Appendix5 Other Relevant Data/Information

Appendix 5-1 Meteorological Data

Table 1 Annual Precipitation of Major Cities in Punjab(1994－1998)

Name of City	1994	1995	1996	1997	1998	Average
Muree	2,220	1,703	2,192	2,307	1,973	2,078
Rawalpindi	1,690	1,615	1,324	1,414	1,412	1.493
Jhelum	1,000	1,158	989	1,336	967	1,090
Sargodha	360	319	447	629	411	433
Faisalabad	191	172	346	807	332	370
Sialkot	1,191	976	1,642	1,388	1,037	1,247
Lahore	542	826	1,189	1,233	403	839
Multan	303	265	211	264	136	236
D.G. Khan	122	87	157	350	-	179
Bahawalpur	246	203	97	304	159	202

Table 2 Monthly Mean Rainfall・Temperature・Humidity in Faisalabad (1961～1990)

Month	Ave. Temperature (℃)		Precipitation (mm)	Relative Humidity (%)
	Max.	Min.		
1	19.4	4.1	11.5	66.0
2	21.9	7.1	20.1	61.2
3	26.7	12.3	25.7	58.2
4	33.5	18.0	16.9	46.5
5	38.4	22.7	16.1	37.5
6	40.5	26.9	27.9	41.7
7	37.1	27.0	115.0	61.5
8	36.1	26.6	89.8	65.9
9	35.7	23.7	28.6	59.9
10	33.0	17.0	3.8	54.7
11	27.2	10.1	3.0	62.7
12	21.4	5.1	8.6	66.5
Ave.	30.9	16.7	372.3	56.8

Appendix 5-2 Field Report on Hydrological Study

1. Objectives

The proposed well fields, Alternative-1 and Alternative-2, are located in the vicinity of Chenab river bed near Chiniot bridge; therefore, there is concern that the installed facilities will be influenced by the floods. In addition, some amount of sewage water flows into Chenab river bed from the city of Chiniot, and it may influence the quality of the water source. The objectives of hydrological study are as follows:

- to evaluate the influence of the flood on the proposed facilities in the proposed well fields, especially the possibility of water inundation and land erosion by the flood.

2. Period 15 December 2002 - 5 January 2003

3. Methodology

Hydrological study was conducted by the following steps:

- Site observation around Chenab river bed and bank near Chiniot and Chenabnagar
- Interview with farmers around Chenab river bank about the situation of the floods in the past
- Data collection on the water level and discharge of Chenab river (33 year data from 1970 to 2002, at Rivaz bridge, Chiniot bridge and Qadirabad barrage, provided by Irrigation and Power Department, Lahore.)
- Analysis of the water level and discharge data of Chenab river, the results of interview with farmers, and the results of site observation

4. Findings

1) Collection of hydrological data

The hydrological data shown in Table-1 were provided by the Irrigation and Power Department, Government of the Punjab, Lahore. Some of the discharge and water level data at Chiniot bridge which are not available were obtained by regression estimate as is mentioned in Note of Table-1.

Table-1 Hydrological data available

Station / Year	Rivaz Bridge (69km downstream of Chiniot bridge)		Chiniot Bridge		Qadirabad Barrage (117km upstream of Chiniot bridge)	
	Water Level	Discharge	Water Level	Discharge	Water Level	Discharge
1970	○	○	○	N.A.[2)]	○	○
1971	○	○	○	N.A.[2)]	○	○
1972	○	○	○	N.A.[2)]	○	○
1973	○	partly N.A.	○	N.A.[2)]	○	○
1974	○	partly N.A.	○	N.A.[2)]	○	○
1975	○	○	○	N.A.[2)]	○	○
1976	○	○	○	N.A.[2)]	○	○
1977	○	○	○	N.A.[2)]	○	○
1978	○	○	○	N.A.[2)]	○	○
1979	○	○	○	N.A.[2)]	○	○
1980	○	○	○	N.A.[2)]	○	○
1981	○	○	○	N.A.[2)]	○	○
1982	○	○	○	N.A.[2)]	○	○
1983	○	○	○	N.A.[2)]	○	○
1984	○	○	○	N.A.[2)]	○	○
1985	○	○	○	N.A.[2)]	○	○
1986	○	○	○	N.A.[2)]	○	○
1987	○	○	○	N.A.[2)]	○	○
1988	○	○	○	N.A.[2)]	○	○
1989	○	○	○	N.A.[2)]	○	○
1990	○	○	○	N.A.[2)]	○	○
1991	○	○	○	N.A.[2)]	○	○
1992	○	○	○	partly N.A.[2)]	○	○
1993	○	○	○	partly N.A.[2)]	○	○
1994	○	○	partly N.A.[3)]	partly N.A.[2)]	○	○
1995	○	○	partly N.A.[3)]	partly N.A.[2)]	○	○
1996	○	○	○	partly N.A.[2)]	○	○
1997	○	○	partly N.A.[3)]	partly N.A.[2)]	○	○
1998	○	○	partly N.A.[3)]	N.A.[2)]	○	○
1999	○	○	partly N.A.[3)]	partly N.A.[2)]	○	○
2000	○	○	partly N.A.[3)]	partly N.A.[2)]	○	○
2001	partly N.A.	partly N.A.	partly N.A.[3)]	partly N.A.[2)]	N.A.	N.A.
2002 (Jan-Sep)	partly N.A.	partly N.A.	○	N.A.[2)]	N.A.	N.A.

Note:
1) The items marked "○" indicate the data provided as raw data by the Irrigation and Power Department, Lahore.
2) The discharge data at Chiniot bridge were estimated by regression curves obtained by the relationship between the raw data of water level and discharge in the years 1992 - 1997 and 1999 - 2000.
3) The water level data at Chiniot bridge from 1994 to 2001 were estimated by regression curves obtained by the relationship between the raw data of Rivaz bridge and Chiniot bridge in the years 1990 - 1993.

2) Outline of flood data in the past

Table-2 and Figure-1 show the maximum water level and discharge data at each station in each year.

According to the table and figure, the years of high water flow are 1973, 1976, 1988, 1992, 1995, 1996 and 1997, while the years of drought are 1987 and 1998. The flood which occurred in 1973 is the severest one in the past 30 years. Maximum flood in each year occurs mostly in July or August, sometimes in September. Comparing the flood data of Rivaz bridge, Chiniot bridge and Qadirabad barrage, the time lag of floods from Qadirabad to Chiniot and from Chiniot to Rivaz is approximately one day (=24 hours) respectively.

Table-2 Flood data of Chenab river in the past 30 years

Year	Rivaz Bridge			Chiniot Bridge			Qadirabad Barrage		
	Maximum Discharge (cusec)	Maximum Water Level (ft)	Date	Maximum Discharge (cusec)	Maximum Water Level (ft)	Date	Maximum Discharge (cusec)	Maximum Water Level (ft)	Date
1970	123,221	518.45	03-Sep-1970	151,593	592.08	02-Sep-1970	237,572	693.82	01-Sep-1970
1971	201,878	518.80	12-Aug-1971	165,731	592.38	04-Aug-1971	305,968	694.80	03-Aug-1971
1972	87,082	517.25	14-Jul-1972	117,783	591.30	08-Aug-1972	178,203	693.70	10-Jul-1972
1973		523.00	12-Aug-1973	1,203,343	597.75	11-Aug-1973	847,249	699.45	10-Aug-1973
1974		517.30	07-Aug-1974	142,517	591.88	18-Jul-1974	198,228	693.70	25-Jul-1974
1975	148,200	517.80	13-Sep-1975	126,046	591.50	12-Sep-1975	198,210	691.20	12-Sep-1975
1976	651,000	521.50	09-Aug-1976	504,961	595.20	08-Aug-1976	577,015	696.00	07-Aug-1976
1977	225,900	518.00	07-Aug-1977	207,591	593.20	17-Jul-1977	452,532	695.60	17-Jul-1977
1978	139,575	518.90	12-Aug-1978	141,180	591.85	23-Jul-1978	293,418	695.50	10-Aug-1978
1979	142,326	518.00	06-Aug-1979	89,291	590.55	19-Jul-1979	240,785	694.10	03-Aug-1979
1980	167,117	518.00	17-Jul-1980	117,783	591.30	17-Jul-1980	97,697	694.10	10-Aug-1980
1981	275,000	520.00	29-Jul-1981	154,835	592.15	31-Jul-1981	505,638	696.90	26-Jul-1981
1982	182,000	518.30	04-Aug-1982	128,156	591.55	03-Aug-1982	225,517	694.50	02-Aug-1982
1983	182,420	518.30	05-Sep-1983	147,929	592.00	04-Sep-1983	283,229	695.95	03-Sep-1983
1984	156,800	517.80	30-Aug-1984	86,698	590.45	15-Aug-1984	90,023	694.30	15-Aug-1984
1985	226,000	519.00	10-Aug-1985	152,516	592.10	09-Aug-1985	213,460	696.40	19-Jul-1985
1986	177,000	518.20	07-Aug-1986	161,899	592.30	06-Aug-1986	244,022	695.40	06-Aug-1986
1987	115,000	516.70	29-Jul-1987	78,254	590.00	28-Jul-1987	96,996	693.10	09-May-1987
1988	550,000	521.20	29-Sep-1988	630,340	595.75	28-Sep-1988	529,664	698.70	27-Sep-1988
1989	245,000	519.30	03-Aug-1989	218,515	593.40	01-Aug-1989	295,085	697.10	01-Aug-1989
1990	250,000	519.40	24-Mar-1990	141,180	591.85	15-Jul-1990	339,191	696.00	23-Mar-1990
1991	226,000	518.80	16-Apr-1991	147,929	592.00	15-Jul-1991	249,663	694.50	15-Apr-1991
1992	475,000	520.50	13-Sep-1992	529,400	595.70	12-Sep-1992	948,530	700.30	11-Sep-1992
1993	282,000	519.80	14-Jul-1993	282,500	594.00	13-Jul-1993	434,754	697.30	12-Jul-1993
1994	274,900	519.80	23-Jul-1994	166,696	592.40	06-Aug-1994	425,567	697.00	21-Jul-1994
1995	620,000	521.60	01-Aug-1995	667,000	596.00	30-Jul-1995	640,577	698.40	29-Jul-1995
1996	785,000	522.60	27-Aug-1996	700,000	596.20	26-Aug-1996	728,432	699.90	25-Aug-1996
1997	587,000	521.80	31-Aug-1997	546,600	595.30	30-Aug-1997	600,246	699.70	29-Aug-1997
1998	126,000	517.00	07-Jul-1998	76,255	589.89	07-Jul-1998	68,983	698.80	01-Aug-1998
1999	97,000	516.30	10-Aug-1999	90,200	590.60	09-Aug-1999	111,102	691.50	08-Aug-1999
2000	186,420	518.50	25-Jul-2000	147,200	592.20	25-Jul-2000	190,640	693.50	24-Jul-2000
2001	115,000	516.70	26-Jul-2001	80,000	590.00	25-Jul-2001			
2002				147,200	592.20	16-Aug-2002			

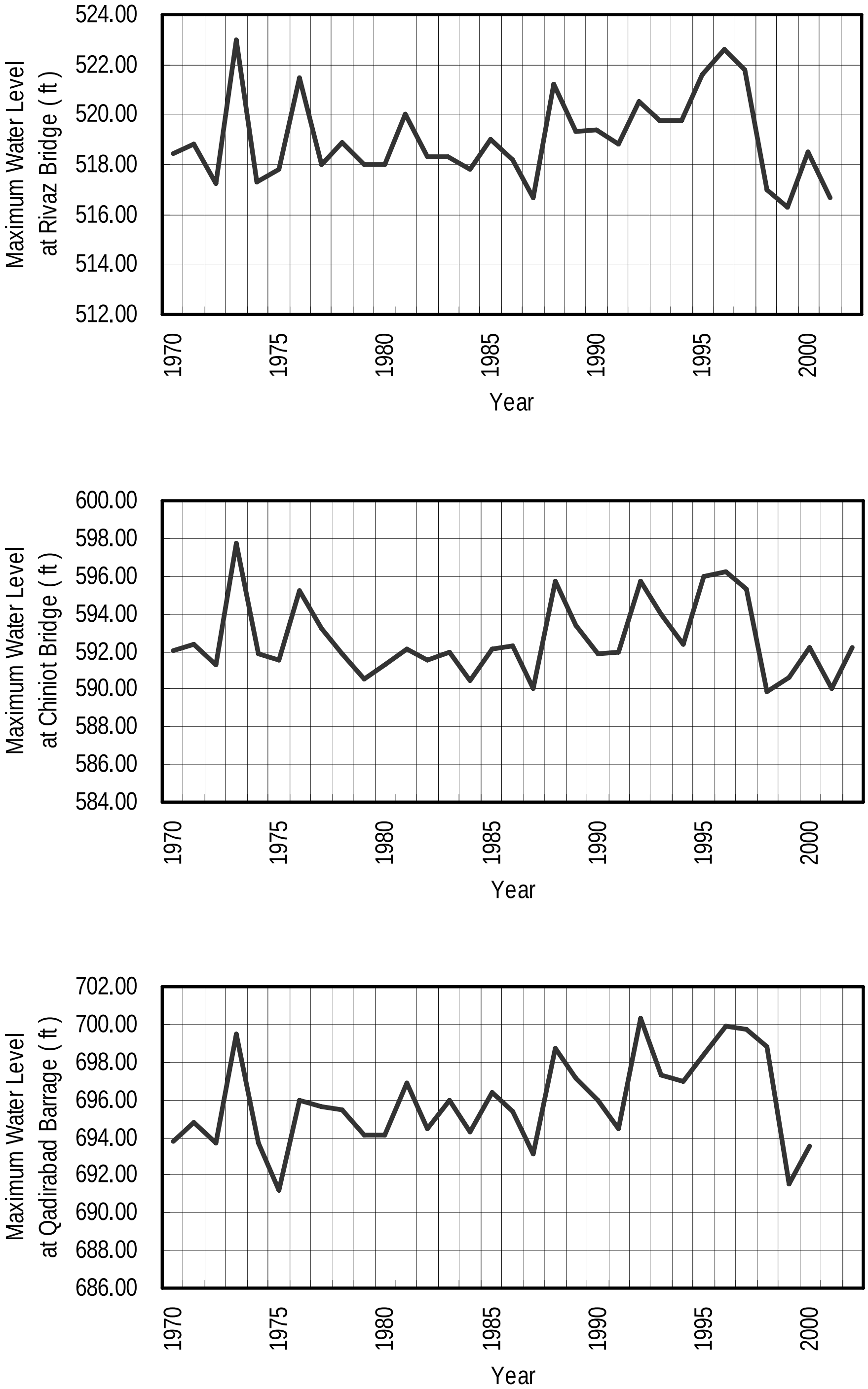


Figure-1 Flood data of Chenab river in the past 30 years

3) Characteristics of the distribution of water level and discharge

Figure-2 shows the annual change of water level and discharge at Chiniot bridge in several typical years of high water flow and drought. In the years of drought, water level usually does not go up to the level of more than 590.0 or 591.0 feet, and discharge does not go up to more than 80,000 or 90,000 cusec. Water level ranges from 583.0 to 591.0 feet in wet season from March to September, from 581.5 to 583.0 feet in other months. On the other hand, in the years of high water flow, several days from July to September show the water level of more than 590.0 feet and the discharge of more than 200,000 cusec.

Figure-3 indicates the distribution of the discharge at Chiniot bridge in several typical years of high water flow and drought. The characteristics of the typical years are shown in Table-3.

Table-3 Characteristics of the years of high water flow and drought

	Typical years of high water flow	Typical years of drought
Maximum water level in a year	> 595.0 feet	< 591.0 feet
Maximum discharge in a year	> 500,000 cusec	< 90,000 cusec
Total discharge in a year	19.0 - 37.0 billion m^3	15.0 - 20.0 billion m^3
Number of days more than 50,000 cusec	35 - 100 days	25 - 70 days
Number of days more than 100,000 cusec	10 - 40 days	0 days
Number of days more than 200,000 cusec	1 - 10 days	0 days

Figure-4 shows the logarithmic normal distribution of annual maximum discharge in the past 33 years. Based on this distribution, the return period for annual maximum discharge in each year is obtained as indicated in Table-4.

Table-4 Return period for annual maximum discharge

Year	Maximum discharge (cusec)	Return period
1973	1,203,343	approximately 50 years
1996	700,000	15 to 20 years
1995	667,000	10 to 15 years
1988	630,343	10 to 15 years
1997	546,600	5 to 10 years
1992	529,400	5 to 10 years
1976	504,961	5 to 10 years

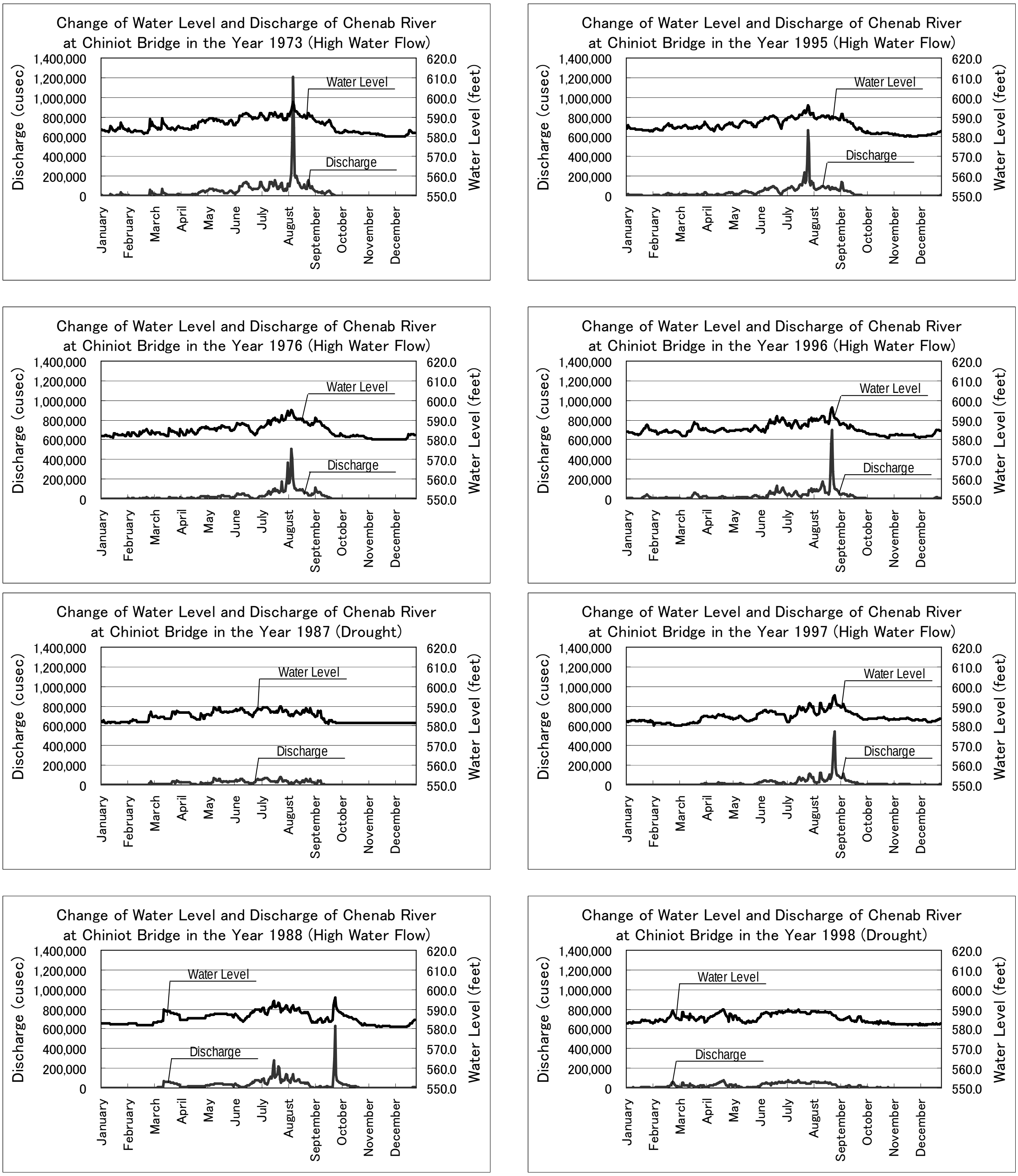
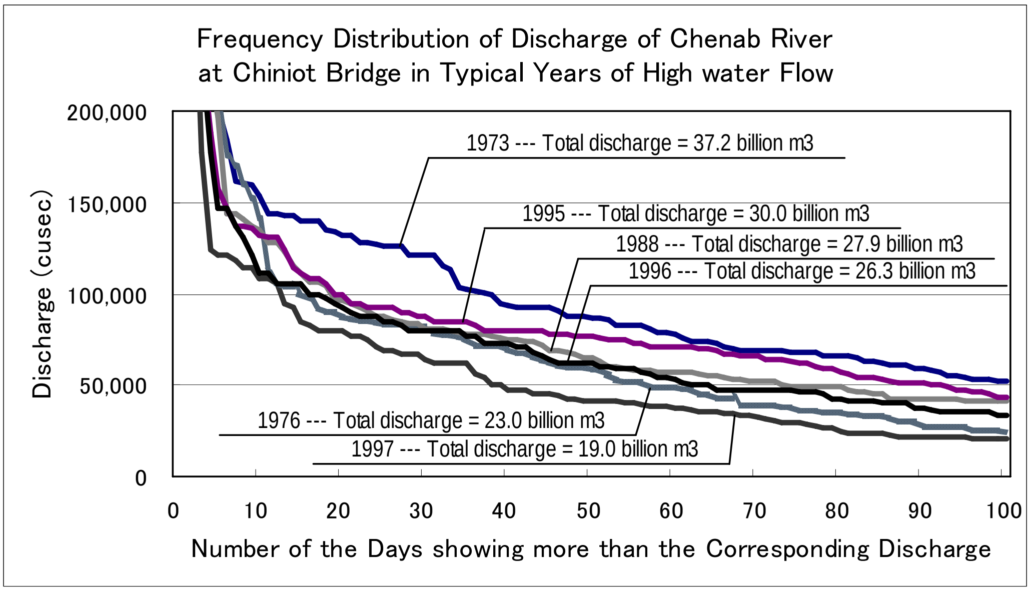


Figure-2 Annual change of water level and discharge at Chiniot bridge

Typical year of high water flow: 1973, 1976, 1988, 1995, 1996, 1997.

Typical year of drought: 1987, 1998.



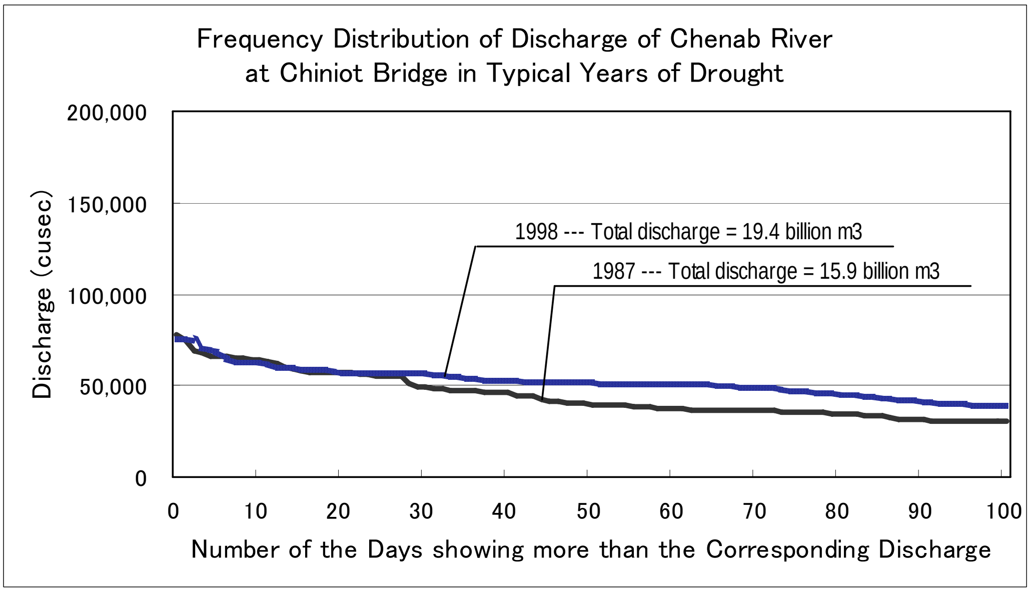
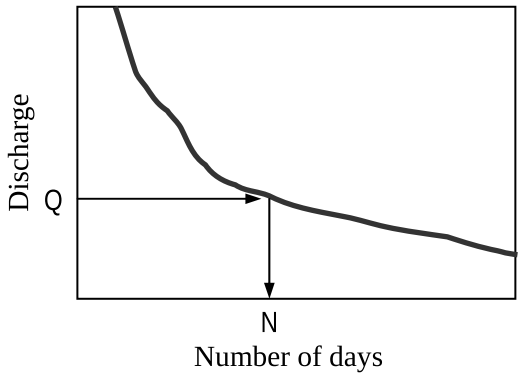


Figure-3 Distribution of discharge at Chiniot bridge

Note: The meaning of the above charts is as follows:



N days in a year have the discharge of more than Q.

Logarithmic-normal Probability Paper

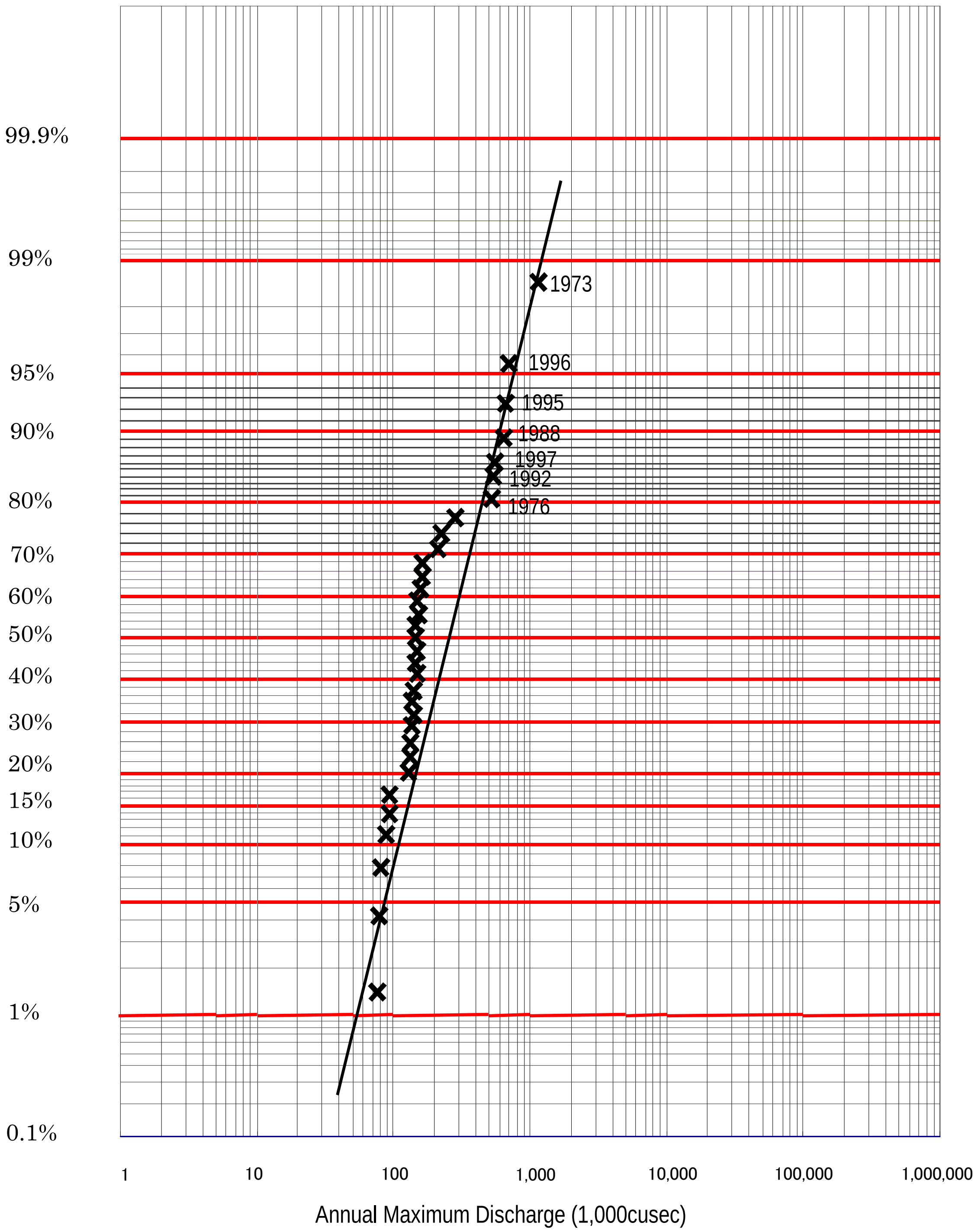


Figure-4 Logarithmic normal distribution of annual maximum discharge

4) Characteristics of the flood flow in the Chenab river basin

Figure-5 shows the situation of floods in the past, that is, the range of flooded area in several years when severe flood occurs, with water depth at several points indicated according to the interview results. As is clarified by the numerical data analysis, the interview results also show that severe flood occurred in the years 1973, 1992, 1995 and 1997. In 1973 when the severest flood in the past 30 years occurred, the flood overflowed the Chiniot-Jhang road, while such severe flood has not occurred in other years.

The left bank of the river is composed of several terraces which are 2 to 4 meter high, and the ups and downs are rather big. Therefore, the left bank is of higher ground level than the right bank side.

The bed and the bank of the river are made of sandy loose soils which compose terraces of 2 to 4 meter height. According to the fact that the main stream of the river is running close to the right bank, and that the right bank is of lower ground level than the left bank, the right bank side has been more easily and more frequently attacked and broken by floods.

The location of the well construction site and the protection measures against the influence of floods should be determined taking into consideration the topographical features of the river bed and the bank. According to the above-mentioned facts, the left bank side, if it is on a higher terrace and far from the river bed, has little fear of being attacked by floods.

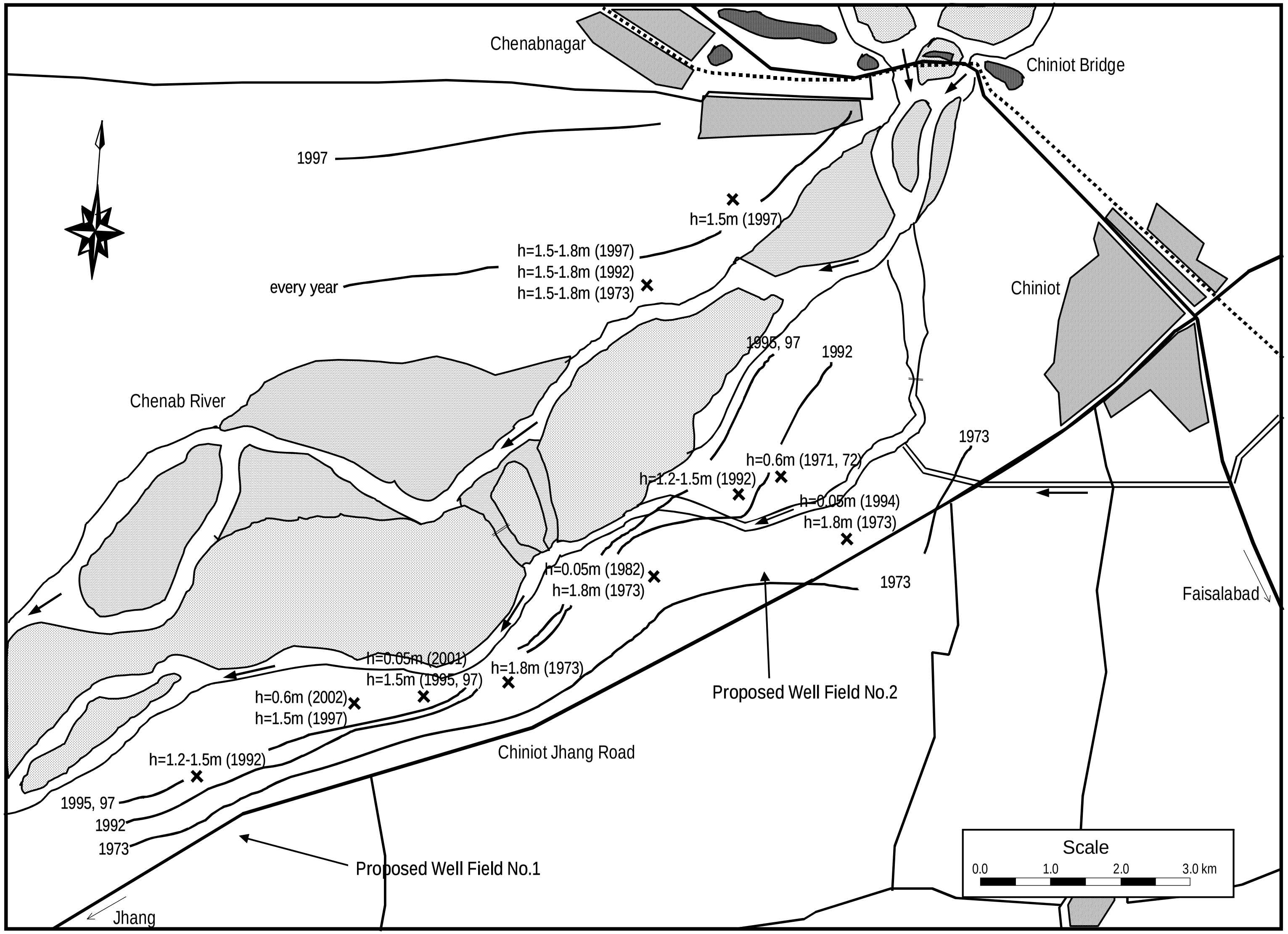


Figure-5 Situation of the flooded area in the past

5. Conclusion

The proposed well fields, Alternative-1 and Alternative-2, are located in the vicinity of Chenab river bed. The left bank sides where the well fields are planned are composed of several terraces, and far from the main stream of the river. Consequently, if the proposed facilities are constructed on a higher terrace, far from the river bed, there is not a worry about the influence of flood on the facilities. It should be noted that the construction site be determined in consideration of the topographic features of the river bed and the river bank.

Appendix 5-3 Geophysical Survey

1) Outline of the Survey

The geophysical survey of the proposed wellfield and its vicinity was carried out in an area about 12km long and 3 km wide along the Jhang Branch Canal to examine the vertical and horizontal continuity of prospective aquifers that can provide information on design for screen length, basic drilling depth in the wellfield, etc..

The outline of the survey was as shown in the following table.

Table 1 Outline of Geophysical Survey

	Item	Description
1.	Period of field survey	Aug. 15 to Aug. 21, 2003
2.	Type of the survey	Surface electrical resistivity survey
3.	No. of resistivity stations	24 stations
4.	Method of the survey	Wenner 4-electrode configuration
5.	Depth of measurement	200m
6.	Layout of stations	a No. 1 track just beside the embankment of left bank (11 stations) b. No. 2 track one km south of No. 1 (9 stations) 3. No. 3 track about one km south of No. 2 (4 stations) (Refer to Fig. 2-2-1-8 for locations.)

(Refer to Fig. 2 for the locations of survey stations.)

2) Survey Results

After the field survey, the data analysis was made, based upon geological information from the records of drilling carried out in and around the survey area. For this study, the following data is available:

a. Lithology of the test well installed by Binnie & Partners at RD259
b. Lithology of the test well installed by REC at RD245
c. Lithology of the test well under this study at RD 245

The results of analysis are summarized as follows:

a. Unconsolidated deposits continue from surface down to 200m and seem to compose a single continued aquifer as a whole. According to the analysis, this aquifer is divided into 3 sections, each one separated with an interbed of clayey formations with relatively low permeability, as follows:

* First section up to 30 m in depth
*Second section up to 76 to 140 m in depth
*Third section up to 170 m in depth

b. The main aquifer is the second section. Although it varies slightly in depth along the track from upstream to downstream, it is uniformly distributed through the area, showing the highest values of resistivity.

c. The occurrence of the third section seems to depend upon the location. Some stations lack this section. For the construction of production wells, it is planned to confirm it with the geophysical survey at the very points where they are to be drilled.

d. The horizontal relation of the second and the sections are confirmed through the analysis. Therefore, drilling depth is recommended to be the average of 150 and 170m, namely 160m.

e. The first section is composed of recent deposits of mainly sand where unconfined groundwater flows through. Irrigation tubewells tap this section, with a part of them further reaching the upper horizon of the second section.

f. The second and the third sections are interpreted to consist of Pleistocene alluvium of fine to medium sand. Each of the aquifers can further be subdivided into 3 to 4 layers from clayey materials to sand. Those showing high resistivity is sand, while those with low one is clayey materials.

The plotted curves of 24 stations are attached herewith, together with a sectional correlation of layers at the 11 stations along the first measurement line. All of the plots show similar trends in the pattern of their curves, indicating similar hydrogeological characteristics of the subsurface conditions along the measurement line. The typical trends of the plotted curves are shown in Fig. 1.

Fig. 1 Typical plot pattern

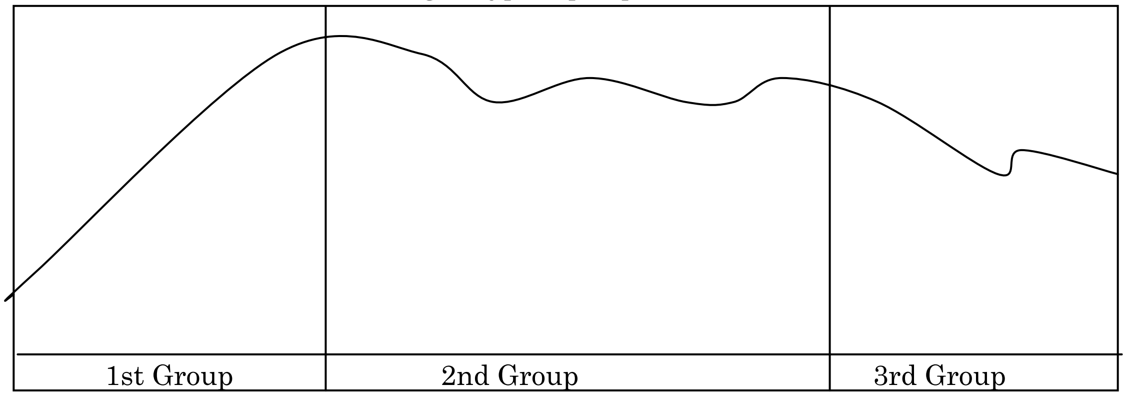


Fig. 2 LOCATION MAP OF GEOPHYSICAL SURVEY

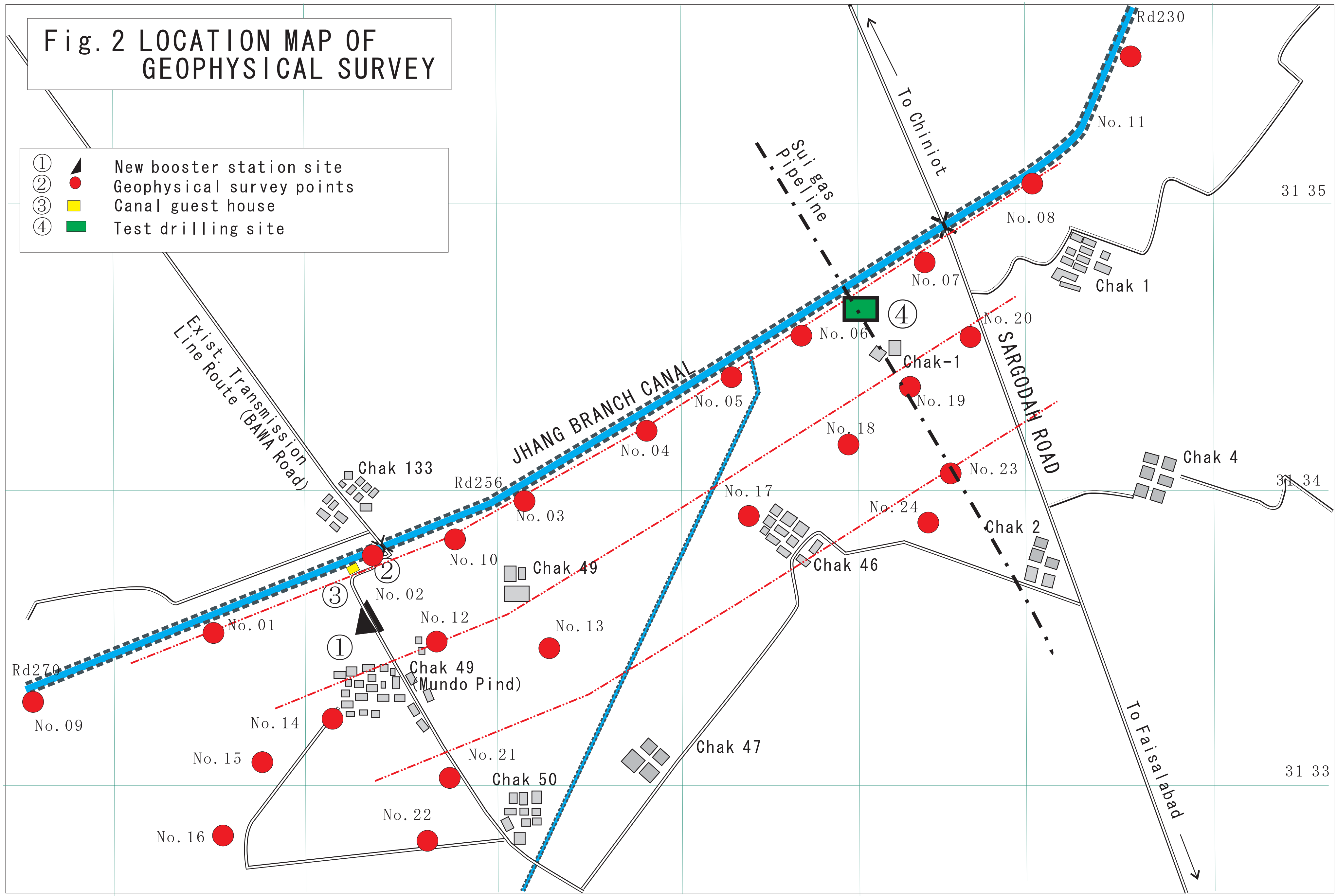
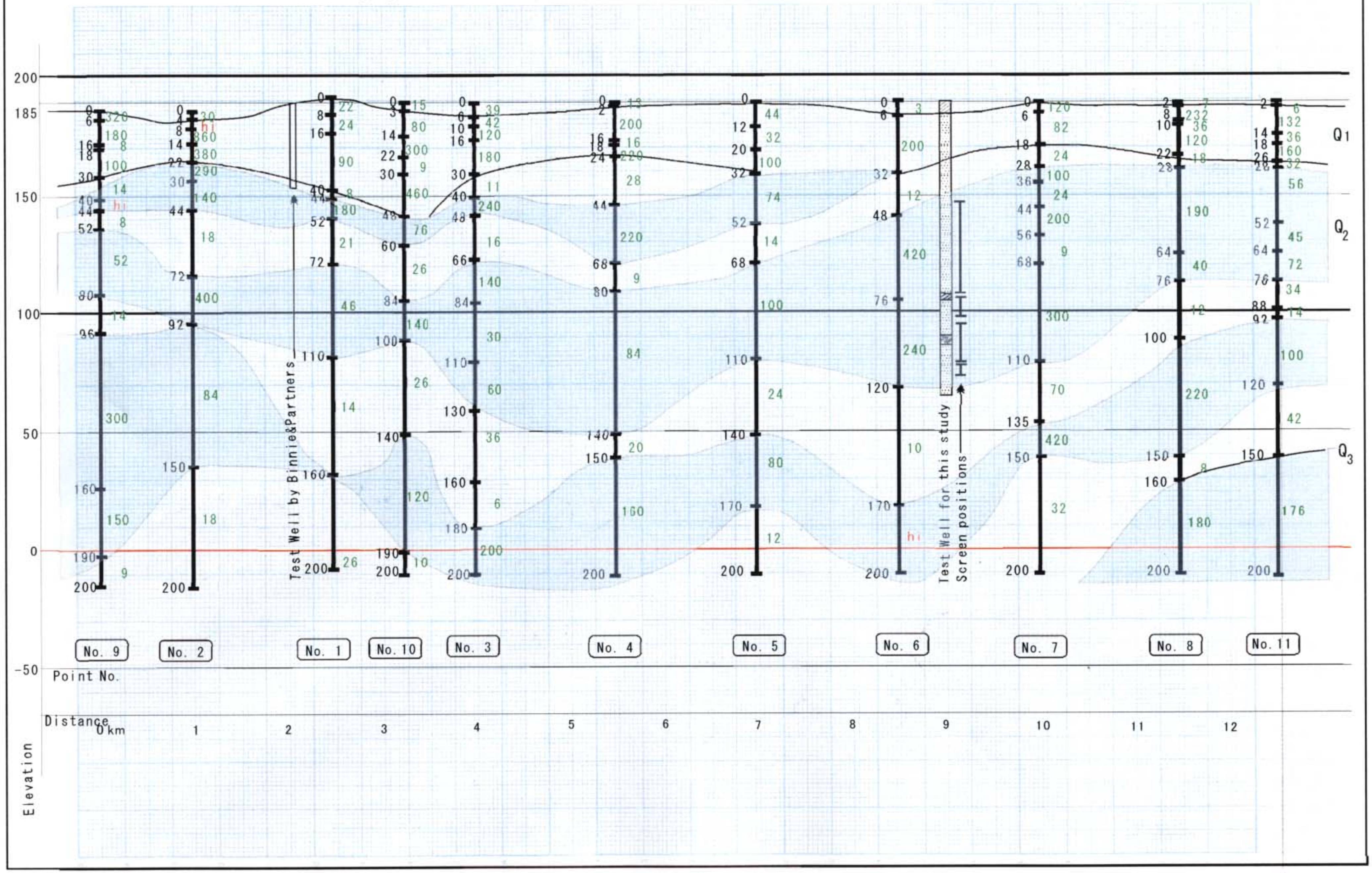
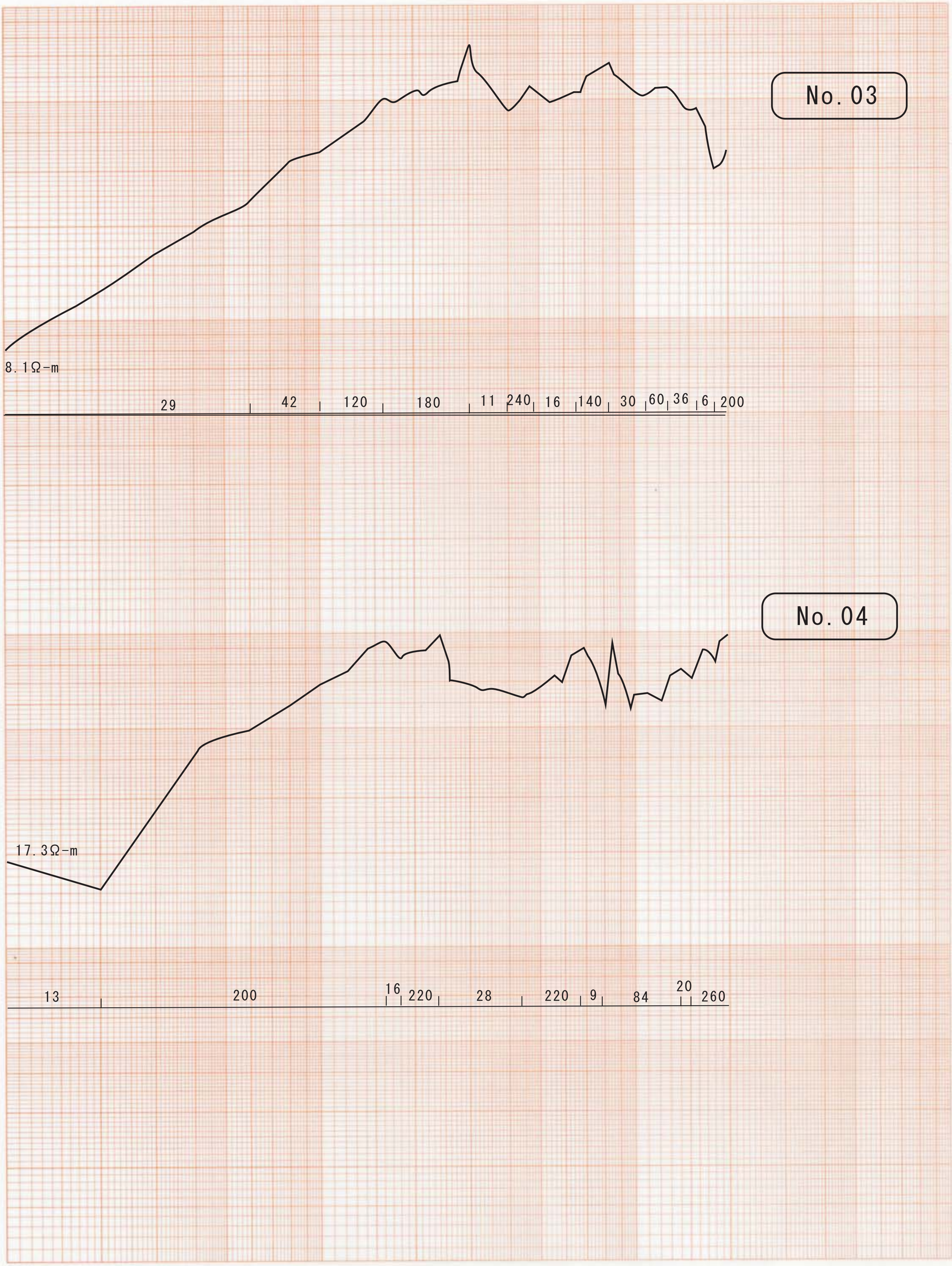


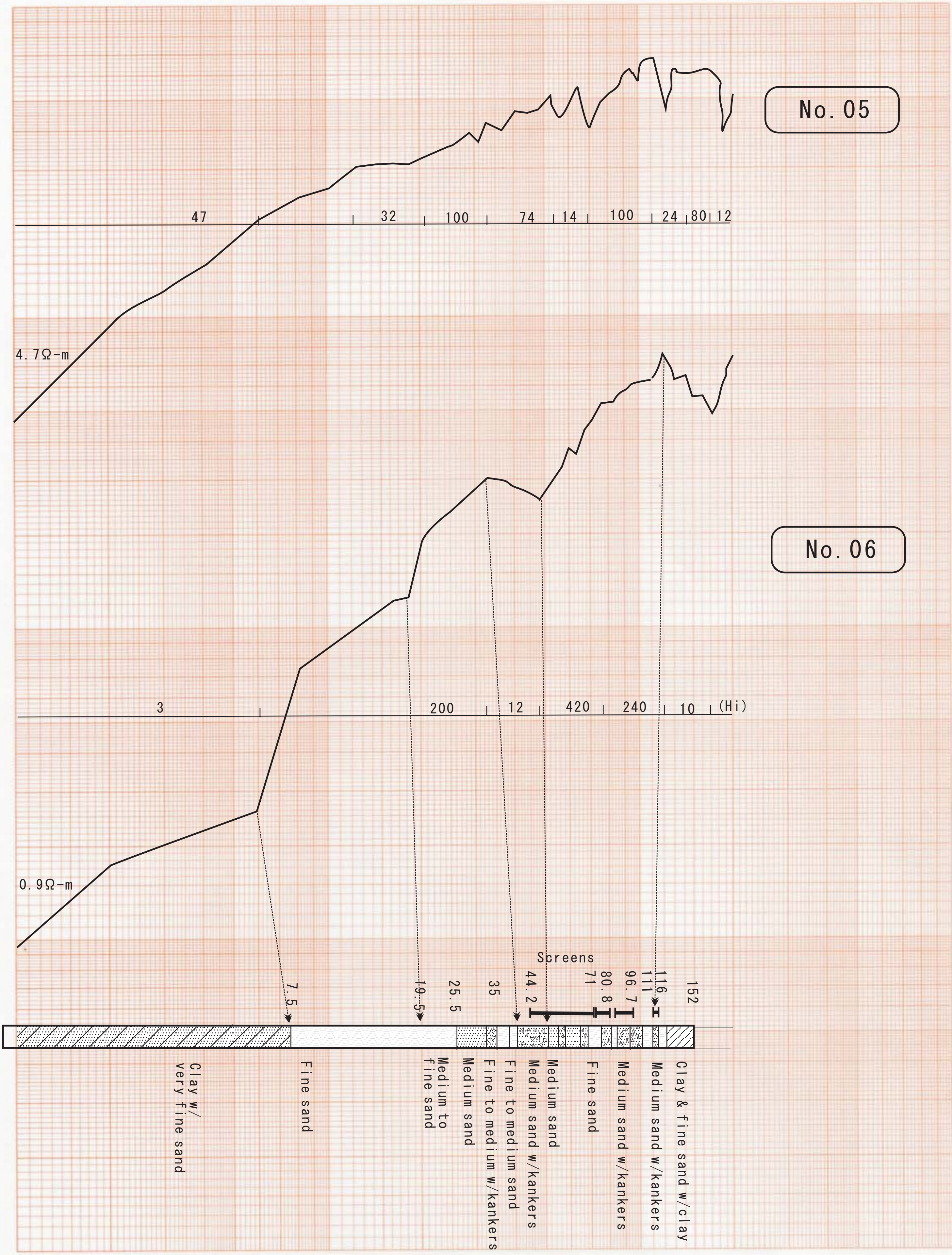
Fig. 3 Correlated Aquifer Section
Based on the Geophysical Survey

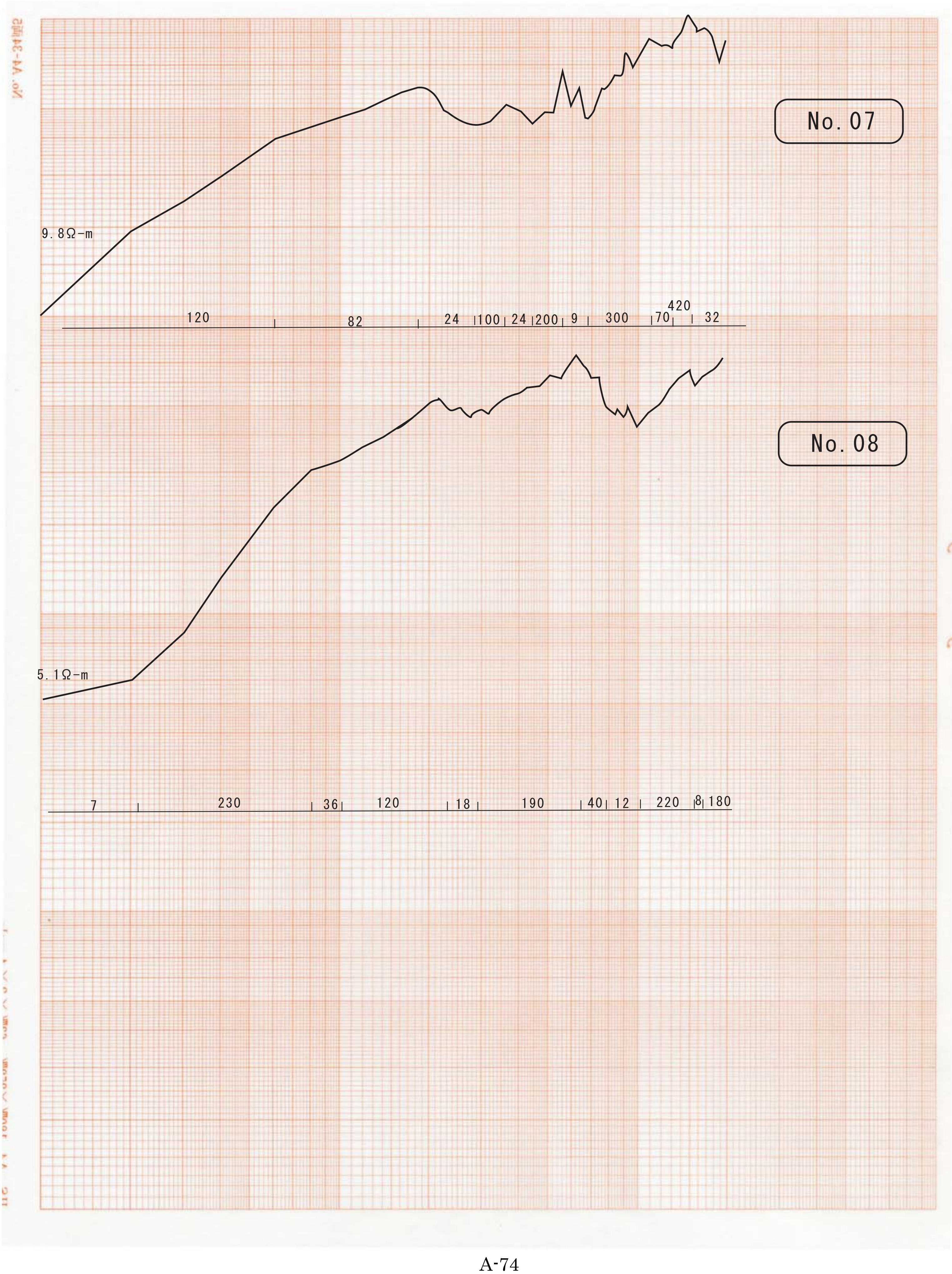
Figures indicate: Black ones, analyzed depth
Green ones: analyzed resistivity



No. 02
22
26
90
8
80
21
96
14
26
2.9Ω-m
No. 01
1.9Ω-m
30
(Hi)
860
380
290
140
18
400
84
18

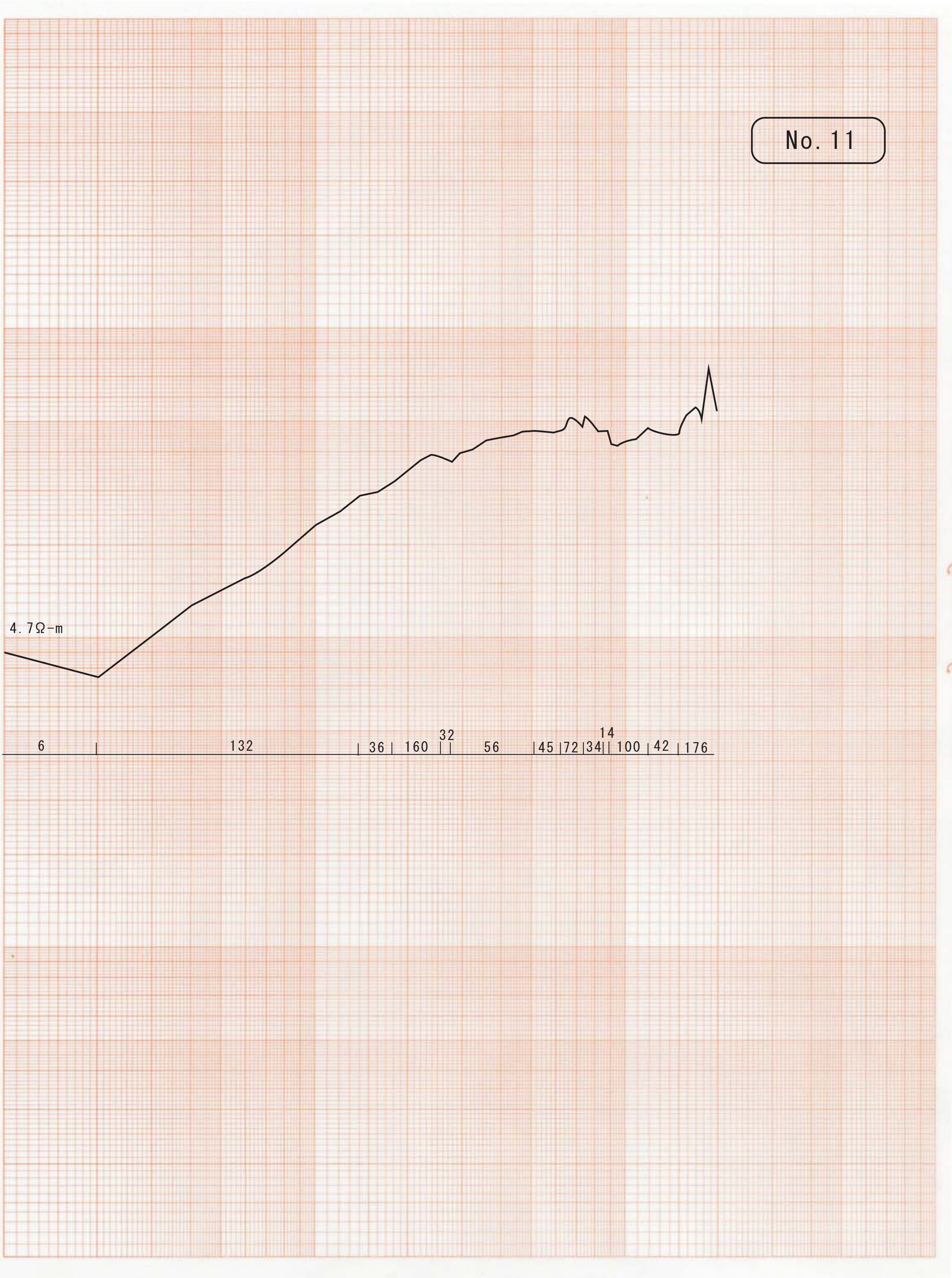

No. 03
8.1Ω-m
29
42
120
180
11
240
16
140
30
60
36
6
200
No. 04
17.3Ω-m
13
200
16
220
28
220
9
84
20
260


No. 05
47
32
100
74
14
100
24
80
12
4.7Ω-m
No. 06
3
200
12
420
240
10
(Hi)
0.9Ω-m
Screens
7.5
19.5
25.5
35
44.2
71
80.8
96.7
111
116
152
Clay w/ very fine sand
Fine sand
Medium to fine sand
Medium sand
Fine to medium sand w/kankers
Fine to medium sand
Medium sand w/kankers
Medium sand
Fine sand
Medium sand w/kankers
Medium sand w/kankers
Clay & fine sand w/clay

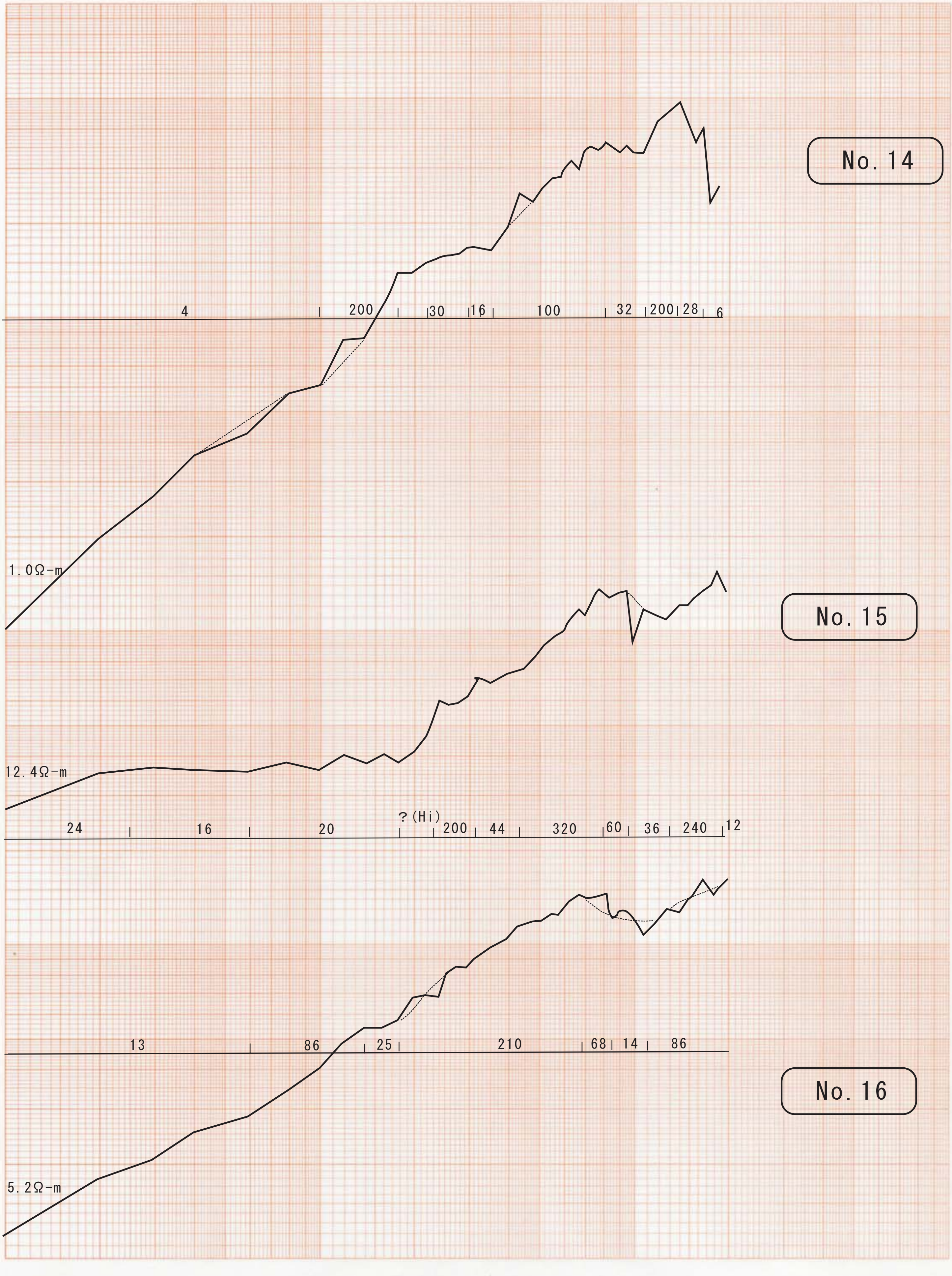

No. 07
9.8Ω-m
120
82
24
100
24
200
9
300
70
420
32
No. 08
5.1Ω-m
7
230
36
120
18
190
40
12
220
8
180


No. 09
8.4Ω-m
320
180
8
100
14
?
8
52
14
300
110
9


No. 10
4.1Ω-m
15
80
300
9
460
76
26
140
26
120
10


No. 11
4.7Ω-m
6
132
36
160
32
56
45
72
34
14
100
42
176


No. 12
9.7Ω-m
4
46
14
52
9
116
11
34
15
3
No. 13
14.4Ω-m
?(Hi)
3
420
22
24
240
28
330
34
110

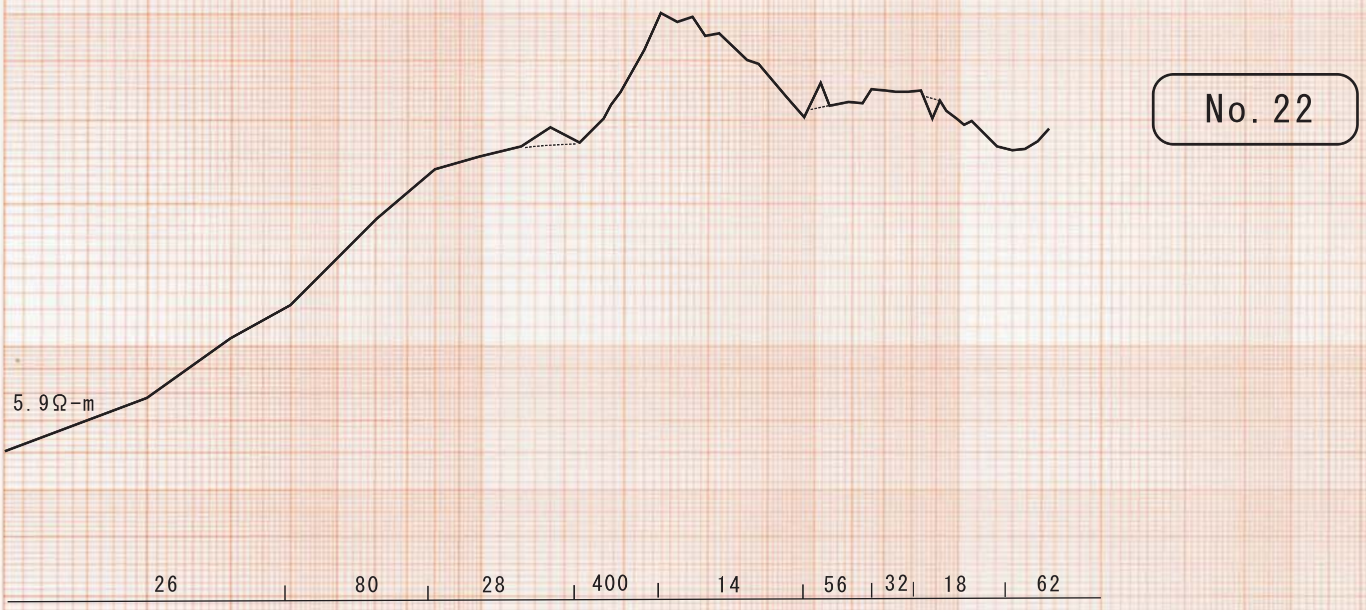

No. 14
4
200
30
16
100
32
200
28
6
1.0Ω-m
No. 15
12.4Ω-m
24
16
20
?(Hi)
200
44
320
60
36
240
12
No. 16
13
86
25
210
68
14
86
5.2Ω-m


No. 17
4.9Ω-m
64
28
22
72
210
12
300
7
No. 18
11.3Ω-m
18
84
12
140
42
120
28
160
10
240
2


12.3Ω-m
No.19
52
25
82
31
16
340
8
34
320
7


11.0Ω-m
No.20
13
42
6
40
10
22
72
22
120
50
12
8
27


No. 21
7. 1Ω-m
3
82
10
34
260
42
28
34
26
46


No. 22
5. 9Ω-m
26
80
28
400
14
56
32
18
62


No. 23
5.6Ω-m
38
120
16
7
18
550
10
No. 24
11.7Ω-m
15
42
22
8
110
14
82
?(Hi)
10

Appendix 5-4　Summary of Test Drilling/Aquifer Test

(1) Test Drilling Program

1) Components of the test drilling program

Test drilling in the second stage of this study consisted of the following works:

Table 1　Components of Test Drilling for the Study

	Test holes	Q'ty	Specification	Pumping test
1	Test well	1	Depth :150m Hole dia. 24" Casing: steel pipe, 20" and 10" Screen:slotted pipe of brass, 10"	a. Step Drawdown test b. Time drawdown test at a constant discharge rate (48hrs) c. Recovery test
2	Observation well	2	Depth: 120m Casing, screen: PVC make	d. Interference test (Simultaneous measurements of drawdown at observation wells)

2) Test drilling site

a. Prior to the commencement of test drilling, WASA took steps to carry out a pumping test with its own fund at an existing test well installed at RD260 by Binnie & Partners in 1970s. The drilled depth of the well is 40, tapping only the upper part of the aquifer. The test results showed it has a similar capacity as irrigation wells, discharging about 1 cusec.

b. The existing well tested by WASA was located in the downstream section of the canal. For this study, therefore, a site in the middle section of the canal was chosen near RD246, where existed another test well installed by REC in 1980s and later closed with sand. This time WASA provided assistance in this program, installing three observation wells at the site. The section of the canal around the site had a wide space of public land, adequate enough for an extensive operation of the testing program.

c. The layout of a test well, and 5 observation wells at the site is referred to in the map of Fig. 2-2 in the main report.

3) Summary of drilling work

The drilling work for the installation of the test well and observation wells showed the following geological features of the wellfield.

a. At the drilling site of the test well, formations composing the main aquifer lie below 40m, featuring alternate beds of medium and fine sand, which corresponds to the second section of the aquifer identified as a result of the geophysical survey.

b. However, the site lacks the third section at the lower section than 120m, where clay and silt is predominant. Accordingly the test well was completed with casing and screen to a depth of 130m.

c. Lithology of the test well is shown in Fig. 2-3 in the main report

(2) Aquifer Test

After the drilling work was completed, the aquifer test consisting of a series of pumping tests was conducted, involving the test well and 5 observation wells. The summary of testing is as follows:

1) Step drawdown test

This test examined the safe yield and the well efficiency of the drilled test well. The test used four different discharge rates (steps) for examining the drawdown at the respective rates as follows:

First step	1.50 cusec
Second step	2.25 cusec
Third step	3.09 cusec
Fourth step	3.75 cusec

As a result of the test, the highest rate is still in the range of safe yield. The capacity of the test well is in the same level or more than those of the existing wells in the Chenab wellfield discharging 4 cusec/well.

2) Time drawdown test

a. The time drawdown test (or "sustained yield test") was carried out at a constant discharge rate of 3.0 cusec for 48 straight hours. While pumping continued, drawdown at the respective observation wells was monitored through the simultaneous measurements of their levels. With a static water level at 5.3m, the drawdown after 48 hours of pumping was 2.5m at the test well.

b. The test showed a remarkable performance of recharge from the nearby canal during the test, with the level at the well stabilizing in about 360 minutes after pumping started. After this time, the well kept the same level until pumping is stopped. That means recharge from the canal equaled to the rate of discharge, 3 cusec as long as pumping continued.

c. As a result of the time drawdown test, the two key factors related to the functions of aquifers, "T"=*coefficient of transmissibility* and "S"=*coefficient of storage*, were calculated to estimate various performances of proposed tubewells and to predict on their influence to the vicinity of the wellfield. The sizes of T and S thus obtained turned out to stand in a similar range as those calculated in the previous studies. Hydraulic calculation in this report employed the factors from the latest test, referring to those in the previous tests.

3) Time recovery test

The time recovery test followed the time drawdown test, with the measurements of water level starting just after the pump is stopped. The measurements of the recovery of level were simultaneously made at the test well and 5 observation wells until their levels returned just or near to the initial ones. "T" and "S" were calculated from the test results in a similar manner as those of the time drawdown test.

4) Water analysis

Samples were taken from the test well at the end of the time drawdown test for analysis by WASA Laboratory, which resulted in an acceptable quality of a TDS concentration of 480 mg/lit. On the left side of the Jhang Branch Canal, however, the quality of groundwater tends to worsen towards the city and WASA set the upper limit of TDS concentration up to 1,000 for tubewells in this area. Since the tubewells are to be installed just beside the left bank of the canal, the better quality is likely to be ensured, thanks to seepage from the canal, as WASA experienced in the wellfield along the Rakh Branch Canal within the city.

Appendix 5-5 Analysis of Step Drawdown Test

1. Purpose of step drawdown test

Step drawdown test was conducted in this study as a part of aquifer test for various purposes. In this test the pumping rate is stepped up successively and drawdown for each step is recorded.

Its purposes are focuses on the following:

(a) to define the most appropriate pumping rate

(b) to calculate head loss of groundwater when it moves from an aquifer towards the well (called "*aquifer loss*": "*B*") and another loss when water is pumped through the well screen (called "*well loss*" "*C*"). The sum of the well loss and the aquifer loss is drawdown of the well.

(c) to determine the well efficiency "*E*" (the ratio between aquifer loss and well loss at a certain pumping rate. If 2 sets of values of "E" at different periods are compared, it is possible to quantify and evaluate the degradation of well capacity.)

B, C and E are the indices of the tubewells as follows:

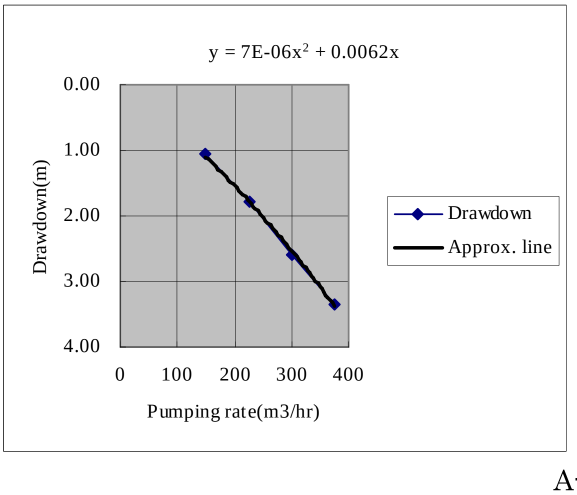
- if B is increasing the loss of the aquifer is increasing
- if C is increasing the loss of the well is increasing
- through the comparison of the ratio C/B, it is possible to evaluate which loss is increasing
- by comparing E, it would be possible to find out how much the efficiency has changed
- larger E indicates better well efficiency

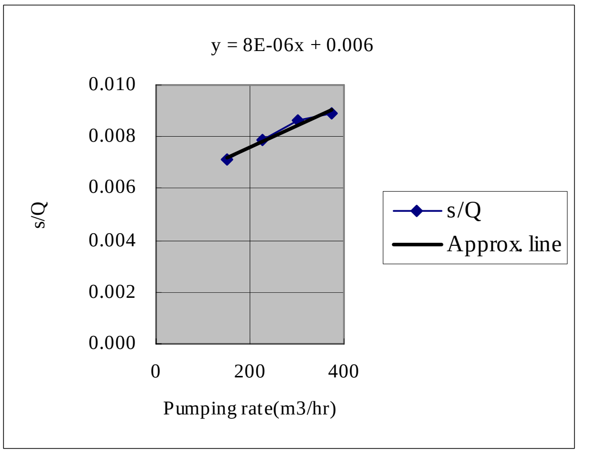
1-1 Analysis of the relationship between pumping rate and drawdown

(1) Evaluation of optimum pumping rate

From the results of the step drawdown test of this survey, the graphs showing the relationship between the pumping rates (steps) and drawdown, and the relationship between the pumping rates and the reciprocal of specific capacity are plotted in Fig. 1.

Fig. 1 Analysis of Step Drawdown Test





In this test the maximum pumping rate was set at 3.7cusec. The analysis showed the change in the pumping rate produced little change in the specific capacity, resulting in the same range of aquifer loss, even in this maximum pumping rate. It indicates the maximum rate is still within a range of safe yield.

(2) Calculation of aquifer loss (B) and well loss (C)

The relationship between B and C is given as below:

$s = BQ + CQ^2$

where

s = drawdown level (measured value)

Q= pumping rate (measured value)

B= aquifer loss coefficient (BQ = aquifer loss)

C= well loss coefficient (CQ^2= well loss)

The results of the calculation are listed in the following table:

Table 1 Coefficient of Losses in the Test Well

Coefficient	unit	Primary approximation
B	hr/m^2	0.0059
C	hr^2/m^5	9.00E-06
C/B	$1/m^3$	1.53E-03

- From the results of the calculations of B and C, it turned out that drawdown was mainly caused by aquifer loss. The drawdown occurs in proportion to the increase of the pumping rate. The limit of the rate is around $600m^3$, and if the pumping rate goes over this volume, drawdown caused by the well loss dramatically increases.
- At a pumping rate of $200m^3$/hr, proposed for the project, aquifer loss is 1.18m and well loss is 0.36m, meaning that the drawdown will be approximately 1.5m. The reason why the drawdown is small even if the pumping rate is great is that the ability of the aquifer is excellent.

1-2 Calculation of well efficiency E

Well efficiency is calculated from B and C, and by comparing with the data of the existing wells in the Chenab wellfield, the performance of the planned tubewells can be evaluated.

From the formulation E=1/ ［1+(C/B)*Q)］, well efficiency at each pumping rate is calculated in the following table

Table 2 Well efficiency of the test well

Pumping rate	Well efficiency E
m^3/hr	E=1/［1+(C/B)*Q)］
0	
150	0.84
225	0.77
300	0.72
375	0.67

In the study area around the planned wellfield, there are existing tubewells such as those of the ADB project with a similar structure to the planned wells, drilled to the depth targeted by this project as well. So the well efficiencies of those wells are compared for analysis as follows:

(1) Comparison of the Sepcification of the Tubewell

Table 3 Comparison of the structure of the test well and existing well

	unit	This survey	ADB 18	ADB 23	NSC
Well depth	m	120	128	the same as No.18	95
Screen material		Brass	Johnson	Johnson	Unknown
Screen length	m	60	48.7	the same as No.18	50
Screen opening area	%	6	15 (not verified)	the same as No.18	12
Slot size	mm	1	1 (not verified)	the same as No.18	1.5

(2) Comparison of Coefficients of Loss

Aquifer loss and well efficiency of the test well and the existing wells are compared in the table below.

Table 4 Comparison of Loss Coefficients

	unit	This study	ADB 18	ADB 23	NSC
B	hr/m^3	0.0059	0.0114	0.0116	0.0095
C	hr^2/m^5	9.00E-06	2.00E-06	1.00E-06	4.00E-06
C/B	1/m^3	1.53E-03	1.75E-04	8.62E-05	4.21E-04

Table 5 Comparison of well efficiency (E)

This study		ADB 18		ADB 23		NSC	
pumping rate	This study	pumping rate	ADB11	pumping rate	ADB11	pumping rate	NSC
m^3/hr	E	m^3/hr	E	m^3/hr	E	m^3/hr	E
0		0		0		225	0.88
150	0.84	300	0.95	300	0.95	306	0.86
225	0.77	400	0.93	400	0.93	356	0.84
300	0.72	500	0.92	500	0.92	397	0.82
375	0.67	600	0.90	600	0.90	459	0.80

- The aquifer loss coefficient of the test well is a half (1/2) of ADB No.18 / 23 wells, and two thirds (2/3) of the NSC well.
- On the other hand, the well loss coefficient of the test well is four times to that of ADB No.18/23 wells and a half of the NSC well.

As a result of the analysis of step drawdown test, the capacity of the planned tubewells along the Jhang Branch Canal is estimated as follows:

- The aquifer around the Jhang Branch Canal is better in performance than that in the Chenab wellfield.

- The test well was installed with screens of brass make with 6 % of open area, The tubewells for the ADB project used wire-wound type stainless steel screen with around 20% of open area and NSC had screens with 12 % of open area. The percentage of open area is proportional to well efficiency.
- Taking the above analysis into account, wired type screen is planned for the well screen in this project. However, it must be noted that well loss coefficient depends on the penetration ratio against the whole thickness of the aquifer, in addition to percentage of the open area. Since the wire-wound type stainless steel screen having enough length will be expensive, so economic factors must be taken into account for final determination.

Appendix 5-6 Examination of Aquifer Coefficients

(1) AQUIFER TEST

The aquifer test in this study consisted of step drawdown test (Appendix 5-5) , time drawdown test followed by time recovery test for estimating the characteristics of the aquifer through hydraulic calculation, employing the coefficients of aquifer, “T” and “S” deriving from the analysis of the results of the latter two tests. These tests were carried out, involving one test well (completed depth, 120m) and 2 observation wells (120m), together with 3 observation wells (18.5m) provided by WASA.
The summary of time drawdown test and time recovery test was as follows:

Table 1 Summary of Aquifer Test

Well for testing	Time drawdown test	Time recovery test
	The test well was pumped at a constant rate of 300m^3/min for 48 consecutive hours (2,880 minutes)	It followed time draw test immediately after the pump was stopped. Duration of test was 720 minutes.
1. Test well	Continuous measurements of discharge and water level	The level recovered to its static water level in 360 minutes after the pump was stopped.
2. Observation well No. 1	Continuous measurements of water level	Residual drawdown was 0.0254m in 720 minutes.
3. Observation well No. 2	Continuous measurements of water level	Recovered to its initial water level in 720 minutes
4. Observation well No. 3	Continuous measurements of water level	Recovered to its initial water level in 720 minutes
5. Observation well No. 4	Continuous measurements of water level	Recovered to its initial water level in 720 minutes
6. Observation well No. 5	Continuous measurements of water level	Residual drawdown was 0.0254m in 720 minutes

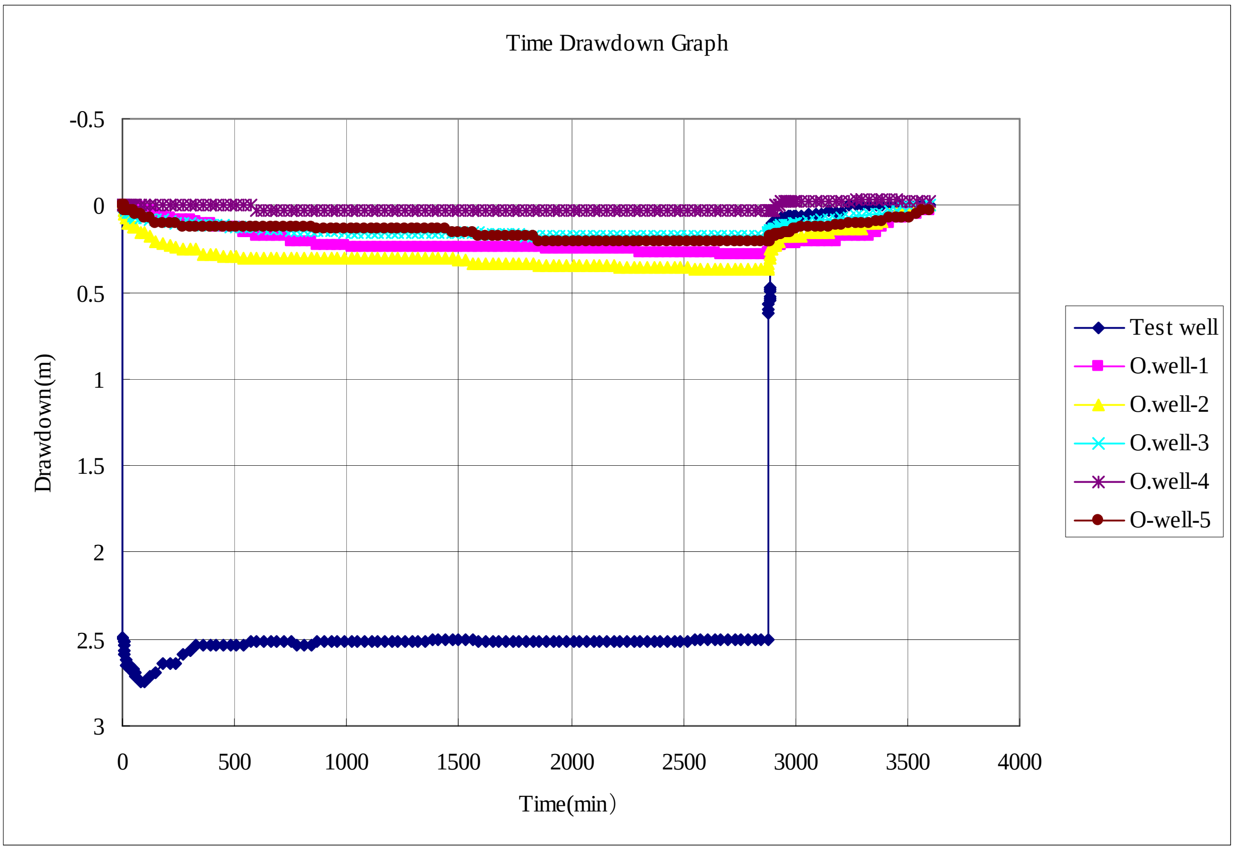
(2) Summary of Test Results

The results of the tests at five wells, relation of time to drawdown, are plotted in Fig. 1.
The specific features of the test results were as follows:

a. The pumping (dynamic) water level at the test well stabilized in 100 minutes after pumping started. Thereafter no further lowering of the water level occurred.

Static Water level	5.334m
Stabilized level	7.8486m
Drawdown	2.50m

Fig. 1 Time drawdown graph



The dynamic water level at the test well was stabilized at a depth of 7.8486 with drawdown of 2.50m. The same level was maintained thereafter until the end of pumping in 2,880 minutes. It indicates that soon after the pumping started, the forced recharge from the canal began as the enlarged cone of influence encountered the source of recharge.

b. The same characteristics were witnessed at the observation wells, notably in No. 1 and No. 2 installed beside the channel of the canal.

c. Such a recharged condition encountered during the test made it difficult to analyze the obtained data. The following data, however, are considered useful for hydraulic analysis.

* Data on the relation ship of time and drawdown at each well before they received recharge, namely test data from the start of pumping to 100 minutes of continuous pumping (However, since the data from a single well was found irrelevant for analysis, the time-distance relationship of 5 wells was employed

* Data of recovery test at the observation wells, No. 2 and No. 3 (Those of the test well and other observation wells were found out irrelevant, with their hydraulic analysis yielding unpractical results.)

d. Since the remarkable effect of recharge affected a greater part of the test results, the hydraulic calculation in this study refers to the results of the past studies, particularly those by REC, carried out in 1980 at the same location along the canal. (The test well was installed just beside the REC's test well, now abandoned due to clogging by sand deposits inside.)

(3) Calculation of Aquifer Coefficients

Based upon the selected data and method as explained in the foregoing section, the coefficients of an aquifer, "T", Coefficient of Transmissibility, and "S", Coefficient of Storage, were calculated.

a. Coefficient of Transmissibility

Coefficient of Transmissibility, T, of an aquifer indicates how much water will move through the formation. It is the rate at which water will flow through a unit vertical section of the aquifer and extending through the full saturated thickness under a hydraulic gradient of 1.00. It equals the *coefficient of permeability, K*, multiplied by the thickness of an aquifer. In this study, the Jacob-Cooper method was employed for the calculation of "T" and "S". The unit is generally m^2/day.

$$T = \frac{0.183Q}{\Delta s} \qquad (1)$$

where,

Q = Discharge (m^3/day)

Δs = Slope of the time-drawdown graph expressed as the change in drawdown between any two values of time on the log scale whose ratio is 0. (m)

b. Coefficient of Storage, S, of an aquifer indicates how much water can be moved by pumping through a unit cross section of an aquifer.

$$S = \frac{2.25Tt_0}{r^2} \qquad (2)$$

where,

T = Coefficient of transmissibility, from equation (1) (m^2/day)

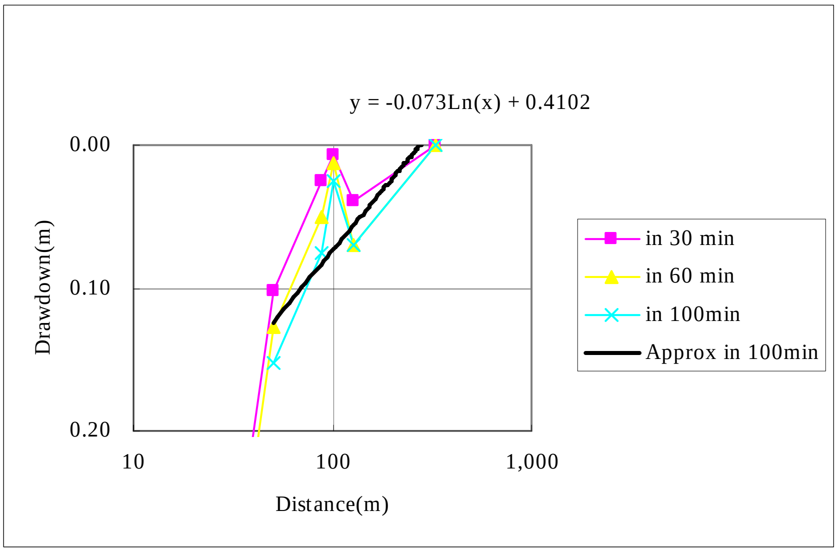
t_0 = Intercept of the straight line at zero drawdown (day)

r^2 = Distance from pumped well to observation well where drawdown measurements were made (m)

1) Calculation –I, based upon the relationship of the test well and the observation wells during time drawdown test

Case I-1: Calculation based upon the distance-drawdown relationship among 5 observation wells, excluding the test well, employing the data for 100 minutes after the pump started, while the influence of recharge remained inconspicuous.

Fig. 2 Approximate Relationship of Drawdown versus Distances of Observation Wells



Through the graphical analysis, the relationship of distances of the observation wells with drawdown is approximated into an equation (slope), y=0.073Ln(x) +0.4102. From this equation, drawdown per a unit log cycle of time is calculated as ⊿s=0.168 m.

As a next step, the calculation of "T" employs the following equation derived from the previous basic equation (1) as follows:

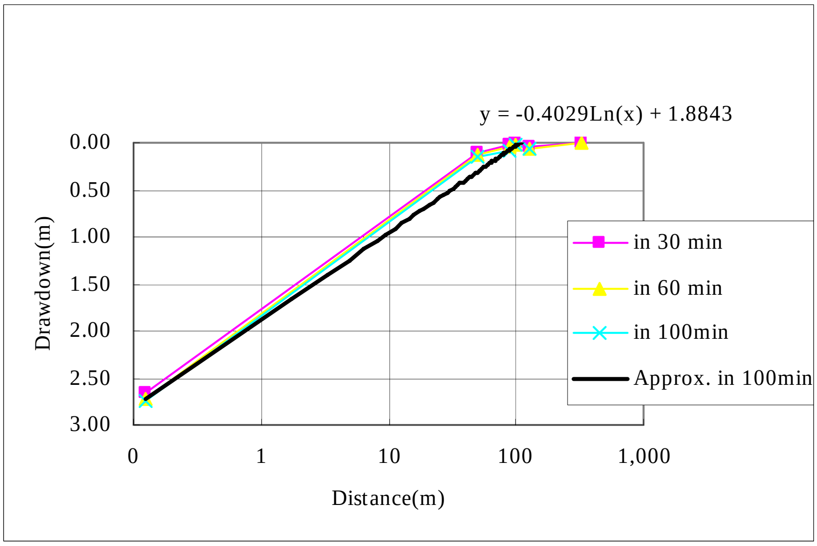
$$T = \frac{0.366Q}{\Delta s} = \frac{0.366 \times 7{,}200 \text{ m3/day}}{0.168} = 15{,}686 \text{ m2/day} = 654 \text{ m2/hr}$$

The calculation of "S" is as follows, based upon the equation (2).

$$S = \frac{2.25xTx\ t}{r_0\,\hat{}\,2} = \frac{2.25 \times 15{,}686 \text{ m2/day} \times 6.94E\text{-}02 \text{ day}}{276\ \hat{}\ 2} = 3.22E\text{-}02$$

2). Calculation –II: Calculation based upon the data at 5 observation wells plus the test well

Fig. 3 Approximate Relation ship of Distances and Drawdown
of Test Well/Observation (Data from the start of pumping to 100 min.)



The same process as for the Calculation-I is applied.

*Approximate relationship of the distances of 1 test well and 5 observation wells and drawdown is :
y = - 0.4029Ln(x) +1.8843

* ⊿s = 2.30 x 0.4029 = 0.928 m

Therefore,

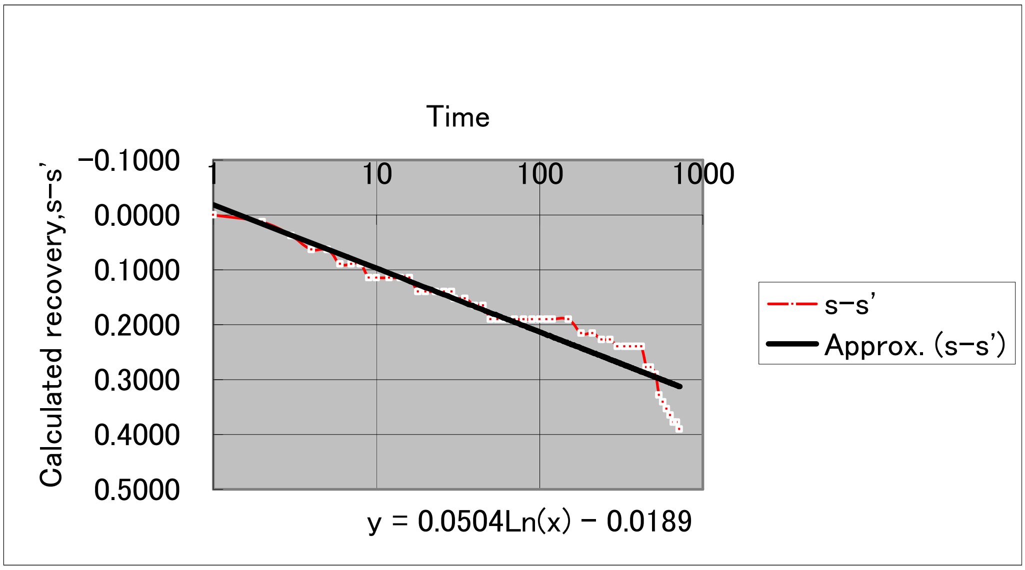
T	＝	0.366Q	＝	0.366	×	7,200	m^3/day	＝	2,840	m^2/day	＝	118	m^2/hr
		⊿s			0.928								

S	＝	2.25xTx t_0	＝	2.25	×	2,840	m^2/day	×	6.94E-02	day	＝	3.88E-02
		r^2						107	^ 2			

3) Calculation-III, based upon the data of recovery test at observation well No. 2

The time-recovery relationship at the observation well No. 2 is plotted in Fig. 4. For the calculation of the coefficients based upon this relation, the calculated drawdown instead of residual drawdown was used. The vertical axis indicates this calculated drawdown (s=extended drawdown assuming pumping continued after it was stopped) – (s'=Residual drawdown)..

Fig. 4 Time Recovery Graph of Observation Well No.2



From the time recovery graph,

*Approximate relationship (s-s')=0.0504Ln(t) - 0.0189

*Drawdown per a unit log time cycle ⊿(s-s')=0.1008m、

*Intercept of the line at zero drawdown

t^0=1.038212min=7.2E-04day

*Distance between the test well and the observation well No. 2

r = 50m

*Discharge at the test well Q=300m^3/hr=7,200m^3/day

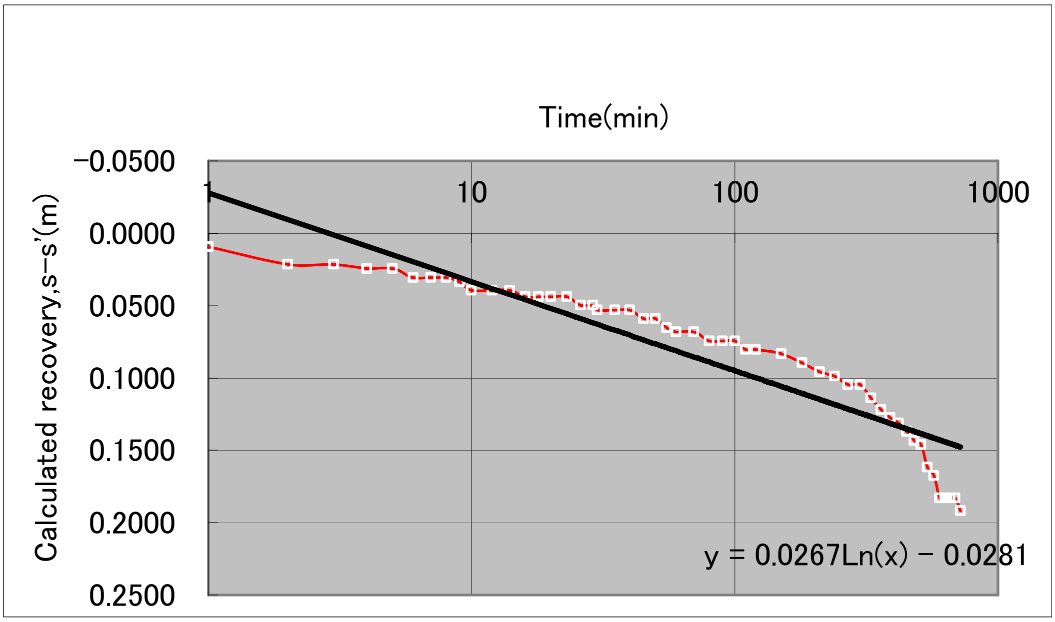
The following equation is used for the calculation of “T” and “S”.

$$T = \frac{0.183Q}{\Delta(s-s')} = \frac{0.183 \times 7,200\ m^3/day}{0.1008} = 13,071\ m^2/day = 545\ m^2/hr$$

$$S = \frac{2.25xTx\ t_0}{r^{\wedge 2}} = \frac{2.25 \times 13,071\ m^2/day \times 7.20E-04\ day}{50\ ^{\wedge}2} = 8.47E-03$$

4) Calculation IV: based upon the data of recovery test at observation well No. 3

Fig. 5 Time Recovery Graph of Observation Well No.3



From the graph in Fig. 5

* Approximate relationship (s-s')=0.0267Ln(t) - 0.0281
* Drawdown per a unit log time cycle ⊿(s-s')=0.0534m、
* Intercept of the line at zero drawdown t^0=2.865min=2.0E-03days
* Distance between the test well and the observation well No. 3 r = 89m
* Discharge at the test well Q=300m^3/hr=7,200m^3/day

The calculation of "T" and "S" is as follows:

$$T = \frac{0.183Q}{\Delta(s\text{-}s')} = \frac{0.183 \times 7,200\ m^3/day}{0.0534} = 24,674\ m^2/day = 1,028\ m^2/hr$$

$$S = \frac{2.25xTx\ t_0}{r^{\hat{}2}} = \frac{2.25 \times 24,674\ m^2/day \times 2.00E\text{-}03\ day}{89^{\ \hat{}\ 2}} = 1.40E\text{-}02$$

(4) Results

The results of the foregoing calculations are shown in the following table, together with the data obtained in the studies in the past, REC's study in 1980 and drilling reports of the ADB project wells.

Table 2 List of Coefficients of Aquifer in the Study Area

Test wells	Method for calculation	Coefficient of Transmissibility, "T" (m^2/day)	Coefficient of Storage "S" (non dimensional)
This study (one test well & 5 observation wells)	*Distance-drawdown analysis(1) of 5 observation wells	15,686	3.22E－0.2
	*Distance-drawdown analysis(2) of one test well/5 observation wells	2,840	3.88E－0.2
	*Recovery method (No. 2 well)	13,071	8.47E－0.3
	*Recovery method (No. 3 well)	24,674	1.40E－0.2
Study by REC	*Recovery method (RTW1 well)	5,312	2.50E－0.2
	*Recovery method (RTW2 well)	7,080	1.27E－0.2
ADB tubewells in the Chenab	*Pumping test (No. 18 well)	11,094	
	*Recovery method (No. 18 well)	9,861	

Note: The data for the ADB tubewells is from the report of completion of drilling works. The coefficient of storage is not calculated. The report shows that an average value of "T" from the pumping tests at 23 tubewells was 12,000m^3 /day.

With the coefficients of the aquifer calculated on the basis of the test in this study, the following points should be taken into due consideration.

a. All of the wells for testing in this study were installed in the site close to the canal channel. The water levels were more or less affected by rapid recharge from the canal. Accordingly, the results of the calculation may involve the influence of recharge, although the data was selected, and their accuracy appears limited to a certain extent.

b. Although the results of the calculation may not honestly represent the characteristics of the aquifer, the values of "T" and "S" are still within a tolerable range, compared to those presented by the past studies.

c. Storage coefficients, which are in the order to the minus 1 st to minus 2nd power of 10, generally indicate the characteristics of an unconfined aquifer, while smaller numbers in the order to the power of the minus 3rd power of 10 are interpreted to mean confined aquifers. Except for the case of No. 3 observation wells, all the calculated values of "S" indicate the aquifer in the study area is classified as unconfined one.

As a result of extensive hydrogeological studies in the study area, it is known that the aquifer there is basically of unconfined nature. The test well tapped the aquifer deeper than 40m underlying beneath an impervious silty bed. It is partly or locally in confined condition, and deeper groundwater may feature a flow pattern differing from the one of shallow groundwater under command of canal recharge. In this view, the aquifer in this area partly bears a nature of semi-confined formations.

Appendix 5-7 Examination of Extent of Influence

The extent of influence by pumping at the planned tubewells is examined, employing the values of "T" and "S" in this section.

(1) Equation for the Calculation

The equation for the calculation of the radius of influence is derived from that for "S", Storage Coefficient.

$$S = \frac{2.25xTx\ t}{r_0\hat{}2}$$

where,
S= calculated value of storage coefficient
T=calculated value of transmission coefficient (m^2/day)
t= Duration of pumping (day)
r_0= Radius of influence (m)

The foregoing equation is modified in the following relationship:

$$r_0 = SQRT \frac{2.25xTx\ t}{S} \qquad (3)$$

(2) Calculation of Radius of Influence

The radius of influence (r_0) is calculated, based upon the equation (3), assuming the following conditions:

*T and S	various values of "T" and "S" calculated in Appendix 4-7
*Q=discharge	the unit rate of discharge of the project wells =200 m^3/hr=7,200m^3/day
*Duration of pumping	20 hours a day in accordance with the plan for the project (However, since a part of values employed the data from the start of pumping to 100 minutes, r_0 in 100 minutes was also calculated for reference.

The results of the calculation are listed in the following table.

Table 1 Calculated Radius of Influence

(Discharge =200m^3/hr、Pumping duration=20hrs/day)

Origin of "T" & "S"	Method of Calculation of T and S	T (m^2/day)	S	Radius of influence (m)	
				in 100	in 20 hrs
This Study: Test well and observation wells	Distance-drawdown method (1), observation wells only	15,686	3.22E-02	277	954
	Distance-drawdown method (2)test and observation wells	2,840	3.88E-02	65	369
	Time-recovery method (Observation well No. 2)	13.071	8.47E-03	299	1,697
	Time-recovery method (Observation well No. 3)	24,674	1.40E-02	320	1,813
REC's Study test well RTW1	Time recovery method	5,312	2.50E-02	111	630
RTW2	Time recovery method	7,080	1.27E-01	56	323

As a result of the calculation in Table 1, the following situation can be estimated:

a. This study proposes 600m for the distances of the respective tubewells. If a radius of influence is less than 300m, half of 600m, the interference of neighboring tubewells will not occur. According to the calculation, all the radii extend beyond this limit at the end of pumping for 20 hours, ranging from a minimum 323 to a maximum 1,813m

b. However, during the test, the water levels at test well and observation were stabilized in about 100 minutes after the start of pumping due to direct recharge from the canal, and they remained at the same depth until the end of pumping for 48 hours. This means the test well received recharge equaling to discharge, and its effect extended to the observation wells. It suggests that the radius of influence of pumping at the respective wells no more enlarges.

   Compared to the radii of influence in 20-hour pumping, those in 100 min are all within 300m with one exception slightly over 300m. Therefore, it is highly possible that the radius of influence can remain within 300 m for a duration of pumping for 20 hours at a rate of 200m^3/hr, with no interference occurring among the neighboring wells..

c. On the other hand, the canals are all closed during the winter season for about one month for their maintenance and repair. There is no recharge during this season, and the first calculation becomes realistic.

Since the closure of canals continues one month, the extent of influence of pumping at the project wells were estimated, employing the same conditions as for the preceding calculation.

Table 2 Extent of Radius Influence during Canal Closure

Origin of "T" & "S"	Method of Calculation of T and S	T (m^2/day)	S	Radius in 30 days(m)
This Study: Test well and observation wells	Distance-drawdown method (1), observation wells only	15,686	3.22E-02	5,736
	Distance-drawdown method (2)test and observation wells	2,840	3.88E-02	2,437
	Time-recovery method (Observation well No. 2)	13.071	8.47E-03	4,914
	Time-recovery method (Observation well No. 3)	24,674	1.40E-02	6,708
REC's Study test well RTW1	Time recovery method	5,312	2.50E-02	3,337
RTW2	Time recovery method	7,080	1.27E-01	3,852

As a result, it is estimated that before the canals restart delivery, the radius of influence enlarges as far as 4.5 km from the wellfield as an average of the calculation results, causing the lowering of regional groundwater level. The amount of lowering in this case is examined in the following section.

(3) Lowering of Groundwater Level during Canal Closure

For the purpose of predicting the lowering of water levels related to distances, the previous approximation of their relationship presented in Fig. 3 in Appendix 5-6 is used. (the distance-drawdown relationship based upon the test well and observation wells)

The equation is as follows:

$$s = -0.4029 \times \mathrm{Ln}(r) + 1.8843 \qquad (4)$$

where

s = drawdown (m)

r = distance from test well (m)

Since the approximation was based upon a discharge rate of 300m$^{3/}$hr, the equation is modified to adapt to a situation at a discharge of 200m$^{3/}$hr, as follows:

* Unit discharge rate 200m$^{3/}$hr
* Duration of daily operation 20 hours
* Daily discharge per well 4,000m$^{3/}$day
* Ratio of planned discharge to testing discharge =4,000/7,200 =0.556

Under these conditions, the slope of the approximation is modified as follows:

$$s = -(0.4029 \times 0.556) \times Ln(r) + 1.8843 = -0.224Ln(r) + 1.8843 \quad (4)'$$

The equation (4)' is expressed in a simple form as follows:

$$s = A \times Ln(r) + C \quad (5)$$

For the calculation, the following conditions are assumed:

* In the radius of influence, drawdown =0m S0 (m)
distance from pumped well to S0r0 (m)
* In the radius of influence, drawdown=1m S1 (m)
distance from pumped well to S1r1 (m)

(Drawdown of 1m is assumed for the calculation, since that amount of drawdown is the critical range for many irrigation tubewells.)

From the equation (5), therefore,

$$s_0 = A \times Ln(r_0) + C$$
$$s_1 = A \times Ln(r_1) + C$$

Combining the two equation,

$$s_1 - s_1 = A \times Ln(r_1) - A \times Ln(r_0) = A \times Ln(r_1/r_0)$$

Therefore,

$$r_1 = r_0 \times EXP(s_1 - s_0/A) \quad (6)$$

The equation (6) thus derived is employed for the calculation of estimated drawdown related to the distance. As a specific condition for the calculation in this study, the drawdown is assumed as 1m. The results of the calculation are shown in the following table:

Table 3 Radius of Influence of Pumping at a Project Tubewell and Predicted Drawdown In the Surroundings in One Month after Canal Closure

(Unit discharge rate =200m^3/day/well、20-hour operation/day)

	Method of calculation T & S	T (m^2/day)	S	Radius of influence (m) Drawdown 0m	0.25m	0.5m	1.0m
This study	Distance-drawdown method (1), 5 observation wells	15,686	3.22E-02	5,736	1.879	615	66
	Distance-drawdown method(2): test and observation wells	2,840	3.88E-02	2,437	727	238	26
	Time-recovery method (Observation well, No. 2)	11.520	7.47E-03	4,914	3,343	1.095	118
	Time-recovery method (Observation well, No. 3)	21,466	1.22E-02	6,708	3.573	1,170	126
REC-	Time recovery method (RTW1 well)	5,312	2.50E-02	3,337	1,240	406	44
	Time recovery method (RTW1 well)	7,080	1.27E-01	3,852	635	208	22

To predict the influence of pumping during the closure of canals based upon the results of calculation in the foregoing table, the two sets of values out of four resulting from this study are selected, as they are in the medium range and nearly correspond to those from the REC's study, namely the results employing T and S derived from the distance-drawdown analysis of observation wells only, and those from the same analysis involving the test well. Since the former is larger than the latter, it is assumed to take the former as a maximum and the latter as a minimum. Reality may further converge into the middle range of these two.

a. In case of the minimum influence, groundwater level will be lowered by 0.25m at a distance of 730 m from a pumped well in 30 days after the canal is closed. In case of the maximum, the lowering of the same level will be seen at a distance of 1,900m from the well. In either case, the range of the radius is more than 300m, the lowering will increase due to interference of adjacent wells. The actual drawdown will be nearly doubled.

b. It is not clear whether the thus lowered levels could be restored to its initial ones before the canals are closed, after canals restarts delivery. Probability is the levels remain at their lowered depths, since the recharge from the canal was assumed to be fully consumed by irrigation wells and the project wells. In case of such a worst scenario, regional groundwater level will continue to go down, and the area at a distance of 1km from them may witness groundwater level has been lowered by 1 m within 5 years after the operation of the project wells started.

(4) Conclusion

The prediction based upon the test results in this study will not necessarily be realized, due to the limit of accuracy in the values of T and S, explained in Appendix 4-7. Moreover, since the hydraulic calculation is based upon lots of assumption, it has its own limit of accuracy. However, in this case the analysis may have yielded more optimistic results with strong aid of canal recharge. Reality may be more severe. If this assumption is true, the influence will be much more than the predicted one.
The conclusion of this analysis is as follows:

a. Pumping by the project wells will hardly affect groundwater level in the surroundings as long as the canal continues delivery.

b. However, during its closure for one month during winter, regional groundwater is likely to be affected. The range of influence is the lowering of 0.25 to 0.5 m in the area at distances of 500 to 1,000m from the project wells for the first year when the operation starts.

c. The recovery of the lowered levels after the restart of canal delivery is unknown. The levels are more likely to remain at the lowered depths, since withdrawal of irrigation wells and the project wells seems to nearly equal to recharge. In such a case, regional groundwater level will continuously be lowered. The calculation indicates that an area 1 km away from the project wells will witness the lowering of 1 m within 5 years.

Finally, aside from the influence of the project wells, this area seems under threat of another influence extending from the existing tubewells in the Chenab wellfield. This risk will be separately being examined in Appendix 5-9.

Appendix 5-8 Examination of Influence by Existing Tibewells in the Chenab Wellfield

(1) Outline of ADB Tubewells

The main water source of WASA's water supply system is currently the tubewells installed under the ADB project in the Chenab wellfield lying 5 to 10 km north of the Jhang Branch Canal. Since the commissioning in 1992, these wells have been producing discharge at a unit rate of 400m^3/hr per well, totaling 200,000 to 160,000m^3/day. The number of tubewells installed by the ADB project was 25, with four added later by WASA.

Compared to the tubewells under planning for this project, those in the Chenab wellfield were designed to discharge a rate twice the one for this project, with their distances spaced at 400m. Immediately after the operation of those well started, groundwater level in the vicinity began to lower, with the influence gradually extending to irrigation wells, and it didn't take long for the residents to find difficulties in pumping their own wells. Conflicts occurred between WASA and neighboring residents. The influence soon developed in a wide range. The water levels at the tubewells themselves have considerably been lowered these ten years.

Since the Chenab wellfield is close to the planned new wellfield along the Jhang Branch Canal with its southern end located 5 km north of the latter, there is a risk their influence might soon extend to the latter. To idenfity foreseeable influence from that area, the conditions of the existing tubewells were examined under this study in relation to its own efforts to minimize the influence by pumping to irrigation tubewells.

(2)Conditions of the ADB wells

2-1 Progress of Level Lowering

The progress of the static water levels at the respective tubewells in the Chenab wellfieldss since its commencement of operation in 1992 to 2002 is shown in the graphs in Fig. 1, in which the plots are approximated to straight lines, allowing to predict their future.

2-2 Features of Levels

1) Initial levels

The range of initial levels at the tubewells is represented by the data on No. 18 well as follows:

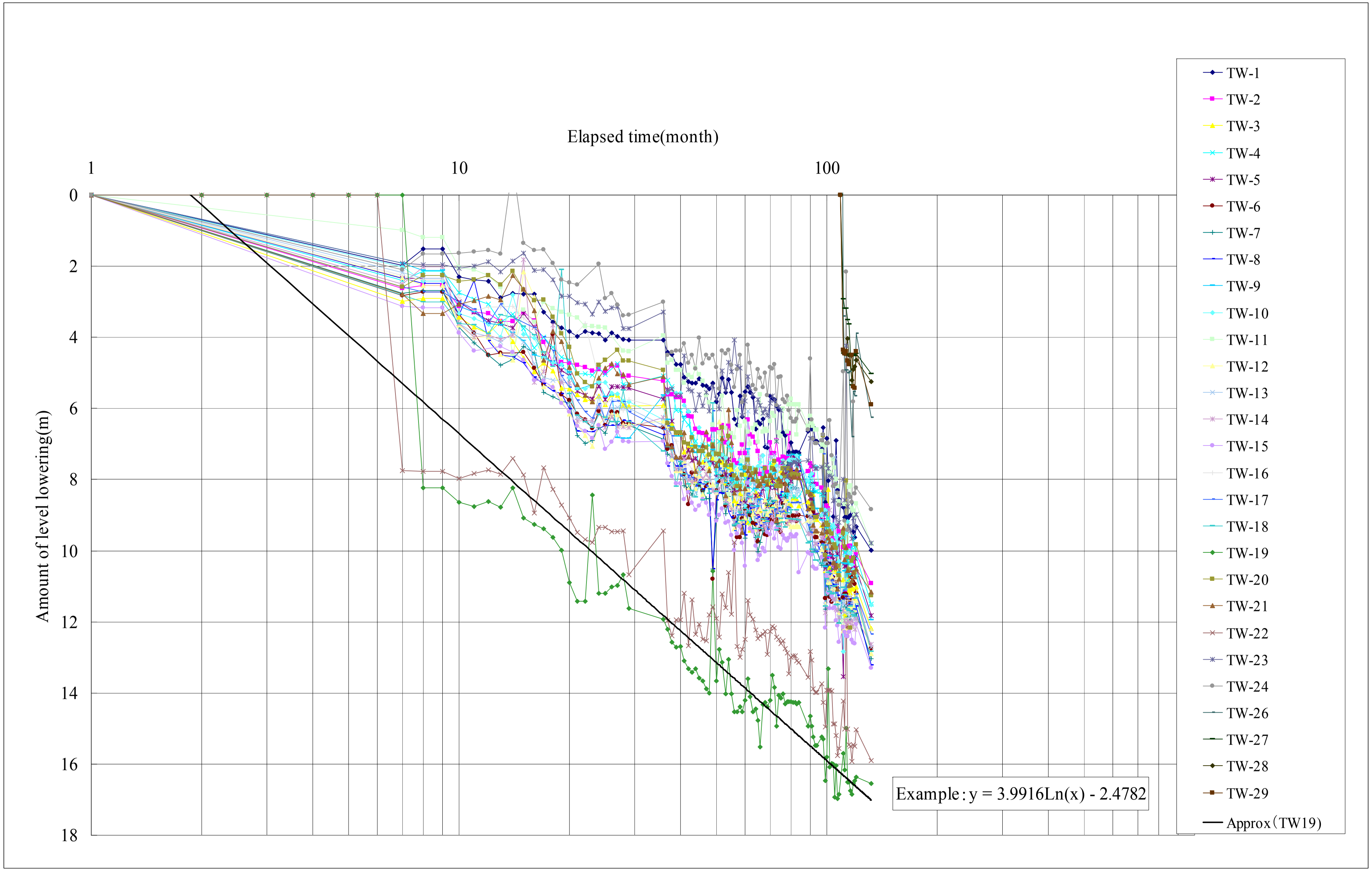
* Static water level at the time of construction 19 ft. 9 in. (6.0236m)

* Dynamic water level	35 ft. 9 in.(10.90m)
* Drawdown	16 ft (4.88m)

2) Features of change in levels

* Immediately after the start of operation, the static water levels lowered by a range of 2m to 5m. (Excessive lowering of 8m was seen at No. 19 and No. 22. The reason for this change is not clear.)
* The difference of levels among the wells was roughly 4m in the initial stage of operation. After 10 years, it enlarged to 6m.
* The progress of level lowering has the following features:
  - The levels were nearly stabilized from January 1996 to December 1998.
  - Progressive lowering occurred in the year 2000.
  - Since 2001, lowering was relaxed.

Fig.1 Progress of Level Lowering at ADB Wells(1992～2002)



2-3 Prospects of Levels in the Future

The change in levels at the tubewells is predicted on the following process.

a. The change in the water levels is approximated to the time-drawdown relationship (Refer to Fig. 1). The levels in the future are derived from the calculation of the approximate equation.

b. The static water level is represented by the one at No. 18 as an average ranges at all the wells.

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Based upon this assumption, the prospective levels of the respective wells are listed in Table 2.

Table 1 shows the list of the tubewells in the order of a larger rate of drawdown. This list indicates the following situation:

* Those installed at the periphery of the wellfield have a gentle slope of drawdown.
* Those located in the central part of the wellfield tend to have a steeper slope of drawdown. (No. 14 to No. 19)

(For the locations of the respective wells, refer to the map in Fig.2-4 of the Basic Design Study Report.)

Table 1

No.	Slope
TW25	2.232
TW24	2.3176
TW11	2.3965
TW01	2.4027
TW10	2.5242
TW02	2.5314
TW23	2.5779
TW09	2.627
TW03	2.7029
TW04	2.7294
TW05	2.72941
TW07	2.7636
TW21	2.7706
TW13	2.7889
TW08	2.7916
TW06	2.8195
TW20	2.8219
TW12	2.9394
TW17	2.9537
TW16	2.9825
TW15	3.04271
TW18	3.0478
TW14	3.0678
TW22	3.47
TW19	3.9916

Table 2 Prospect for Future Drawdown & Water Level of ADB Wells

		10 years			Level	20 years			Level	30 years			Level	40 years			Level	50 years			Level
		Drawdown			6.02	Drawdown			6.02	Drawdown			6.02	Drawdown			6.02	Drawdown			6.02
Well	Slope	m	y	Log e(m)	(m)	m	y	Log e(m)	(m)	m	y	Log e(m)	m	m	y	Log e(m)	(m)	m	y	Log e(m)	m
TW01	2.4027	120	10	11.50	17.52	240	20	13.17	19.19	360	30	14.14	20.16	480	40	14.83	20.85	600	50	15.37	21.39
TW02	2.5314	120	10	12.12	18.14	240	20	13.87	19.89	360	30	14.90	20.92	480	40	15.63	21.65	600	50	16.19	22.21
TW03	2.7029	120	10	12.94	18.96	240	20	14.81	20.83	360	30	15.91	21.93	480	40	16.69	22.71	600	50	17.29	23.31
TW04	2.7294	120	10	13.07	19.09	240	20	14.96	20.98	360	30	16.07	22.09	480	40	16.85	22.87	600	50	17.46	23.48
TW05	2.72941	120	10	13.07	19.09	240	20	14.96	20.98	360	30	16.07	22.09	480	40	16.85	22.87	600	50	17.46	23.48
TW06	2.8195	120	10	13.50	19.52	240	20	15.45	21.47	360	30	16.60	22.62	480	40	17.41	23.43	600	50	18.04	24.06
TW07	2.7636	120	10	13.23	19.25	240	20	15.15	21.17	360	30	16.27	22.29	480	40	17.06	23.08	600	50	17.68	23.70
TW08	2.7916	120	10	13.36	19.38	240	20	15.30	21.32	360	30	16.43	22.45	480	40	17.23	23.25	600	50	17.86	23.88
TW09	2.627	120	10	12.58	18.60	240	20	14.40	20.42	360	30	15.46	21.48	480	40	16.22	22.24	600	50	16.80	22.82
TW10	2.5242	120	10	12.08	18.10	240	20	13.83	19.85	360	30	14.86	20.88	480	40	15.58	21.60	600	50	16.15	22.17
TW11	2.3965	120	10	11.47	17.49	240	20	13.13	19.15	360	30	14.11	20.13	480	40	14.80	20.82	600	50	15.33	21.35
TW12	2.9394	120	10	14.07	20.09	240	20	16.11	22.13	360	30	17.30	23.32	480	40	18.15	24.17	600	50	18.80	24.82
TW13	2.7889	120	10	13.35	19.37	240	20	15.28	21.30	360	30	16.42	22.44	480	40	17.22	23.24	600	50	17.84	23.86
TW14	3.0678	120	10	14.69	20.71	240	20	16.81	22.83	360	30	18.06	24.08	480	40	18.94	24.96	600	50	19.62	25.64
TW15	3.04271	120	10	14.57	20.59	240	20	16.68	22.70	360	30	17.91	23.93	480	40	18.79	24.81	600	50	19.46	25.48
TW16	2.9825	120	10	14.28	20.30	240	20	16.35	22.37	360	30	17.56	23.58	480	40	18.41	24.43	600	50	19.08	25.10
TW17	2.9537	120	10	14.14	20.16	240	20	16.19	22.21	360	30	17.39	23.41	480	40	18.24	24.26	600	50	18.89	24.91
TW18	3.0478	120	10	14.59	20.61	240	20	16.70	22.72	360	30	17.94	23.96	480	40	18.82	24.84	600	50	19.50	25.52
TW19	3.9916	120	10	19.11	25.13	240	20	21.88	27.90	360	30	23.49	29.51	480	40	24.64	30.66	600	50	25.53	31.55
TW20	2.8219	120	10	13.51	19.53	240	20	15.47	21.49	360	30	16.61	22.63	480	40	17.42	23.44	600	50	18.05	24.07
TW21	2.7706	120	10	13.26	19.28	240	20	15.18	21.20	360	30	16.31	22.33	480	40	17.11	23.13	600	50	17.72	23.74
TW22	3.47	120	10	16.61	22.63	240	20	19.02	25.04	360	30	20.42	26.44	480	40	21.42	27.44	600	50	22.20	28.22
TW23	2.5779	120	10	12.34	18.36	240	20	14.13	20.15	360	30	15.17	21.19	480	40	15.92	21.94	600	50	16.49	22.51
TW24	2.3176	120	10	11.10	17.12	240	20	12.70	18.72	360	30	13.64	19.66	480	40	14.31	20.33	600	50	14.83	20.85
TW25	2.232	120	10	10.69	16.71	240	20	12.23	18.25	360	30	13.14	19.16	480	40	13.78	19.80	600	50	14.28	20.30
Ave	2.80	120	10	13.41	19.43	240	20	15.35	21.37	360	30	16.49	22.51	480	40	17.29	23.31	600	50	17.92	23.94
Max	3.99	120	10	19.11	25.13	240	20	21.88	27.90	360	30	23.49	29.51	480	40	24.64	30.66	600	50	25.53	31.55
Min	2.23	120	10	10.69	16.71	240	20	12.23	18.25	360	30	13.14	19.16	480	40	13.78	19.80	600	50	14.28	20.30

2-3 Examination of Drawdown in the Future

As shown in Fig. 2, the slope of drawdown goes gentle as time elapses. For the first 10 years, it was 10 to 19 meters. In 30 years the prospect is 16.5 to 23.5m. The levels in the future are summarized in the following table:

Table 3 Prospected Water Levels of ADB Wells in the Future

		2002		2012		2022		2032		2042	
	Slope	A	Level	A	B	A	B	A	B	A	B
Ave	2.80	13.41	19.43	15.35	1.94	16.49	3.08	17.29	3.88	17.92	4.51
Max	3.99	19.11	25.13	21.88	2.77	23.49	4.39	24.64	5.53	25.53	6.42
Min	2.23	10.69	16.71	12.23	1.55	13.14	2.45	13.78	3.09	14.28	3.59

Note: A= Amount of lowering since the start of operation in 1992

B= Cumulative drawdown after 2002

(3) Examination of the Extent of Influence

3-1 Distance-Drawdown Relationship

In order to examine the extent of influence by pumping at the ADB wells, the approximation of the distance-drawdown relationship was made in the following process.

1) Assumption

To define the distance-drawdown relationship, No. 1 well is assumed as a pumping well and No. 25 well 400 m away from No. 1, as an observation well.

2) Estimate of coefficients of approximate relationship

Drawdown is expressed by the following relationship:

Amount of level lowering = $-$ a × log e (Time) + b (A)

Amount of level lowering = $-$ a' × log e (Distance) + b (B)

(a' = 2 x a)

Coefficients (a) and (b) in the above relationship are calculated, based upon the assumption in (1), as follows:

* In the relationship (A), the value of "a" (slope of the time-drawdown relationship) at No. 25 by pumping at No. 1 is already known by the preceding approximation, which was 2.23.
*Therefore, the value of "a'" in the relationship (B) is ("a" x 2) = 4.46
*Once "a'" is defined, the value of "b" is calculated from (B), since the distance from the pumped

well No. 1 is 400m as follows

b = Amount of level lowering (each year) +"a" x log e(distance)

("b" is a variable, since the level of No. 1 as the observation well is continuously lowering.

*Radius of influence = exp^ (b/a)

The values of "a'", "b" and the radius of influence are listed in the following table:

Table 4 List of Parameters of Approximate Relationship

	0 year level	10-year level	2-year level	30-year level	40-year level	50-year level	
Slope of time-drawdown at No25	0.691	0.691	0.691	0.691	0.691	0.691	timeΔs
Slpe of time-distance at No.25	1.382	1.382	1.382	1.382	1.382	1.382	a
b	8.28	11.59	12.07	12.35	12.55	12.71	b
Radius=x at y=0	400	4,396	6,219	7,619	8,800	9,840	e ^ (b/a)

4) Long term forecast for the extent of influence

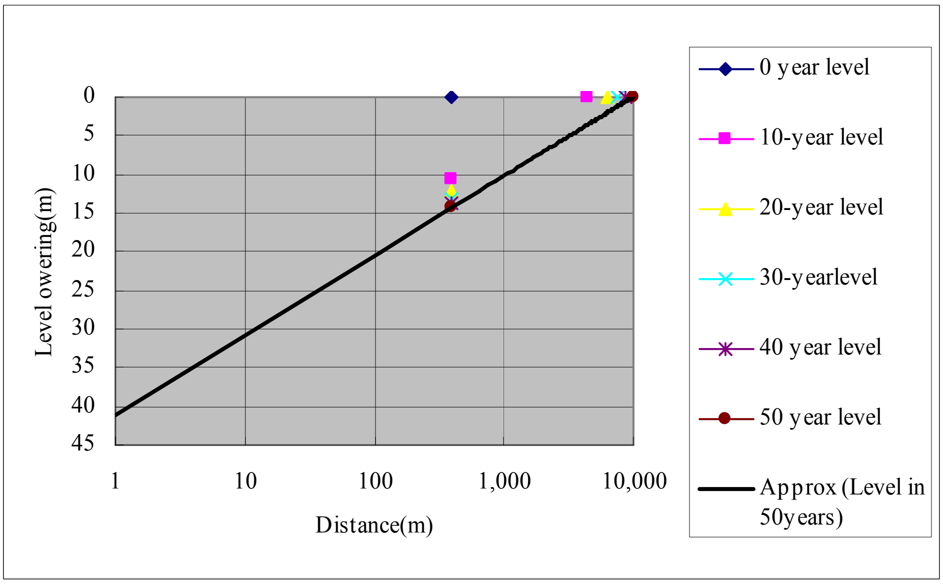
Based upon the defined approximate relationship, the radius of influence (x) in each decade is calculated in the following table:

Table 5 Forecast for Radius of Influence of the ADB Wells

Distance from No. 1Well (m)	0 year level	10-year level	20 year level	30-year level	40 year level	50 year level	
1							
10							Av
100							Max
400	0	3.31	3.79	4.07	4.27	4.43	Min
1,000							
4,391		0					
6,212			0				
7,610				0			
8,788					0		
9,826						0	

Ten years have passed since the tubewells started the operation. The calculated radius of influence is about 4.4 km. Compared to the water level contour map in Fig. 3 in the main report, it almost corresponds to the actual extent of influence.

Fig. 2 Progressive Change of Radius of Influence



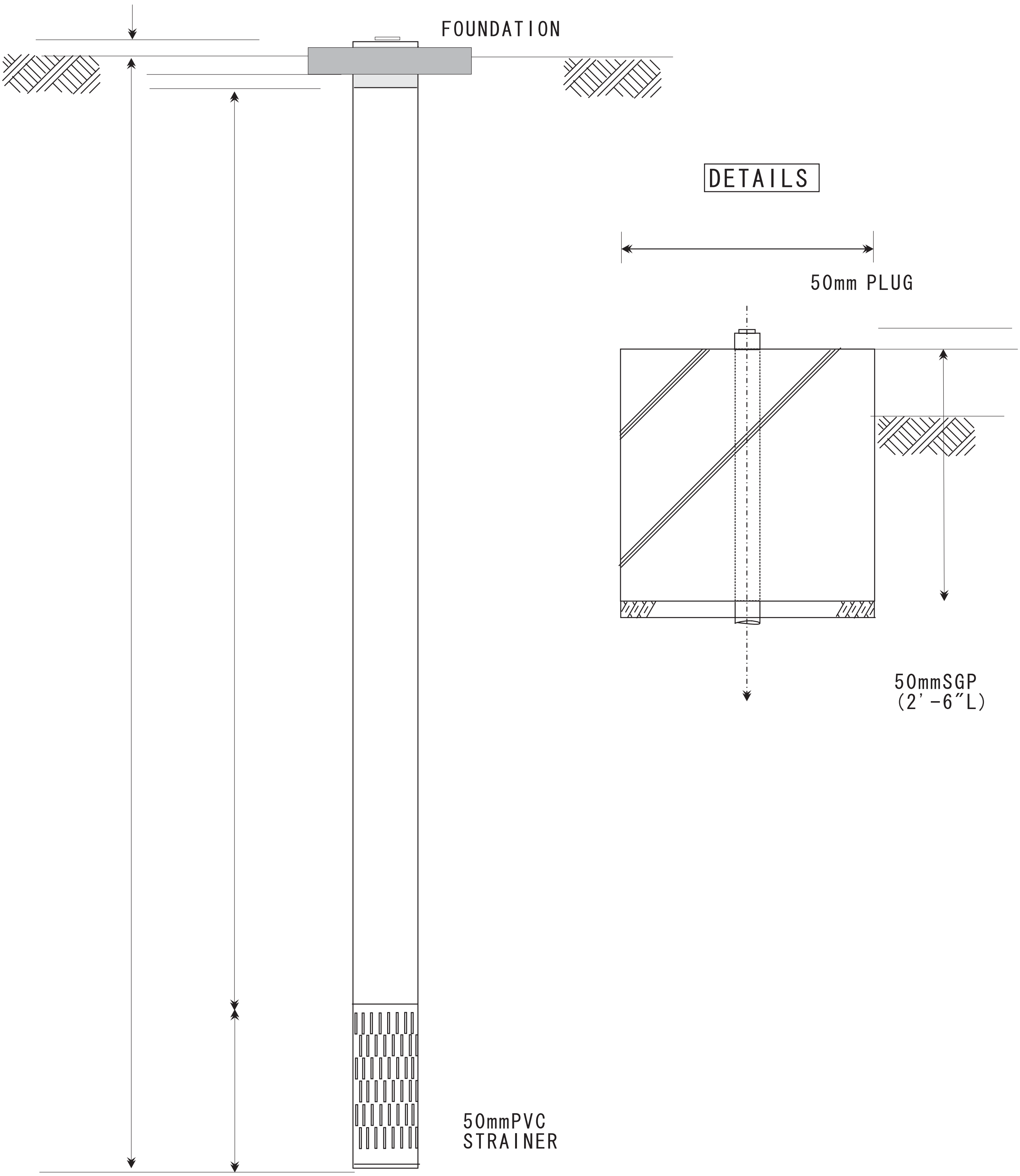
(4)Conclusion

The foregoing analysis is entirely based upon the record of the water levels measured by WASA. Since the Chenab wellfield is close to that along the Jhang Branch Canal, there is a high risk warned by this analysis.

The calculation indicates that the radius of influence from the Chenab wellfield enlarged by about 4 km in 10 years after the operation there started. Since the planned wellfield is located about 5 km from the southern periphery of the wellfield, there is possibility of the influence affecting the tubewells in the wellfield for the project.

In this view, proper measures for risk management will be required to avoid conflicts with local communities in the future.

APPENDIX 5-9 STRUCTURE OF WASA'S STANDARD MONITORING WELL



Appendix 5-10 Water Analysis on Site and by WASA Laboratory of Samples from Tubewells

(1) WATER ANALYSIS AT IRRIGATION WELLS

1) For the locations of tubewells, refer to Fig. 2-2-1-13 for the well number.

2) Affix "N" for the well No. means the tubewells in the area north of the Jhang Branch Canal, surveyed during the first stage (Dec. 2002)

3) The numbers without affix mean tubewells south of the canal. No. 101 is the test well installed for the study in the second stage. This area was surveyed both in the first and second stages (Dec. 2002 and Aug. 2003)

4) Analyses by WASA laboratory has a mark of circle in the column of remarks。

5) Water analysis of WASA tubewells in the Chenab wellfield is shown in Section 2.

	Well No.	Time of analysis	T ℃	p H	EC μM/cm	Turbidity NTU	TDS mg/ℓ	Ca mg/ℓ	Mg mg/ℓ	Total hardness mg/ℓ	Cl mg/ℓ	Total Nitrogen mg/ℓ	NO3 mg/ℓ	NH4 mg/ℓ	Total Phosphorus mg/ℓ	Re-marks
1	N01	Dec. 2002	22.4	7.36	1450	0	986	51	163	780	183	0	0	0	0.03	○
2	N02	Dec. 2002	23.2		1934											
3	N03	Dec. 2002	24.4		1700											
4	N04	Dec. 2002	24.1		1336											
5	N05	Dec. 2002	22.1	7.49	1047	0	695	48	77	428	94	0	0	0	0.039	○
6	N06	Dec. 2002	23.1		1041											
7	N07	Dec. 2002	22.1	7.28	1291	0	869	52	106	556	64.3	0	0	0	4.0	○
8	N08	Dec. 2002	23.3	7.60	351	0	287	26	59	300	29.7	0	0	0	0.06	○
9	N09	Dec. 2002	23.8		1945											
10	N10	Dec. 2002	24.0		941											
11	N11	Dec. 2002			750											
12	N12	Dec. 2002	24.2	7.09	896	5	613	62	52	360	41.0	0	0	2	0.01	○
13	N13	Dec. 2002	24.3		726											

	Well No.	Time of analysis	T ℃	p H	EC μM/cm	Turbidity NTU	TDS mg/ℓ	Ca mg/ℓ	Mg mg/ℓ	Total hardness mg/ℓ	Cl mg/ℓ	Total Nitrogen mg/ℓ	NO3 mg/ℓ	NH4 mg/ℓ	Total Phosphorus mg/ℓ	Re-marks
14	N14	Dec. 2002	24.4		672											
15	N15	Dec. 2002	24.7		555											
16	N16	Dec. 2002	24.1	7.95	867											
17	N17	Dec. 2002	25.2		1003											
18	N18	Dec. 2002	25.0		1464											
19	N19	Dec. 2002	22.1	7.07	1358	4	957	64	72	448	156	0	0	0	0.04	○
20	N20	Dec. 2002	24.8		807											
21	N21	Dec. 2002	25.2	7.19	940	0	641	44	94	484	50	0	0	0	0.07	○
22	N22	Dec. 2002	24.4		1209											
23	N23	Dec. 2002	25.9		794											
24	N24	Dec. 2002	25.0		1228											
25	N25	Dec. 2002	25.3	7.31	794	0	535	56	98	532	32	0	0	0	0.05	○
26	N26	Dec. 2002	24.4		994											
27	N27	Dec. 2002	25.6	7.24	782	0	538	60	101	556	25	0	0	0	0.06	○
28	N28	Dec. 2002	25.0		796											
29	N29	Dec. 2002	25.9		906											
30	N30	Dec. 2002	25.4		806											
31	N31	Dec. 2002	26.1	6.96	944											
32	N32	Dec. 2002	23.8	7.47	1115	0	840	77	121	680	69.3	0	0	0	0.05	○
33	N33	Dec. 2002	24.7		1135											
34	N34	Dec. 2002	24.7	7.2	2960	0	1940	46	150	720	222	2	7	2	0.1	○
35	N35	Dec. 2002	24.9		899											

36	N36	Dec. 2002	23.2	7.68	1168	0	747	39	52	308	49.5	0	0	0	0.07	○
	Well No.	Time of analysis	T ℃	p H	EC μM/cm	Turbidity NTU	TDS mg/ℓ	Ca mg/ℓ	Mg mg/ℓ	Total hardness mg/ℓ	Cl mg/ℓ	Total Nitrogen mg/ℓ	NO3 mg/ℓ	NH4 mg/ℓ	Total Phosphorus mg/ℓ	Re-marks
37	101	Sep. 2003	24.0	7.8	230	1.5	480	24	10	100	36	0	0	0		Lahore
38	102	Dec. 2002	22.3	7.91	260	0	194	32	41	244	29.7	0	0	0	0.09	○
39	103	Aug. 2003	24.7	8.30	232		200	54	26	240	50	0	0	0	0	○
40	104	Aug. 2003	26.3	8.7	1410		920	53	80	456	92	0	0	0	0.08	○
41	105	Dec. 2002	26.4	7.33	1882		1220	80	145	780	149	1	1	3	0.04	○
42	106	Dec. 2002	26.5	7.64	1265		834	58	71	428	99	0	0	0	0.05	○
43	107	Sep 2003	25.3	8.7	1374		928	53	71	416	92	0	0	0	0.09	○
44	108	Dec. 2002	24.8	8.6	1478		1030	55	98	528	95	0	0	0	0.13	○
45	109	Dec. 2002	26.5	8.7	1277		766	42	36	244	40	0	0	0	0.10	○
46	110	Dec. 2002	26.5	8.7	1277		766	42	35	244	40	0	0	0	0.10	○
47	111	Dec. 2002	25.6	8.7	1215		766	44	38	260	44	0	0	0	0.05	○
48	112	Dec. 2002	27.8	8.6	1708		1238	72	65	440	185	1	3	1	0.03	○
49	113	Dec. 2002	23.1	7.62	1275		846	96	98	632	74.3	0	0	0	0.01	○
50	114	Aug. 2003	26.7	8.6	1567		1104	36	30	210	176	0	0	0	0.03	○
51	115	Dec. 2002	25.8		1136											
52	116	Dec. 2002	26.2		993											
53	117	Dec. 2002	25.6		1477											
54	118	Dec. 2002	25,9		1175											
55	119	Dec. 2002	25.0		1442											

(2) WATER ANALYSIS FOR WASACHENAB TUBEWELLS

Item	T.D.S (mg/ℓ)				Ca (mg/ℓ)				Cl (mg/ℓ)			
Time	98Feb	00Jun	01Jul	02Jun	98Feb	00Jun	01Jul	02Jun	98Feb	00Jun	01Jul	02Jun
Well No.												
TW-1	340	368	372	390	40	32	36	30	38	30		
TW-2	440	434	380	375	52	48	32	29	46	46		60
TW-3	620	586	562	520	55	56	54	41	92	88		60
TW-4	466	444	436	470	47	52	50	35	66	62		99
TW-5	506	450	400	490	49	54	48	36	56	44		108
TW-6	400	384	392	345	52	50	50	35	38	40		106
TW-7	370	370	400	295	54	49	50	35	38	38		74
TW-8	368	410	440	350	52	52	52	26	40	42		60
TW-9	360		386	395	50		50	35	36			45
TW-10	428	460	446	485	54	51	53	28	38	70		84
TW-11	528	482	500	445	55	54	54	51	43	70		79
TW-12	480	456	466	425	56	52	54	74	44	66		84
TW-13	470	400	410	465	56	50	49	33	42	40		99
TW-14	400	432	466	435	52	54	52	40	45	44		89
TW-15	402	398	432	460	48	48	48	50	36	38		43
TW-16	330	386	434	400	46	50	51	46	34	50		47
TW-17	324	420	400	360	42	48	49	41	25	36		40
TW-18	306	332	340	320	40	38	42	34	26	32		25
TW-19	320	348		322	36	43		32	24	30		24
TW-20	318	350	330	316	34	44	32	30	23	28		22
TW-21	322	328	312		34	40	40		21	26		
TW-22			312	300			30	29				21
TW-23		330	316	310		36	28	28		24		23
TW-24												
TW-25		582	430			52	68			86		

Appendix 5-11 Comparison of Population Projections

Year	Actual Figures			Rate of Change Formula (Adopted)			Rate of Change Formula			Power Law Curve Formula	Logistic Curve Formula
	Collected Data	Adjusted Figure	Difference	Calculated Figure	Difference	Growth Rate	Calculated Figure	Difference	Growth Rate		
1991	1,583	1,583		1,607			1,583			1,583	1,598
1992		1,656	73	1,666	59	3.5%	1,636	53	3.2%	1,661	1,661
1993		1,729	73	1,725	59	3.4%	1,691	55	3.2%	1,727	1,724
1994		1,802	73	1,783	59	3.3%	1,747	57	3.2%	1,790	1,787
1995	1,875	1,875	73	1,842	59	3.2%	1,806	58	3.2%	1,851	1,848
1996		1,916	41	1,901	59	3.1%	1,866	61	3.2%	1,910	1,908
1997		1,956	41	1,960	59	3.0%	1,929	62	3.2%	1,968	1,966
1998	1,997	1,997	41	2,019	59	2.9%	1,993	65	3.2%	2,024	2,024
1999		2,065	68	2,077	59	2.8%	2,060	67	3.2%	2,080	2,079
2000		2,132	68	2,136	59	2.8%	2,129	69	3.2%	2,136	2,133
2001	2,200	2,200	68	2,195	59	2.7%	2,200	71	3.2%	2,190	2,184
2002				2,254	59	2.6%	2,274	74	3.2%	2,244	2,234
2003				2,313	59	2.5%	2,350	76	3.2%	2,298	2,282
2004				2,371	59	2.5%	2,428	79	3.2%	2,351	2,327
2005				2,430	59	2.4%	2,510	81	3.2%	2,403	2,371
2006				2,489	59	2.4%	2,594	84	3.2%	2,456	2,412
2007				2,548	59	2.3%	2,680	87	3.2%	2,508	2,452
2008				2,607	59	2.3%	2,770	90	3.2%	2,559	2,489
2009				2,666	59	2.2%	2,863	93	3.2%	2,610	2,524
2010				2,724	59	2.2%	2,958	96	3.2%	2,661	2,557
2015				3,018	59	1.9%	3,488	113	3.2%	2,912	2,696
Correlation Coefficient							0.993			0.998	0.997

Average Rate of Change Equation y=a×x+b a=58.80606, b=1,548.16364、

Since collected data were incomplete, the missing data were calculated by proportional distribution.

Adjusted power law was simplified. Refer to Guideline for Design of Waterworks (Projection Method)

Appendix 5-12 Amount of Water Production and Water Supply

(1) 1ST Study (Dec./2002 ～ Jan./2003)

Operation Record of Water Production Amount of Chenab Wellfield and Water Supply Amount distributed from Terminal Reservoir are shown in the next table.

(December/2001 and Jun/2002 are chosen as an example of recent characteristic data of winter and summer season water supply)

1) Dec./2001

	Chenab Wellfield				Terminal Reservoir			Remarks
Dec./ 2001	Number of working wells	Monthly working hour	Daily working hour (hour/day)	Amount of Water Production (m3/day)	Distribution Pump working hour (hour/day)	Water Supply hour by Gravity Flow (hour/day)	Amount of Water Supply (m^3/day)	
1	21	468	22.3	183,784	10	14	173,100	
2	21	464	22.1	180,385	9	15	175,300	
3	21	460	21.9	176,529	10	14	168,200	
4	21	461	22.0	172,873	10	14	171,700	
5	22	448	20.4	173,626	10	14	169,800	
6	21	418	19.9	161,818	8	16	159,800	
7	21	454	21.6	174,096	11	13	175,500	
8	21	456	21.7	179,061	11	13	178,500	Max.
9	20	441	22.1	170,481	10	14	176,700	
10	20	438	21.9	166,189	10	14	171,400	
11	22	426	19.4	159,757	10	14	168,000	
12	22	412	18.7	159,780	10	14	150,600	
13	21	406	19.3	155,912	9	15	155,800	
14	23	408	17.7	160,605	8	16	153,000	
15	20	396	19.8	154,355	8	16	154,400	
16	21	402	19.1	154,496	8	16	154,500	
17	21	408	19.4	149,762	7	17	157,600	
18	20	393	19.7	150,223	8	16	150,300	
19	20	398	19.9	153,549	8	16	156,200	
20	23	392	17.0	149,319	9	15	154,200	
21	23	420	18.3	157,987	9	15	153,400	
22	23	491	21.3	149,130	8	16	152,800	
23	22	396	18.0	154,490	9	15	149,000	
24	21	364	17.3	141,117	8	16	147,500	
25	22	387	17.6	153,933	7	17	145,700	
26	21	392	18.7	151,892	10	14	154,100	
27	22	402	18.3	157,021	9	15	150,400	
28	23	410	17.8	161,611	9	15	106,200	
29	23	313	13.6	119,998	5	19	116,700	
30	21	388	18.5	149,334	7	17	138,200	
31	21	370	17.6	144,210	5	19	145,200	
Total	664		602.9	4,927,323	270	474	4,833,800	
Daily Ave.	21.4	28wells)	19.4	158,946	8.7	15.3	155,929	98.10%

2) Jun/2002

	Chenab Wellfield				Terminal Reservoir			Remarks
Jun/ 2002	Number of working wells	Monthly working hour	Daily working hour (hour/day)	Amount of Water Production (m3/day)	Distribution Pump working hour (hour/day)	Water Supply hour by Gravity Flow (hour/day)	Amount of Water Supply (m^3/day)	
1	23	445	19.3	168,701	8	16	168,200	
2	23	450	19.6	169,738	8	16	160,700	
3	23	448	19.5	171,875	8	16	168,700	*
4	22	438	19.9	169,492	8	16	156,000	
5	26	457	17.6	173,296	8	16	162,100	
6	25	398	15.9	135,272	9	15	151,600	
7	23	449	19.5	173,965	8	16	161,200	
8	24	433	18.0	173,089	8	16	165,300	
9	24	443	18.5	169,198	8	16	168,900	
10	22	437	19.9	167,289	8	16	165,100	
11	25	337	13.5	130,189	6	18	126,900	
12	24	450	18.8	169,683	8	16	163,400	
13	24	406	16.9	154,120	8	16	150,200	
14	23	437	19.0	164,114	8	16	156,000	
15	24	447	18.6	168,720	8	16	162,400	
16	22	425	19.3	160,338	8	16	161,800	
17	23	441	19.2	165,646	8	16	159,300	
18	22	443	20.1	168,546	8	16	165,000	
19	23	425	18.5	159,651	8	16	160,000	
20	24	421	17.5	156,863	8	16	164,700	
21	24	395	16.5	145,995	6	18	137,800	
22	26	421	16.2	162,260	6	18	165,200	
23	26	437	16.8	170,990	8	16	163,300	
24	24	449	18.7	164,981	8	16	165,500	
25	23	427	18.6	157,057	8	16	159,200	
26					8	16	163,300	
27	24	425	17.7	164,829	8	16	160,800	
28	23	446	19.4	170,215	8	16	160,200	
29	24	416	17.3	158,252	8	16	162,000	
30	24	444	18.5	160,187	8	16	157,300	
Total	687		528.8	4,724,551	235	485	4,792,100	
Daily Ave.	23.7	(In 28wells)	18.2	**162,916**	7.8	16.2	**159,737**	98.05%

(2) 2nd B/D Study (Aug./2003)

Water supply amount from Terminal Reservoir in summer season（Jun/2003 – Aug./2003）

1. Water supply amount from Terminal Reservoir（Operation record: meter reading）

Jun.		July		Aug.		Remarks
Day	Water supply amount	Day	Water supply amount	Day	Water supply amount	
1		1	151,200	1	162,500	
2		2	149,900	2	165,000	
3		3	150,800	3	163,500	
4		4	153,500	4	163,500	
5		5	160,500	5	163,500	
6		6	150,000	6	161,500	
7		7	148,000	7	161,000	
8		8	149,000	8	*(159,000)	Day of 7－8
9		9	151,000	9	*(169,000)	Day of 8－9
10		10	140,000	10		
11		11	149,500	11	163,500	OHR Inflow was measured
12		12	156,000	12	164,500	
13		13	153,500	13	164,500	
14		14	152,000	14	160,000	Arterial Main Flow was measured in this term
15		15	151,500	15	165,500	
16		16	151,300	16	164,500	
17		17	148,800	17	160,000	
18		18	146,400	18	169,000	
19		19	150,000	19		
20		20	147,500	20		
21	144,500	21	159,500	21		
22	144,300	22	158,000	22		
23	119,900	23	162,000	23		
24	105,100	24	164,000	24		
25	141,500	25	154,500	25		
26	146,000	26	153,500	26		
27	146,000	27	163,000	27		
28	153,000	28	163,000	28		
29	140,500	29	162,800	29		
30	152,600	30	159,500	30		
31	-	31	159,200	31		

2. Water Production（Operation record of existing inline booster pump station）

Aug.		
Day	Water Production	Remarks
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		
11	171,450	
12	168,660	
13	168,247	
14	174,943	
15	163,798	Transmission Main Pressure was measured in this term
16	173,378	
17	164,000	
18	164,038	
19		
20		
21	159,050	
22	169,409	
23	174,049	
24		
25		
26		
27		
28		
29		
30		
31		

※ Flow Measurement: from 13:00 to 13:00

Appendix 5-13 Study on Existing Water Supply Facilities

For planning of water supply facilities, the survey on the present situation of the city's existing water supply system was carried out during the study. The major findings of the survey concerning the facilities planning are described hereunder.

(1) Topography of the supply area

The topography of the city area is mostly flat, with the northeast zone being slightly higher and the level falling a couple of meters toward southwest. The area where the T/R is located is the highest (GL185m) in the city, although a part of the east side is at the same level (GL 185m - 183m). The west side zone ranges from GL185m to 181m. The HWL of T/R is GL 188m, with its LWL at 3m lower than the HWL (GL 185m), which is the same as the ground level.

(2) Method of distribution from T/R

One of the remarkable features of WASA's water supply is that distribution by pumps to the city is limited to 4-6 hours a day with pumps being run only during time zones when demand rises to peak. For the rest of the day, the water is fed by gravity into arterial mains depending on the ground level at the T/R. Since water pressure along the lines remains low through duration of gravity-flow (slightly over negative pressure), the substantial supply is achieved simply while pumps are run.

Table 1 Current practice of WASA water supply at T/R（Dec. 2002）

Supply method		Time for pumped supply（Dec. 2002）	Duration (hrs)	Hourly supply rate（m^3/hr）	Daily maximum supply rate(m^3/day）
	Morning	6：00－7：30	1.5-2hrs	14,000～ 17,000	
	Afternoon	12：00－13：00	1.5-2hrs		
	Evening	17：00－18：30	1-2hrs		
Total outflow of pumped supply			4-6hrs		100,000
Total outflow by gravity		Rest of the day	16-20hrs	4,000～	80,000
Total outflow				Ave.7,500 m^3/h	180,000

＊Rated head of the existing pumps：45m
＊Rated pump discharge： 2,250m^3/hrs×7 units、2,070 m^3/hrs×3 units、plus 1 standby
Maximum 14,000m^3/hr、335,000 m^3/day

The current condition of WASA's water service by pumps and by gravity in Faisalabad in December 2002 is summarized in Table1. (The duration of pump operation and supply rate of the pumps vary day to day). The current practice of water supply for the city is far from normal 24- hour service, and the main reasons for this situation involve the insufficient production of water sources, lack of reservoir capacity and further pump malfunctioning due to the imbalance of demand and supply. (Although the existing reservoir has an adequate capacity for regular supply, its lower half remains unused since its inception apparently due to the defective design in the structure of pump suction.)

3) Present state of the distribution network

The production in the Chenab wellfield accounts for nearly 80% of water supply currently available to WASA. It is transmitted from the wellfield about 20 - 25km north east of the city via the booster pump station to the T/R, and then is distributed from the T/R to the service area of the city through the arterial mains (primary network system). All of these facilities were constructed in the preceding project completed in 1992. The arterial mains consist of 1,600mm to 500mm-diameter ductile cast iron pipes approximately 50km long in all. The lines gradually reduce their sizes as they run from the T/R at the western fringe down to the east side of the city, and at the eastern end, the water pressure remarkably shrinks.

①Water pressure of the arterial mains

The water pressure of the arterial mains in the city ranges from under 0.5kg/cm^2 to 2.5kg/cm^2. (The discharge pressure of the T/R supply pumps was designed at 4.5kg/cm^2, but presently their working pressure is around 3kg/cm^2).

- West side zone of the city: The water pressure through the arterial mains is about 2.5kg/cm^2-1kg/cm^2 in this area.
- East side: Over the east side zone, which is separated from the west side by the channel of the Rakh Branch Canal and the railroad running in parallel, the water pressure in the lines decreases to a low range of 0.5 kg/cm^2 while the pumps are run at T/R. It finally falls to 0 when the pumps are stopped. The pressure zero condition persists while the pumps are idling.

The following table shows the results of pressure measurements during this study at main sections of the arterial mains with auto-recording electromagnetic pressure meter.

Table 2 Results of the pressure survey (Dec. 2002)

		T/R	West side			East side		
			Up-stream area	Central area	End of down-stream	Up-stream area	Up-stream area	End of down-stream
Junction Node No.		101	3	36	31	45	57	70
Max	Registered pressure (m)	3.2	2.5	1.2	1.5	0.5	0.5	0.5
Min	Registered pressure(m)	0.1	0.5	0	0	0	0	0.1

The pumps are run daily for 4 to 6 hours in total in time to peak demand within the city. While they are run, a greater part of the west side can enjoy a satisfactory level of water service and even the east side can receive the delivery of supply although pressure is quite low (0.5kg/cm^2). One of the measures to improve the water supply is the continuous operation of the pumps, although the shortage of supply from the existing sources makes it difficult at present. If additional water sources are ensured through the implementation of this project, the duration of pump operation can be extended, eventually resulting in the change in the pattern of citizens' consumption, which is now keenly concentrated in intermittent hours while the pumps are run.

②Water supply ratio to the service areas

The daily rate of supply differs largely between the west side close to the T/R and the east side away from it. During the second stage of this study, water flow was measured with an ultrasonic flow meter along 2 sections of arterial mains of 600 mm and 800mm in diameter, which transport the water flow to the network loops in the east side. (August 2003).

The served populations of the west and the east sides were estimated on the basis of the water service area map prepared by WASA (Fig. 2-12 in the Basic Design Study Report). The results are as follows:

Table 3 Estimate of served populations of the service areas

	West side	East side	Total
Ratio of the service area	65%	35%	100%
Ratio of total served population	35.7%	19.3%	55%
Estimated served population	825,000	448,000	1,273,000

The flow measurements revealed that the inflow to the east side was 31,000m^3/day out of the total supply of 163,000m^3/day from the T/R relying upon the production of the Chenab source. Comparing the rates of total daily supply to both areas obtained through the field measurements and the respective served population estimated in the above table, the ratio of supply to the east side is calculated at 64% against 100% to the west side. This proves the present unbalanced situation of water distribution even though the east side is fed with supplementary supply from Jhal Khanuana Head Waterworks located in the east side. The detailed record of flow measurements and calculation results are shown in Tables (1) and (2) attached.

③ Existing pipelines for distribution

The total extension of the arterial mains, secondary mains and branches for distribution of 75mm and larger in size is now about 1,000km. The arterial mains composed of ductile cast iron pipes with the diameters ranging from 500 to 1,600mm accounts for 5%. 88% consists of asbestos cement pipes

(ACP) of diameters of 600mm and under, and 7%, PVC pipe less than 400mm in size.

Since 1998, WASA was engaged in a project for extending and reinforcing the existing network, in which it installed some 40 km of pipeline (or 4% of total extension) financed by the Punjab government.

The list of the existing lines for distribution is shown in the table below.

Table 4 Extension of main and branch pipes

<table>
<tr><th>Arterial mains, secondary mains, branches of 80 mm and larger in diameter</th><th>Type</th><th>Extesion（km）</th><th>Ratio</th></tr>
<tr><td>1,600mm to 500mm</td><td>DIP</td><td>50</td><td>5%</td></tr>
<tr><td>Distribution lines smaller than 600mm</td><td>ACP</td><td>833</td><td>87%</td></tr>
<tr><td>Main pipes under 400mm</td><td>PVC</td><td>77</td><td>8%</td></tr>
<tr><td rowspan="2">Total</td><td>Installed before 1998</td><td>About 960km</td><td>100%</td></tr>
<tr><td>Present
（＋40km since 1998）</td><td>About 1,000km</td><td></td></tr>
</table>

The east side shares 43% of the total extension of the arterial mains (Population ratio is 35%). Refer to the table below.

Table 5 Extension and average diameter of arterial mains

(Diameters ranging from 500mm to 1,600mm)

Unit	Extension(m)	Ratio	Ave. diameter (mm)	Pipe capacity ratio
1. West side	28,000	57%	890	89%
2. East side	21,000	43%	580	11%
Total	49,000	100%		100%

④ Examination of the arterial mains for future distribution

In order to examine the capacity of the existing arterial mains for distributing increased supply after the implementation of the project, the network analysis were conducted assuming 3 types of design hourly maximum. The following table outlines the results with details shown in Fig. (4) Result of Arterial Main Network Calculation attached.

Table 6　Network analysis summary

			T/R	West side			East side		
				Up-stream	Central zone	Down-stream	Upstream (North)	Upstream (South)	Down-stream.
	Node No		101	102	31	34	46	49	70
Ongoing	Case□ Hourly max.=. 1.9	Water head (m)	30	28	17	16	8	8	4
After the project	Case□ Hourly max.= 1.9	Water head (m)	40	33	9	8	-8	-15	-19
	Case□ Hourly max.=. 1.5	Water head(m)	40	37	21	20	11	7	5
	Case□ Time coeff. 1.7	Water head(m)	35	30	11	9	-2	-8	-9

From the results shown in the above table, the supply condition of the existing network can be estimated as follows:

*Maximum daily water supply rate　　230,000+91,000=321,000m^3/day

*Target design maximum hourly distribution rate

Case 1:　　321,000/24 (hrs) x 1.9= 25,400 m^3/hr

Case 2:　　321,000/24 (hrs) x 1.5= 20,000 m^3/hr

Case 3:　　321,000/24 (hrs) x 1.7= 22,700 m^3/hr

*Calculation results

For cases of hourly maximum of 1.9 or 1.7, the pressure will be negative at the downstream of the network in the east side, possibly resulting in an extremely poor service condition there. Only in the cases of hourly maximum rate of 1.5 the pressure can be retained in a similar range as at present, allowing the supply, but the unbalanced service between the west and the east sides will persist. If the water sources are increased through the project, water supply may be improved compared to the present situation, but without any improvement on the existing system, the conditions close to the one shown in case 3 is likely to occur.

4) Storage capacity

The number of water storage facilities of WASA within the city totals 38, consisting of T/R, underground and overhead tanks. While the total capacity of storage amounts to 86,000m^3, the capacity of working tanks of 13 in number is limited to 46,000m^3. The table below shows the comparison between the existing capacity and the ongoing working capacity.

Table 7 Present situation of storage facilities

		Existing facilities		Working facilities	
		Number	Capacity (m^3)	Number	Capacity (m^3)
1, Terminal Reservoir		1	48,000	1	24,000
2. Underground/ Overhead tanks		37	38,000	13	22,000
	West side	23	23,000	4	9,000
	East side	14	15,000	9	13,000
Total capacity		38	86,000	14	46,000
Storage capacity			for 9 hours		for 4.8 hours

Most of these tanks scattered around the city were constructed with local funds long before 1992 when the main facilities of the present system were completed by Phase 1 financed by the ADB. They were used for supply of groundwater from the existing tubewells within the city, mainly along the Rakh Branch Canal penetrating the city. Since the completion of Phase 1, the existing tanks have had a different function to compensate the low water pressure of the arterial mains, particularly in the east side zone. In the west side where pressure is generally adequate for direct supply from the network to households, most of the tanks have ceased working. Some of them have capacity totaling over 3,000m^3, but most are a combination of an underground tank of less than 1,000m^3 and an overhead one mostly of 230 m^3 in capacity with a booster pump station. When the T/R pumps are operated, the water flows into the underground tanks to be pumped up to the overhead tanks by the booster pumps), and then it is supplied from the latter to households for about 2 hours in 3 times (totaling 6 hours) everyday. There are many families who have set their own tanks on top of the roof together with booster pumps. A list of the existing storage facilities in the city is shown in Table (3) attached.

5) Issues in the distribution system

As a result of the survey on the existing facilities, the distribution system has issues as mentioned before, which causes the unbalanced water supply condition in the city. To deal with these issues, there are needs to revise the water service master plan according to the future development policy of the city, and to take actions having consistency with the whole plan.

The main purpose of this project is to increase the water supply as a top priority. However, even after the augmentation of water supply is achieved through the implementation of the project, it is most likely that the unbalanced water supply now prevailing is left unimproved as examined through the network analysis (Refer to 3 ④ this section). Therefore, various measures for the improvement were examined and were discussed with WASA during the survey, and at the end of the second stage of the study, WASA proposed its own measures for the improvement of system as stated in Appendix 1 “Technical Note" in the Basic Design Study Report. To enhance the effectiveness of the project, it is necessary to take an effective measure to contain difficulties in distribution. Through the review of WASA's proposal, this project intends to include an appropriate measure for the improvement of the existing system as one of its components. The details are described in section 2-2-2-3-(2)-5) "Plan for Improvement of Distribution System in the City” in the Basic Design Study Report.

Table (1) Measurement of Water Flow in Arterial Mains in the East Side Area

: Dia800mm, Flows from TR

:Pump operation

Date	Hour	Min.	Ave 30min. Hourly flow rate		No. of operating pump	Pump operating time	Water supply amount	Pump Head	Water level TR	No. of operating wells
			(m^3/hr)			(hour)	(m^3/hr)	(kg/cm^2)	(HWL 6m)	
14-Aug	12	0	921		1		12,200	2.0	5.3	16
Pump operating in noon time		30	2,372		5			2.0		
	13	0	1,568		5	1.3	7,000	2.0	4.6	16
		30	394							
	14	0	436		0		3,500	0	4.4	16
		30	452							
	15	0	442		0		3,600	0	4.7	16
		30	490							
	16	0	510		0		4,000	0	5.0	16
		30	588							
	17	0	652		0		7,800	0	5.3	18
Pump operating in the evening		30	1,054		1					
	18	0	2,767		6		18,000	3.0	5.5	23
		30	2,608		6			3.0		
	19	0	2,480		6		17,000	3.0	4.4	23
		30	2,486		5	2.3		2.4		
	20	0	686		0		3,200	0	3.5	23
		30	492							
	21	0	562		0		4,000	0	3.9	23
		30	594							
	22	0	640		0		4,000	0	4.6	23
		30	644							
	23	0	650		0		3,800	0	4.9	12
		30	652							
15-Aug	0	0	514		0		4,000	0	4.9	12
		30	636							
	1	0	642		0		4,000	0.0	4.9	12
		30	634							

	2	0	634		0		4,000	0	4.9	12
		30	636							
	3	0	654		0		4,000	0	4.9	12
		30	648							
	4	0	678		0		9,000	0	5.3	22
Pump operating In the morning		30	1,246		1					
	5	0	**2,786**		6		17,200	3.1	5.5	22
		30	2,688		6			3.1		
	6	0	2,610		6		**17,900**	3.0	4.4	22
		30	2,748		6	2.3		3.0		
	7	0	462		0		2,200	0	**＊3.3**	22
		30	-68							
	8	0	-104		0		14,000	0	3.9	22
		30	8							
	9	0	143		0		1,800	0	3.9	18
16-Aug	9	30	10							
	10	0	128		0		3,700	0	4.6	18
		30	334							
	11	0	418		0		4,400	0	5.0	18
		30	442							
Daily		**Total**	**22,334**	m^3/day		**5.9**	163,000			
Hourly Ave.		T.F=3.0	**931**	m^3/hr		T.F=2.6	6,792	Capacity =4.5		
			Detective value		Operation record of TR					

(2) Measurement of Water Flow in the Arterial Mains in the East Side Area

: Dia600mm, Water flows from the TR

:Pump operation

Date	Hour	Min.	Ave 30min. Hourly flow rate (m^3/hr)		No. of operating pump	Pump operating time (hour)	Water supply amount (m^3/hr)	Pump Head (kg/cm^2)	Water level TR (HWL 6m)	No. of operating wells
17-Aug-03	16	0	228				4,200	0	5.0	16
		30	268							
	17	0	298				7,000	0	5.3	16
Pump operating in the evening		30	526		1					
	18	0	**838**		6		17,100	3.0	5.3	23
		30	830		6					
	19	0	818		6		17,700	3.0	4.1	23
		30	726		5	2.3		2.4		
	20	0	206				3,200	0	3.3	23
		30	232							
	21	0	216				3,300	0	4.4	23
		30	228							
	22	0	230				4,000	0	4.9	23
		30	252							
	23	0	276				4,000	0	4.9	12
		30	280							
17-Aug-03	0	0	276				4,000	0	4.9	12
		30	262							
	1	0	254				4,000	0	4.9	12
		30	254							
	2	0	256				4,000	0	4.9	12
		30	250							
	3	0	256				4,000	0	4.9	12
		30	236							
	4	0	238				9,200	0	5.0	16

Pump operating in the morning		30	406		1					
17Aug03	5	0	826		6		16,900	3.2	5.2	22
		30	616		6					
	6	0	770		6		16,500	3.0	4.3	22
		30	798		6					
	7	0	660		6	2.5	800	3.0	**＊3.0**	22
		30	126							
	8	0	132				1,000	0	3.7	16
		30	132							
	9	0	184				1,900	0	4.3	16
		30	280							
	10	0	372				3,700	0	4.9	16
		30	364							
	11	0	330				4,600	0	5.2	16
		30	324							
	12	0	444		1		12,500	2.0	5.3	16
Pump operating in noon time		30	740		5					
	13	0	472		3	1.3	6,600	2.6	4.6	16
		30	206							
	14	0	246				3,500	0	4.4	16
		30	258							
	15	0	274				3,500	0	4.7	16
		30	280							
		Total	**8,987**	**m^3/day**		6.1	**157,200**			
Hourly Ave.			**374**	m^3/hr			11,229	Capacity =4.5		
			Detective value		Operation record of TR ＊3.0m:Minimum Water Level					

(3) List of Existing Reservoirs in the City

Tank No.	Location	Overhead reservoir	Ground reservoir	total		Node No.	Operating condition
West side		(MG)	(MG)	(MG)	(m^3)		
W-01	Civil Line Bagh-e-Jinnah	0.05	0.1	0.15	682		×
W-02	Gujjiar Basti	0.05	0.15	0.2	909	A/M NO.7	×
W-03	Dhobi Ghat	0.03	0.1	0.13	591		×
W-04	Muhammad Pura	0.1	0.1	0.2	909	A/M NO.9	×
W-05	Jinnah Colony	0.05	0.13	0.18	818	A/M NO.10	×
W-06	Karkhana Bazar	0.05	0.03	0.08	364		×
W-07	Gulberg	0.05	0.2	0.25	1,137	A/M NO.11	×
W-08	Afghan Abad	0.2	0	0.2	909	A/M NO.11	×
W-09	G.M.Abad Water Works	0	0.5	0.5	2,273	A/M NO.14,16	×
W-10	Latif Chowk/Chohar Majra	0.05	0.2	0.25	1,137	A/M NO.17	×
W-11	Admn Chowk	0.05	0	0.05	227	A/M NO.9	○
W-12	Kanak Basti	0.025	0	0.025	114	A/M NO.13	×
W-13	Islam Nagar	0.1	0	0.1	455		×
W-14	Jhang Bazar	0.05	0	0.05	227		×
W-15	212 R.B./Chamra Mandi	0.05	0.2	0.25	1,137	A/M NO.51	○
W-16	213 R.B.	0.05	0	0.05	227		×
W-17	Amin Pur Bazar	0	0.05	0.05	227		×
W-18	Gulistan Colony	0.5	1	1.5	6,819	A/M NO.2	○
W-19	Gulistan Colony- II	0.05	0.2	0.25	1,137		×
W-20	Nazim Abad	0.05	0.1	0.15	682		×
W-N1	Kaleem haheed Colony No.1	0.04	0.05	0.09	409	A/M NO.18	×
W-N2	Kaleem haheed Colony No.2	0.04	0.05	0.09	409	A/M NO.18	×
W-21	Gulfishan Colony	0.1	0.05	0.15	682		○
Total volume of reservoir in western zone (MG)		1.735	3.21	4.945			
Total volume of reservoir in western zone (m^3)		7,887	114,593		22,480		
Total volume of reservoir in western zone under operation (MG)		**0.70**	**1.25**	**1.95**			
Total volume of reservoir in western zone under operation (m^3)		**3,182**	**5,683**		**8,865**		

East Side			(MG)	(MG)	(MG)	(m^3)	
E-01	Abdullah Pur	0.05	0	0.05	227	A/M NO.40	×
E-02	Peples Colony OHR-1	0.05	0.2	0.25	1,137	A/M NO.43,45	○
E-03	Peples Colony OHR-2	0.05	0	0.05	227	A/M NO.63	×
E-04	Head Water Works Jhal	0.05	1.2	1.25	5,683	A/M NO.32	△
E-05	Waris Pura	0.05	0.1	0.15	682	A/M NO.70	○
E-06	Baber Chowk / Batala Col.	0.05	0.1	0.15	682	A/M NO.70	○
E-07	Allama Iqubal Colony OHR +W.W.	0.1	0.2	0.3	1,364	Tube Well (106,106/A)	○
E-08	D-Type Colony	0.05	0	0.05	227	Tube Well (106,106/A)	○
E-09	Ahamed Nagar	0.03	0.05	0.08	364		○
E-10	Samanabad (Qadri Chowk)	0.03	0	0.03	136	A/M NO.49	×
E-11	Samanabad (OHR No II)	0.05	0	0.05	227	A/M NO.67	×
E-12	OHR 17-W	0.5	0.25	0.75	3,410	A/M NO.50	○
E-13	Madina Town-	0.1	0	0.1	455	Tube Well (1,2,3,4,5)	×
E-14	Madina Town-	0.1	0	0.1	455	Tube Well (1,2,3,4,5)	○
Total volume of reservoir in eastern zone (MG)		1.26	2.1	3.36	15,275		
Total volume of reservoir under operation in eastern zone (MG)		**0.90**	**2.05**	2.95			
Total volume of reservoir under operation in eastern zone (m^3)		**4,091**	**9,319**		**13,411**		
Total volume of reservoir in whole city (MG)		3.00	5.31	8.3			
Total volume of reservoir in whole city (m^3)		**13,615**	**24,139**		**37,755**		
Total volume of reservoir under operation in whole city						Total volume of water supply	
Total volume of reservoir under operation in whole city (m^3)					**22,300**	230,000	(m^3/d)
Total volume of terminal reservoir under operation (m^3)					**24,000**	Detention time	
Total (m^3)					**46,300**	4.8	(hour)

① RESULT OF ARTERIAL MAIN NETWORK CALCULATION (PRESENT CONDITION)

Arterial Main Network in Faisalabad City

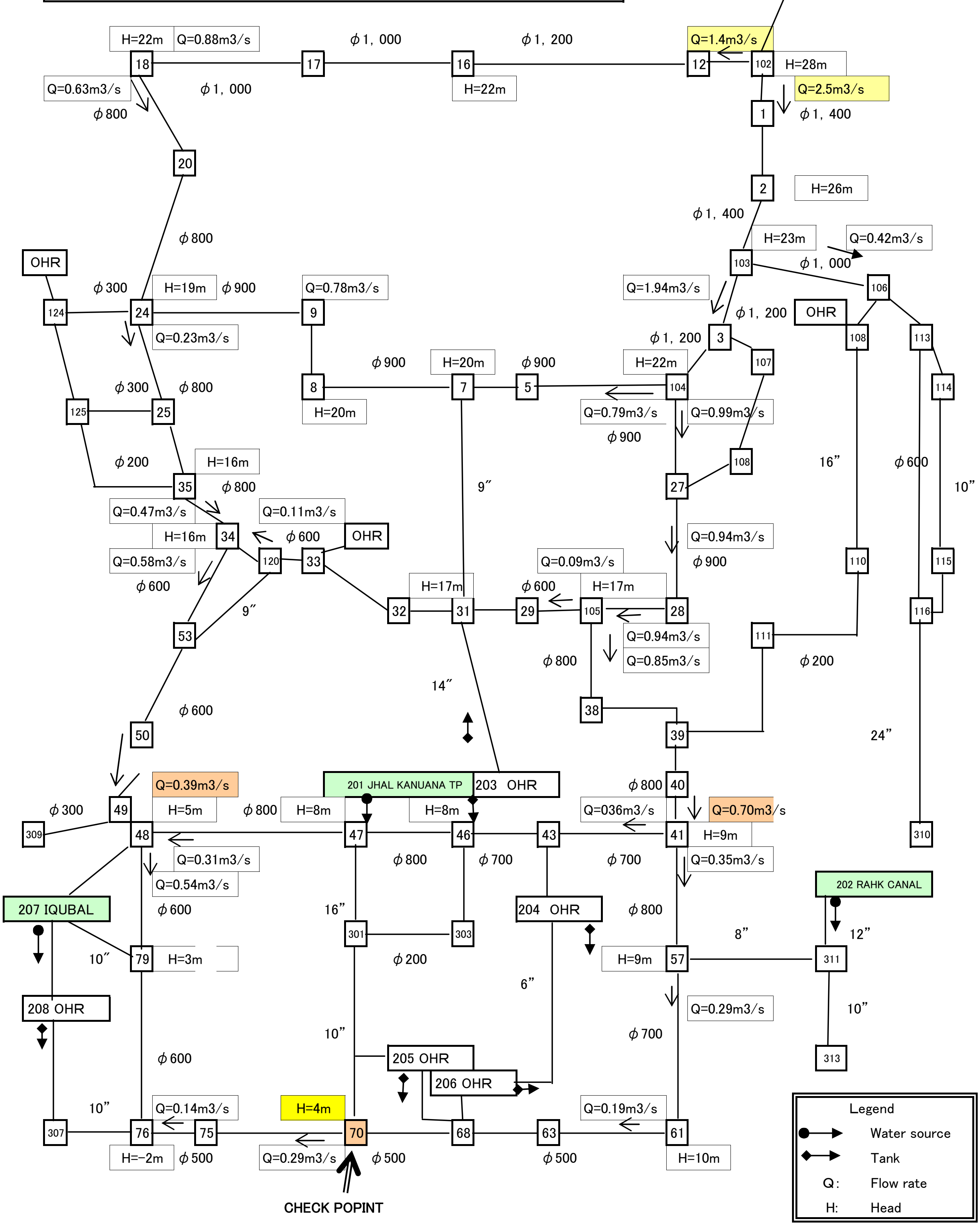
Present Condition

Total Q=190,000m3/day (Chenab+JBC+J.K TP＋OTHERS)

(q=49,600m3/day (26%) in East Zone)

(Hourly Peak Factor=1.9)

(TR Pump Head: H=30m)



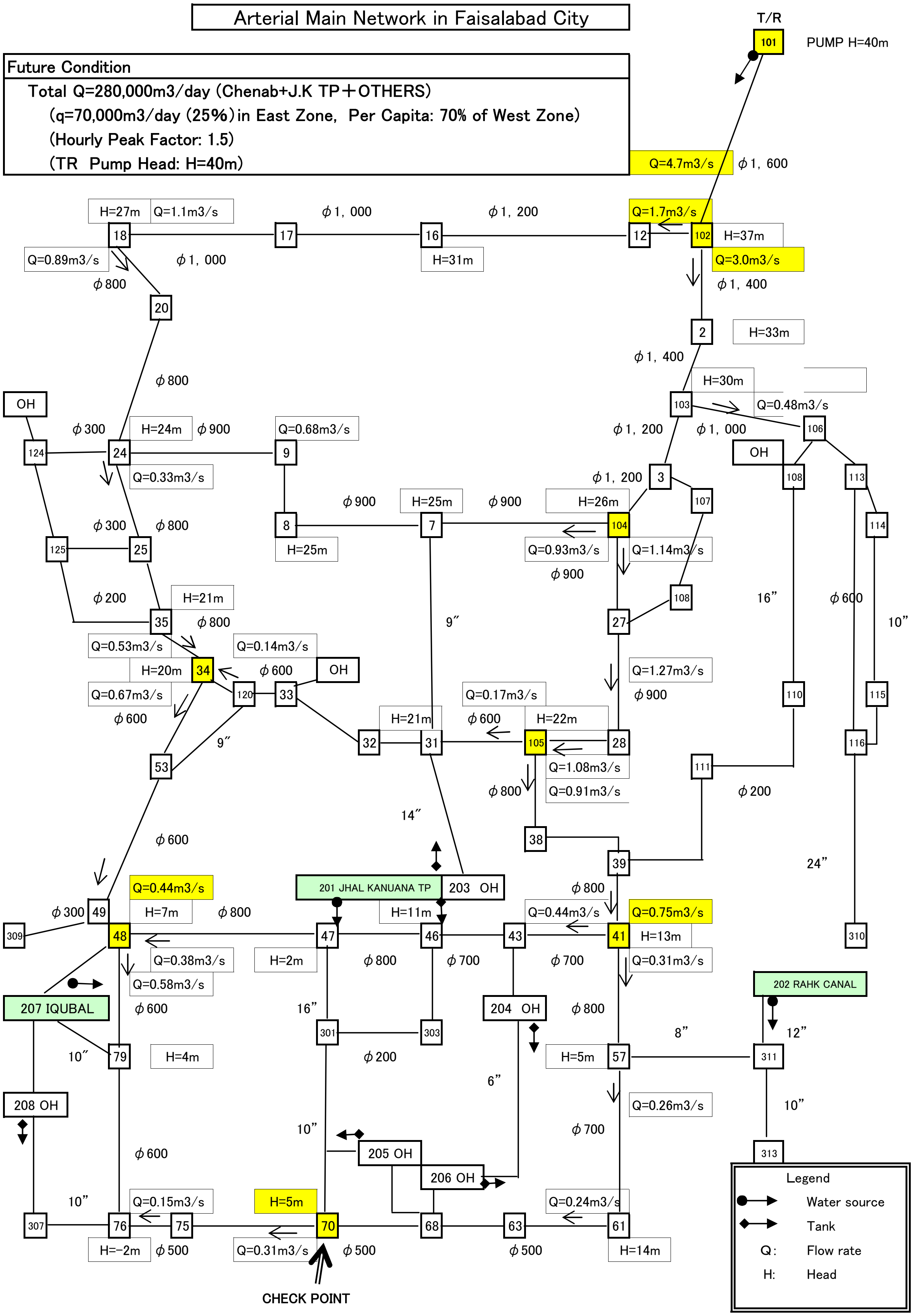
② RESULT OF ARTERIAL MAIN NETWORK CALCULATION

*AFTER WATER SOURCE DEVELOPMENT

*PRESENT PIPELINE

Arterial Main Network in Faisalabad City

Future Condition
Total Q=280,000m3/day (Chenab+J.K TP＋OTHERS) (q=70,000m3/day (25%) in East Zone, Per Capita: 70% of West Zone) (Hourly Peak Factor: 1.5) (TR Pump Head: H=40m)



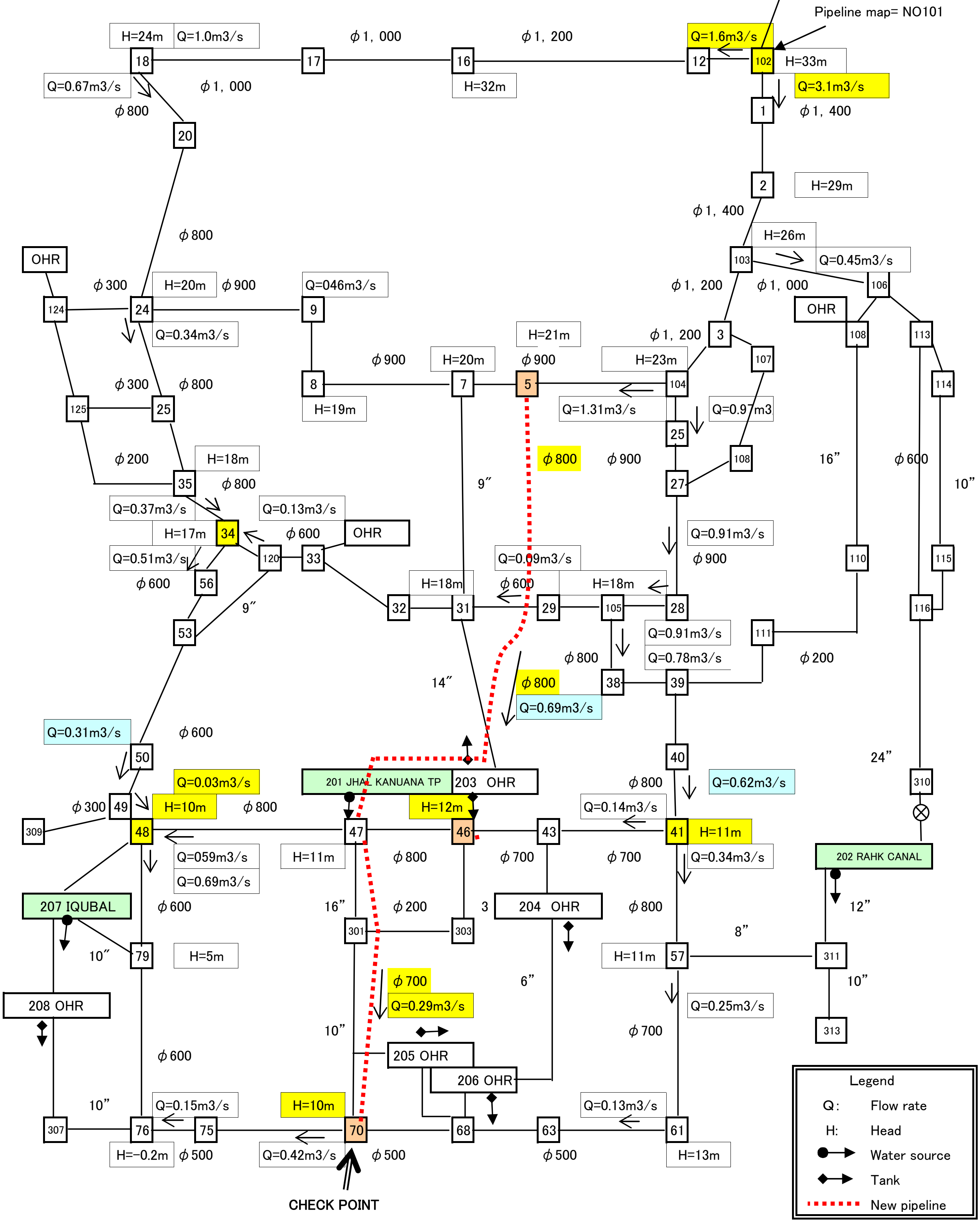
③ RESULT OF ARTERIAL MAIN NETWORK CALCULATION

*AFTER WATER SOURCE DEVELOPMENT

*AFTER PIPELINE STRENGTHEN

Arterial Main Network in Faisalabad City

Future Condition
Total Q=280,000m3/day (Chenab+JBC+J.K TP＋OTHERS) (q=100,000m3/day (36%) in East Zone) (Hourly Peak Factor=1.5) (TR Pump Head: H=40m)



Appendix 5-14　Comparison of pipeline network calculation

Strengthen Pipeline route		Node NO & Hydrodynamic Head（m）								Remark
		West Zone						East Zone		
		NO102	NO5	NO 18	NO 29	NO34	NO50 -51	NO47	NO70	
		1600mm branch	Start popint of Strengthen Pipe				West zone pipe end	End of Strengthen Pipe	Eest Zone pipe end	
1. Present condition										
	① Actual Head						**5**	**5**	**2～5**	Insufficient water supply
	②Calculation result（Per Capita Supply in East Zone）	28	20	22	17	16	8	7	4	
2. After project(equal per capita supply whole the city)										
	① Present Pipeline	**37**	**26**	**28**	**22**	**21**	**1**	**3**	**-6**	
(1) WASA proposal	②「Route 1」（φ700,l=3.5km）	37	25	28	23	21	11	16	4	Booster Pump is necessary In East Zone
	③Route 3（φ800,l=2.5km）	40	27	29	24	22	5	4	-5	
	④Route1 + Route3 (φ700-φ800,l=5.7km)	38	26	29	26	22	12	17	4	
	⑤Route1 + Route2 (φ700-φ800,l=7.6km）	37	28	29	26	23	14	18	5	
(2) Adopted Plan (Alternative)	**⑥「Route1」＋[Route ; Inside of East Zone]（φ800- φ700, l=6km）**	**33**	**21**	**24**	**18**	**17**	**10**	**11**	**10**	**Good**
(3) Reference	Direct supply from TR to Jhal Kanuana T.P. (φ1,000,l=11km)	39	32	32	31	28	20	29	15	Good

Appendix 5-15 Socio-economic Survey on 2nd Basic Design Study (1) and Activities after the completion of the survey

(1) Background and Objectives of the Survey

In 1998 JICA dispatched a study team to Pakistan for 2nd Basic Design Study for the project for improvement of water supply in Faisalabad. It was, however, cancelled on the way due to the economic sanctions imposed on Pakistan due to its execution of nuclear testing. Before the study ceased, the team had been faced with an intense rejection against test drilling by residents in and surrounding the proposed wellfield site along the Chenab river. After that incident, WASA, executing agency of the project, took steps to procure a piece of land for testing in an alternative site about 5 km upstream the initially proposed site. For the present study undertaken in 4 years since the initial study, therefore, WASA had an intention to target the second wellfield site for groundwater development.

The proposed sites along the Chenab river were under jurisdiction of *Tehsil* Chiniot of the Jhang District, while Faisalabad city, the target of the project, is in another district of Faisalabad. Taking such a specific local condition into account, the renewed study proposed to carry out the social survey of households in both of the proposed sites (No. 1 and No. 2) for the wellfield. The objectives of the survey were (a) to examine the views of stakeholeders in both sites, (b) to confirm the conditions, if any, for their approval of the project, and (c) to propose to WASA findings and recommendations for securing agreement with them in either of alternative sites, since the Japanese side asked for definite verification of stakeholders' approval for the project implementation. The survey intended to support WASA's efforts for that target.

It turned out, however, that despite such a strategy of the current study, the residents' rejection popped up while it was underway, this time in No. 2 wellfield site. Immediately on completion of the household survey, WASA found itself in a position to enter direct negotiations with local stakeholders. At the end of January 2003, this process ended up with WASA's decision to suspend an addition development of the Chenab wellfield and to move the site from the initially proposed Chenab area to a new one along the Jhang Branch Canal some 15 km south of the river, as is outlined in Section (4).

As a result, those targeted directly in this social survey have now turned into just neighbours. However, their socio-economic situation revealed through the survey represents a section of general characteristics of those engaging in agricultural production throughout the region where WASA has been continuing the operation of existing tubewells, and their views should be referred to in planning the project in a newly-proposed site.

(2) Survey Areas

This social survey for the development of a new wellfield targeted No. 1 and No. 2 wellfield sites along the Chenab river initially proposed by WASA for the project. The No. 1 site is represented by the village of Bukharian and the No. 2, about 5 km upstream, by Metha.

Both sites belong to Tehsil Chiniot of the Jhang District, while Faisalabad city is the capital of the adjoining Faisalabad District. WASA already has an existing wellfield in the Chenab area, which was commissioned in 1992 and has since been the major sources for water supply for Faisalabad. The villages covered by the present study involved 18 in total including Bukharian and Metha, as shown in the attached map (Fig.1).

(3) Survey Period

1) Field survey including discussions with WASA
From 18 December 2002 to 24 December 2002

2) Preparation of the report
From 24 December 2002 to 27 December 2002

(4) Survey Team

Since a great majority of villagers understands only Punjabi, a local language commonly spoken over a vast area throughout the Punjab province, the survey was entrusted to a local authority as follows:

Supervisor overseeing the survey:

Dr. Mohammed Zakaria Zakar
Chairman, the Social Department of Punjab University, Lahore

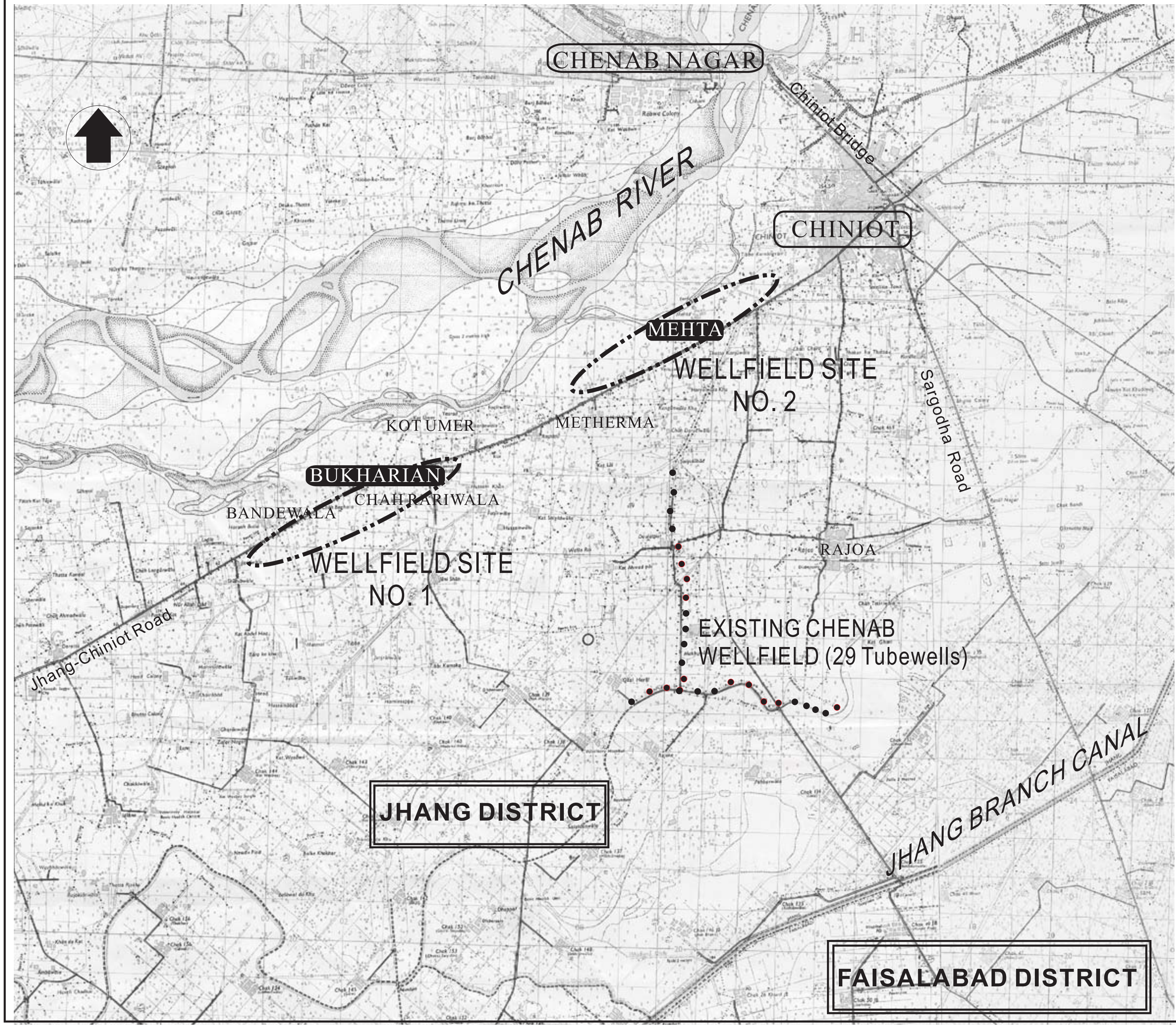


Fig.1
LOCATION MAP OF RELATED COMMUNITIES FOR PROPOSED WELLFIELDS

(5) Methodology

An extensive field research was conducted to find out baseline information and attitude of targeted villagers towards the project. The following methodologies were used.

□Sample survey

A sample survey was conducted against almost 10% of total household in the project area by semi-structured interview schedule. Population, size of household, no. of villages and no. of respondents are as below.

Table 1 Population, size of household, no. of villages and no. of respondents

Project sites	Total villages	No. of respondents	No. of household	Population
Bukharian	10	100	1,121	5,670
Mehta	8	100	904	7,400
Total	18	200	2,025	13,070

An accidental sampling procedure was adopted to approach the respondents for interviewing. Tools for data collection were constructed in English. However the local consultants as field researcher interviewed with respondents in local language.

□Village profile concerning socio-economic conditions in each village

Socio-economic conditions and demographic data of each affected village were profiled by interview schedule and village survey.

□Group Focus Discussion: GFD

GFD was conducted for villagers in each of 18 villages in the targeted areas. The above semi-interview schedule with questionnaire provided quantitative data, while GFD was conducted to get qualitative data in order to complement the former approach.

□Interviews with the Local Leaders, Influentials and Politicians

This survey was conducted to interview with politicians voted by the will of villagers, influentials such as landlords who have strong social influence and local leaders who have a sense of their obligation to look after the perceived interests of the people. From each village at least 2 representatives such as village leaders/influentials ware interviewed, and at the local center of Chiniot city, the *tehsil nazim* (chief of Tehsil Chiniot), police master, etc, were interviewed.

(6)Summary of social survey results

From social survey results, socio-economic profile provided perceived interests of people regarding construction of tube wells.

1) General socio-economic situation

People are mainly depended on agriculture for their livelihood. From results of sample survey, 63% of respondents were farmers by occupation. In case that livestock holding is added, the data become 71%. Then 9.5% of respondents were business; 9.0 of respondents were labour; and 6.5% of respondents were employed persons. 72% of respondents owned self cultivated land, with 52.5% having land holdings ranging from 1 to 12 acres. However, 13.5% of big landlords hold more than 25 acres and are infulentials in the project area. Though 28% of respondents were landless farmers, 21.8% of them were employed by landlords for their income. And 95.2% of households keep livestock.

The annual incomes of 74% of households are lower than Rs. 40,000 (about ¥80,000). Since the average number of household members is 7 in Tehsil Chiniot, based on the 1998 census, an average person is in extreme poverty, living on less than ¥30 a day. The survey results exemplify the typical social structure of the region under control of a small number of landlords over an overwhelming majority of people in a level of extreme poverty

Regarding the situation of infrastructure in the project area, 90.12% of houses are electrified. However, 95.06% of housing units of the project area are without latrines and telephone facilities. It is noteworthy that the civic facilities like gas, water and sewerage/drainage systems are nonexistent in the entire project area.

Their livelihood depends on agriculture by irrigation in the project area. Out of 18 villages only 5 villages are nominally (10%-20%) dependent on river pumping and canal water for irrigation, whereas an overwhelming majority of remaining 13 villages are dominantly (80%-100%) irrigating their agricultural lands from privately installed tube wells. The entire population of the project area is significantly (80%-90%) dependent on ground water pumped by electric motor pumps or hand pumps.

According to sample survey data, 41.5% of the respondents and their wives (88.6%) are illiterate, and 36% of the respondents and their wives (4.7%) in educational background are more than primary level.

2) Perceived interests regarding construction of tubewells

There are four dominant castes in the project area; these include Khokhar, Wainse, Ansari and Sehdhan. The caste system play a major role in the collective and consensus decision making and formulating the public opinion. Blood and social relations within the caste are very strong and closely knit the entire area and its vicinity. Therefore any threat to any village/community became a matter of concern for the whole area.

82% of the respondents have been aware of WASA planning for additional tubewell construction.

83.0% of respondents perceive the scope of the project and reported that the beneficiaries are citizens of Faisalabad. (9.5% of respondents: the beneficiaries are government. 7.5% of respondents: don't know)

87.5% of respondents have no willingness to provide land to government regarding to construction of tubewell; 73% among them is against due to the lowering of water level. The remaining 27% is against due to opposition to landlords that agree to construction of tubewells.

The background of objections by people is in the commonly shared experience that groundwater level in private wells had been lowered due to continuous pumping of tubewells installed under the ADB project in the past. Their livelihood depended on agriculture by irrigation from pumping up private tube-well in the project area. They had a common fear that the lowering of groundwater level would further increase cost of their economical activities. In addition, as their domestic life and livelihood also depended upon groundwater from private wells, their fear was aggravated. The following table shows social, economical and environmental impact assumed by respondents on village life resulting from the construction of tubewells.

Table 2 Social, economical and environmental impact assumed by respondents on village life resulting from the construction of tubewells

Nature of Impact	Frequency	Ratio（%）
Drought like conditions will happen	136	68.0
Agriculture and livestock would perish	28	14.0
Population would be compelled to migrate	5	2.5
Cost of agricultural farming will increase	19	9.5
Hunger and thirst will prevail	7	3.5
Eco-system will be destroyed	5	2.5
total	200	100

68.0% of respondent feared “Drought like conditions will happen”. As “cost of agricultural farming will increase”, respondents assume the reinstallation of tubewells and motor pumps due to the lowering of water level. In the past, respondents used the hand pumps. However, people reinstalled motor pumps in the individually owned tube-wells by themselves due to the lowering of water level by the ADB project. Respondents shared the same experience. Therefore almost all answers from respondents attributed the lowering and shortage of ground water to the former WASA project. 96.5% of the respondents (had fear about the effects of tubewells on groundwater level. (No effect: 3%, Don't know: 0.5%)

The respondents thought that there would be no natural way to compensate the shortage of water caused by tubewells installed by a new project. However, the respondents had come up with some proposals to minimize the worst effects of installation of tubewells causing shortage of water in the project area by measures listed in the following table:.

Table 3 Proposal remedial measures

Proposal remedial measures	frequency	Ratio（%）
Supply of sufficient canal water	43	21.5
Construction of dam on river Chenab	39	19.5
Non installation of tube-wells	45	22.5
Reliance on nature's reaction	23	11.5
Rain flood water	11	5.5
No substitute	39	19.5
Total	200	100.0

58% of respondents proposed various measures like supply of sufficient canal water to the project area (43%), construction of dam (19.5%), reliance on nature's reaction (11.5%) and rain flood water (5.5%) to make up the anticipated shortage of water in their area. These constructive answers are worthy of remark. The project is able to reach the settlement with villagers in case that sufficient compensations and correspondences would be provided to them.

The difficulties of settlement with villagers are suspected to be caused by their mistrust in WASA and insufficient correspondences in the past. The affects of the area were not consulted and took into confidence while the installation of tubewells during the ADB project in the past. They feared that the same practice would be repeated.

Comments of the respondents regarding the installation of tubewell were as follws.

Table 4 Comments of the respondents regarding the installation of tubewell

Comment	Frequency	Ratio（%）
The Government may only install the tubewells by using state power, disregarding the opposition of the affectees	17	8.5
The Government will provide only temporary compensation for pacifying the resistance of local people	25	12.5
The Government is not trustworthy because in past no meaningful compensation was provided to the affectees owning private tubewells in the project area	129	64.5
The Government never consulted the local population	29	14.5
total	200	100

Comments of local leaders were the same as those in the above table.

3) Conclusion of the survey

Summing up the results of the survey, the recommendations for promoting the project in the area where hostility has been lingering are as follows:.

 A relationship of trust needs to be established between WASA and local population through direct dialogues.

 WASA with support and cooperation of the Government should present effective proposals to local population to compensate the probable lowering of groundwater levels.

A possibility to reach agreement with local population might be created through WASA's continuous efforts.

(7) Activities after the completion of the survey

1) On completion of the field survey of the households in the targeted villages, the Consultant discussed with the WASA's representatives on the survey results, requesting them to take initiatives in the matter, starting the negotiations with the local population, first with the regional leaders such as Nazim Tehsil Chiniot, influential landlords, etc.

2) Notwithstanding such movement on the side of the execution agency, people's resistance broke out on December 23, 2002 when a drilling machine was delivered to the site for soil testing under the Basic Design Study. A crowd of some 100 residents around the No. 2 wellfield site, led by a provincial parliament member, gathered near the site and demanded to withdraw the equipment.

3) Faced with this incident, WASA was abruptly thrown into direct negotiations with local population. The subsequent events turned and twisted around persistent resistance of people, along with the interference of the government. The situation proceeded as presented in the following table:

Table 5 The situation proceeded as presented

Year	Date	Event	Description
2002	Dec. 23	Resistance of local population came to surface.	A gathered crowd of some 100 demanded to withdraw the study team to withdraw survey machine.
	Dec. 24	Dialogue between WASA and local population	In Chiniot city, Chairman and Deputy chairman of WASA had talks with representatives of Tehsil Chiniot and local residents
	Dec. 31	Local conference for discussions on the matter	At the public office of Metherma village, near the No. 2 site, representatives of related villages held a meeting to discuss on the matter with presence of WASA top officials and Nazim Tehsil Faisalabad.

2003	Jan. 8	Conference at Lahole chaired by the Chief Secretary, provincial government	Under the direction of the Chief Minister, the Chief Secretary called for the meeting of local authorities concerned about the matter. The conference concluded to support to continue the study.
	Jan. 18	No. 3 wellfield site was offered by an influential landlord.	As a solution for dispelling local resistance, a new land for the study (No. 3 site) was offered to WASA by an influential landlord in Metherma village located between the No. 1 and No. 2 sites. WASA and the study team inspected the proposed site, which was judged to be a relevant site for the study.
	Jan. 20	Mobilization of equipment to No. 3 site was cancelled due to resistance of people.	While preparing for mobilization of testing equipment to No. 3 site, a crowd of people gathered around the site to demonstrate their objection Chiniot police interfered in confrontation and WASA accepted police advice not to force mobilization. As a result of this incident, WASA decided to suspend the development of an additional wellfield in Chenab area for fear of stirring violence among resistant local population, and proposed to the study team to move the site to an alternative site along the Jhang Branch Canal.
	Jan. 22	Japanese side decided to cease the study for groundwater development	Since the new proposal by WASA had no assurance of agreement of stakeholders in a new site and lacked technical feasibility, the Japanese decided to cease the study for this part.
	Jan. 22	Site visit by the Minister and Secretary of HUD/PHD Dept.	Top officials of the HUD/PHD Dept visited Chiniot, in the hope of solving the matter through direct talks with local representatives around the site. The situation was found unfavourable for the project.
	Jan. 24	Conference between the Pakistani and the Japanese sides at EAD Islamabad	An official meeting of responsible officials on both sides was held at EAD in Islamabad under chair of Joint Secretary of EAD with the Pakistani side represented by the Minister of HUD/PHE Dept, Chairman of WASA, etc and the Japanese side by officials from the Embassy of Japan and JICA. During the meeting, the Pakistani side proposed to the Japanese side to continue the study in an alternative site along the Jhang Branch Canal, while the Japanese side informed of its intension to close the study this time since a new site had no guarantee for security as well as technical feasibility. The conference was closed with a conclusion that the Pakistani side will renew an official request for the study in the alternative site, supported by documents verifying consensus of stakeholders in the new site as well as technical feasibility.

Appendix 5-16 Socio-economic Survey on 2nd Basic Design Study (2)

1. Introduction

Project area was situated near Faisalabad city along the left bank of Jhang Canal on Faisalabad-Sargodha road. It was consisted of nine villages/towns stretched in almost 36 sq kilometers (12kmx3km). Land of the project area was fertile and almost all the area was under cultivation. The present project area under study was selected for the installation of tube wells for water supply to Faisalabad city.

Naturally, people might have their concern about the depletion of ground water level and socioeconomic implications on their produce and livelihood. Given this context, this socioeconomic survey was conducted to get baseline information about knowledge and perceptions of the people about the installation of the tube wells in their area, and its likely implications on their existing socioeconomic support systems. Furthermore, the nature of water resources, water needs of the people and the extent of their dependencies on the existing water resources in connection with their socioeconomic conditions were major concern of this study.

2. Methodology

The nine villages of the project area and three localities of Faisalabad city were selected for conducting survey. A team of local experts and social researchers carried out the survey to explore perceptions and concerns of the population of the study area. (Refer to the attached map)

<u>Site-A. Project Area</u>: Nine villages/towns along the Jhang Canal on Sargodha road near Faisalabad city where tube wells of the project are proposed to be installed.
<u>Site-B. Faisalabad City Area</u>: Three selected localities with different socioeconomic profile.

A representative sample of 220 households from nine villages of the project area was selected and interviewed by using quota-sampling procedure. An accidental random sampling was adopted to approach the head of the households/respondents of the villages. The village-wise detail of population, number of households and sample size of respondents is given in Table 1.
Similarly, three localities of Faisalabad City area with different socioeconomic characteristics were selected by adopting purposive sampling procedure. A sample of 82 households were selected by random sampling and interviewed by following quota-sampling procedure. The localities-wise sample size is shown in Table 2.

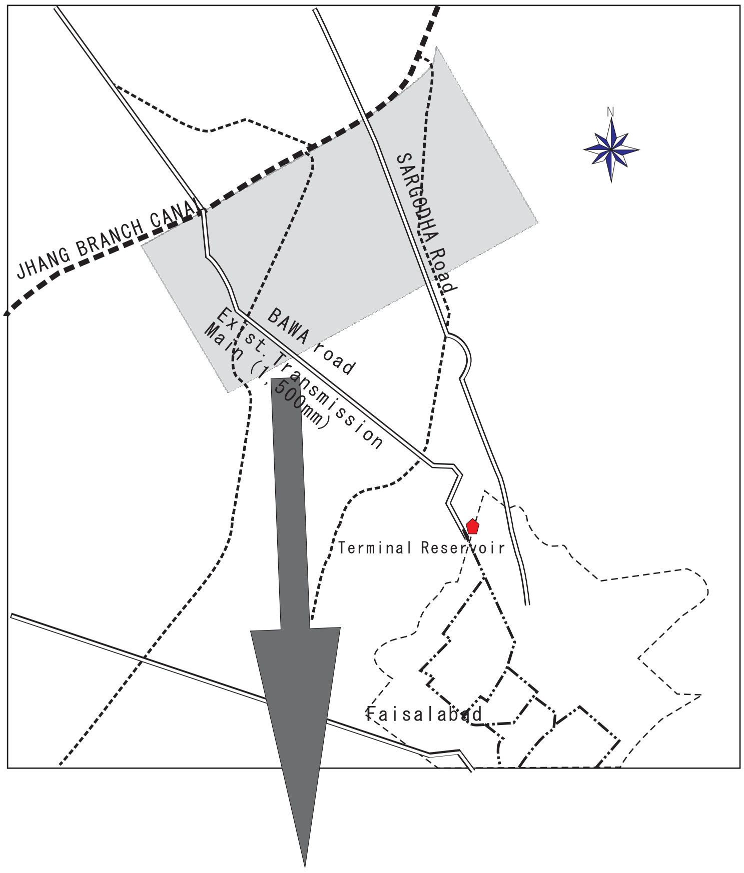


Fig.1
LOCATION MAP OF VILLAGES FOR THE SOCIAL SURVEY IN THE 2nd STAGE

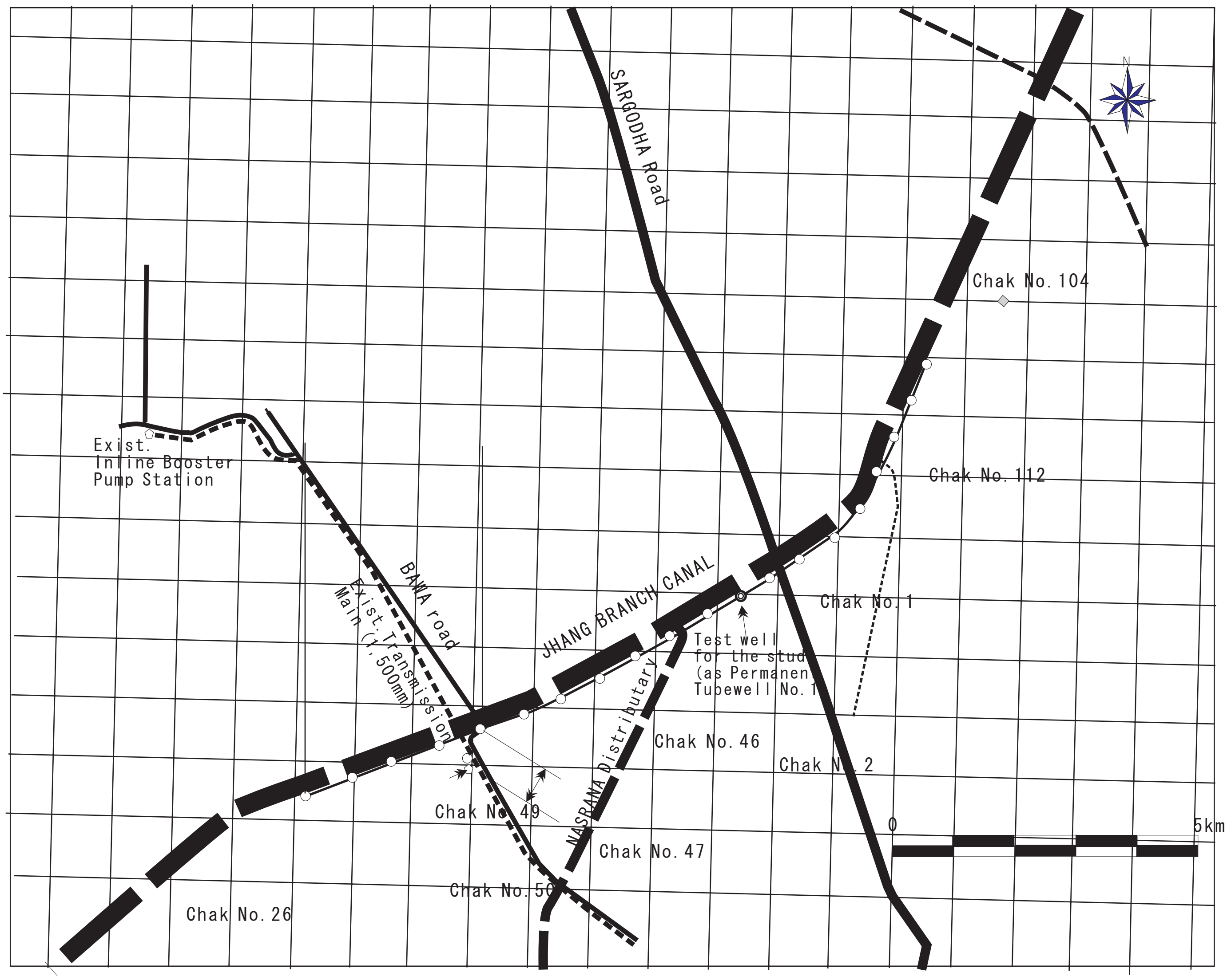


Table 1 Village-wise sample size of the households of project area

No	Chack No. / Village name	Population	Households	Sample Size	% of Households
1	1/R.B (Rasool Pur)	3,527	490	11	5.0
2	2/R.B (Ram Dewali)	4,713	733	14	6.4
3	26/R.B (Hargobind Pura)	10,654	749	24	10.9
4	46/R.B (Dhandra)	3,936	573	13	5.9
5	47/R.B	3,441	467	20	9.1
6	49/R.B (Munda Pind)	10,892	2.133	58	26.3
7	50/R.B (Sathyala)	8,463	1,783	48	21.8
8	104/R.B (Harmoay)	5,212	740	18	8.2
9	112/R.B (Kharral Wala)	3,639	542	14	6.4
	Total	54,477	8,210	220	100.0

Table 2 Sample size of the households from selected localities of Faisalabad City

Name of Locality	Sample Size	Percent (%)
Gulistan Colony	34	41.5
Hajvery Town	24	29.3
People's Colony	24	29.3
Total	80	100.0

The following tools and techniques of data collection were used to collect comprehensive and detailed information related to the project.

(a) Sample survey

A representative sample comprising of 320 respondents including 20 local leaders/influential from both the sites i.e. nine villages of the project area and three selected localities of Faisalabad city were interviewed.

(b) Interviewing and interview schedule (Questionnaires)

By applying interview technique for data collection for two different sites, two separate semi-structured interview schedule (questionnaires) were used. A separate interview guide (checklist) for interviewing the influential of the project area was used to conduct their in-depth interview. Copies of the questionnaires and checklist are attached in the appendix-I and II.

(c) In-depth interviews with the local leaders/influential

From each village of the project area at least two local readers/influential were interviewed by using interview guide as tools for data collection (copy attached as Appendix-III). It enable to have free and frank conversation with the influential of the project area to extract their understanding and perceptions about the installation of tube wells of the project for water supply to Faisalabad city.

3-1 Salient Findings (Site-A Village Area)

(1) Age and occupation of the respondents

83.1% of the respondents was in age-bracket of 21-60 years and 81.4% were the head of their households. 74.1% of the respondents were engaged with agricultural farming by occupation demonstrating their dominant agrarian economic activities. Less than one tenth of the respondents were doing business whereas rest (18.2%) was either employed in private sector or working as labor most probably in nearby city Faisalabad.

(2) Employment status of family members

92.2% of respondents had large family size composing of 4-15 family members. It indicated a dominant feature of a joint family system prevalent in the project area. Further, no family member of 60.9% of the respondents was employed. Rather families of two fifth of the respondents (41.8%) had no student (any family member attending any educational institution). It was an indication that mostly poor people of the project area had least preference to send their children to school.

(3) Major sources of family income and expenditure

60.9% of reported their monthly income less than Rs 5,000/- and correspondingly almost the same percentage (64.6%) mentioned their monthly expenditure less than Rs 5,000/-. It depicted the dominant prevalence of poverty in the project area. 81.8% of the respondents ware engaged in agriculture and (36.4%) in cattle farming. Almost a little less than one third (31.0%) of the respondents reported labor activities as their major source of income. Majority of them was working in the textile factories in the nearby city Faisalabad. Table 3, 4 and 5 are evident to these findings.

Table 3 Monthly expenditure

Monthly Expenditure	Frequency	Percent (%)
3,000 and below	86	39.1
3,001 – 5,000	70	31.8
5,001 – 10,000	53	24.1
10,001 – 15,000	9	4.1
15,001 – 20,000	2	0.9
Total	220	100.0

Table 4 Monthly income

Monthly Income	Frequency	Percent (%)
3,000 and below	73	33.2
3,001 – 5,000	69	31.4
5,001 – 10,000	54	24.5
10,001 – 15,000	16	7.3
15,001 – 20,000	4	1.8
Above 20,000	4	1.8
Total	220	100.0

Table 5 Majority source of family income

Response	Frequency	Percent (%)
Agriculture farming	180	81.8
Cattle Farming	80	36.4
Farming Labor	10	4.5

Business/Trade	21	9.5
Other (i.e. labor on looms in Fsd.)	58	26.4

(4) Agriculture land, farming and livestock/ Means of living

61.8% of the respondents had self-owned small cultivated land holdings ranging from one to five acres. Mostly, being the self-cultivators of their agricultural land, 83.6% of the respondents was engaged in agricultural farming. However, a little less than one fifth (18.6%) of the respondents were either tenants or lessees of the agricultural land. They usually cultivate wheat and fodder for self-consumption and for their livestock respectively. Sugarcane was their major cash crop that needed plenty of water round the year.

Cattle farming were the second major source of income of the people in the project area. 74.1% of the respondents owed their cattle and majority had 2-3 buffaloes and cows for selling milk to supplement their family income. However, goats and sheep were the second major strength of the livestock of the project area.

(5) Water resources for domestic and farming use

Hand pumps and electric motor pumps were the major source of water for domestic use and livelihood whereas canal water and tube wells were the only two sources of water for irrigation of agricultural land of the project area. 75% of the respondents, however, reported the non-availability of canal water found the year and correspondingly almost equal percentage (74.5%) regarded tube wells as the only major substitute source of canal water for irrigation of their agricultural land. However, 67.7% of the respondents considered tube wells as an expensive and not affordable source of water for irrigation, and 39.5% of them attributed the high expenses of tube well water to costly instruments and maintenance. Thus, considering a cheaper option, people showed their preference for canal water supply for their agriculture instead of ground water.

(6) Lowering of ground water level

However, 48.6% of the respondents were able to relate the lowering of table/ground water level with the installation of privately owned tube wells for extracting water for irrigation of agricultural land, and 41.8% linked it with drought like conditions due to no rains in the entire country during the previous year (Table 6). At the same time, 80.9% of the respondents were able to relate rain and nearby canal water with recharging of the ground water level in the project area (Table 7).

Table 6 Knowledge about causes of lowering the ground water level

Response	Frequency	Percent (%)
No rain	92	41.8
More tube wells (over pumping)	107	48.6
Other (specify)	5	2.3
Total	204	92.7

Table 7 Knowledge about source of recharging the ground water level

Response	Frequency	Percent (%)
Rain water	163	74.1
Nearby canal water	15	6.8
Don't know	21	9.5
Other (specify)	12	5.5
Total	211	95.9

(7) Installation of privately owned tube wells in project area

95% of the respondents were aware about the installation of the privately owned tube wells in the project area and they identified the local landowners as the installers. According to 61.4% of the respondents any one who was in need of more water for irrigation installed tube wells and expect 10% of the respondents no one had objected on the installation of privately owned tube wells in the project area. Rather, they were least concerned to install tube wells in their self-owned land.

(8) Reasons of no objection on installation of private tube wells

Three major reasons of such no objection on the local installation of tube wells in the project area were the common need of tube well water for irrigation of agricultural land (sharing of tube well water by purchasing), installation of tube well in self-owned land and "no reason." A little less than half (46.4%) of the respondents reported the purchase of tube well water and almost an equal percentage (43.6%) considered it as an expensive source of water. At the same time an overwhelming majority of the respondents was not satisfied with the available quantity of canal water and tube well water to fulfill their present needs of water for irrigation. Table 8 represented the aforesaid findings.

Table 8 Objection on installation of private tube wells

Response	Reason	Frequency	Percent (%)
Yes	Water level will decrease	22	10.0
No	Sharing water from tube wells on payment	85	38.6
	Due to installation of tube wells In self-owned land	48	21.8
	No reason	36	16.4
	Other (Specify)	5	2.3
	Not applicable	24	10.9
	Total	220	100.0

(9) Knowledge about the installation and benefits of the project tube wells

83.6% of the respondents were aware about the project site for the tube wells installation and almost an equal percentage (70.9%) reported the purpose of the project tube wells was water supply to Faisalabad City. One fifth (20.9%) of the respondents, however, were not familiar about the installation of tube wells of the project in their area.

Table 9 Knowledge about the purpose of installing the project tube wells

Response	Frequency	Percent (%)
Water supply for Faisalabad	156	70.9
Water supply for native land/people	10	4.5
Don't know	46	20.9
No response	8	3.6
Total	220	100.0

Almost one third (35.4%) of the respondents foresaw job opportunities for the local people during and after installation work of tube wells of the project. Considering a water supply project almost one fifth of respondents expected water supply to their village, and two fifth (41.8%) reported no benefit for the project area rather they foresaw loss for their area.

(10) Social organization and stratification

People of the project area had great regard to their elders and traditional authority figures. Normally they prefer consensus-based decision making and collective response to the issues by following the norms of mutual consultations. They showed their trust and respect to their local leadership. Anyhow, they also watched carefully the conduct and integrity of their local leaders to ensure their collective interests.

Due to small land holdings and farming patterns of typical agrarian society, people of the project area had many socioeconomic commonalities among them. They had almost similar nature of sources of income and dependencies on the local support systems. Such a common socioeconomic features depicted the homogeneous character of the local population. Given this context, they had common problems and concerns with the installation of tube wells of the project.

(11) Local perceptions about the installation of tube wells

Shortage of canal water compelled the farmers of the project area to install tube wells for irrigating their agricultural land. Their increasing dependency on ground water led them to install more tube wells in the project area. Ultimately, it caused lowering of ground water level

Understandably, almost majority of the respondents anticipated shortage of ground water by linking it with lowering of ground water level due to the installation of tube wells of the project. Anyhow their concern was only with the quantity of canal water for irrigation of their agricultural land regardless tube wells costly source of water for irrigation with negative effects on the fertility of their lands. Canal water was considered with twofold benefits i.e. irrigation and improvement of fertility of agricultural land.

(12) Local apprehensions about the installation of tube wells

Considering the installation of tube wells in their area, village people of the project area anticipated without sufficient supply of canal water they would be deprived of their only major substitute source of water i.e. ground water. With such understanding people of the project area foresaw serious set back to their agriculture and livestock, which were their major source of income, livelihood and habitat.

At the same time, they were equally conscious about the water needs of the citizens of Faisalabad City and had feelings of in-group with them. They showed their conditional consent for supplying water to Faisalabad City from area if they were provided sufficient canal water for irrigation of their agricultural land.

3-2 Salient Findings (Site-B Faisalabad City)

(1) Water supply and its use

Public water supply was the only source of water especially for drinking purposes. People reported acute shortage of water supply especially in shanty-towns (poor localities). Duration and pressure of public water supply was not adequate to fulfill the water needs of the citizens. Similarly, people had complaints about the quality of water as well as quantity of water.

(2) Awareness about the project

Nearly half of the respondents (53.7%) living in the city area were not aware about the installation of tube wells of the project under study. Knowing about it through the interview, however, all the respondents welcomed such a project meant for water supply to Faisalabad City. Relevant table is attached below.

Table 10 Knowledge of respondents of Faisalabad City about the project

Item	Response	Frequency	Percent (%)
Installation of tube wells	Yes	38	46.3
	No	44	53.7
Name of execution agency	Yes	6	7.3
	No	56	68.3
Area of tube well installation	Yes	22	26.9
	No	60	73.1
Source of public water supply	Yes	43	52.5
	No	39	47.5
Purification of public water supply	Yes	21	25.6
	No	61	74.4
Distribution of public water supply	Yes	27	32.9
	No	55	67.1

Although, they were not satisfied with the performance of WASA regarding its responsibilities to supply sufficient water to the city.

4. Conclusion and Recommendation

(1) Conclusion

(a) The two major sources of income of the people of the project area are agriculture and employment/labor work in Faisalabad City. Their agriculture depends on the source of water, and their employment/labor work is connected with the people of Faisalabad City. Both of the means of income are equally important to them.

(b) There is efficient network of canal water distribution in the project area with insufficient supply of canal water. Ground water is the secondary and costly source of water for irrigation of the agricultural land of the people. People of the project area preferred canal water to ground water for irrigation.

(c) People of the project area had in-group feelings with the citizens of Faisalabad City. They had no objection on installation of tube wells by the local agricultural landowners in their land. It may equally be applicable for government project if the tube wells would be installed in governmental owned land.

(d) People of the project area considered the installation of tube wells as a public welfare project meant for supplying drinking water to the citizens of Faisalabad City. However, if their canal water needs for irrigation of their agricultural land is fulfilled, they had no objection on the installation of tube wells of the project.

(2) Undertakings/countermeasures by the project

(a) A workable dialogue with the representatives of the locals of the project area is needed to minimize the chance of misunderstanding amongst the locals and the execution of the project.

(b) Participation of the locals in the project activities during its execution and after completion must be ensured so that they may have share in economic benefits and employment opportunities of the project.

(c) A regular and institutionalized interaction of the project execution agency with the locals of the project area be maintained. For this purpose, establishment of Project Liaison Committee (PLC) is recommended which should manage and monitor the implication of the project.

(d) The nominee of the project execution agency and the representatives of the local population may represent the PLC. A local norm of consultation and consensus-based decision making must be adhered to ensure the participation of the local population to own the decisions about the project.

(3) Recommendations to Pakistan government

(a) Viable substitute source of water especially the canal water should be ensured for the fulfillment of irrigation needs of the project area.

(b) The genuine socioeconomic problems especially the sewerage system of the villages be addressed by taking the matter with the agencies concerned.

(c) A meaningful progress of the mitigation measures regarding the plentiful supply of canal water to the project area may be helpful in trust building and winning the cooperation of the population of project area. Therefore mitigation measures of the likely adverse impacts of the project on the local population are recommended to be the integral part of the project activities.

Appendix 5-17 WASA's Water Tariff

(Revised on March 2004, Applied from January 2004)

1. Tariff for Water Supply

	Category	Meter	Dia.	Plot size	Conversion into metric system	Rate/Month	
1)	Domestic	Without	1/4″	Up to 2−1/2 *Marla*	60m^2	72	Rs
				2−1/2 ˜ 3−1/2 *Marla*	～88.5 m^2	108	
			(Above 1/4″ will be charged according to rates at category 5)	3−1/2 ˜ 5 *Marla*	～126.5 m^2	126	
				5 ˜ 7 *Marla*	～177 m^2	210	
				7 ～10 *Marla*	～250 m^2	210	
				～ 20 *Marla*	～500 m^2	280	
				＜40 *Marla*	～1,000m^2	560	
				>=40 *Marla*		840	
			Note	1 *Marla*=272 ft2=25.3 m^2			
				1 *Kanal* = 20 *Marla*			
				1 acre = 8 *Kanal*			
2)	Domestic	With				Per 1,000 gallon	=/4.55 m^3
				Up to 5,000gallon/month	～22.75m^2/month	34	Rs
				5,000～10,000/month	22.75～45.5m^3	35	
				>10,000 gallon/month	>45.5 m^3	42	
3)	Industrial commercial & other non residential properties	Without	1/4″	＜＝3 *Marla*	76 m^2	280	Rs
				3< <= 6 *Marla*	76 ～ 150 m^2	420	
				6< <=10	150～250 m^2	700	
				10*Marla*< <1 *Kanal*	250～500 m^2	1,120	
				1 *Kanal* <2 *Kanal*	500 ～1,000 m^2	2,100	
				>=2 *Kanal*	> 1,000 m^2	2,800	
4)	Industrial commercial & other non residential properties	With		Per 1,000 gallon	Per 4.55 m^3	46.5	
				Incase of defected meter, the ferule size connection will be charged according to rates at category 6			
				Incase of temporary disconnection, consumer have to pay min.15% of total bill			

5)	Industrial Commercial	Without	Above 1/4"	<10 *Marla*	<250 m^2	700	
				10<= <20 *Marla*		1,120	
				20 *Marla*<=	1.000 m^2<=	1,960	
6)	Industrial Commercial	Without		3/4"		2,100	Rs/month
				1"		2,800	
				1-1/2"		8,400	
				2"		16,800	
				3"		42,000	
				4"		84,000	
				6"		280,000	
				More than 6"connection size, the rate will be charged as per below mentioned formula.			
				6"/month x d x d x 4 (d=connection size in ft)			
7)	The Government registered religious/charity units/departments & Mosques will be charged as 70% of domestic rates						
8)	Aquifer charges(Fee on tube wells)						
	a. Industrial, Commercial, Government, Semi Government, Corporation, Irrigation Dept					11,200	Rs/cusec= 1.7m^3/min
	b. Textile Processing and Hosiery units					9,100	Rs

2. Tariff for Sewer/Drainage

	Category	Plot size	Conversion into metric system	Rate/Month	
1)	Domestic	Up to 2-1/2 *Marla*	60m^2	42	Rs
		2-1/2 ~ 3-1/2 *Marla*	～88.5 m^2	56	
		3-1/2 ~ 5 *Marla*	～126.5 m^2	105	
		5 ~ 7 *Marla*	～177.1 m^2	175	
		～10 *Marla*	～250 m^2	210	
		～ 20 *Marla*	～500 m^2	350	
		＜40 *Marla*	～1,000m^2	490	
		>=40 *Marla*		770	
		The above mentioned rates wiil be chargd upto three story's. On above 33%of the rate will be charged to each story.			

2)	The Government registered religious/charity units/departments & Mosques will be charged as 70% of domestic rates			
3)	Commercial	Shop, Shopping centers, Department store and Areades (per poit having one toilet/ wash basin/ sink tap etc.)	105	Rs
		Hotel (per bed/bath/tap/wash basin/toilet/sink etc.)	70	
		Restaurant (per wash basin/toilet/sink/ bath/tap etc.)	70	
		Private Hospital, Clinic, Clinical laboratories (per bed/bath/ wash basin/ tap/sink etc.)	50	
		Car services station (per lift/bay etc.)	1,260	
		Motor cycle service station etc.	170	
		Hair cutting saloon, Beauty Parlor, Hamam etc. (per bath/ wash basin/ sink/tap etc.)	50	
		Multi-story commercial plaza, Marriage hall (per $1000ft^2$=$92.9m^2$ covered area)	350	
		Governmental offices (per $1000ft^2$ covered area)	175	
		Private education dept./Schools/ Colleges/Institution (per $1000ft^2$ covered area)	252	
		Other units/ Departments (per $1000ft^2$ covered area)	252	
		Four star & five star hotels (per $1000ft^2$ covered area)	2,800	
			Per year,$1ft^2$	
4)	Industrial	Limited waste/used water discharge factories (through toilets/sink/point/wash basin etc.)	3.0	Rs/year
		Waste/used water discharge of small units (through toilets/sink/point/wash basin etc.)	1.5	
5)	Industrial	Bulk waste/used water discharge (per cusec=$1.7m^3$/min)	40,600	Rs/month

3. Fee for New Connection

	Category	Detail	Rate	
1)	Water connection	Domestic 1/4″ ferrule size	420	Rs
		1/2″ ferrule size and above	2,800	
2)	Sewer/ Drainage Connection	Domestic	280	Rs
		Commercial	700	
		Industrial	2.800	
3)	Reconnection	Re-connection fee for water and sewer will be half of connection fee.		