.

A cumulative drawdown of 11.0 m, noted in the centres of both the Sukhna Choa and the Patiali Rao well-fields is the result of continuous pumping (about 20 hours a day) during the 10 years between 1967 and 1976.

C. Calculation of exploitable ground-water resources

The optimum yield from each well-field has been calculated by the method developed by N. N. Bindeman and L. S. Yasvin (1). Such an approach involves determination, in stages, of:

(1) The total draft from the well-field during the previous 10 years;

(2) The aquifer parameters, including values for permeability (K) and transmissivity (Km); and

(3) The radius of the "big well" - a single imaginary well, located at the centre of the well-field, which would cause the same drawdown as all the existing

tubewells together - and a calculation of reduced time (t_r), i.e. the time (in days) which the big well would take to pump out the total quantity of water so far withdrawn from the well-field at the latest rate of discharge.

1. Sukhna Choa well-field

The approximate ground-water draft from the Sukhna Choa well-field from 1967 to 1976, estimated on the basis of data provided by the Chandigarh Public Health Department and/or reported in Myslik's 1976 report (35) is as shown in table 9.

Table 9. Ground-water draft from Sukhna Choa well-field, 1967 - 1976

(m^3/d)

Year	Ground-water draft
1967	15 340
1968	23 010
1969	30 680
1970	38 356
1971	42 670
1972	43 505
1973	46 800
1974	54 319
1975	62 102
1976	73 623

The coefficient of permeability or hydraulic conductivity (K) has been calculated on the basis of the following formulas using Dupuit's equation with corrections introduced by N. N. Vozigin (1). The equation for unconfined aquifers is

$$K = \frac{0.73\ Q(\ln R/ro + 0.217\ \xi)}{(2\ H - S)\ S} \qquad \text{Equation 5.1}$$

and that for confined aquifers is

$$K = \frac{0.366\ Q\ (\ln R/ro + 0.217\ \xi)}{mS} \qquad \underline{\text{Equation 5.2}}$$

where Q = discharge of tubewell during pumping test (m^3/d);

R = radius of influence of tubewell (m);

l = total length of all the screened portions of the tubewell assembly (m);

H = total thickness of unconfined aquifer (m);

m = total thickness of confined aquifer (m); and

ξ = correction factor. The correction factor (ξ) takes into account the partial penetration of the aquifer, the value for which depends on l and H/m and can be obtained from a standard table given in any hydrogeology textbook.

The values of K and other variables, as thus determined for the wells at the Sukhna Choa well-field, are given in table 10. From this table, the average parameters of the aquifer at the Sukhna Choa well-field can be established as

H = 36 m;

K = 38.2 m/d; and

Km = 1,374 x m^2/d.

The radius of the "big well" (R_{bw}) is determined by the Bochever formula

$$R_{bw} = 0.1\ P \qquad \underline{\text{Equation 5.3}}$$

where P = well-field perimeter (roughly 17,689 m in the present instance).

Thus

$$R_{bw} = 0.1 \times 17{,}689$$

$$= 1{,}769 \text{ or (rounded off) } 1{,}770 \text{ m}$$

Table 10. Sukhna Choa well-field: calculations of permeability (K) and transmissivity (Km)

Tube-well No.	H/m (m)	l (m)	Q (m^3/d)	S (m)	ξ	K (m/d)	Km (m^2/d)
FJ-10	22.9	14.6	2 815	7.9	3.24	32.3	739
FJ-12	30	18	2 048	8.8	3.24	15.6	468
FJ-13	32	24	4 476	3.7	0.846	61.3	1 960
FJ-14	29.8	18.4	4 910	9.8	3.24	28.9	861
FJ-15	36	23	2 627	9.8	4.01	15.3	551
FJ-16	47.9	23.7	10 736	7.9	9.64	68.8	3 293
FJ-17	32	17	3 948	6.4	4.01	38	1 218
FJ-18	33	16	3 681	7.9	9.64	35.7	1 178
FJ-19	23.5	16.3	1 091	3.7	3.24	23.4	549
FJ-20	32	6	1 089	2.9	17.7	35.2	1.125
FJ-21	40.8	14	2 564	2.6	21.8	79.6	3 246
FJ-23	36	13	3 828	7.6	9.64	34.8	1 254
FJ-24	41	13	1 529	5.5	21.8	23.2	951
R-1	46	22	3 828	5.2	9.64	37.8	1 737
R-3	39.9	18.6	4 452	3.4	9.64	76.2	3 041
R-4	48	26	4 450	3.7	9.64	58	2 785
R-10	45.1	27.2	3 470	2.7	4.01	52.2	2 355
R-11	45	24	5 564	6.7	4.01	35.4	1 595
R-12	37	22	4 199	8.8	3.24	25.1	929
R-13	43	23	6 549	12.2	4.01	25.9	1 112
R-14	26.5	17.5	1 691	11.9	0.664	10.5	277
R-16	43	23	5 685	8.5	4.01	30.7	1 319
R-17	25	14	4 277	9.1	3.24	34	854
R-19	31	20	3 637	6.1	0.846	32.6	1 010
R-20	28	26.2	3 779	5.5	0.0	39.7	1 112
RN-17	42	24	3 793	7.3	4.01	24	1 011
RN-18	42.4	21.6	4 910	5.5	9.64	50	2 122
RN-19	32.3	16.7	3 791	7.6	3.24	30	970

Table 10 (continued)

Tube-well No.	H/m (m)	l (m)	Q (m^3/d)	S (m)	ξ	K (m/d)	Km (m^2/d)
RN-20	30	18	4 778	7.0	3.24	38.3	1 148
RN-21	31.4	15.8	4 910	7.3	7.86	50.5	1 585
RN-22	40	24	4 908	4.0	3.24	55.4	2 217
RN-23	23.5	16.4	3 600	7.6	0.664	36.4	855
RN-24	23.5	13.3	4 124	7.3	0.664	36.5	857
RN-25	39	23.5	5 346	5.2	4.01	50.2	1 958
RN-26	40	21	6 005	4.3	9.64	82	3 281
RN-27	36	17	5 504	6.1	3.24	47	1 690
RN-29	28	19	4 910	5.5	3.24	54.9	1 537
RN-46	12.5	9.95	644	6.1	2.67	14.2	177
RN-47	28	19	1 418	6.1	3.24	16	448
RN-48	49	30	3 819	4.6	4.01	31.6	1 548
RN-49	46	29	2 290	6.1	4.01	15.5	715
RN-50	39	23	4 622	6.1	4.01	37.5	1 461
RN-52	58.5	26.5	4 255	2.2	9.64	74.9	4 383
RN-58	33	18	2 687	7.6	3.24	20.8	685
RN-59	29.6	19	2 402	6.1	3.24	22.9	678
RN-61	47	27	3 292	7.6	4.01	17.8	838

If the "big well" is pumped at the rate of 73,623 m^3/d, i.e. at the latest (1976) rate of discharge, then the reduced time (t_r) over which the "big well" would have pumped the entire quantity of water that has been produced by the well-field can be calculated by the formula

$$t_r = \frac{\sum_{i=1}^{n} Q_i \, t_i}{Q_n} \qquad \text{Equation 5.4}$$

where Q_i = well-field discharge during the time interval t_i (in m^3/d)and

Q_n = most recent discharge figure from the well-field (in m^3/d).

Thus $t_r = \left[(15,340 + 23,010 + 30,680 + 38,356 + 42,670 + 43,505 + 46,800 + 54,319 + 62,102 + 73,623) \times 365\right] \left[73,623\right]^{-1}$

$= 5.85$ years or 2,134 days.

The water-level conductivity or hydraulic diffusivity of the aquifer can be determined by the following formula (L. S. Yasvin):

$$S = \frac{Q}{4 \Pi Km} \ln \frac{2.25\ a\ t_r}{R^2_{bw}} \qquad \underline{\text{Equation 5.5}}$$

Where S = drawdown in the centre of the well-field (= 11.0 m in the present instance);

Km = aquifer transmissivity (m^2/d);

a = water-level conductivity (hydraulic diffusivity) of the aquifer (m^2/d); and

R_{bw} = radius of "big well".

To determine the value of (a), equation 5.5 can be rewritten as follows:

$$\ln a = \frac{4 \Pi Km S}{Q} + 2 \ln R_{bw} - \ln 2.25 - \ln t_r \qquad \underline{\text{Equation 5.6}}$$

$$\ln a = \frac{4 \times 3.14 \times 1,374 \times 11}{73,623} + 2 \ln 1,770 - \ln 2.25 - \ln 2,134$$

$$= 9.06$$

or $a = 8,624\ m^2/d$

The maximum permissible drawdown (S_{mp}) in the case of an unconfined aquifer can be given by the formula S = 0.7 H, where (H) is the thickness of the unconfined aquifer, thus (S_{mp}), in the present case, would be:

$$S_{mp} = 0.7 \times 36$$

$$= 25.0 \text{ m.}$$

On the basis of the data obtained from working the tube-wells for the last 10 years it is possible to calculate drawdown in the centre of the well-field for any future date and at any given rate of discharge.

The estimated water-supply requirement for Chandigarh for 1991 is 408,000 m^3/d. This requirement cannot be met from this well-field; the quantity of water that can be made available from the Sukhna Choa well-field in 1991 will be as follows:

(1) The reduction time (t_r) for the period from 1967 to 1991 (= 25 years) can be calculated by equation (5.4):

$$t_r = [15,340 + 23,010 + 30,680 + 38,356 + 42,670$$

$$+43,507 + 46,800 + 54,319 + 62,102 + (16 \times 73.623)$$

$$\times 365] \quad [73,623]^{-1}$$

$$= 7,609 \text{ days}$$

$$= 20.85 \text{ years.}$$

(2) The drawdown after 25 years (1967 to 1991) can be calculated from equation (5.5):

$$S = \frac{73,623}{4 \times 3.14 \times 1,374} \ln \frac{2.25 \times 8,624 \times 7,609}{1,770 \times 1,770}$$

$$= 16.4 \text{ m}$$

This is less than the permissible drawdown of 25 m.

The drawdown in 1991, if water is withdrawn at the rate of 110,000 m^3/d, will be 23.9 m, according to the following calculations:

(3) The reduced time (t_r) for 1991 with discharge of 110,000 m^3/d since 1977 can be calculated using equation (5.4):

$$t_r = [15,340 + 23,010 + 30,680 + 38,356$$

$$+ 42,670 + 43,507 + 46,800 + 54,319 + 62,102$$

$$+ 73,623 + (15 \times 110,000) \times 365] \ [110,000]^{-1}$$

$$= 6,903 \text{ days}$$

$$= 18.9 \text{ years.}$$

(4) The drawdown at the centre of the well-field in 1991, using equation (5.5), will be

$$S = \frac{Q}{4 \Pi \, km} \ln \frac{2.25 \, a.t_r}{R \, bw^2}$$

$$= \frac{110,000}{4 \times 3.4 \times 1,374} \ln \frac{2.25 \times 8,624 \times 6,903}{1,770 \times 1,770}$$

$$= 23.9 \text{ m}$$

Thus, the ground-water draft from the Sukhna Choa well-field can be increased from 73,623 m^3/d (1976) to 110,000 m^3/d during the 15 years from 1977 to 1991, with an increase in drawdown from 11.0 to 23.9 m in the centre of the well-field.

2. Patiali Rao well-field

Calculations of the exploitable ground-water resources from the Patiali Rao well-field have been carried out along the same lines as those for the Sukhna Choa well-field.

The approximate figures for ground-water draft from the well-field for the period 1967-1976, as estimated on the basis of data supplied by the Chandigarh Public Health Department and from Mylil's 1976 report, are shown in table 11.

Table 11. Ground water draft from Patiali Rao well-field, 1967 - 1976

(m^3/d)

Year	Ground-water draft
1967	12 057
1968	18 086
1969	24 115
1970	30 137
1971	33 580
1972	36 780
1973	36 770
1974	42 679
1975	48 749
1976	58 565

The aquifer parameters were determined using equations (5.1) and (5.2), as shown in Table 12. The average parameter values, as indicated in Table 12, are as follows:

(H) = 26.8 m;

(K) = 22.7 m/d; and

(Km) = 605 m^2d.

The radius of the "big well" is determined using equation (5.3), (P) in this case being about 25,139 m (figure 14):

$$R_{bw} = 0.1\ P = 0.1 \times 25{,}139$$

$$= 2513.9\ m$$

The reduced time (t_r) was calculated for the 1976 discharge (see table 11) for the period from 1967 to 1976, using equation (5.4):

$$tr = (12{,}057 + 18{,}086 + 24{,}115 + 30{,}137$$

$$+33{,}580 + 36{,}780 + 36{,}770 + 42{,}679 + 48{,}794$$

$$+ 58{,}565)\ (58{,}565)^{-1}$$

$$= 2{,}128 \text{ days}$$

$$= 5.83 \text{ years.}$$

Using equation (5.6), the water-level conductivity (a) for the well-field is as follows:

Q = 58,565 m^3/day (1976);

S = 11.0 m (in centre of well-field).

Thus

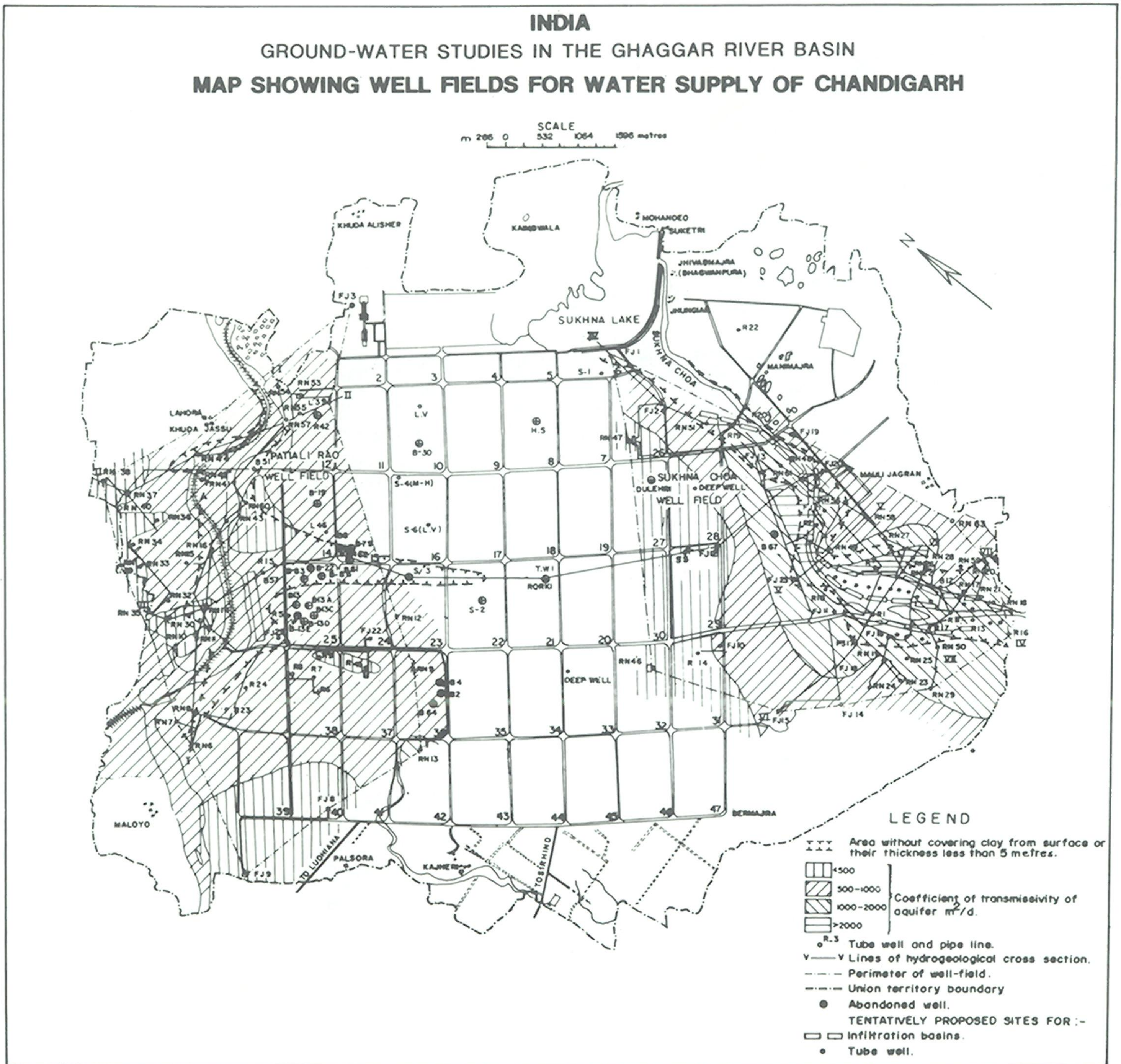
$$\ln a = \frac{4 \times 3.14 \times 605 \times 11}{58{,}565} + 2 \ln 2{,}514$$

$$- \ln 2.25 - \ln 2{,}128$$

$$= 8.61$$

Hence

$$a = 5{,}504\ m^2/d$$


INDIA
GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN
MAP SHOWING WELL FIELDS FOR WATER SUPPLY OF CHANDIGARH
SCALE
m 266 0 532 1064 1596 metres
SUKHNA LAKE
SUKHNA CHOA
PATIALI RAO WELL FIELD
SUKHNA CHOA WELL FIELD
MANIMAJRA
MALOYO
PALSORA
TO LUDHIANA
TO SIRHIND
LEGEND
Area without covering clay from surface or their thickness less than 5 metres.
<500
500-1000
1000-2000
>2000
Coefficient of transmissivity of aquifer m²/d.
Tube well and pipe line.
Lines of hydrogeological cross section.
Perimeter of well-field.
Union territory boundary
Abandoned well.
TENTATIVELY PROPOSED SITES FOR :-
Infiltration basins.
Tube well.

3237.8x

Table 12. Patiali Rao well-field: calculations of permeability (K) and transmissivity (Km)

Tube-well No.	H/m (m)	1 (m)	Q (m^3/d)	S (m)	ξ	K (m/d)	Km (m^2/d)
FJ-8	25.6	16.5	2 182	10.1	3.24	14.5	372
FJ-9	13.1	9	1 888	8.5	2.67	28.4	372
FJ-22	31.4	18.1	3 950	10.7	3.24	24.3	764
FJ-25	29	17.4	3 022	8.2	3.24	25.4	737
R-5	20.7	28.3	2 946	5.2	0.664	41.5	859
R-6	33.2	26.8	2 848	6.7	0.664	21.5	715
R-7	23.8	17.7	2 848	7.0	0.664	30.3	722
R-8	31.4	23.5	2 848	7.0	0.664	22.0	693
R-9	33.8	27.4	2 275	7.9	0.664	14.6	493
R-15	25	22	2 048	6.4	0	21.4	536
RN-7	29.6	14	2 073	7.9	7.86	21.3	631
RN-12	36.4	19.5	3 791	9.1	9.64	25.5	929
RN-13	31.7	14	3 055	6.1	7.86	36.4	1 154
RN-14	22	16	2 272	9.1	0.664	21.6	476
RN-15	25.9	15.5	2 673	10.7	3.24	20.9	540
RN-16	31	20	2 316	8.8	3.24	17	526
RN-30	30	20	2 471	9.8	3.24	14.5	434
RN-31	19.2	17.7	2 145	6.1	0.664	27.8	533
RN-32	19.8	15.2	2 247	6.1	0.664	33.3	659
RN-33	20.4	17.7	1 435	5.5	0.664	19.4	396
RN-34	32	21.9	2 073	6.1	3.24	20.2	646
RN-35	30	24	2 687	8.2	0.664	16.6	496
RN-36	18.3	14.6	1.637	6.1	0.664	22.2	407
RN-37	14	8	449	6.1	6.5	10.4	146
RN-38	30	25	1.702	6.1	0.846	15.8	474
RN-39	20.7	17.7	1 200	6.1	0	16.3	337
RN-40	30.2	13.3	1 698	6.1	7.86	21.4	646
RN-41	24	22	2 817	6.1	0.664	29.2	700

Table 12 (continued)

Tube well No.	H/m	1	Q (m^3/d)	S (m)	ξ	K (m/d)	Km (m^2/d)
RN-42	29.9	21.5	2 402	6.1	3.24	22.7	678
RN-43	33	19	2 402	6.1	3.24	22.6	745
RN-44	28	20	2 687	6.1	3.24	27	758
RN-45	26.8	15.8	982	6.1	3.24	11.6	311
RN-53	23.8	23.8	3 796	6.1	0	38.2	909
RN-54	33	30	2 938	6.1	0.846	24.5	810
RN-55	33	26	3 292	6.1	3.24	28	928
RN-60	20	16	1 581	7.6	0.664	15.8	315
RN-62	32	22	3 292	11.3	0.664	16.7	535
L-39	49	33	2 341	6.7	4.01	13.1	644

The maximum permissible drawdown (S_{mp}) is given by the formula

$$S_{mp} = 0.7 \ H$$

Where

H = 26.8 m works out to be 0.7 x 26.8 = 18.76 or 19.0 m.

If the ground-water draft from this well-field continues at the 1976 rate (i.e. 58,565 m^3/d), then (t_r) for the "big well" at the end of 1991 will be as determined according to equation (5.4):

$$\begin{aligned} t_r &= [12{,}057 + 18\ 086 + 24{,}115 + 30{,}137 + 33{,}580 \\ &\quad + 36{,}780 + 36{,}770 + 48{,}794 + 42{,}679 \\ &\quad + (16 \times 58{,}565) \times 365]\ [58{,}565]^{-1} \\ &= 7{,}603 \text{ days} \\ &= 20.83 \text{ years.} \end{aligned}$$

Using the value of 7,603 days for (t_r), and assuming that the ground-water draft from the well-field continues at the 1976 rate of 58,565 m^3/d, the drawdown in the centre of the well-field at the end of 1991 will be predicted by equation (5.5):

$$S = \frac{Q}{4 \pi Km} \ln \frac{2.25\ at_r}{R^2 bw}$$

$$= \frac{58,565}{4 \times 3.14 \times 605} \ln \frac{2.25 \times 5,504 \times 7,603}{2,514 \times 1,514}$$

$$= 20.81 \text{ m.}$$

Since this drawdown, 20.8 m, is greater than the permissible maximum of 19.0 m, the draft from the well-field will have to be reduced.

If the draft is lowered to 52,000 m^3/d for the 15 years from 1977 to 1991, the drawdown in 1991 will be 18.76 m according to the following calculations:

(1) According to equation (5.4), the reduced time (t_r) for 1991 with a draft of 52,000 m^3/d will be

$$t_r = [12,057 + 18,086 + 24,115 + 30,137 + 33,529 + 34,184 + 36,770 + 42,679 + 48,794 + 58,565 + (15 \times 52,000) \times 365] \ [52,000]^{-1}$$

$$= 7,872 \text{ days}$$

$$= 21.57 \text{ years.}$$

(2) The drawdown (S) at the end of the year 1991 will be according to equation (5.5),

$$S = \frac{52,000}{4 \times 3.14 \times 605} \ln \frac{2.25 \times 5,541 \times 7,872}{2,514 \times 2,514}$$

$$= 18.76 \text{ m}$$

Thus, within the maximum permissible drawdown it will be feasible to withdraw between 110,000 m^3/d and 52,000 m^3/d of ground water from the Sukhna Choa and Patiali Rao well-fields for the 15 years from 1977 to 1991, i.e. a total of 162,000 m^3/d as against the present total draft of 132,185 m^3/d. However, as the screen lengths in some of the tubewells may not be sufficient to allow the calculated drawdown, the yield, in actual practice may be somewhat less than 162,00 m^3/d.

But even this increased draft (162,000 m^3/d) will fail to meet the projected requirement of 408,000 m^3/d for 1991. It is possible, however, to increase the ground-water resources by artificial recharge. As discussed below, hydrogeological conditions in the Sukhna Choa and Patiali Rao well-fields are suitable for artificial recharge by the basin method.

D. Calculation of exploitable ground-water resources with artificial recharge

The method used below for calculation of ground-water resources with artificial recharge was developed by N. A. Plotnikov and K. Y. Sycheov (21).

1. Sukhna Choa well-field

The necessary aquifer parameters for the Sukhna Choa well-field are as follows:

Length of well-field along the Sukhna Choa	6,000 m
Average aquifer thickness (H)	36 m
Permeability (K)	38.2 m/d
Average transmissivity (Km)	1,374 m^2/d
Water-level conductivity (a)	8,604 m^2/d
Maximum permissible drawdown	25.0 m

It is proposed to excavate 24 recharge basins each 250 x 60 m in area and between two and five metres deep; the depth may vary, depending on the thickness of the surface clay deposits, which must be removed entirely. Twenty-three such basins will be in continuous use; the single remaining basin will be used as a standby.

Thus, the following figures are already known:

Number of basins to be excavated	24
Number of working basins	23
Length (l_b) and (b_b) of each basin	250 and 60 m, respectively

Total length of all 23 basins(Σl_b)	$250 \times 23 = 5{,}750$ m
Distance from a tubewell to the infiltration basin (X_o)	100 m
Distance from centre of basin to tubewell	$0.5\ l_b + X_o = 30 + 100 = 130$ m
Assumed distance between any two adjacent tubewells (σ)	200 m
Average infiltration rate (V_i)	1 m/d
Calculated time (t_i) for recharge through basins	10,000 days (= 27.4 years).

The value for specific recharge (q_b), i.e. the rate of infiltration from the basin in m^3/d per metre length of the basin, is given by the equation

$$q_b = \frac{V_i\ b_b\ \Sigma\ l_b}{2\ l} \qquad \text{Equation 5.7}$$

$$= \frac{1 \times 60 \times 5{,}750}{6{,}000}$$

$$= 57.5\ m^3/d/m$$

The radius of the cone of depression produced by pumping tubewells during exploitation of the well-field is (L_p). Since the tubewells are arranged along a line they behave like a single gallery, thus

$$L_p = 1.128\ \sqrt{at_i} \qquad \text{Equation 5.8}$$

Where (a) is the water-level conductivity, the value for which in this case has already been determined as 8,604 m^3/d.

Thus,

$$L_p = 1.128\ \sqrt{8{,}604 \times 10{,}000} = 10{,}463\ m$$

The radius of the cone of impression created as a result of recharge from all 23 basins is (L_b). L_b can be calculated by the formula:

$$L_b = 2\sqrt{at_i} \quad P\left[\frac{x}{2\sqrt{at_i}}\right] \qquad \underline{\text{Equation 5.9}}$$

Where

$$X = X_o + 0.5b_b = 100 = 0.5 \times 50 = 130 \text{ m}$$

$$P\left[\frac{X}{2\sqrt{at_i}}\right] = P\left[\frac{130}{2\sqrt{8,604 \times 10,000}}\right]$$

$$= p\ (0.007) = 0.564.$$

The value of p is seen from an already published table (14); therefore

$$L_b = 2\sqrt{8,604 \times 10,000} \times 0.564$$

$$= 18,552 \times 0.564 = 10,463 \text{ m.}$$

The above calculations assume a continuous gallery for water extraction, but since in fact the water will be withdrawn through a system of tubewells, a certain degree of hydrodynamic resistance will be offered by the aquifer between the tubewells. This resistance, which is denoted as L_{kt}, can be calculated as

$$L_{kt} = \frac{\sigma}{2\Pi} \ln \frac{\sigma}{2\Pi r_o} \qquad \underline{\text{Equation 5.10}}$$

Where r_o is the radius of the screened portion of the tubewell and σ is the distance between any two tubewells.

$$L_{kt} = \frac{200}{2 \times 3.14} \ln \frac{200}{2 \times 3.14 \times 0.1} = 183 \text{ m}$$

The value for specific discharge, i. e. discharge (m^3) per unit length of the well-field per day (q_w), can be calculated by the formula

$$q_w = \frac{9_b \ L_b}{L_p + 2\ L_{kt}} \qquad \underline{\text{Equation 5.11}}$$

$$Q_w = \frac{57.5 \times 10,463}{10,463 + 2 \times 183} = 55.6 \text{ m}^3\text{/d/m}$$

With the total length of the tubewell system given as 6,000 m, the total ground-water draft from the Sukhna Choa well-field will be Q = qw x 21 = 1 55.6 x 6,000 = 33,600 m^3/d or 3,860 l/sec.

The proportion of water that can be exploited in practice through tubewells to the total quantity of recharged water is denoted by α_b, which can be calculated as follows

$$\alpha_b = \frac{L_b}{L_p + 2 L_{Kt}} \qquad \underline{\text{Equation 5.12}}$$

$$\alpha_b = \frac{10,463}{10,463 + 2 \times 183}$$

$$= 0.97 \text{ or } 97\%$$

Assuming that each tubewell can discharge 70 l/sec, then the total number of tubewells required to exploit 3,860 l/sec of water will be 3,860 $\div$ 70 = 55 wells.

As there are already 54 operating tubewells in the Sukhna Choa well-field, only one additional tubewell will be required.

2. Patiali Rao well-field

The parameters of the Patiali Rao well-field are as follows:

Length of well-field	4,000 m
Coefficient of water-level conductivity (a)	5,541 m^2/d
Distance between any two adjacent tubewells	100 m
Distance from a tubewell to infiltration basin (X_o)	100 m
Length (l_b) and breadth (b_b) of each basin	250 and 60 m respectively
Total number of basins	7
Number of operating basins	6
Total length of operating basins	250 x 6 = 1,500 m
Average infiltration rate (V_i)	1 m/d
Calculated time (ti) for recharge through basins	10,000 days (= 27.4 years)

Specific recharge (q_b) is determined using equation (5.7):

$$q_b = \frac{1 \times 60 \times 1,500}{4,000} \quad 22.5 \ m^3/d/m$$

The radius of the cone of depression (L_p) is determined using equation (5.8):

$$L_p = 1.128 \sqrt{5,768 \times 10,000} = 8,567 \ m$$

The radius of the cone of impression (L_b) is determined using equation (5.9)

$$L_b = 2\sqrt{5,768 \times 10,000} \quad P\left(\frac{130}{2\sqrt{5,768 \times 10,000}}\right)$$

$$= 151 \times p(0.0086) = 15,189 \times 0.564$$

$$= 8,567 \ m$$

Hydrodynamic resistance (L_{kt}) is calculated using equation (5.10):

$$L_{kt} = \frac{100}{2 \times 3.14} \ln \frac{100}{2 \times 3.14 \times 0.1} = 81 \ m$$

The specific discharge (q_w) is calculated using equation (5.11):

$$q_w = \frac{22.5 \times 8,567}{8,567 + 2 \times 81} = 22 \ m^3/d/m$$

With the total length of the well-field 4,000 m, the total draft works out to be

$22 \times 4,000 = 88,000 \ m^3/d$, or 1,018 l/sec.

The proportion of water that can be exploited through tubewells to the total quantity of water recharged through the basins (αb) is calculated using equation (5.12):

$$\alpha b = \frac{8,567}{8,567 + 2 \times 81}$$

$$= 0.98, \text{ or } 98 \text{ per cent}$$

Assuming that each tubewell gives a discharge of 35 l/sec, for a total withdrawal of 1,018 l/sec we will require 1,018/35 = 29 tubewells.

The area where there is no overlying clay deposits, or where the clay covering is less than 5 m, is suitable for construction of recharge basins. In the Patiali Rao well-field such an area exists in the line of tubewells Nos. RN54, RN57, RN41, RN60, RN43, RN16, B57 and RN15 (figure 14). Therefore, all seven basins will have to be constructed in this area and only these eight tubewells can be used for exploitation, while pumping from the other tubewells should therefore be stopped. It will be necessary to drill 21 new tubewells along the same line as the above eight tubewells.

3. Possibility of exploiting the Sukhna Choa well-field alone

To obtain the total required quantity of water from the Sukhna Choa well-field alone, the number of infiltration basins would have to be increased from 24 to 30, so that 29 would always be in use with one reserved for standby. The revised calculation for 29 operating basins would be as follows:

$$\sum l_b = 29 \times 250 = 7{,}250 \text{ m}$$

$$q_b = \frac{1 \times 60 \times 7{,}250}{6{,}000} = 72.5 \text{ m}^3/\text{d}$$

$$L_p = 10{,}463\text{m};\ L_b = 10{,}463 \text{ m};\ L_{Kt} = 183 \text{ m}$$

Therefore

$$Q_w = \frac{72.5 \times 10{,}464}{10{,}453 + 2 \times 183} = 70 \text{ m}^3/\text{d}$$

The length of the well-field is 6,000 m. The total discharge obtainable from the well-field will therefore be $70 \times 6{,}000 = 420{,}000$ m^3/d or 4,860 l/sec(= 4.8 m^3/sec). The projected discharge, 420,000 m^3/d, is a little more than the projected requirement of 408,000 m^3/d for the year 1991 and therefore quite adequate.

If the discharge for each tubewell is 70 l/sec, then 4,860 ÷ 70 = 70 tubewells will be required to meet this demand.

The drawdown (S) in each tubewell, taking into consideration constant recharge for the basins and a discharge of 70 l/sec, will be:

$$S = \frac{q_w}{Km} L_{Kt} \qquad \text{Equation 5.13}$$

$$S + \frac{70}{1,374} \times 183 = 9.3 \text{ m}$$

Thus, about 20 additional tubewells would have to be drilled in the Sukhna Choa well-field, which could then entirely meet the water-supply needs of Chandigarh until the year 1991.

E. Conclusions and recommendations

1. An artificial recharge system near the Sukhna Choa well-field is recommended for the water supply to Chandigarh. The discharge thereby obtainable from the well-field would be 420,000 m^3/d, and the Patiali Rao well-field could then be abandoned. Nearly 20 additional tubewells, each with a discharge of 70-75 l/sec, will have to be drilled in the Sukhna Choa well-field. The tubewells must be placed at a distance of about 100 m from basins and not more than 200 m from one another. The tubewells should penetrate the entire thickness of the aquifer (100-130 m).

2. The water to be used for artificial recharge should meet the following quality specifications: p^H should be between 6.5 and 8.5; quantity of plankton should not exceed 1,000 cells/l; turbidity should be no greater than 20 mg/l for aquifer with effective grain size of 0.5-2 mm and not more than 10 mg/l for aquifer with effective grain size of 0.15 - 0.3 mm; the colour index should be 60 to 40 degrees on the cobalt scale; biochemical oxygen demand should be low; quantity of oxygen consumed by permanganate should not be more than 15 mg/l and by dichromate should not be more than 30 mg/l; bacterial pollution should be low (i.e. 10,000 E. Coli units per litre of water and bacteria count not higher than 5,000 per litre); the value for the various radicals should be FE $\leq$ 3 mg/l, Pb $\leq$ 0.1 mg/l, Cu $\leq$ 1 mg/l, phosphate $\leq$ 1 mg/l, oil $\leq$ 0.3 mg/l and phenol $\leq$ 0.001 mg/l.

3. The source of water for artificial recharge, because of the high turbidity values, cannot be the flood-waters of the Patiali Rao, Sukhna Choa or Ghaggar rivers. Moreover, the perennial flow of the Ghaggar River (0.16-1.81 m^3/sec; see table 2) is not adequate for the purpose. For these reasons it is proposed that the clean water of the Bhakra Canal be brought to Chandigarh through a lined canal, which could be fed directly into the recharge basins.

4. The recharge basins will be constructed along the Sukhna Choa River for a distance of about 600 m, approximately along the line of tubewells Nos. RN51, R19, R20, FJ13, RN48, RN61, FJ20, RN56, FJ21, R2, RN49, R3, RN22, R12, RN17, RN21, RN18, R1, FJ16, R17, R11, R13 and R16.

The distance between adjacent basins should be about 10-15 m or less and basins may be in two or three rows, as shown approximately on figure 14.

The size of each basin will be 250 x 60 x (2-5) m, the depth depending on the thickness of the overlying clay layer, which has to be fully removed.

The slopes of the basin should be of concrete or cement in order to avoid their erosion and consequent silting. The basins should be flat-bottomed and surfaced with a clean bed of sand about 0.5-0.8 m thick. The water column in the basins should be 1-1.5 m.

The basins should be provided with an arrangement for the control and measurement of discharge, and the water should be poured into the basins in such a manner as not to disturb the sand bed at the bottom.

The basins should be cleaned when the infiltration rate falls below 0.5 m/day, i.e. approximately every two to three months. At the time the basins are cleaned, the top 3-4 cm of sand should be replaced with new, clean sand.

Before constructing the basins, the subsurface lithology below the water table should be studied by drilling an adequate number of boreholes or, at the minimum, at least one in the centre of each basin.

The infiltration rate may be confirmed by actual field tests in at least three representative experimental pits. The depth of the pits should be equal to the proposed depth of the basin.

A zone of sanitary protection should be built around the basins and tubewells in order to prevent bacterial and industrial pollution of the water.

5. It is suggested that water-level fluctuations should be measured in at least three piezometers (not pumping wells), one of which should be at the centre of the well-field, another at the upstream edge and the third at the downstream limit of the well-field; water levels should be measured once every 10 days, and discharge measurements should be taken in all the pumping wells in the well-field.

VI. PROJECT RESULTS: NARWANA BRANCH CANAL AREA

A. General

All essential hydrogeological studies related to the Narwahana Branch Canal area were conducted at Dabkheri, about 3 km upstream from the Canal Rest House at Jyotisar (figure 16).

The purpose of these studies was threefold: to show, experimentally, the possibilities of artificially recharging a confined aquifer by injection of water through tubewells under pressure; to determine the period for which water could be continuously injected into a well without need for cleaning or development; and to verify whether untreated canal waters could be used for injection through the wells.

The Dabkheri site was chosen for the experiment because the local extraction of ground water by a number of augmentation tubewells along this canal, in Haryana, has already resulted in the threat of overdraft. The Narwana Branch Canal crosses the Project area from north to south with a discharge of about 113 m^3/sec at full capacity. Crossing the Project area in Haryana, the canal takes about 2.6 m^3/sec of ground water from the 80 augmentation tubewells situated along about 48 km of the canal's banks.

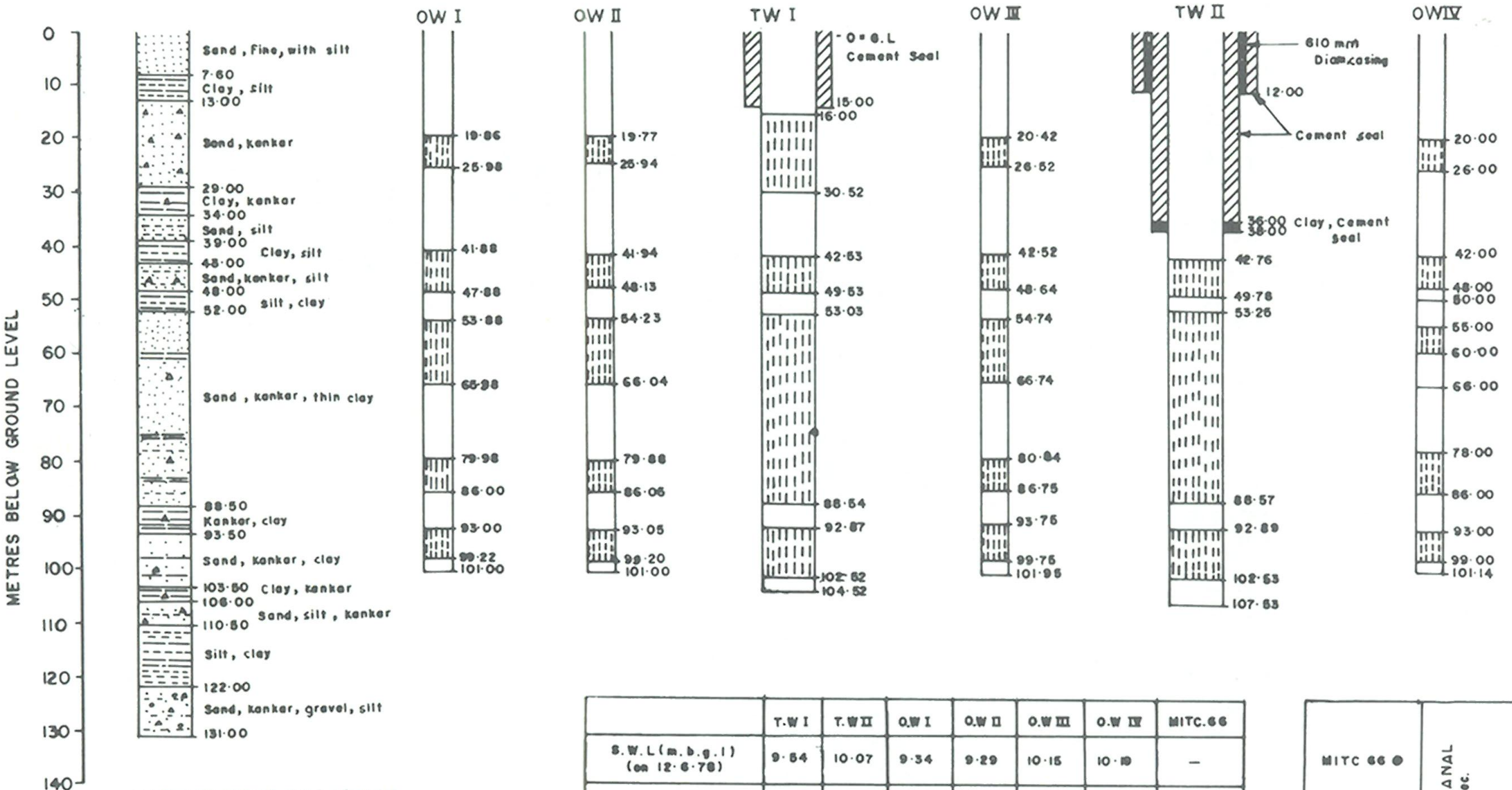
During the rainy season, when demand for canal water is small, some of these augmentation tubewells are used for artificial recharge of the aquifer by feeding canal water into them under gravity. The exact quantity of water thus replenished has not been calculated with certainty, and the aquifer, being under confined conditions, may not accept much water under gravity. Water draft and recharge are therefore not equally balanced.

B. Hydrogeological conditions

An alluvial aquifer is found in the area along the Narwana Branch Canal. It consists of sand and gravel with intercalated lenses and layers of clay, loam and sandy loam (figure 17). The thickness of these intercalations varies from 0.2 to 30 m and they are more frequent towards the northern part of the canal near the present course of the Ghaggar River. Their

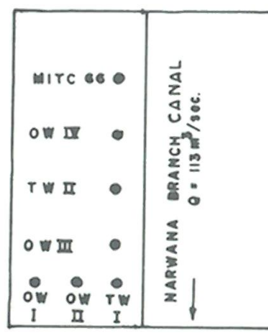
FIGURE 17

INDIA
NARWANA BRANCH AREA: SYNTHETIC LOG OF DABKHERI SITE



(BASED ON PIOLET HOLE DRILLING FOR T.W. I)

	T.W I	T.W II	O.W I	O.W II	O.W III	O.W IV	MITC.66
S.W.L (m.b.g.l) (on 12·6·78)	9·54	10·07	9·34	9·29	10·15	10·19	—
ASSEMBLY DIAM. (mm)	355	305	100	100	100	100	—
SLOT SIZE (mm)	1·6	3·2	1·6	1·6	1·6	1·6	—
DISTANCE FROM TW I	X	50m	150m	60m	30m	75m	130m
DISTANCE FROM TW II	50m	X	152·70m	74·50m	20m	25m	80m



3237.11x

thickness decreases towards the south and near the village of Kirmach (RD-315) where they are missing altogether. The thickness of the aquifer also varies from about 40 m in the northern to about 119 m in the southern part of the area; average aquifer thickness (without clay) is about 79.6 m (figure 18). The aquifer also has good filtration characteristics, the average coefficient of permeability (K) being 24.3 m/d and the average transmissivity (Km), 1,936 m^2/d. The aquifer is confined over most of the area and it attains a piezometric head ranging from 2 to 10 m bgl. In the southern part of the area the aquifer is unconfined; depth to water level is about 6 m bgl.

Discharges from the tubewells vary from 28.9 to 80.7 l/sec, with drawdowns ranging from 5.6 to 18.9 m.

At Dabkheri, the aquifer parameters were as follows: aquifer thickness, 72,5 m; test well discharge, 69 l/sec; drawdown, 5.6 m; transmissivity, 2,424 m^2/d; and water-level conductivity, 1.31 x 10^5 m^2/d.

The groundwater at the Dabkheri site is fresh, mineralization being 266 mg/l (HCO_3 = 251 mg/l, Cl = 10 mg/l, SO_4 = 19 mg/l, Ca = 30 mg/l, Mg = 11 mg/l, Na = 53 mg/l, K = 3.5 mg/l, NO_3 = 1 mg/l, F = 0.86 mg/l, B = 0.24 mg/l, SiO_2 = 19 mg/l and total hardness as $CaCO_3$ = 121 mg/l).

Ground water is widely used in this area for water supply, irrigation and as a source of replenishment of canal water, during the dry season, through the augmentation tubewells.

C. Determination of Hydrogeological parameters

Exploratory drilling was carried out at the Dabkheri site down to a depth of 131 m. A test well and four observation wells were then constructed, each to about 101 m depth.

A long-duration pumping test was conducted from 24 to 26 October 1977. Water-level measurements were made in the test well, in the four observation wells constructed by the Project and in another tubewell, HMITC tubewell No. 66 (figure 19).

The pumping test data were processed according to the C. E. Jacob method (15), plotting S versus log t, S versus log r, S versus log t/r^2 and S* versus log t/t+T. Transmissivity (Km) and water-level conductivity (a) were calculated by the following formulas given in table 13.

Table 13. Dabkheri: methods of processing pumping test data

$S/\log t$	$S/\log r$	$S/\log \frac{t}{r^2}$	$S^*\ \log/\ \frac{t}{t+T}$
$Km = \frac{0.183\,Q}{C_t}$	$Km = \frac{0.366\,Q}{C_r}$	$Km = \frac{0.183\,Q}{C_k}$	$Km = \frac{0.183\,Q}{C}$
$\log a = 2 \log r - 0.35 + A/C_t$	$\log a = \frac{2A}{C_r} - 0.35 - \log t$	$\log a = \frac{A}{C_k} - 0.35$	$\log a = 2 \log r - 0.35 + \frac{S_{max}}{C} - \log T$

Where A is the intercept on the ordinate axis when log t = 0, log r = 0 or log $t/r^2 = 0$

$$C_t = \frac{S_2 - S_1}{\log t_2 - \log t_1}$$

$$C_r = \frac{S_1 - S_2}{\log r_2 - \log r_1}$$

$$C_k = \frac{S_2 - S_1}{\log (t/r^2)/2 - \log (t/r^2)\ t}$$

and where

Km = transmissivity (m^2/d);

a = water-level conductivity (m^2/d);

Q = discharge of test well (m^3/d);

r = distance between test well (TW) and observation well (OW) (m);

t = pumping time (days); and

S = drawdown (m).

For the plot of recovery ($S^*/\log \frac{t}{t+T}$)

$$C = \frac{S^*_2 - S^*_1}{\log (t/t + T)2 - \log (t/t + T)1}$$

where

S = rise in water level above pumping level since start of recovery (m);

T = duration of pumping (days); and

S_{max} = maximum drawdown during pumping test (m).

1. Pumping test analysis (S/log t)

Control time for reaching the quasi-stationary regime of filtration, as per the Jacob criterion, was determined for each well by the formula $t_k = \frac{r_2}{0.4a}$. The results, in minutes, for each of the wells were as follows:

Test well	OW No. 1	OW No. 2	OW No. 3	OW No. 4	OW No. 66
8.4×10^{-4}	413	81	18.7	126.6	411.1

As seen in figures 20 and 21, representative parts of the graphs are rather long. The first parts of the graphs show the effects of mutual leakage from one granular zone to the other within the aquifer, and have therefore not been considered for the determination of the hydrogeological parameters. After the stabilization of this leakage, i.e. after about 1,100 minutes of pumping, the aquifer is seen to behave as a single unit, as seen on the graphs.

The values for (Km) as thus determined ranged from 1,364 to 2,479 m^2/d and those for (a) ranged from 1.48×10^5 m^2/d to 1.96×10^5 m^2/d for the various observation wells. From the test well data (Km) works out to 1,364 m^2/d, almost half of what is found from the observation well data, because of well losses and the "skin effect" in and around the test well. The value for (Km) obtained from the test well has therefore not been considered. Similarly, determination of (a) from the pumping test data from the test well is also not possible, as well losses and the "skin effect" cause the graph plot to be displaced along the ordinate axis. The values for (Km) and (a) as determined by this method are summarized in table 14.

Figure 20. Dabkheri test well: pumping test (S/log t)

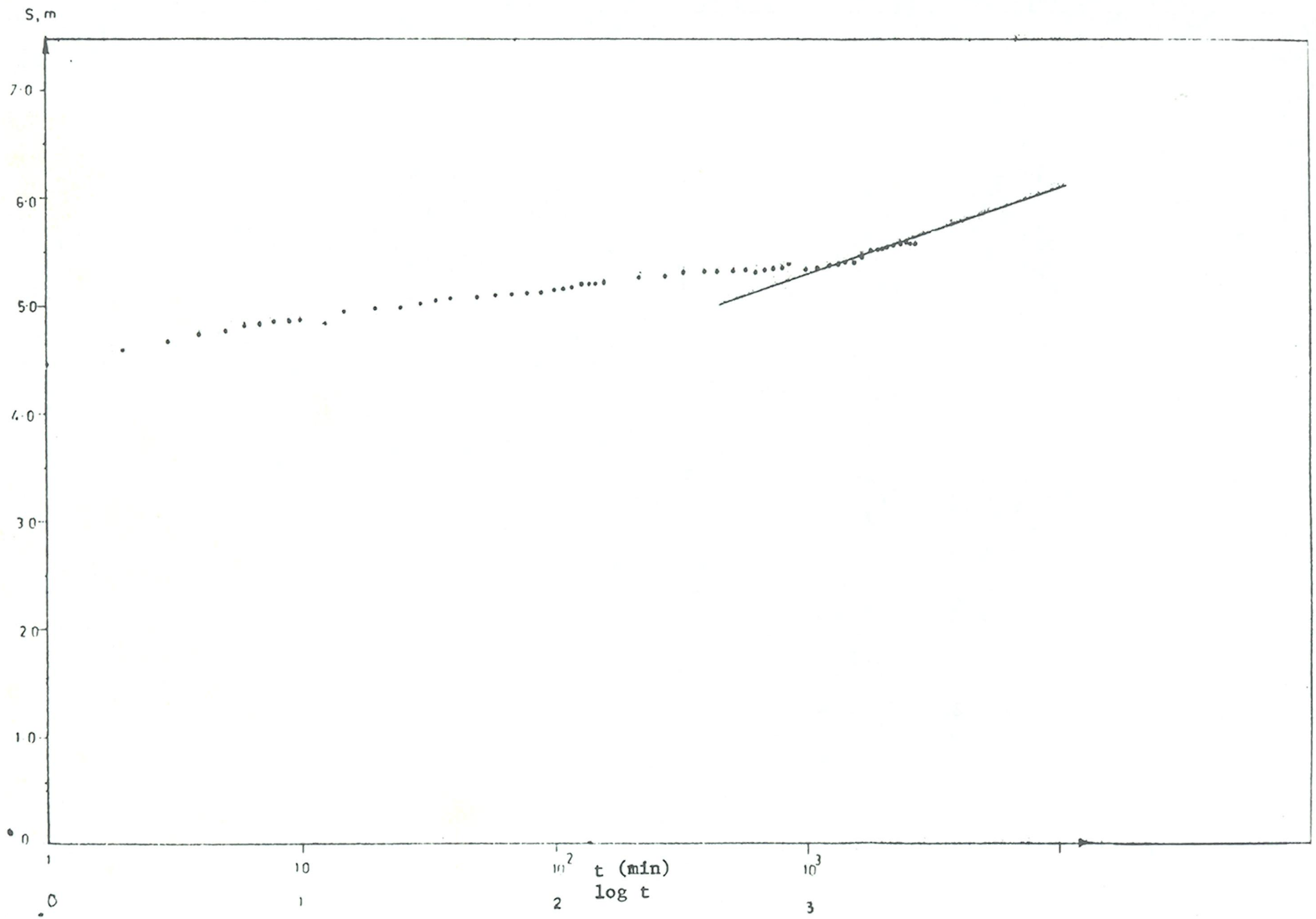


Figure 21. Dabkheri observation wells: pumping test (S/log t)

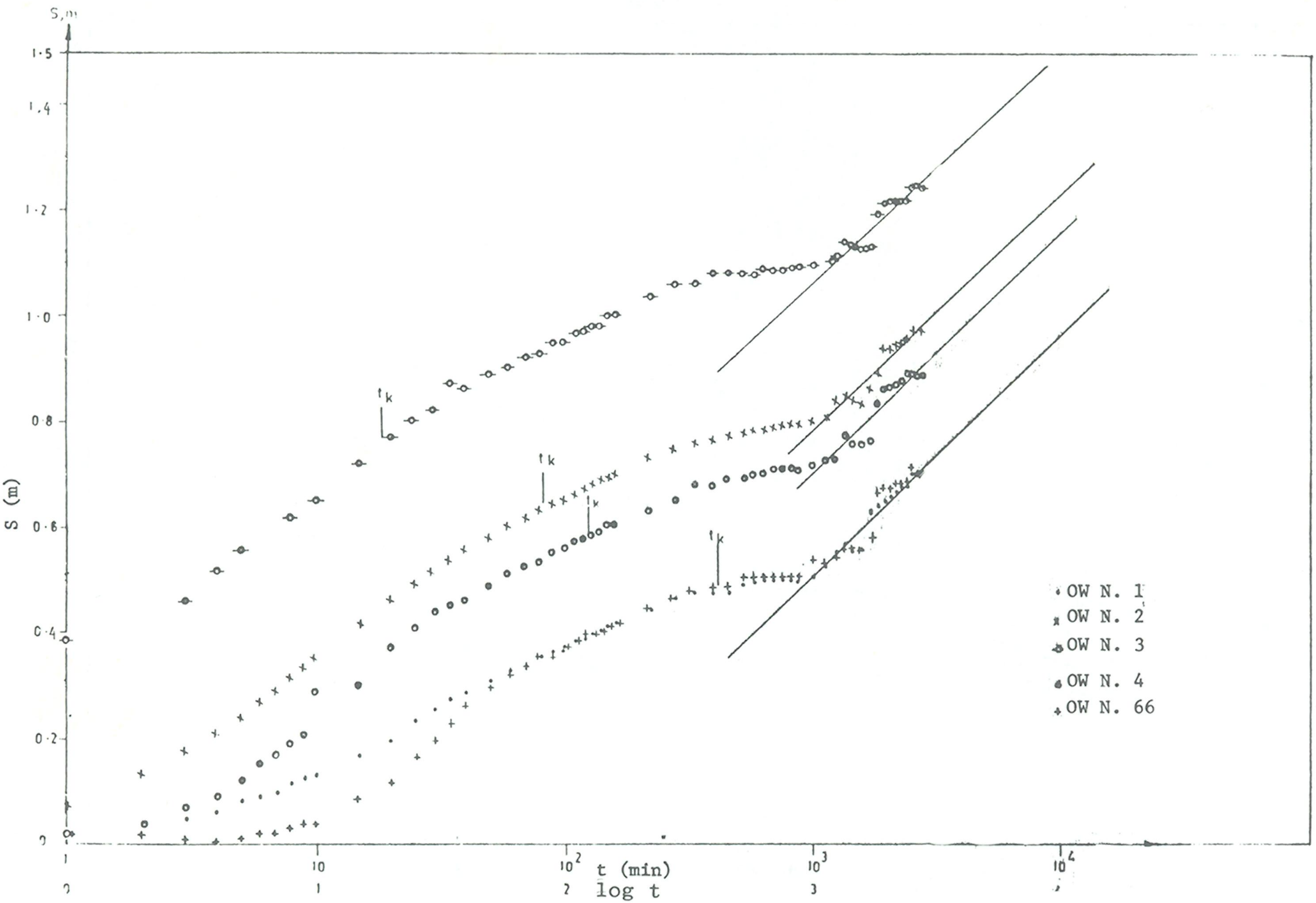


Table 14. Dabkheri: results of pumping test (S/log t)

Well	Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
Test well	5 962	0.8	--	1 364	--
OW No. 1	--	0.45	0.84	2 424	1.96×10^5
OW No. 2	--	0.45	0.52	2 424	1.6×10^5
OW No. 3	--	0.44	0.23	2 479	1.73×10^5
OW No. 4	--	0.46	0.62	2 372	1.6×10^5
OW No. 66	--	0.45	0.84	2 424	1.48×10^5

2. Pumping test analysis (S/log r)

A graph in coordinates S versus log r was plotted for two periods of time: 1,360 minutes and 2,800 minutes after the start of pumping (figure 22). Valid hydrogeological parameters have been obtained for observation wells Nos. 2, 3 and 4 and HMITC tubewell No. 66.

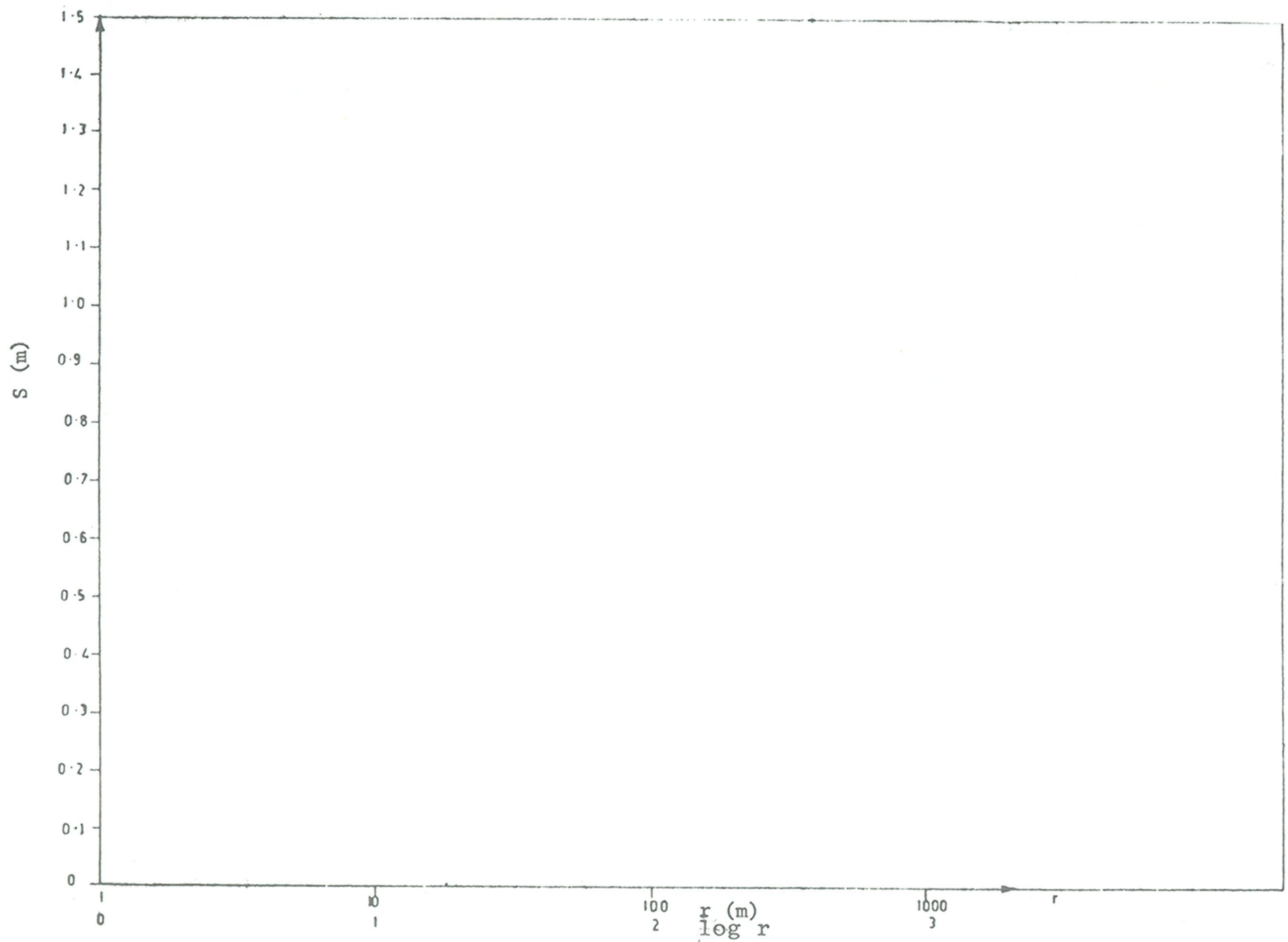
Results from OW No. 1 cannot be used, as the drawdown in this well was somewhat higher than in the other observation wells. Transmissivity for the rest of the observation wells works out to 2,424 m^2/d and water-level conductivity to between 0.82 and 2.8×10^5 m^2/d.

The results obtained by this method are summarized in table 15.

Table 15. Dabkheri: results of pumping test (S/log r)

Minutes since start of pumping	Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
1 360	5 962	0.9	2.6	2 424	2.8×10^5
2 800	5 962	0.9	2.5	2 424	0.80×10^5

Figure 22. Dabkheri observation wells: pumping test (S/log r)



3. Pumping test analysis $(S/\log \frac{t}{r^2})$

As may be seen from figure 23, the curve plots of the drawdowns in the observation wells had all converged to a straight line, even though the duration of the pumping test was rather short, showing that the quasi-stationary regime of filtration had already been reached. The straight-line part of the curves could therefore be used for the determination of (Km) and (a). The values thus found were: Km = 2,424 m^2/day; and a = 1.31 x 10^5 m^2/d.

It may be stated that this method gives the most objective results insofar as it estimates the parameters as the average for nearly the entire area of the cone of depression. Table 16 shows the values for (Km) and (a) as determined by this method.

Table 16. Dabkheri: results of pumping test $(S/\log \frac{t}{r^2})$

Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
5 962	0.45	1.04	2 424	1.31 x 10^5

4. Pumping test analysis: recovery $(S^*/\log \frac{t}{t + T})$

The picture of the rise in water level after the pumping has been stopped is practically the same as that for drawdown. At the beginning of the recovery, a "back-leakage" effect is seen on the graph (figures 24 and 25). The latter part of the recovery graph, which represents equilibrium in the aquifer, has been used for calculation of (Km) and (a). The values thus determined are: Km = 2,424 m^2/d; a = (5.5 - 7.1) x 10^4 m^2/d. These values were obtained from the plot of all the wells, as summarized in table 17.

The hydrogeological values are summarized in table 18.

Figure 23. Dabkheri observation wells: pumping test $(S/\log \frac{t}{t + T})$

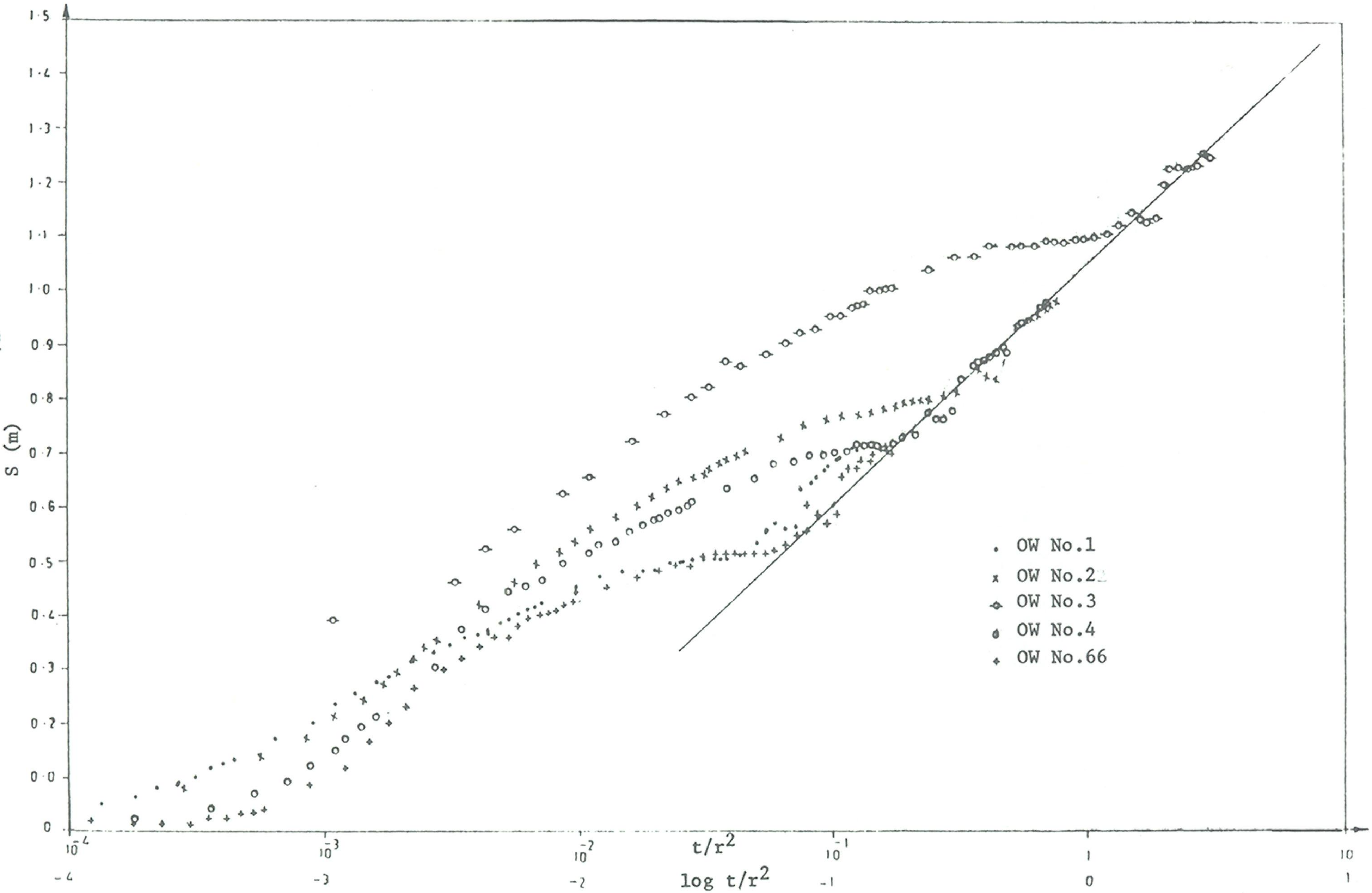


Figure 24. Dabkheri test well: recovery ($S^*/\log \frac{t}{t + T}$)

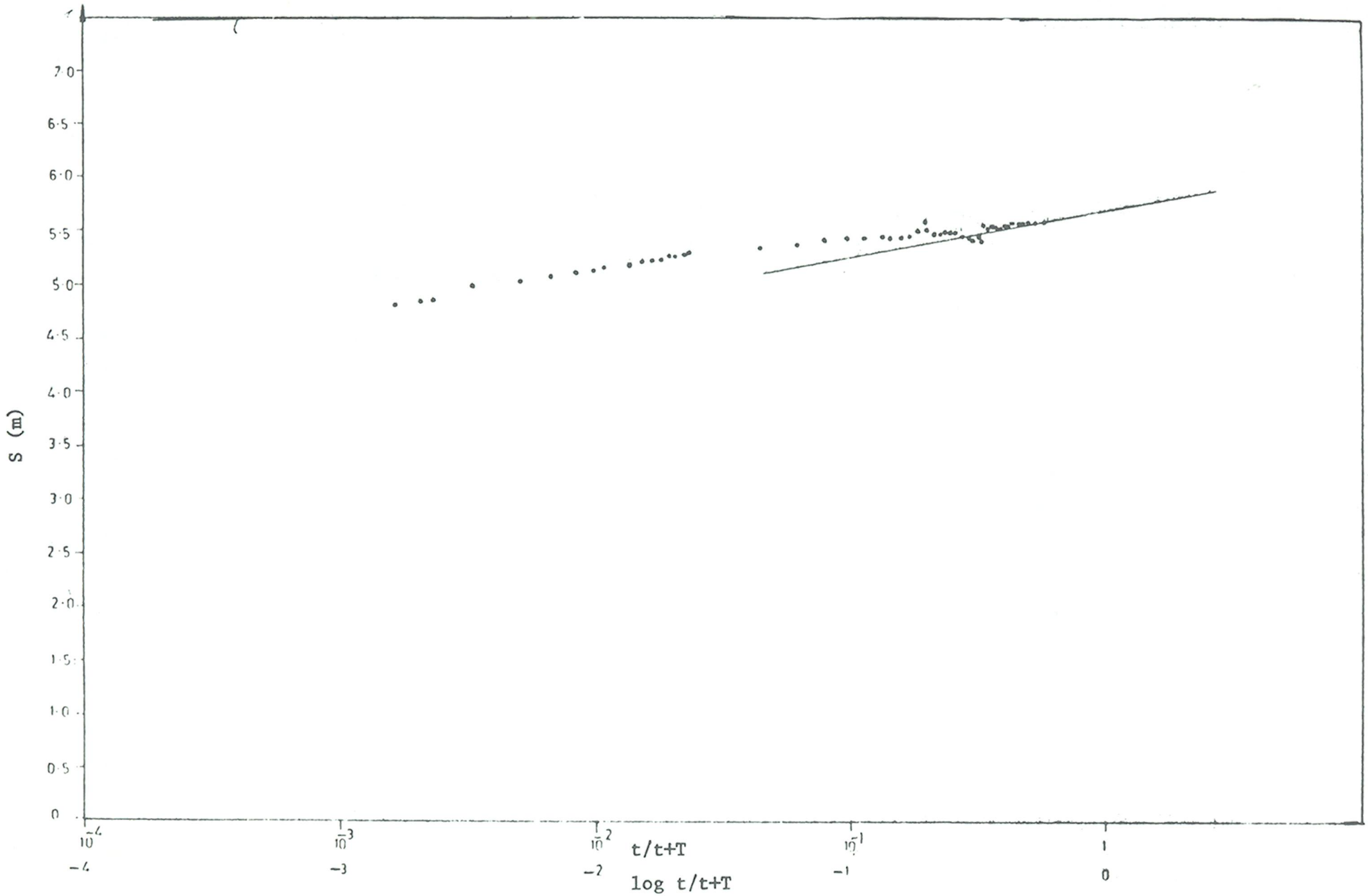


Figure 25. Dabkheri observation wells: recovery $(S^*/\log \frac{t}{t + T})$

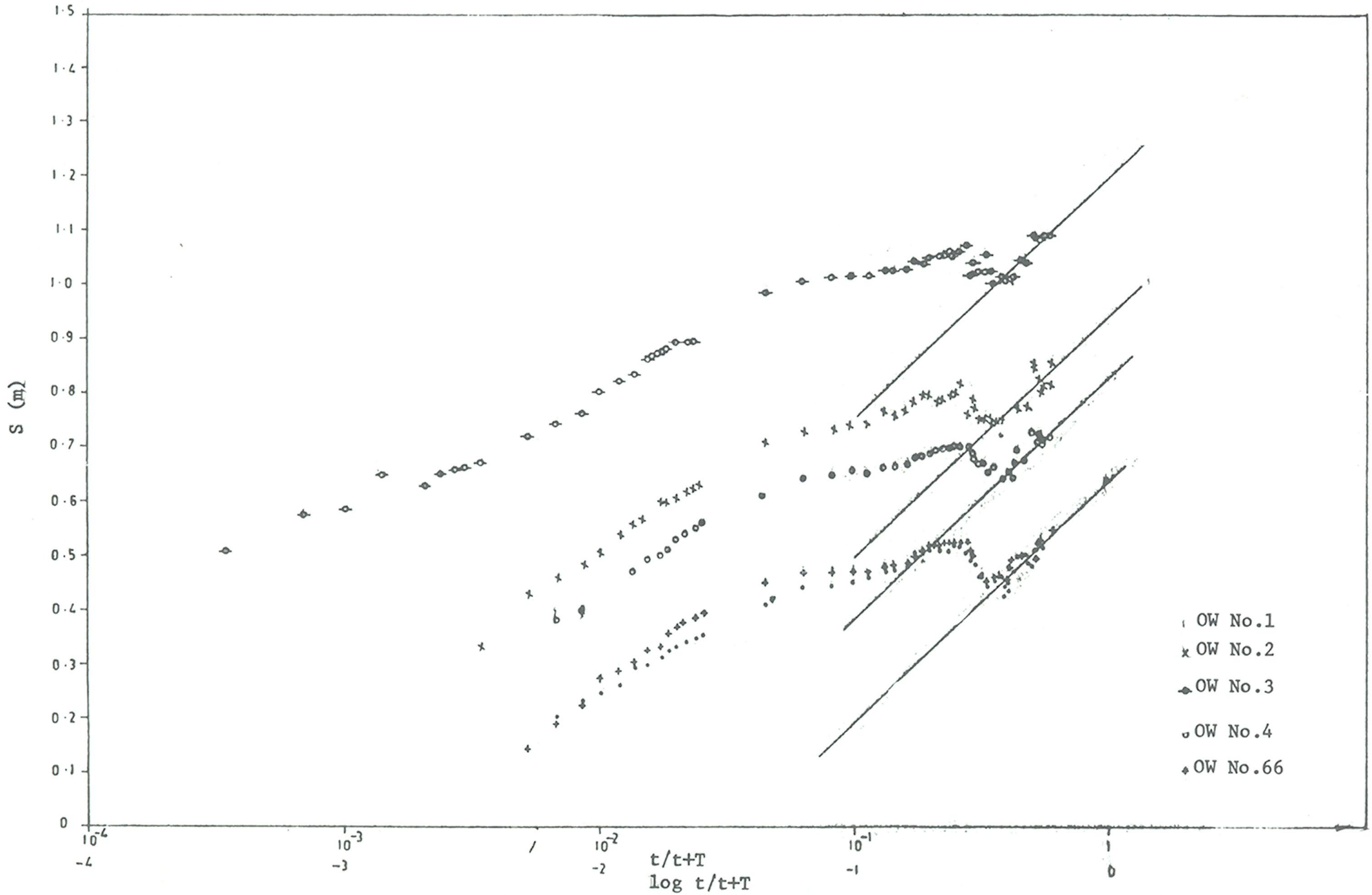


Table 17. Dabkheri: recovery ($S^*/\log \frac{t}{t+T}$)

Well	Q (m^3/d)	C	S_{max} (m)	Km (m^2/d)	a (m^2/d)
Test well	5 962	0.45	5.585	2 424	-
OW No. 1	--	0.45	0.535	2 424	7.7×10^4
OW No. 2	--	0.45	0.815	2 424	5.2×10^4
OW No. 3	--	0.45	1.1	2 424	5.3×10^4
OW No. 4	--	0.45	0.73	2 424	6.1×10^4
OW No. 66	--	0.45	0.545	2 424	6.1×10^4

For a control check, the value of (a) may be verified by determining the value of (μ), storavity, with the help of N. N. Bindeman's (1963) formula. According to this formula,

$$\mu = 0,824 \frac{Q \cdot t}{r_1^2 \left(S_1 - S_2\right)} \left(\frac{r_1}{r_2}\right)^{\frac{2 S_1}{S_1 - S_2}} \times \log \frac{r_2}{r_1}$$

Where

Q = tubewell discharge (m^3/d);

t = duration of pumping (days);

r_1, r_2 = distances between test well and any two observation wells (m); and

S_1, S_2 = drawdowns in observation wells 1 and 2 (m).

The results of the subsequent calculations are shown in table 19.

Table 18. Summary of hydrogoelogical parameters (m^2/d)

Well	(Km) Method of determination				(a) Method of determination			
	S/log t	S/log r	$S/\log\frac{t}{r^2}$	$S^*/\log\frac{t}{t+T}$	S/log t	S/log r	$S/\log\frac{t}{r^2}$	$S^*/\log\frac{t}{t+T}$
Test well	1 364	2 424	2 424	2 424	-	-	-	-
OW No. 1	2 424	2 424	2 424	2 424	1.96×10^5	a/	1.31×10^5	7.7×10^4
OW No. 2	2 424	2 424	2 424	2 424	1.6×10^5	a/	1.31×10^5	5.2×10^4
OW No. 3	2 479	2 424	2 424	2 424	1.73×10^5	a/	1.31×10^5	5.6×10^4
OW No. 4	2 372	2 424	2 424	2 424	1.6×10^5	a/	1.31×10^5	5.3×10^4
OW No. 66	2 424	2 424	2 424	2 424	1.48×10^5	a/	1.31×10^5	6.1×10^4
Accepted values all wells	Km = 2 424				$a = 1.31 \times 10^5$			

a/ Values range from 0.82 to 2.8×10^5.

Table 19. Dabkheri observation wells: storativity and conductivity (Bindeman formula)

Observation wells	Storativity (μ *)	Water-level conductivity (m^2/d)
No. 1 and No. 2	0.006	4×10^5
No. 3 and No. 4	0.0178	1.36×10^5
No. 3 and No. 66	0.022	1.1×10^5
No. 4 and No. 66	0.02	1.21×10^5

Thus, it is seen that the values indicated in figure 23 and table 16 (Km = 2,424 m^2/d and a = 1.31×10^5 m^2/d, which were obtained by plotting S against log t/r^2, are trustworthy.

As was shown earlier, the piezometric pressure in the aquifer, regionally, is about 2-19 m, with an average value of about 8.8 m. This pressure is maintained for a rather short time during intensive ground-water exploitation, which is the reason the aquifer works as an unconfined aquifer when subjected to a prolonged and intensive pumping regime. The actual value for water-level conductivity (a) should, accordingly, be lower than that obtained from the pumping test data.

The value of (a) for the unconfined aquifer can be determined with the help of the formula

$$a = \frac{Km}{\mu}$$

where

$$\mu = 0.117 \sqrt[7]{K} \quad \text{and}$$

$$K = \frac{Km}{m} = \frac{2{,}424}{72.5} = 33.4 \text{ m/d}$$

$$\mu = 0.117 \sqrt[7]{33.4} = 0.19$$

Thus

$$a = \frac{2{,}424}{0.19} = 1.27 \times 10^4 \text{ m}^2\text{/d.}$$

In addition, data for hydrogeological parameters of the aquifer for five HMITC tubewells are also available and are given in table 20.

Table 20. Narwana Branch area:
hydrogeological data from HMITC tubewells

	Hydrogeological Parameters				
Tubewell	Discharge (l/sec)	Drawdown (m)	Thickness of aquifer (m)	Transmis-vity (m^2/d)	Storavity (μ)
Khera (RD-176)	-	-	40.0	-	-
Thol (RD-209)	28.9	18.8	69.0	1 521	0.00266
Jansa (RD-223.5)	75.1	13.7	78.0	1 297	-
Darsi (RD-255L)	47.9	13.6	74.0	1 500	0.00156
Jyotisar (RD-287.75)	80.7	11.5	105.0	3 042	0.00209
Kirmach (RD-315)	80.7	13.5	119.0	1 834	0.0009

On the basis of these data and the data for Dabkheri, the average storage coefficient value is $\mu_{av} = 0.005$.

Thus, the average values for hydrogeological parameters along the entire 48 km length of the Narwana Branch are as follows:

Average transmissivity (Km) $= (1{,}521 + 1{,}297 + 1{,}500 + 3{,}042 + 1{,}834 + 2{,}424) \div 6$

$= 1{,}936 \ m^2/d$

Average aquifer thickness (m) $= 40 + 69 + 78 + 74 + 105 + 119 + 72.5) \div 6$

$= 79.6 \ m$

Average coefficient of permeability (K) $= \dfrac{Km}{m} = \dfrac{1{,}936}{79.6}$

$= 24.3 \ m/d$

Average specific yield (μ) $= 0.117 \sqrt[7]{24.3} = 0.184$

Average water-level conductivity (a) $= \dfrac{1,936}{0.184} = 1 \times 10^4\ m^2/d$

These accepted values will henceforth be employed for all further calculations.

D. Results of infiltration experiments

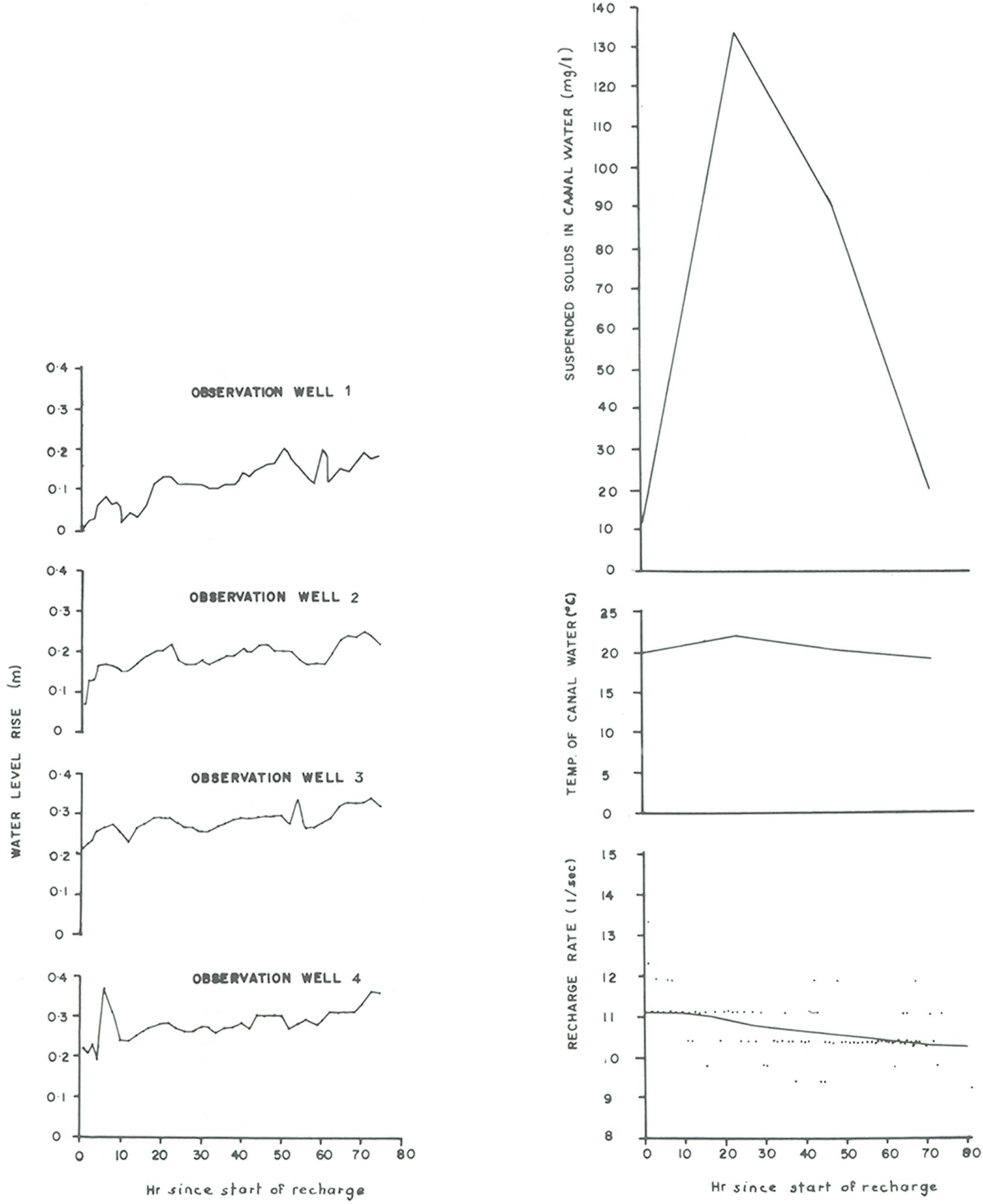
The artificial recharge experiment, by injection of canal water, was conducted in the first test well at the Dabkheri site. The experiment was conducted for about 415 minutes, including a non-injection period of about 45 minutes. The injection rate was 43.8 l/sec; injection pressure was 1 atm at the beginning of the experiment and rose to 1.6 atm after 10 minutes, 1.98 atm after 40 minutes and to 2 atm after 295 minutes, remaining at 2 atm until the end of the experiment. A rise in water level was recorded in the observation wells during the injection test with the maximum rise of 0.42 m being recorded in OW No. 3 (30 m from the test well) and the minimum rise of 0.15 m in HMITC tubewell No. 66 (125 m from the test well) at the end of the test (415 minutes of injection).

The experiment, despite its limitations, proved, first, that it is possible to carry out artificial recharge at a high injection rate (43.8 l/sec) in the confined aquifer in this area and, second, that it is possible to use canal water for this purpose.

In addition, the short-duration injection test showed that the aquifer reacts immediately to the injection of water and that the cone of impression is almost the mirror image of the cone of depression that would form were the well to be pumped at the same rate (43.8 l/sec), which shows that the well is capable of taking injection at a high rate, as it is capable of discharging at a high rate. Hydrological parameters can be determined from the injection test data, as they can from the pumping test data.

After the attempts to carry out the injection under pressure were abandoned, the gravity recharge method was tried. The recharge test by gravity lasted for 101 hr. The recharge rate was greater than 5 l/sec at the start of the test and decreased gradually to 3.5 l/sec by the end. A rise in water levels was recorded in the observation wells for the first 57 hr, at which time pumping was started in a nearby well and the observation wells started to show a decline in water levels. The maximum rise observed was 0.14 m in observation well No. 3 (30 m from the test well).

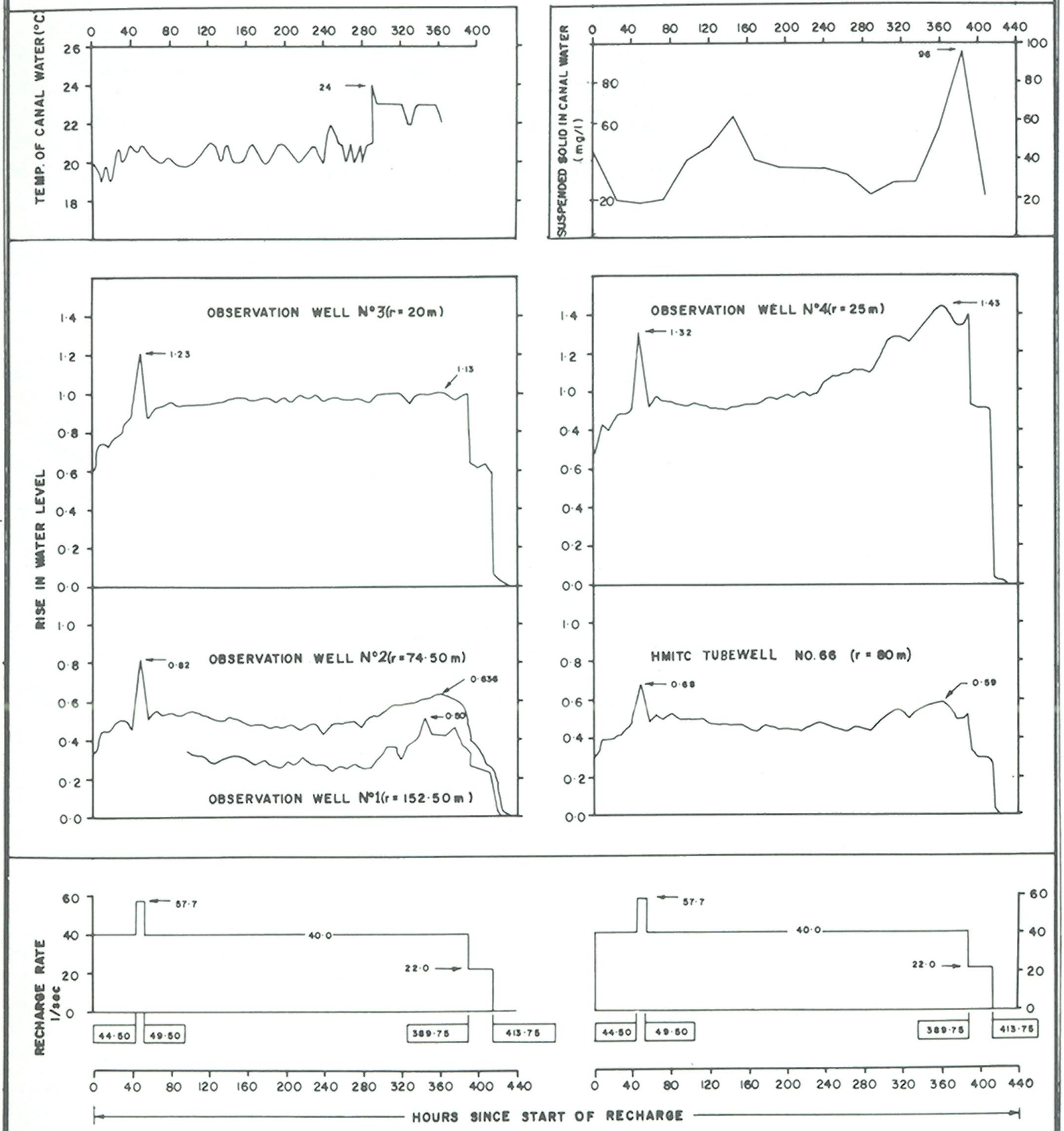
INDIA
DABKHERI TEST WELL II: RESULTS OF GRAVITY TEST



3237.14x

FIGURE 27

INDIA
DABKHERI WELL TEST II: RESULTS OF INJECTION TEST
(r = distance from test well II)



3237.15x

From these results it can be seen that the recharge rate obtainable with injection under pressure is about 10 times greater than the rate obtainable by gravity flow.

In order to achieve maximum results a second injection well (test well II) was constructed between observation wells Nos. 3 and 4. The new test well had its screen at 40 m bgl rather than at 15 m bgl as in the first well; special measures were also taken to ensure the efficiency of the cement seal above the gravel packing.

The recharge of water under gravity into test well II started at 0800 hours on 24 May 1978 and continued for 74 hr. The recharge rate varied from 9.4 to 12.3 l/sec. The maximum water-level rise in observation wells Nos. 1, 2, 3 and 4 (located at distances of 152.50, 74.50, 20.0 and 25.0 m, respectively from the test well) was 0.20, 0.25, 0.34 and 0.36 m, respectively (figure 26). The water level in the test well at the end of the test was about 1.70 m bgl. The suspended-solid content of the canal water during the test period ranged from 12 to 134 mg/l and the temperature of the canal water was between 19° and 20° C.

An injection test was begun on test well II at 1330 hours on 6 December 1978 and continued for 413.75 hr. The injection rate was about 40 l/sec. During the injection test the maximum rise in water levels in observation wells Nos. 1, 2, 3 and 4 and in HMITC tubewell No. 66 was 0.52, 0.64, 1.13, 1.43 and 0.57 m respectively (figure 28).

The water level in the test well during the injection period fluctuated between 0.52 and 2.50 m bgl.

The suspended-solid content in the canal water during the test varied from 18 to 96 mg/l; the temperature of the canal water ranged between 19° and 24° C.

E. Calculation of exploitable ground-water resources with artificial recharge

Since the aquifer functioned as a confined aquifer without hydraulic boundaries during the pumping test, the period of exploitation after which it should behave as an unconfined aquifer is determined, according to F. M. Bochever's formula (4) as follows:

$$S = \frac{Q\,\Sigma}{4\,\Pi\,Km} \ln \frac{2.25\,at}{R^{2}_{bw}} \qquad \text{Equation 6.1}$$

From the above,

$$\ln t = \frac{4 \Pi Km}{Q \Sigma} - \ln 2.25 - \ln a + \ln R^2_{bw} \qquad \underline{\text{Equation 6.2}}$$

Where

Km = average transmissivity (1,936 m^2/d);

S = average pressure head above the roof of the aquifer (8.8)m;

$Q \Sigma$ = total exploitable discharge of well-field (7,776,000 m^3/d);

a = average water-level conductivity for a confined aquifer; and

R_{bw} = radius of the "big well" (= 0.1 x length of well-field = 0.1 x 48,000 = 4,800 m).

Thus

$$\ln t = \frac{4 \times 3.14 \times 1{,}936 \times 8.8}{777{,}600} - 2.25$$

$$-\ln 387{,}200 + \ln 4{,}800^2$$

$$= 3.55$$

Therefore

$$t = 34.8.$$

This period, roughly 35 days, is very short indeed as compared to the 10,000-day period that is taken as the period of exploitation for calculation of reserves. The pressure head can therefore be ignored and the aquifer treated as an unconfined aquifer.

The calculation of maximum drawdown at the end of the 10,000 day exploitation period is carried out with the help of Musket's formula:

$$S = H - \sqrt{H^2 - \frac{Q\ red}{\Pi K}\left(\ln \frac{\lambda}{2 \Pi r_o} + \frac{3.55 \sqrt{at}}{\lambda}\right)} \qquad \underline{\text{Equation 6.3}}$$

where H = average thickness of aquifer (m);

K = average coefficient of permeability (m/d)

λ = distance between adjacent wells (m);

r_o = radius of well assembly intake section (m);

a = water-level conductivity (m^2/d);

t = period of exploitation (d); and

Q_{red} = "reduction" discharge of each well (m^3/d).

The value for (Q_{red}) is calculated as

$$Q_{red} = \frac{Q_d \times t_d - Q_{rech} \times t_{rech}}{365} \qquad \underline{\text{Equation 6.4}}$$

where

Q_d = discharge per well (m^3/d);

t_d = total discharge period per year (d);

Q_{rech} = injection recharge per well (m^3/d); and

t_{rech} = total recharge period per year (d).

The following parameter values have been adopted for calculations:

Q_d = 6,480 m^3/d (75/l/sec);

t_d = 275 d/y;

Q_{rech} = 5,184 m^3/d (601/sec);

t_{rech} = 90 d/y;

K = 24.3 m/d;

H = 79.6 m;

λ = 400 m;

r_o = 0.2 m;

a = 1×10^4 m^2/d; and

t = 10,000 days.

The calculations are based on a well-field of 120 wells at a distance of 400 m from one another to cover the full length of the canal, i.e. 48,000 m. As assumed above, all wells will be pumping 275 days per year, and during the 90 days of the rainy season injection of canal water will be effected under pressure into the same tubewells. Water draft per pumping day will be 120 x 6,480 = 777,600 m^3/d; recharge per day of injection will be 120 x 5,184 = 622,080 m^3/d, or 7.2 m^3/sec.

Annual reduction discharge of each well, per equation (6.4), is

$$Q_{red} = \frac{6,480 \times 275 - 5,184 \times 90}{365}$$

$$= 3,604 \ m^3/d$$

Maximum drawdown in the centre of the well-field, after 10,000 days of exploitation, as per formula (6.3) is now given as

Equation 6.5

$$S = 79.6 \sqrt{79.6^2 - \frac{3,604}{3.14 \times 24.3}\left[\ln \frac{400}{2 \times 3.14 \times 0.2} + \frac{3.55\sqrt{10^4 \times 10^4}}{400}\right]}$$

$$= 36.3 \ m$$

Maximum permissible drawdown (S_{mp}) is given as

$$S_{mp} = 0.7H = 0.7 \times 79.6 = 55.7 \ m$$

Radius of influence of the well-field (R) is then

$$R = 1.5 \sqrt{at} = 1.5\sqrt{10^4 \times 10^4} = 15,000 \ m$$ Equation 6.6

The necessary pressure (P) for injection of canal water through the well (as determined by Musket and Leybenson) is

$$P = \frac{0.366 \ Q_{rech} \gamma}{Km}\left[\log \frac{\lambda}{2 \cdot \Pi \ r_o} + \frac{1.54\sqrt{at_{rech}}}{\lambda}\right]$$ Equation 6.7

where

γr = volumetric weight of the water being injected.

Therefore,

$$P = \frac{0.366 \times 5,184 \times 1.00177}{1,936}\left[\log \frac{400}{2 \times 3.14 \times 0.2} + \frac{1.54\sqrt{10^4 \times 90}}{400}\right]$$

$$= 6 \ atm.$$

F. Conclusions and recommendations

1. Artificial recharge of the confined aquifer along the Narwana Branch canal is possible by means of injection of water under pressure through tubewells.

2. The confined aquifer is capable of accepting large quantities of the canal water. The control of sediment load in the canal water is, however, absolutely essential to prevent clogging of the aquifer. There should be practically no suspended sediment in the water used for injection.

3. The calculations above show that it is possible to pump 777,600 m^3/d (9.0 m^3/sec) of water from 120 tubewells along the 48-km length of the canal for the 275 days of the dry season. During the remaining 90 days of the year, i.e. during the rainy season, injection of water could be undertaken through the same 120 wells at a total rate (for all 120 wells) of 622,080 m^3/d (7.2 m^3/sec). In this case drawdown in the centre of the well-field at the end of 10,000 days (about 27 years) of exploitation will be 36.3 m, or 45.6 per cent of the average thickness of the aquifer.

If the recharge of water through the same 120 wells is to be done under gravity only (without pressure), it will be possible to have a total recharge of about 72,000 m^3/d (0.83 m^3/sec). In this case, drawdown in the centre of the well-field will be 57.8 m or slightly greater than the permissible maximum (55.7 m).

At present, the annual ground-water draft rate is about 225,000 m^3/d (about 2.6 m^3/sec), or only about 30 per cent of the discharge calculated.

4. Exploitation of ground water must be from 120 wells, evenly distributed along the canal at a distance of about 400 m from one another. The depth of the wells should be between 120 and 140 m to a full penetration of the aquifer. The annular space, above the gravel packed area, should be cemented with a special "tamponage" cement down to about 55-60 m, i.e. almost to the top of the intake section of the casing assembly. The intake section of the assembly should be about 400 mm (16") in diameter and must have an open area of not less than 20 per cent. The housing assembly should be able to accomodate either a single double-action pump (for pumping and injection), or a separate pump for each function capable of delivering about 6,480 m^3/d (75 l/sec) discharge and about 5,184 m^3/d (60 l/sec) recharge under pressure.

5. Water from the Narwana Branch canal may be used for injection but a daily quality-control regime should be maintained. The following criteria should be observed: (1) injection water must be free from suspended matter (litter, water plants etc.); (2) sediment load should be no greater than 2 mg/l; if it is any greater, repeated development and cleaning of the well will be required; (3) injection water should be free of air bubbles; and (4) the quantity of toxic substances in the water must be below the permissible limit for drinking water; using polluted water for injection will lead to pollution and clogging of the aquifer.

6. Pressure build-up from injection will be about 6 atm at the end of 90 days, if the water is clean. Constant control of pressure should be maintained during injection. A rise in pressure beyond 6 atm, or a sudden rise at any time during injection, will indicate clogging and a need for development and cleaning of the well.

7. Observation wells should be constructed and maintained at space intervals to keep a check on the development of the cone of depression and the cone of impression, i.e. to observe the areal extent of influence and magnitude of the discharge and artificial-recharge operations.

VII. PROJECT RESULTS: GHAGGAR RIVER WELL-FIELD, TATIANA SITE

A. General

The main purpose of the hydrogeological investigations at the Tatiana site was to study the possibility of creating induced recharge from the Ghaggar River into the alluvial (unconfined) aquifer in order to increase ground-water extraction during the dry season. If carried out along the entire length of the river, such recharge would also, indirectly, decrease the intensity of flooding in the river.

The test site is situated on the left bank of the Ghaggar River, on its first terrace, near the village of Tatiana (figures 1, 28).

The altitude of the land surface at this site ranges from 238.5 to 239.4 m above msl. The water table, which is dependent on the river, fluctuates between 232.8 and 232.6 m above msl. The riverbed itself is about 6 m below the general level of the land surface, its left bank being steep while the right one slopes moderately. The Ghaggar is a perennial stream in this area, its bed being about 15-20 m wide in the vicinity of the Tatiana site. The discharge, about 0.05 m^3/sec, occurs during April-May (1977 measurement); the maximum, about 343.6 m^3/sec, is in August (1976). During the periods of heavy rainfall and consequent floods, the first terrace, on which the test site is located, is covered for a short time by flood water, making this a very favourable situation for recharge of the aquifer and thereby for the replenishment of ground-water resources.

B. Hydrogeological conditions

Exploratory drilling revealed the presence of three aquifer zones, the first within 26 m of the land surface. The second from 150 to 175 m bgl and the third from 210 to 225 m bgl. Only the first aquifer is in direct hydraulic contact with the surface water.

The first, uppermost aquifer, which was chosen for study, comprises fine- to medium-grained sand with kankar and kankar and quartzite gravel. A thin layer of sandy clay is present between 17.0 and 17.8 m bgl. The bottom of the aquifer is sandy loam, about 6.8 m thick overlying a thick layer of clay (figure 28). The aquifer is also overlain by a layer of sandy loam, about 2.8 m thick. The aquifer is in excess of 4 km in width on the left bank of the river alone.

The ground water occurs in the aquifer under water-table conditions; depth to water level being about 6 m. The hydraulic gradient is towards the river during the dry season. During the rainy season the gradient is reversed, as the water level in the river rises and the aquifer obtains replenishment/recharge from the river.

The thickness of the aquifer is about 18 m. Its filtration properties are good: coefficient of permeability is 22.2 m/d, transmissivity about 400 m^2/d, specific yield (μ) about 0.18 and water-level conductivity or hydraulic diffusivity about 2,253 m^2/d. The test well was pumped at a discharge rate of 20 l/sec during the pumping test, causing a maximum drawdown of 2.63 m in the piezometer at a distance of 1.5 m from the test well.

The resistance of the riverbed (Δ L) determines the degree of hydraulic connexion between the ground-water body and the surface water via the river bed. The bed of the Ghaggar is fine sand, partially clogged by particles of clay, which prevent complete interconnexion; Δ L = 94 m.

Groundwater at the site is used for water supply and irrigation through numerous tubewells and dug wells.

C. Determination of hydrogeological parameters

One test well and four observation wells were constructed for the determination of the aquifer parameters, including the thickness of aquifer (H), coefficient of permeability (K), transmissivity (Km), water-level conductivity (a), specific yield (μ) and hydraulic resistance of the riverbed (Δ L).

The distance between the test well and the water-stream in the river was 76 m. Observation wells Nos. 1 and 3 were constructed along a line perpendicular to the river at 20 m and 50 m, respectively, from the test well. Observation wells Nos. 2 and 4 were placed on a line parallel to the river, also 20 m and 50 m, respectively, from the test well.

Observation wells Nos. 1 and 3 were used for the determination of the hydraulic resistance of the riverbed, whereas Nos. 2 and 4 were used for the determination of the other aquifer parameters.

Processing of the pumping test data was done according to the Jacob method (15), plotting S against log t, S against log r, S against log t/r^2 and S* against log t/t + T), as already described for the pumping tests at Dabkheri (table 13).

1. Pumping test analysis (S/log t)

The control time for the aquifer to reach the quasi-stationary regime of filtration (Jacob criterion), was determined for each well with the help of the formula

$t_k = \frac{r^2}{0.4a}$. The results, in minutes were as follows:

Test well	OW No. 1	OW No. 2	OW No.3	OW No. 4
0.036	639	639	3,995	3,995

As seen from the curves in figures 29 and 30, representative parts of the graph are rather long. At the same time the plots for observation wells Nos. 1 and 3 cannot be used for the calculation of hydrogeological parameters, as their water levels are influenced differently by the river; OW No.3 is nearer the river, and therefore is affected more than OW No. 1, which is farther away. The values for (Km) and (a), if calculated from these wells would therefore be too high. Values for (Km) and (a) as determined from the test-well data (632 m^2/d and 3.8 x 10^4 m^2/d, respectively) also cannot be used on account of the influence of well losses and the "skin effect".

Observation wells Nos. 2 and 4, on the other hand, give reliable values for (Km) and (a), both wells being equally distant from (and therefore equally influenced by) the river. The values for (Km) and (a), as determined from all the wells are summarized in table 21.

Table 21. Tatiana: results of pumping test (S/log t)

Well	Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
Test well	1 728	0.5	0.71	632	3.8 x 10^4
OW No. 2	-	0.81	1.67	390	2.23 x 10^3
OW No. 4	-	0.79	2.22	400	2.49 x 10^3
OW No. 1	-	0.75	1.45	422	3.1 x 10^3
OW No. 3	-	0.7	1.9	452	3.1 x 10^3

Figure 29. Tatiana test wells: pumping test (S/log t)

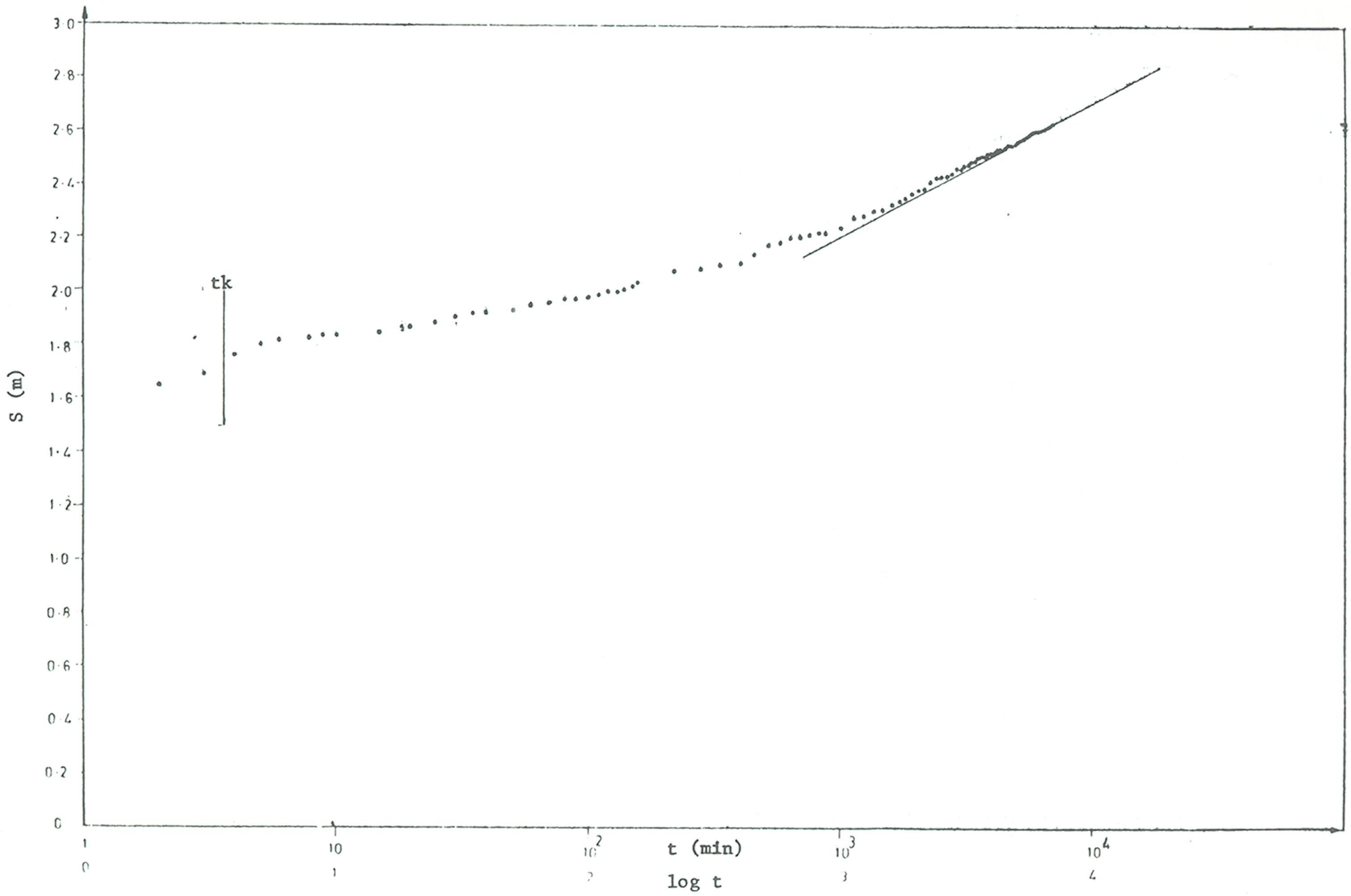
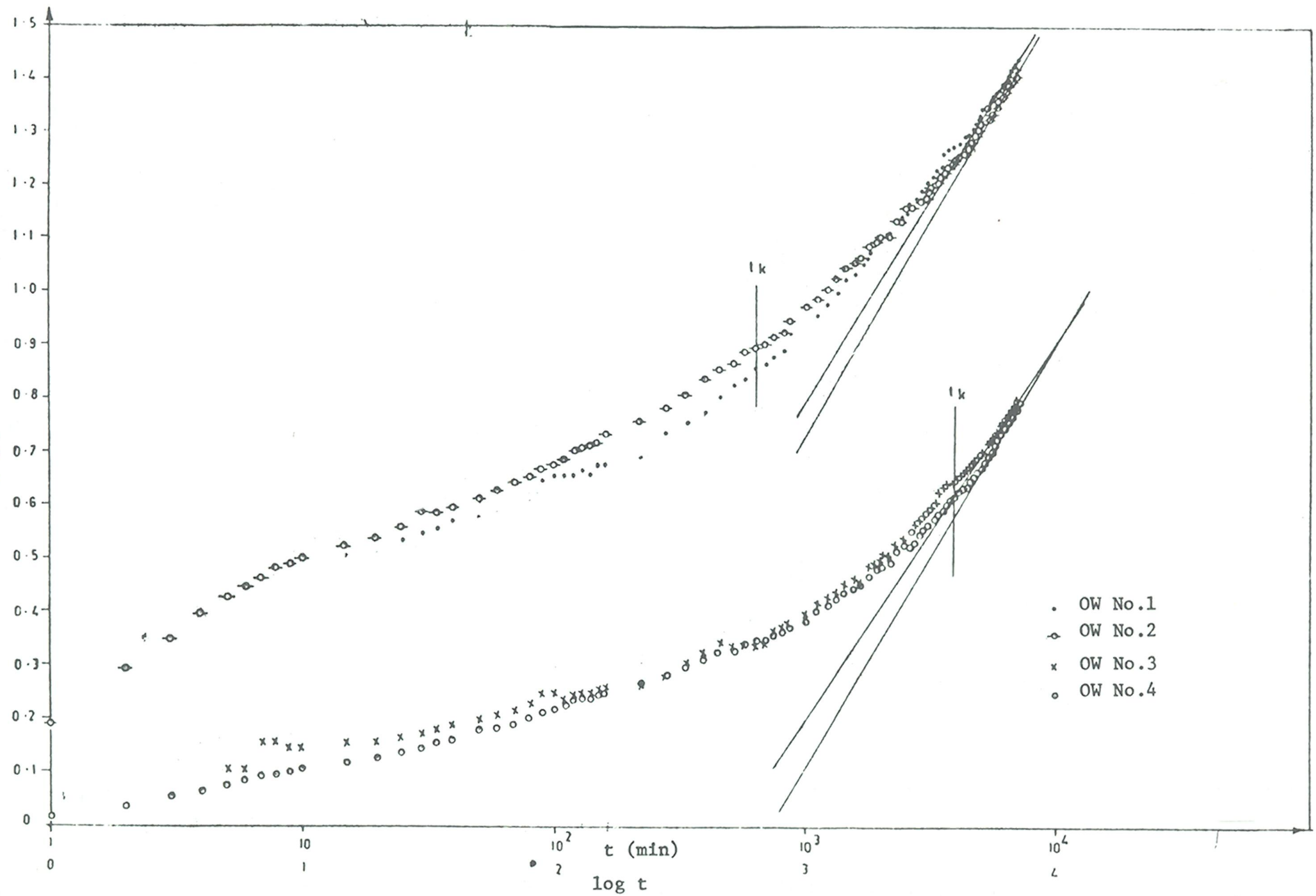


Figure 30. Tatiana observation wells: pumping test (S/log t)



2. Pumping test analysis (S/log r)

Drawdown (S) was plotted against the log of the dis-distance of the observation well from the test well (r) for two periods of time: 5,080 and 7,000 minutes, i.e. for the periods of quasi-stationary regime.

Figure 31 shows that the points for observation wells Nos. 1 and 3 do not fall on the same line as these for OWs Nos. 2 and 4, again showing the variable influence of the positive boundary (the river) on wells 1 and 3. Only the graphs based on data from wells Nos. 2 and 4 have been used for the calculation of (Km) and (a). The results are summarized in table 22.

Table 22. Tatiana: results of pumping test (S/log r)

Minutes since start of pumping	Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
5 080	1 728	1.57	3.32	403	2.15×10^3

3. Pumping test analysis (S/log $\frac{t}{r^2}$)

The end parts of the graphs plotted for S against log t/r^2 for OW No. 2 and OW No. 4 (figure 32) converge to one straight line. This would mean that the quasi-stationary regime has already been reached. For all practical purposes, the graphs for OWs Nos. 1 and 3 also converge to the same straight line, except for the few deflections that, again indicate the unequal influence of the positive boundary on these wells. The values obtained by this method are summarized in table 23.

Table 23. Tatiana: results of pumping test (S/log $\frac{t}{r^2}$)

Q (m^3/d)	C	A	Km (m^2/d)	a (m^2/d)
1 728	0.79	0/43	400	2.25×10^3

Figure 31. Tatiana observation wells: pumping test (S/log r)

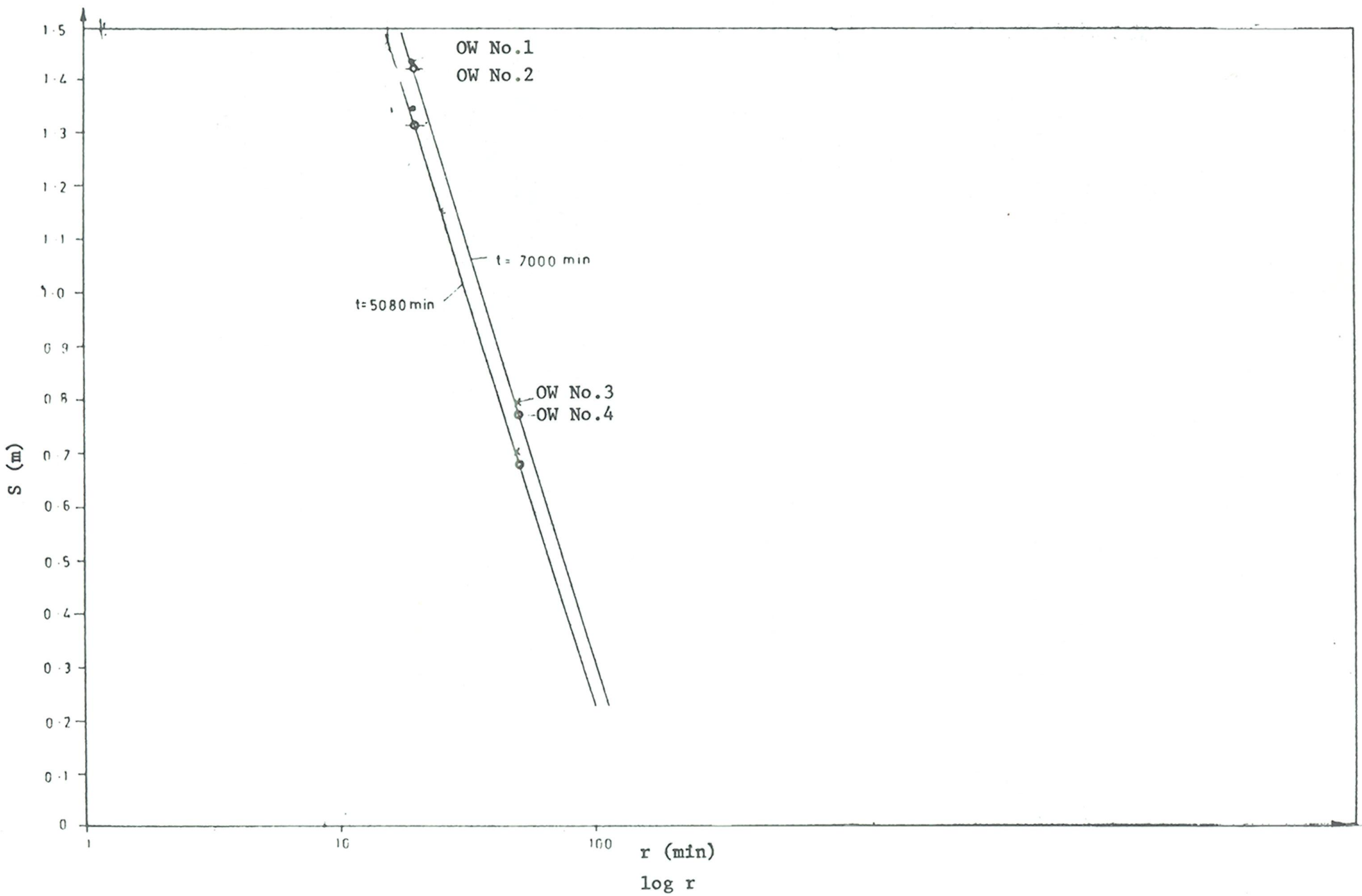
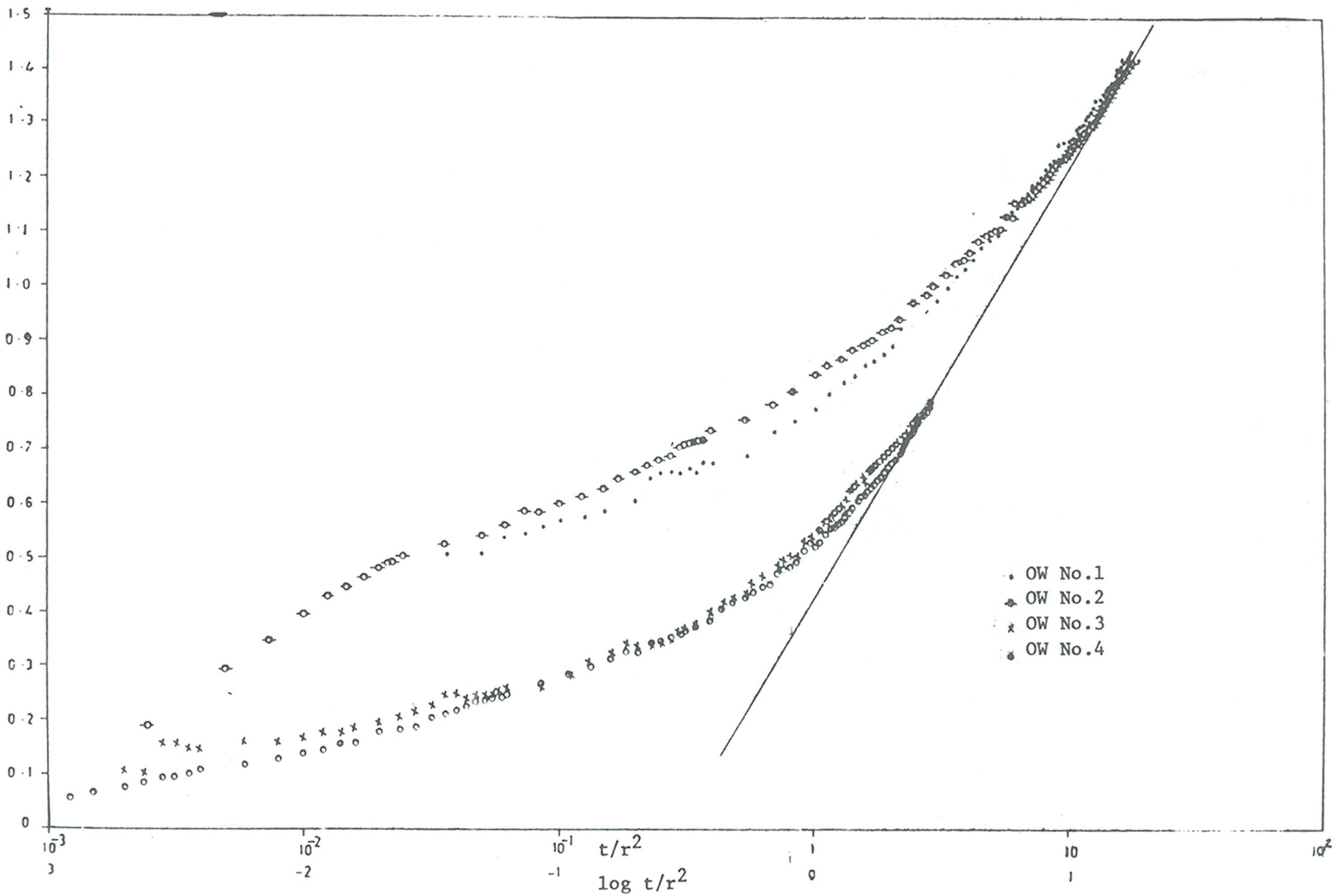


Figure 32. Tatiana observation wells: pumping test $(S/\log\frac{t}{r^2})$



4. <u>Pumping test analysis:</u> recovery ($S^*/\log \frac{t}{t + T}$)

The duration of recovery (t) after pumping was stopped was more than 0.1 of the duration of pumping ($t > 0.1T$), while the maximum drawdown (for the piezometer near the test well) was less than 20 per cent of the total thickness of the aquifer ($S < 0.2H$). Accordingly, the graphs have been plotted for coordinates S^* against $\log t/t + T$ for the test well and all the observation wells with no modification of the data (figures 33 and 34).

The representative part of the graph in case of the test well is seen towards the end of the recovery period. Because of the prolonged period of recovery, the observation wells show the influence of the positive boundary (the river) in addition to that of the actual regime.

The results of the evaluation of the recovery data are given in table 24.

Table 24. Tatiana: recovery ($S^*/\log \frac{t}{t + T}$)

Well	Q (m^3/d)	C	A (m)	Km (m^2/d)	a (m^2/d)
Test well	1 728	0.79	2.63	400	0.4×10^3
OW No. 2	--	0.81	1.41	390	1.97×10^3
OW No. 4	--	0.58	0.782	545	4.98×10^3
OW No. 1	--	0.75	1.44	422	2.97×10^3
OW No. 3	--	0.41	0.794	771	19.3×10^3

A summary of the values of the parameters, as found by different methods, is given in table 25.

Figure 33. Tatiana test well: recovery ($S^*/\log \frac{t}{t + T}$)

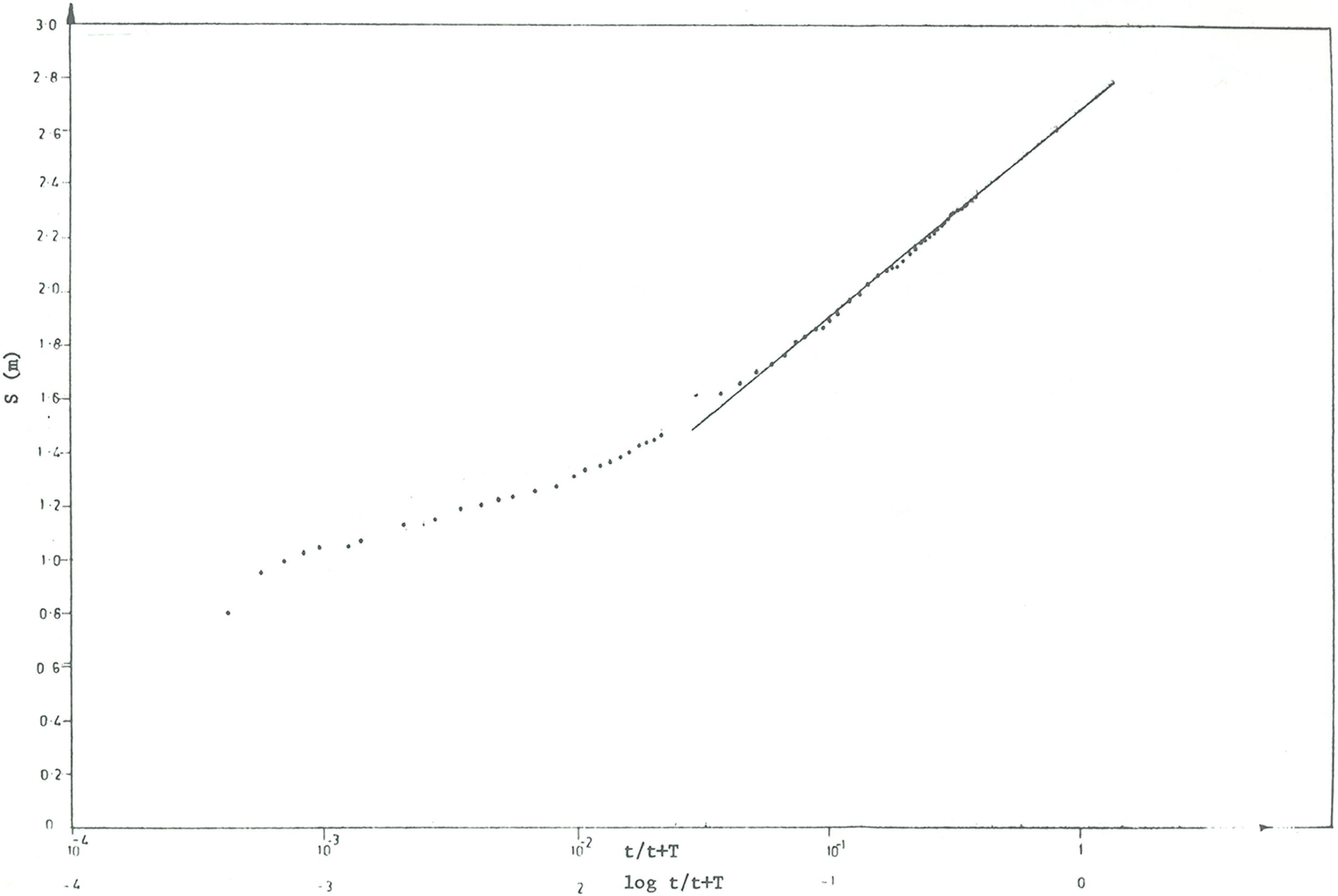


Figure 34. Tatiana observation wells: recovery ($S^*/\log \frac{t}{t+T}$)

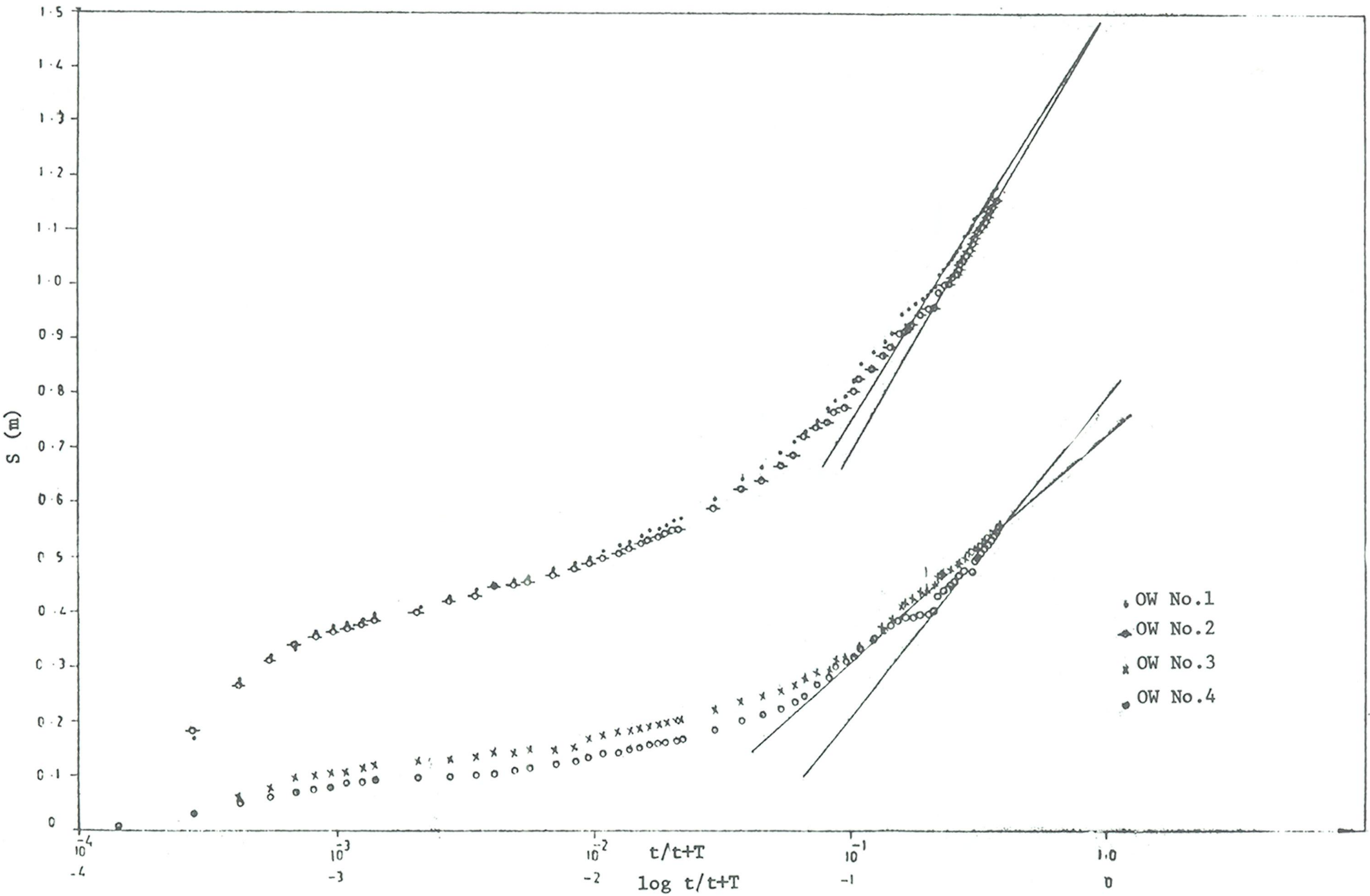


Table 25. Tatiana: summary of hydrogeological parameters (m^2/d)

Well	(Km) Method of determination				(a) Method of determination			
	$S/\log t$	$S/\log r$	$S/\log \frac{t}{r^2}$	$S^*/\log \frac{t}{t+T}$	$S/\log t$	$S/\log r$	$S/\log \frac{t}{r^2}$	$S^*/\log \frac{t}{t+T}$
Test well	632	-	-	400	3.8×10^3	-	-	0.4×10^3
OW No. 2	390	-	-	390	2.23×10^3	2.15×10^3	2.25×10^3	1.97×10^3
OW No. 4	400	403	400	545	2.49×10^3	2.42×10^3	2.25×10^3	4.98×10^3
OW No. 1	432	-	-	422	3×10^3	-	2.25×10^3	2.97×10^3
OW No. 3	452	-	-	771	3.1×10^3	-	2.25×10^3	19.3×10^3
Accepted value, all wells	Km = 400				$a = 2.25 \times 10^3$			

Thus, for further calculations, the accepted values for (Km) and (a) are those derived on the basis of the graph obtained by plotting S against log t/r^2.

The thickness of the aquifer is 18 m; accordingly, the coefficient of permeability (K) is 400/18 = 22.2 m/d. The coefficient of permeability can be control-checked by the E. L. Minkin formula (20), which states

$$K = \frac{Q \ln {}^{r_4}/r_2}{\Pi S_2 (2\,H - S_2) - \Pi S_4 (2H - S_4)}$$ <u>Equation 7.1</u>

where

Q = discharge of test well (m^3/d);

r_2 and r_4 = respective distances (m) between the test well and OWs Nos. 2 and 4;

S_2 and S_4 = respective drawdowns in observation wells Nos. 2 and 4 (m); and

H = thickness of aquifer (m).

In the present case, the value of (K), as per the above formula, will be

$$K = \frac{1{,}728 \times \ln 50/20}{3.14 \times 1.411\ (2 \times 18 - 1.411) - 3.14 \times 0.782\ (248 - 0.782)}$$

$$= 23.7 \text{ m/d}$$

Specific yield (μ) is given as

$$\frac{Km}{a} = \frac{400}{2.25 \times 10^3} = 0.18$$

The same value for specific yield is obtained using the P. A. Betsinskyi formula:

$$\mu = 0.117 \sqrt[7]{K} = 0.117 \sqrt[7]{22.2} = 0.18$$

5. <u>Hydraulic resistance (Δ L)</u>

Stabilization of water levels could not be attained in the observation wells during the pumping test, because the very high well losses prevented a very high drawdown being reached in the aquifer. All available formulas for determination of Δ L are based on steady flow, which could not be attained in the present case; it was not possible, therefore, to calculate Δ L on that basis. However, because the water levels were stable before the pumping test started, Δ L could be calculated according to the formula of V. M. Shestakov (25):

$$L = \frac{H_3 - H_r}{H_1 - H_3} (X_1 - X_3) - X_3$$

where

H_1, H_3, H_r = water levels (m above msl) in OW No. 1, OW No. 2 and the river respectively; and

X_1 and X_3 = respective distances from the water in the river (m) to OW No. 1 and OW No.3.

According to this formula,

$$\Delta L = \frac{232.907 - 232.835}{232.925 - 232.907} (56 - 26) - 26 = 94 \text{ m}$$

This value of Δ L (94 m) shows a rather high degree of clogging of the riverbed.

D. Calculation of exploitable ground-water resources

The aquifer of the Ghaggar River well-field has one positive boundary: the river. Intercommunication between the surface and ground water is a little difficult because of the clogging of the riverbed by clay particles. The second (negative) boundary is rather far away, at a distance of about 4 km. The optimum disposition of the well-field will be along the river. It is assumed that the river has a permanent flow, which will always be greater than the discharge from the future well-field.

Because the velocity of water from the river to the wells in the well-field will be rather high, there is some risk of clogging of the aquifer by clay particles, especially during the rainy season. For this reason the wells should be located at a safe distance - not less than 100 m - from the river.

The well-field will consist of a number of wells constructed in a line parallel to the river with a distance of 200 m between adjacent wells; all wells will be pumped at the same rate of discharge, approximately 20 l/sec (1,728 m^3/d).

Under these circumstances, the drawdown in the centre of the well-field will be given by the Musket and Leybenson formula (1.4):

$$S = H - \sqrt{H^2 - \frac{Q}{\Pi K}\left[\ln \frac{\lambda}{2\ \Pi\ r_o} + \frac{2\ \Pi\ L}{\lambda}\right]} \qquad \underline{\text{Equation 7.2}}$$

where H = thickness of aquifer (m);

Q = discharge per well (m^3/d);

K = coefficient of permeability (m/d);

λ = distance between wells (m);

L = $L_o + \Delta L$, in which L_o is the distance between well-field and riverbed (m), L is the hydraulic resistance of the river bed (m) and

r_o = the radius of the intake section of the well (m).

Calculations of water resources have now been carried out, both under natural conditions and under conditions of artificial recharge measures.

1. Exploitable ground-water resources under natural conditions

The maximum permissible drawdown in the aquifer (S_{max}) equal to 0.7 H, with the hydrogeological parameters as follows:

H = 18 m; Q = 1,728 m^3/d; K = 22.2 m/d; λ = 200 m; L_o = 100 m; ΔL = 94 m; and r_o = 0.2 m. Since S_{max} = 0.7 x 18 = 12.6 m, the drawdown (S) in the centre of the well-field works out to

$$S = 18 - \sqrt{18^2 - \frac{1,728}{3.14 \times 22.2}\left[\ln \frac{200}{2 \times 3.14 \times 0.2} + \frac{2 \times 3.14 \times 194}{200}\right]}$$

$$= 11.1 \text{ m}$$

Thus, with the conditions of construction of well-field and the discharge per well being as above, the maximum drawdown in the centre of the well-field (11.1 m) is still within the limits of the permissible drawdown (12.6 m). This is possible even without artificial recharge measures.

In order to calculate when the river water will first reach the well in the well-field, E. L. Minkin's formula (20) is applied:

$$T_o = \frac{\mu . H . L}{q}\left[\frac{\sqrt{\frac{Q}{\Pi L q}}}{\sqrt{\frac{Q}{\Pi L q} - 1}}\left(\text{arctg}\,\frac{1}{\sqrt{\frac{Q}{\Pi L q} - 1}} - \text{arctg}\,\frac{1 - \frac{L_o}{L}}{\sqrt{\frac{Q}{\Pi L q} - 1}}\right) - \frac{L_o}{L}\right]$$

Equation 7.3

Where T_o = time when the river water first reaches the well (days);

μ = specific yield;

H = thickness of aquifer (m);

L = $L_o + \Delta L$, in which L_o is the distance between well-field and riverbed (stream-front) and ΔL is the hydraulic resistance of the river-bed (both in m);

q = K x I x H, where I is the hydraulic gradient;

K = coefficient of permeability (m/d); and

Q = discharge per well (m^3/d).

The values of all parameters are known except for I, which, on the basis of the static water levels in OW No. 1 and OW No. 3, works out at I = 0.006.

Therefore

$$T_o = \frac{0.18 \times 0.18 \times 194}{2.4}\left[\frac{\sqrt{\frac{1{,}728}{3.14 \times 194 \times 2.4}}}{\sqrt{\frac{1{,}728}{3.14 \times 194 \times 2.4} - 1}}\left(\text{arctg}\,\frac{1}{\sqrt{\frac{1{,}728}{3.14 \times 194 \times 2.4} - 1}} - \text{arctg}\,\frac{1 - \frac{100}{194}}{\sqrt{\frac{1{,}728}{3.14 \times 194 \times 2.4} - 1}}\right) - \frac{100}{194}\right]$$

= 94 days

That is to say, water from the river will reach the wells for the first time only after 94 days of exploitation of the well-field.

With increased duration of well-field exploitation, the discharge in the river will be affected to some optimum level and will be diverted to replenish the aquifer to the extent to which water has been withdrawn from it. The quantity of water that will thus flow to the aquifer is given by the Hantush formula (12):

$$Q_{riv} = Q\,\mathrm{erfc}\left(\frac{L_o}{2\sqrt{at}}\right) \qquad \underline{\text{Equation 7.4}}$$

Where Q = discharge per well (m^3/d),

L_o = distance between well-field and river (m),

a = water-level conductivity (m^2/d), and

t = period of exploitation of the well-field (days),

with Q = 1,728 m^3/d, L_o = 100 m, $a = 2.25 \times 10^3$ m^2/d and t = 10,000 days.

$$\text{Thus } Q_{riv} = 1{,}728\ \mathrm{erfc}\left(\frac{100}{2\sqrt{2.25 \times 10^3 \times 10{,}000}}\right)$$

= 1,689 m^3/d or 97.7 per cent of the discharge per well.

2. <u>Exploitable ground-water resources with artificial recharge</u>

With even the simplest artificial recharge measures, such as periodically scraping the clay film from the riverbed so as to remove the upper (clogged) part of the aquifer and thereby decrease the hydraulic resistance, the discharge per well can be increased with the same drawdown as in the first case.

Thus with Q = 2,333 m^3/d (27 l/sec) rather than 1,728 m^3/d (201 l/sec), as earlier, and with $\Delta L = 0$ and the other parameters being the same as before, the drawdown in the centre of the well-field (equation 7.1) works out as follows:

$$S = 18 - \sqrt{18^2 - \frac{2{,}333}{3.14 \times 22.2}\left(\ln\frac{200}{2 \times 3.14 \times 0.2} + \frac{2 \times 3.14 \times 100}{200}\right)}$$

$$= 11 \text{ m}$$

The increase in discharge under these new conditions is therefore about 35 per cent. Under these conditions (with $L = 0$ the time for the first portion of the river water to reach the wells, according to equation (7.3) is

$$T_o = \frac{0.18 \times 18 \times 100}{2.4}\left[\frac{\dfrac{2{,}333}{3.14 \times 100 \times 2.4}}{\sqrt{\dfrac{2{,}333}{3.14 \times 100 \times 2.4} - 1}}\left\{\text{arctg}\,\frac{1}{\sqrt{\dfrac{2{,}333}{3.14 \times 100 \times 2.4} - 1}} - \text{arctg}\,\frac{1 - \dfrac{100}{100}}{\sqrt{\dfrac{2{,}333}{3.14 \times 100 \times 2.4} - 1}}\right\} - \frac{100}{100}\right] = 39.6 \text{ days}$$

or approximately 40 days. The contribution of the river to the discharge obtained from the aquifer under exploitation after 10,000 days (equation 7.4) is

$$Q_{riv} = Q\ 2{,}333\ \text{erfc}\left(\frac{100}{2\sqrt{2.25 \times 10^3 \times 10{,}000}}\right)$$

$$= 2{,}307\ m^3/d$$

or 98.9 per cent of the total discharge per well.

E. Conclusions and recommendations

1. The studies conducted in this area of the Ghaggar River well-field have revealed that the alluvial aquifer extends from almost the ground surface (zone of aeration) down to about 25 m below ground level. The aquifer occupies a large area and is connected hydraulically with the river water.

The aquifer has the following parameters: thickness, 18 m; transmissivity, 400 m^2/d; water-level conductivity, 2.25×10^3 m^2/d; specific yield, 0.18.

Intercommunication between the aquifer and the surface water is obstructed by clogging of the riverbed, hydraulic resistance (ΔL) of the riverbed deposits being 94 m.

Under natural conditions, therefore, it is possible to have a discharge of 1,728 m^3/d per tubewell with a maximum drawdown, in the centre of the field, of about 11 m, well below the maximum permissible drawdown figure for this aquifer of 12.6 m.

2. It is possible, however, to draw more discharge from each of the wells in the field by practising some artificial recharge measures. The most effective and economical measures will be to periodically scrape off the clay film and the clogged upper part of the aquifer in the riverbed, as well as widening and levelling the riverbed. Adoption of this measure alone will make it unnecessary to use only clean water for recharge in this part of the river, even during the rainy season.

Scraping away the clogged portion of the aquifer beneath the river would considerably reduce the hydraulic resistance offered by the riverbed and, consequently, the specific capacity of the individual wells would be increased. Moreover, the widening and levelling of the riverbed would also increase the area of recharge.

This measure should be put into practice, however, only along the well-field, preferably after the floods or at such other time as the clay film is formed.

3. The well-field should be parallel to the river on each bank, at a distance of not less than 100 m from the river. The distance between the individual wells should be 200 m. Under these conditions, and with hydraulic resistance being nil due to artificial recharge measures, wells can be planned for this area with a discharge of 2,333 m^3/d and with a drawdown of not more than about 11 m.

The diameter of the intake section of the well assemblies should not be less than 0.4 m (16"), and the total tubewells depth should be great enough to house the pumps below the base of the aquifer.

4. Further exploration is absolutely essential to define the area of the well-field in which similar induced recharge can be carried out. The exploration may be conducted in two stages, preliminary and detailed. Investigations at the preliminary level should involve geophysical surveys for selection of sites for detailed hydrogeological investigation. Resistivity profiles should be made across the river at intervals of about one km and with a penetration not less than 50 m.

The detailed hydrogeological investigations should cover all aquifer parameters, including the riverbed resistance, etc. which should be determined by constructing test wells and observation wells and carrying out pumping tests. The studies should also cover the hydrogeological characteristics of the river and the chemical quality of both ground and surface waters. Special attention must be given to the process of clogging of the riverbed and to the evaluation of hydraulic resistance of the riverbed deposits.

VIII. PROJECT RESULTS: OTHER AREAS

A. Areas of old river courses

Old river courses cover a considerable area of the central and eastern parts of the Project area (figure 1). The eastern part of the Project area is traversed by old courses of the Ghaggar-Sarswati-Jamuna river system, while in the central part of the Project area there is an old course of the Sutlej river. Both these areas are characterized by the presence of good permeable deposits extending downwards from the surface. As a rule, the aquifers in these areas are unconfined, of great thickness and with good hydrogeological parameters. These areas thus have great potential for ground-water development and are also suitable for application of artificial recharge methods.

The most suitable method for development of ground water in these old river-course areas is to construct well-fields in which a number of wells are concentrated, rather than to dispose the wells at great distances from one another. This pattern of well-field development is suggested so that replenishment of ground water by artificial recharge can be done most economically, by using surplus surface water during the monsoon season.

Before any specific recommendation as to artificial recharge can be made for these areas, it will be necessary to study the hydrogeological conditions of the well-field areas in detail, to estimate the exploitable ground-water resources both with and without artificial recharge, the quality and quantity of water available for recharge and the source of such water. It is also important to know the present ground-water draft and the future requirements, so that the required quantum of recharge can be planned. Artificial recharge can be carried out in these areas by both the basin method and the induced recharge method.

The basin method can be used in areas where there is good permeable material at the surface or where the layer of nonpermeable surface material is less than 4-5 m thick and can be removed easily during construction of basins. Basins can be of any size. The methodology of calculating water resources by the basin method, the construction details of the basins and the quality of water required for recharge through basins is the same as discussed in chapter V with reference to the water supply to Chandigarh. It is important to stress here that artificial recharge by basins in these areas should be accompanied by simultaneous and proportionate ground-water draft through pumping wells, as recharge without ground-water withdrawal will result in waterlogging.

The induced-recharge method can be used in areas with rivers or unlined canals and with suitable hydrogeological conditions. This method has been discussed in detail in chapter VII with reference to the well-field along the Ghaggar River near the village of Tatiana and the methodology remains the same. It should be kept in mind, however, that a successful application of this method requires that the river and/or canal beds be cleaned periodically to retard clogging of the bottom by clay.

B. Sand-dunes area

A large area of the western part of the Project is desert and occupied by sand dunes. The water table in this area is generally between 30-40 m bgl; the aquifer is unconfined and contains brackish to saline water, though fresh water occurs in small lenses near the Ghaggar River, the canals and around the Suratgarh depressions. Filtration characteristics of the aquifer in the sand-dune areas are rather good. A series of natural depressions between the sand dunes have been filled with the flood waters of the Ghaggar River and, since there is no exploitation of ground water in the area, waterlogging conditions have been created in some places near these depressions.

Two methods of artificial recharge can be used in the sand-dune areas. The first is the induced recharge method. Well-fields can be constructed along the Ghaggar River and the unlined canals, where there is good intercommunication between the surface water and the ground water. These well-fields can be utilized to supplement the water-supply needs of the nearby towns and villages. Exploitation of artificially created lenses of fresh water floating on saline water must be carried out carefully; if such sources are over-exploited, saline water will be drawn into the wells. Detailed hydrogeological investigations are therefore required before exploitation is carried out.

The basin method of artificial recharge can be used near the lined canals in order to create fresh-water lenses, which can be exploited to meet the water-supply needs of the nearby areas.

It is also suggested that more of the natural depressions located between sand dunes be filled with the Ghaggar flood waters and with excess water from the already existing depressions near Suratgarh. Following the necessary hydrogeological investigations, exploitation of ground water from the well-fields created by these depressions by induced recharge method is suggested. This will both avert waterlogging and improve the fresh ground-water resources of the area.

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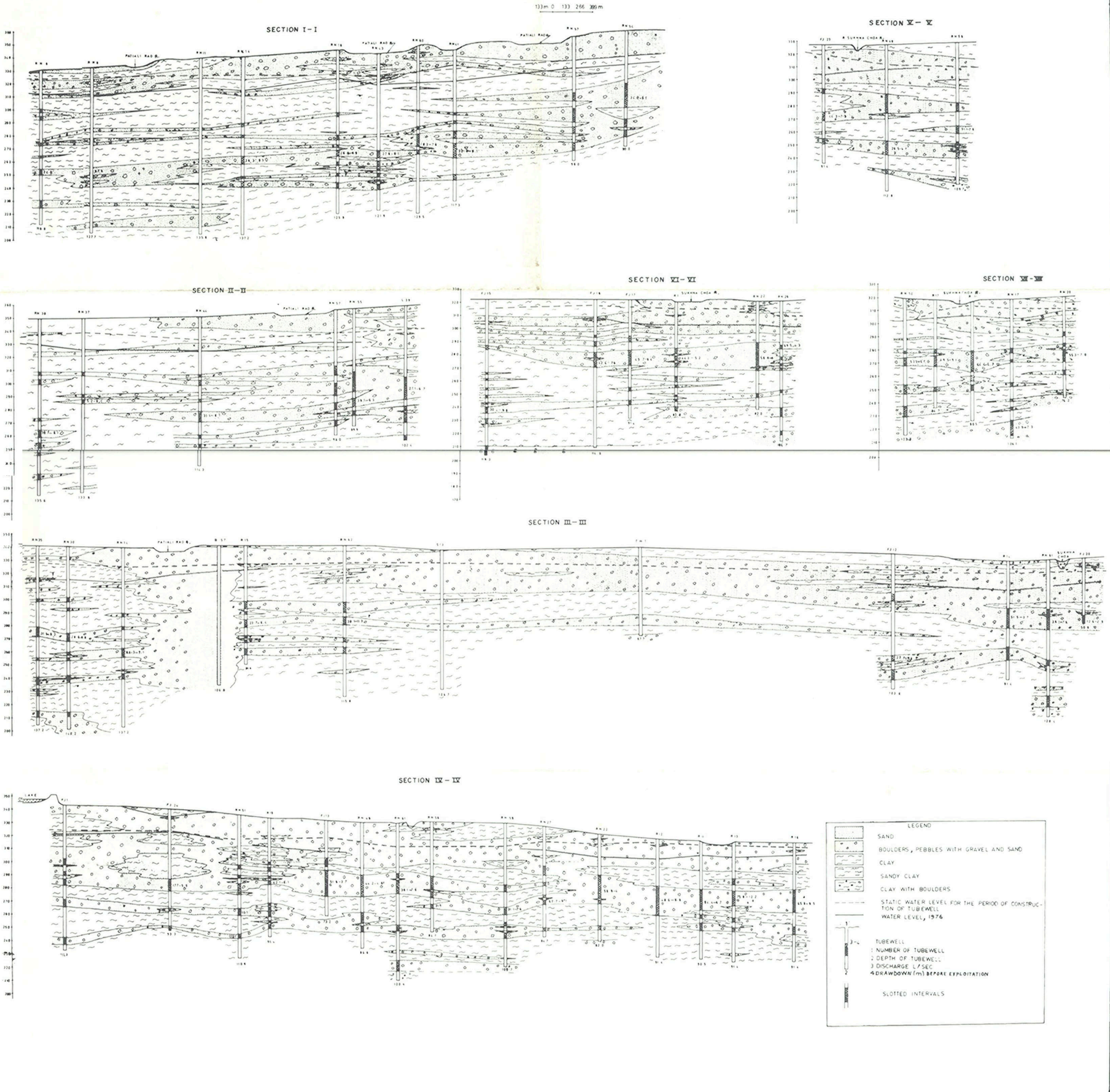
FIGURE 15

INDIA

GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN

HYDROGEOLOGICAL CROSS-SECTIONS I THROUGH VII



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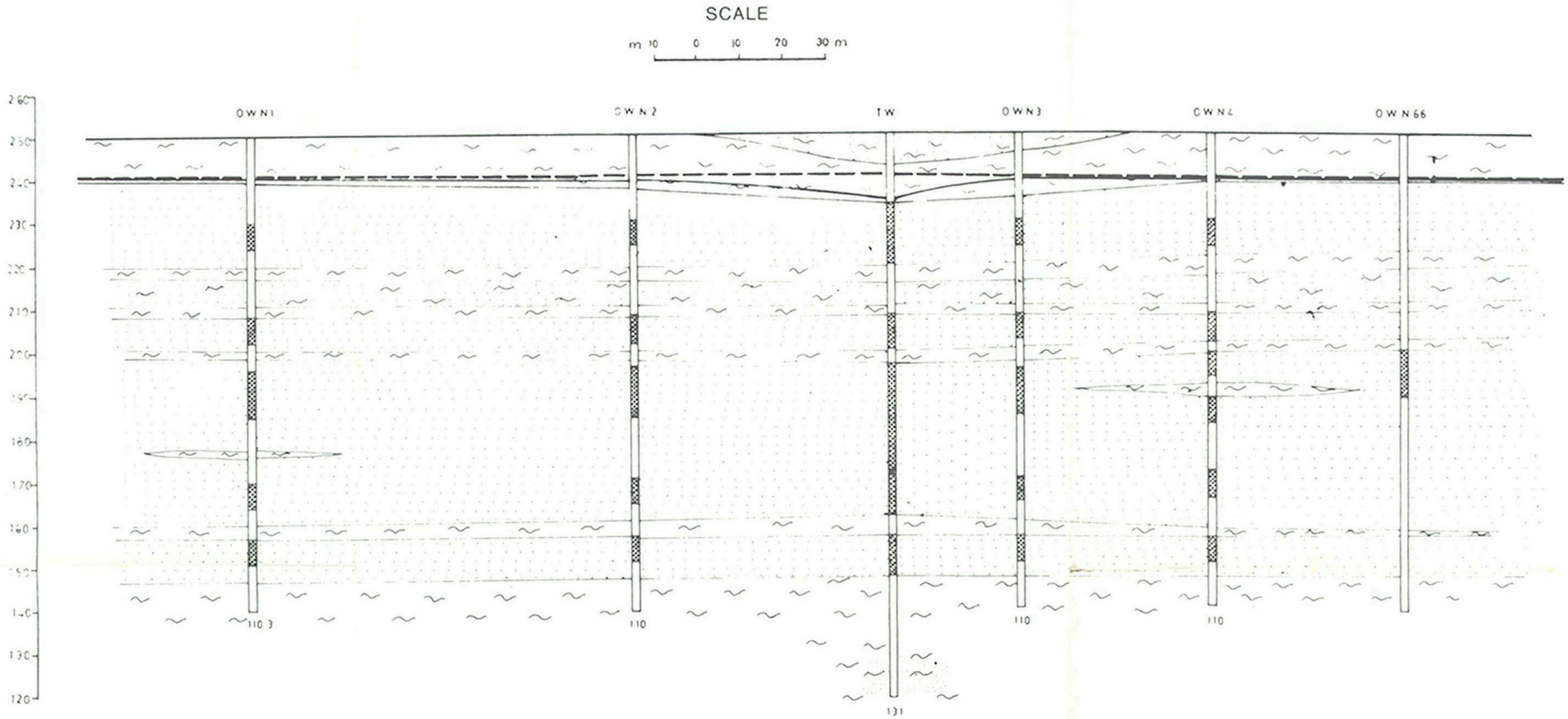
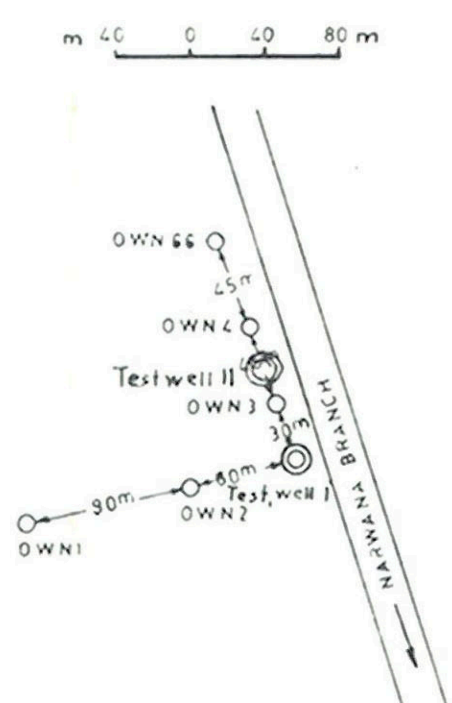
INDIA

GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN

NARWANA BRANCH CANAL AREA: HYDROGEOLOGICAL CROSS-SECTION OF DABKHERI SITE



SITE PLAN, SHOWING LOCATION OF TEST WELL AND OBSERVATION WELLS



INDIA

NARWANA BRANCH AREA: RESULTS OF INFILTRATION EXPERIMENTS AT DABKHERI SITE

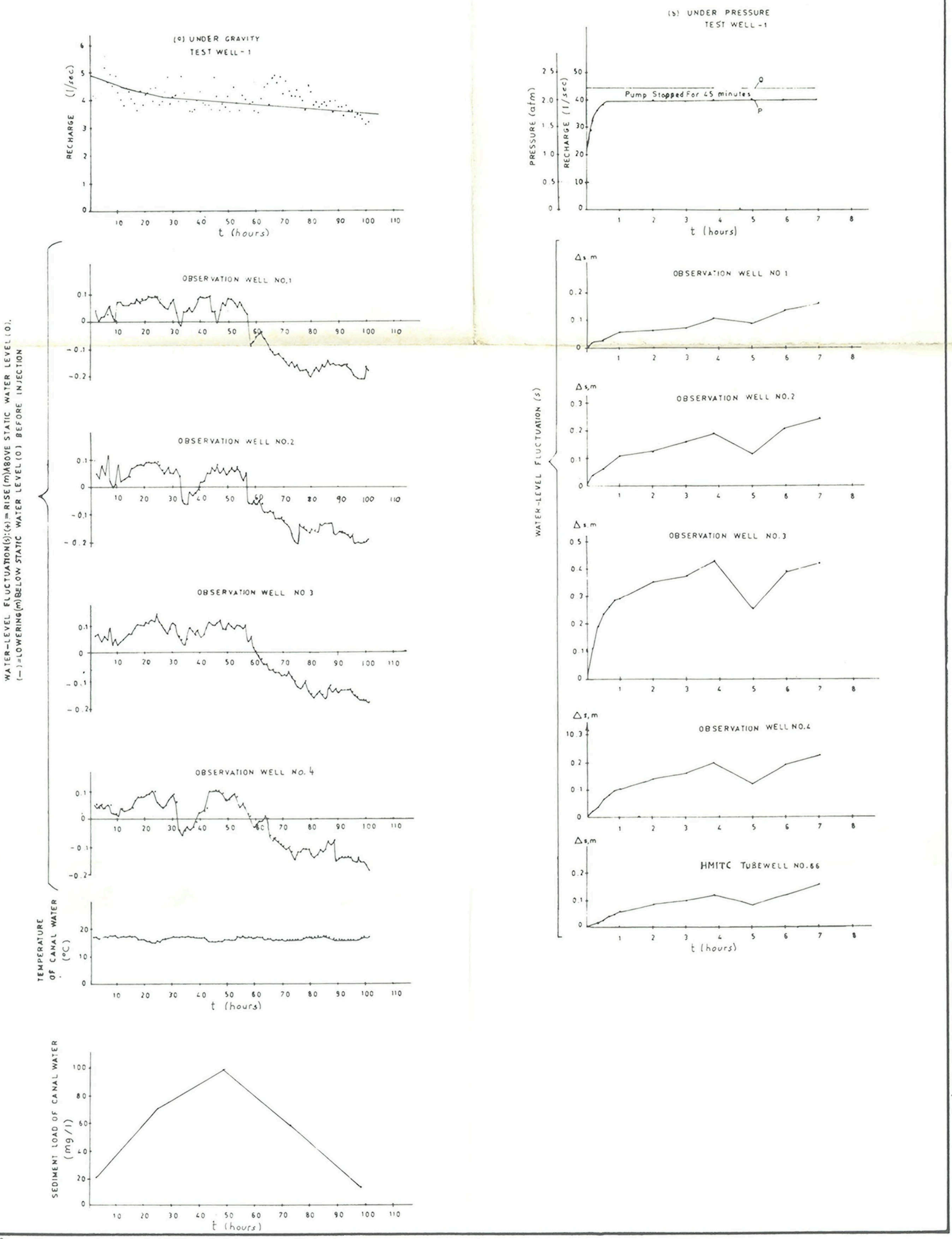


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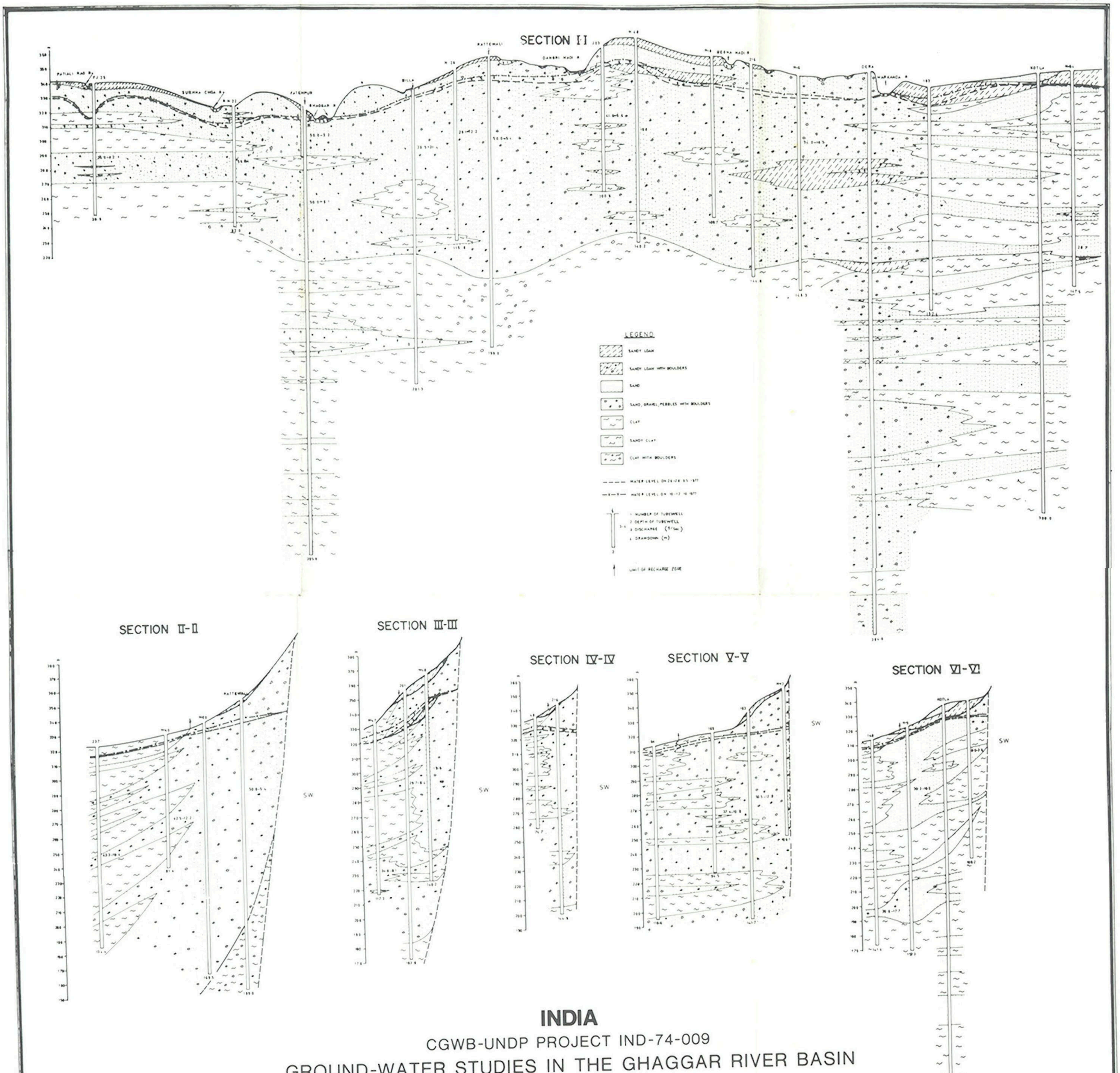
FIGURE 7



INDIA

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GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN

KANDI AREA: HYDROGEOLOGICAL CROSS-SECTIONS

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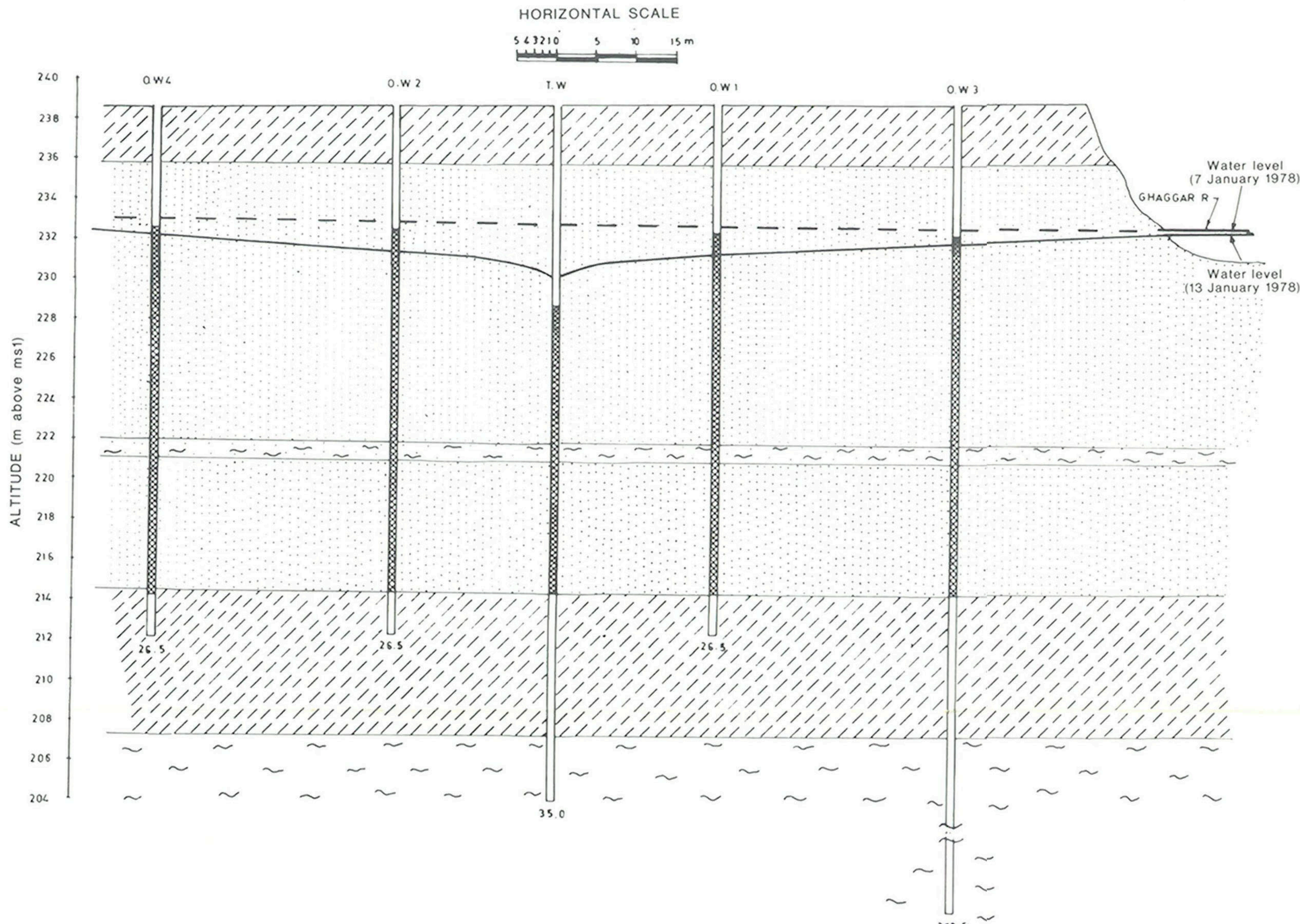
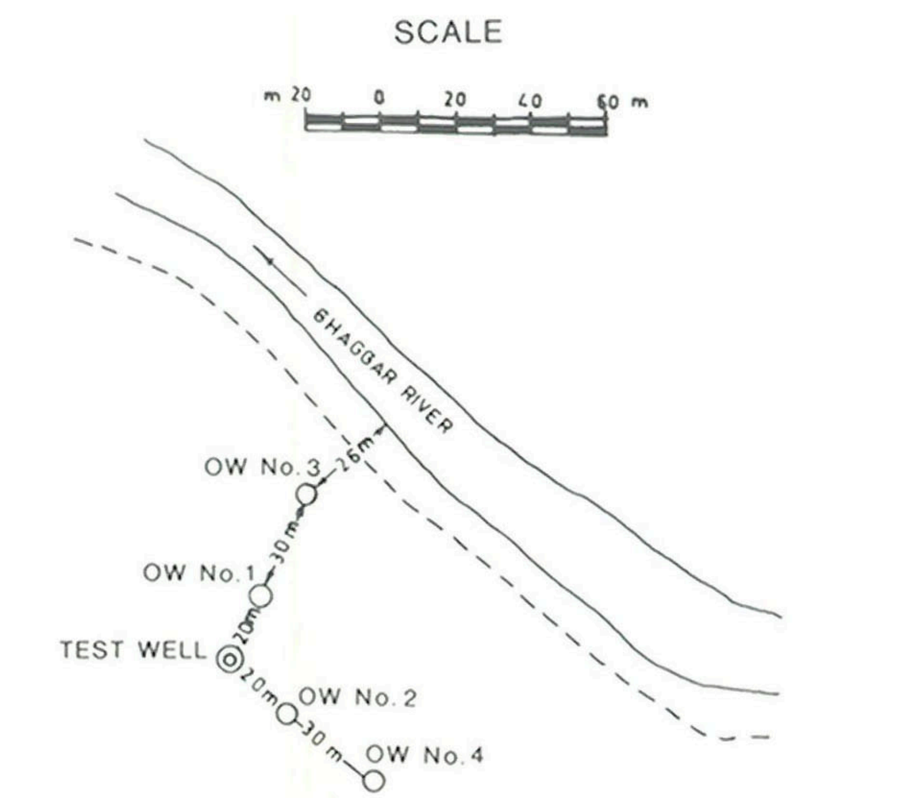
INDIA

GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN

HYDROGEOLOGICAL CROSS-SECTION OF TATIANA SITE



SITE PLAN, SHOWING LOCATIONS AND OBSERVATION WELLS OF TEST WELL



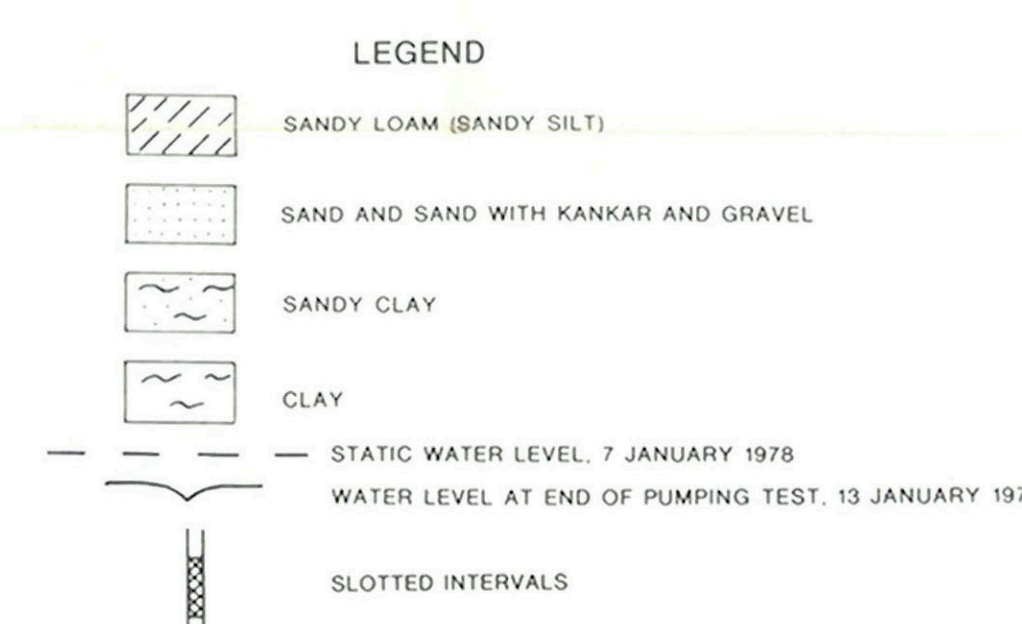


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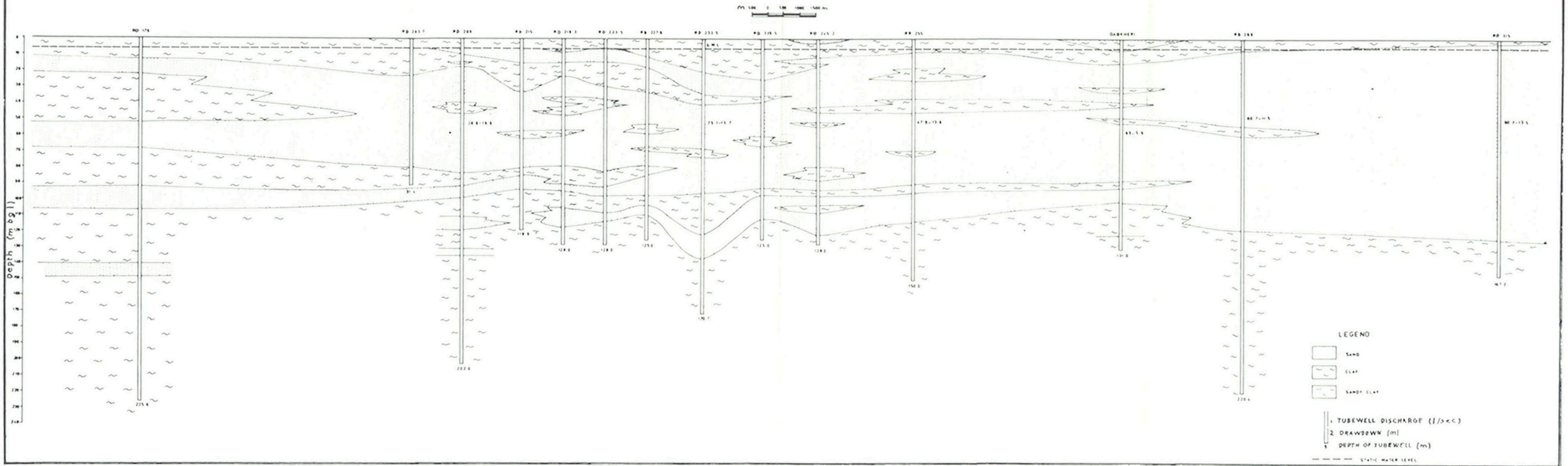
FIGURE 18

INDIA

GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN

NARWANA BRANCH AREA: HYDROGEOLOGICAL CROSS-SECTION ALONG THE NARWANA BRANCH CANAL

HORIZONTAL SCALE



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KANDI BELT: HYDROGEOLOGICAL CROSS-SECTION OF EXPLORATORY WELL SITES

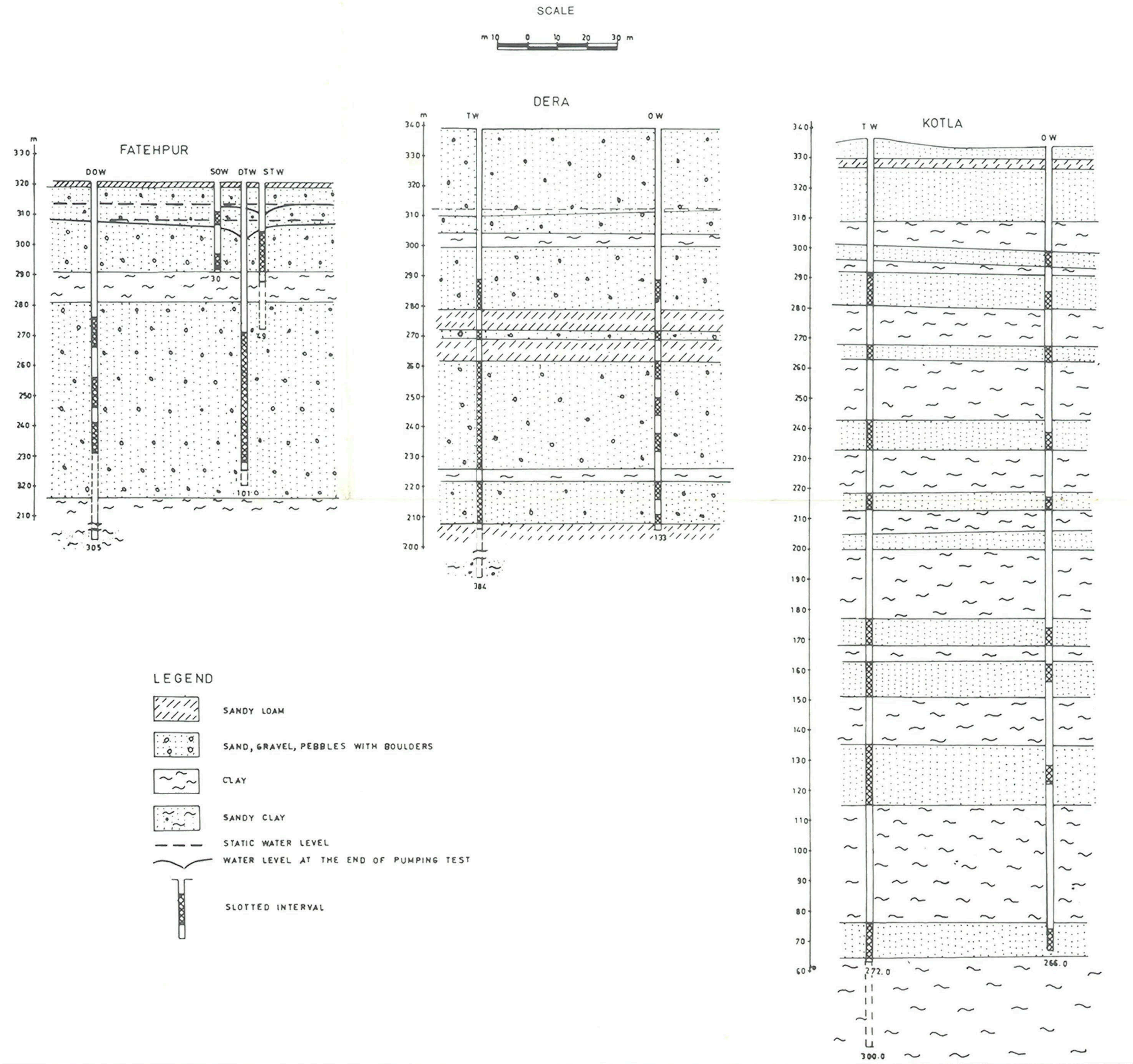

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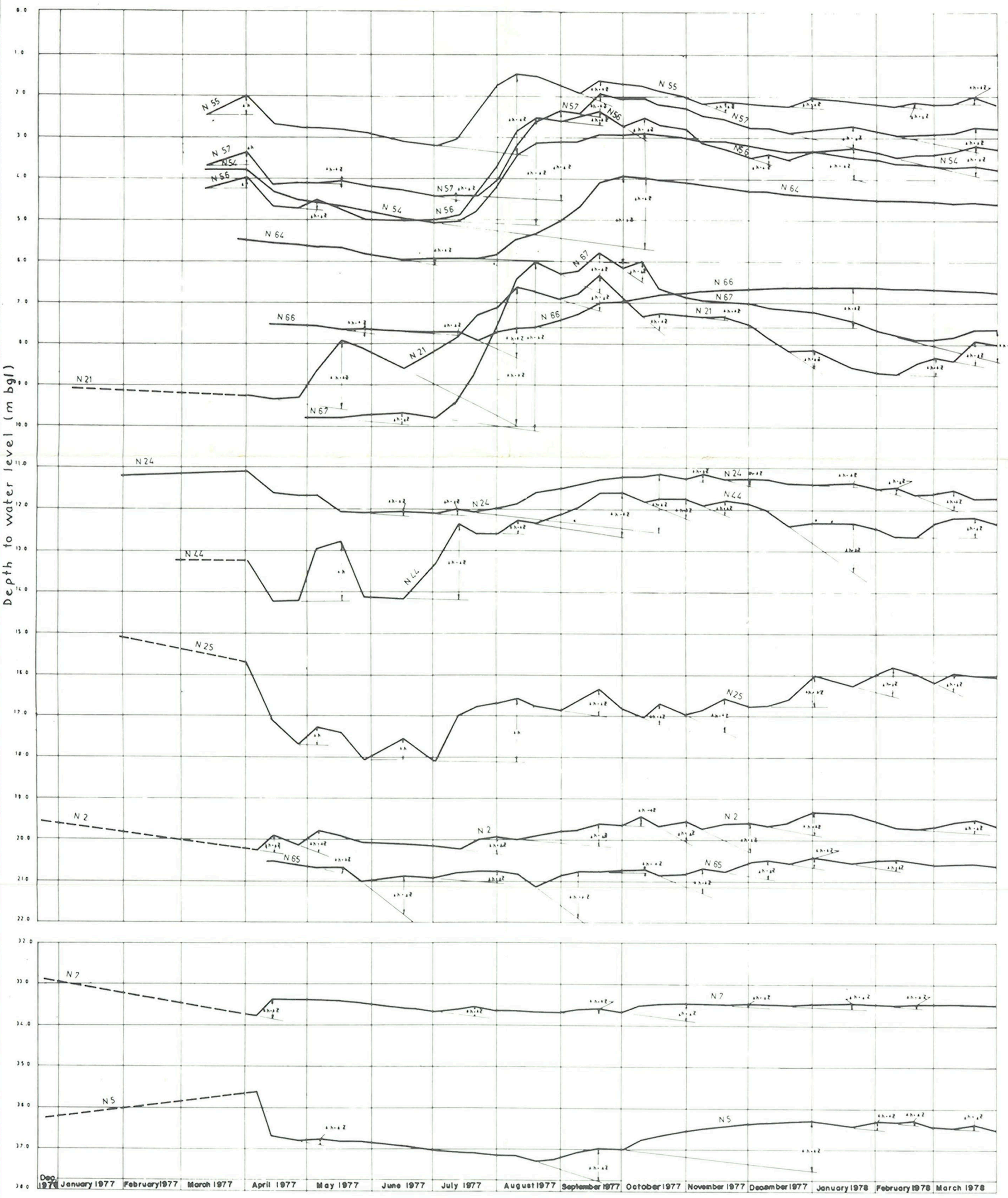
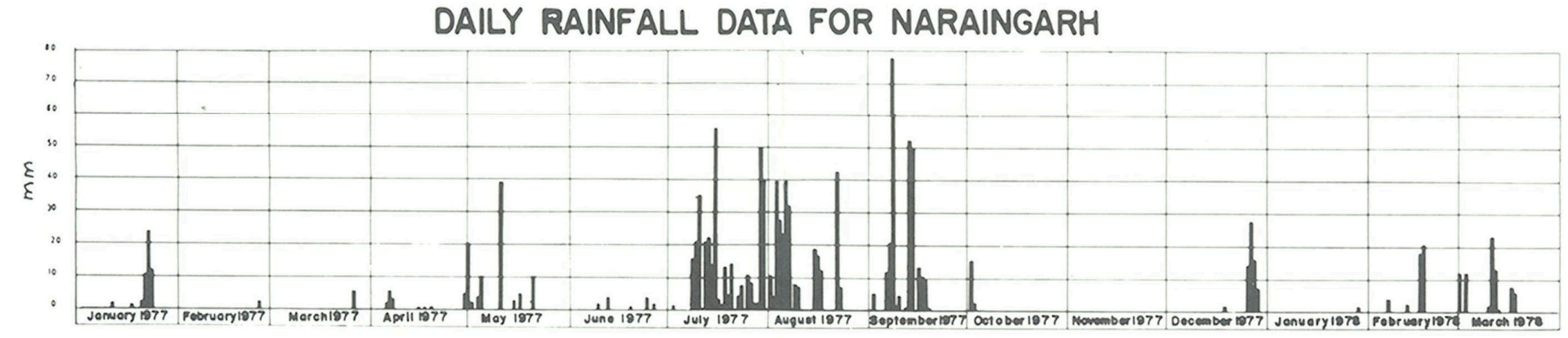
FIGURE 8

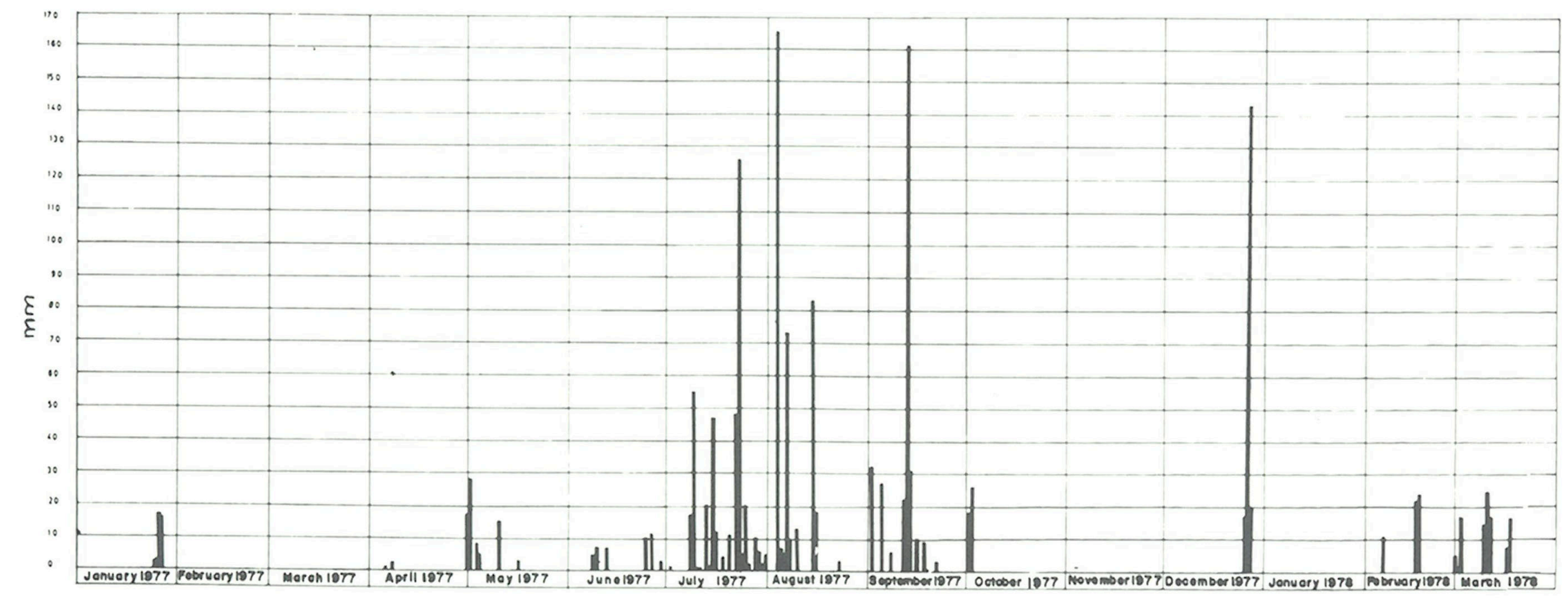
INDIA
GROUND-WATER STUDIES IN THE GHAGGAR RIVER BASIN
KANDI BELT: FLUCTUATIONS IN GROUND-WATER LEVEL



DAILY RAINFALL DATA FOR NARAINGARH



DAILY RAINFALL DATA FOR CHANDIGARH



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