### 2-2-1-4 Policy for local contractors/products

This project will be implemented, employing locally produced basic materials for civil and architectural works such as cement. Moreover, since the country now produces high-quality industrial products that can be employed for the project such as pumps and piping materials as a result of efforts for industrialization promoted by the government policy, suitable materials and equipment are expected to be selected from the local market.

The preceding ADB project, Phase 1, was accomplished by local contractors selected as a result of the international tenders. With Faisalabad located close to the provincial capital, Lahore, the market in this economic zone is poised to provide qualified and capable contractors for drilling, civil and architectural works. For successful implementation of the project, the employment of local contractors is highly recommended. A typical example of cooperation by the local contractors can be seen in the stage of Phase 1 of this project, in which the Japanese contractor employed two local companies specialized in civil works. That stage was successfully completed in March 2006.

### 2-2-1-5 Policy for grade of facilities and equipment

All the facilities and equipment comprising the planned water supply system in this project are of similar types and grades to those installed under the preceding project by ADB, which have satisfactorily been operated and maintained by WASA since 1992. This policy will ensure consistency of the combined operation of the existing and the new systems by WASA.

The study found all the existing pumps for tubewells and pump stations properly functioning for intended operation. (The existing tubewell pumps are of vertical shaft type locally manufactured with license of a European maker, while those for the pump stations are of double-suction volute type of Japanese make.) Similar types of pumps are proposed for this project as well.

The existing and the newly completed systems can be operated independently until T/R, to which all water from the existing and the new wellfields is transmitted and then pumped out to the arterial mains for distribution to the city with the existing and the new pumps simultaneously working. Parallel operation of pumps transforms the performance of the pumps into a combined one. Difficulty arises here in adapting pump operation to extreme concentration of consumption during supply hours, which is currently limited to approximately 6 hours per day. The pumps become over-burdened, resulting in low pressure, which is one of the causes of cavitations. It is feared that the simultaneous operation of the existing and new pumps may risk a similar malfunctioning since the trend in city's consumption is unknown and appears difficult to control. In order to minimize such a risk, the pumps to be installed under the project are to be equipped with protective measures as follows:

a. The new pumps are designed to be installed at a lower level than the low water level of the new reservoir so that the suction pressure could become positive.

b. Control valves specially intended to avoid cavitation will be equipped with new pumps manual speed control of the electric motors.

Although a computerized speed-control system of motors is the most suitable arrangements for adapting operation to the demand fluctuation, WASA has no experience in this sophisticated equipment, and may pose difficulty in operation and maintenance. Instead the proposed measures are taken to contain malfunctioning of the pumps to a minimum level.

## 2-2-2 Basic Plan

### 2-2-2-1 Water sources planning

This project was formulated based on the Basic Design Study which was conducted in 2003. Some of the details were confirmed during the Detailed Design Study in 2005(e.g. electrical prospecting data of the tubewell construction sites etc). Since it was confirmed during this Implementing Review Study, that no major changes had occurred in the natural environment of the project wellfield, it was considered appropriate to adopt the plan of Basic Design Study for this Project.

1) Hydrogeological study for the project

(1) Objectives

A broad study for groundwater development for this project was performed during the 2003 basic design study, for collecting relevant information and data on the following elements for water sources planning:

①Optimum rate of discharge from one tubewell
②Drawdown at a design rate of discharge
③Extent of influence of pumping at group tubewells to the surrounding area
④Layout of group tubewells
⑤Drawdown at simultaneous discharge of group tubewells
⑥Prospect on regional groundwater level lowering
⑦Groundwater quality at the planned tubewells
⑧Situation of Jhang Branch Canal water flow

(2) Components of the Study

The groundwater development study consisted of the following components:

Table 2-2 List of Components of the Study for Groundwater Development

| | Components | Study period | Contents |
|---|---|---|---|
| 1 | Hydrogeological study | 1st & 2nd stages | Includes inspection of more than 100 existing tubewells |
| 2 | Geophysical survey | 2nd stage | Surface electrical resistivity survey at 24 points along the canal |
| 3. | Test drilling | 2nd stage | Test well (150 m x 1 No.)<br>Observation wells (120m x 2 Nos.) |
| 4. | Aquifer test | 2nd stage | A series of pumping tests were undertaken at the test well and 2 observation wells plus 3 observation wells provided by WASA |
| 5 | Water analysis | 1st & 2nd stages | Field test by the study team and analysis by WASA Laboratory |

WASA and the study team agreed that the results of the former studies in the area should be referred to. The studies included the following:

a. Study by WAPDA in 1960s
b. Study by Binnie and Partners in 1975
c. Study by Republican Engineering Co. (hereafter called "REC") in 1981

Further it was agreed between both sides that the results of seepage test undertaken by WASA in 2003 for the project tested by Irrigation Research Institute of the Irrigation Department of the Punjab province should closely be examined, as an important source of information of recharge quantity in the project area.

(3) Outline of the study results

a. Geophysical survey

* The survey was made at 24 stations to a depth of 200m in the planned wellfield along the left side of the Jhang Branch Canal. According to the analysis of the survey results, the subsurface geological section to a depth of 200m is largely divided into 3 groups, among which the second section appears the main aquifer of the area.
* The survey results tell that a basic drilling depth in the well field is 160m on average. The exact depths of the respective tubewells should be confirmed during the detailed design stage by the geophysical study at the very points decided for drilling. (The detailed design study was carried out in 2006 in the preceding stage of the project.)

b. Test drilling

* The locations of the test well with observation wells are shown in Fig. 2-2.
* The geological section and the structure of the test well installed under the study are shown in

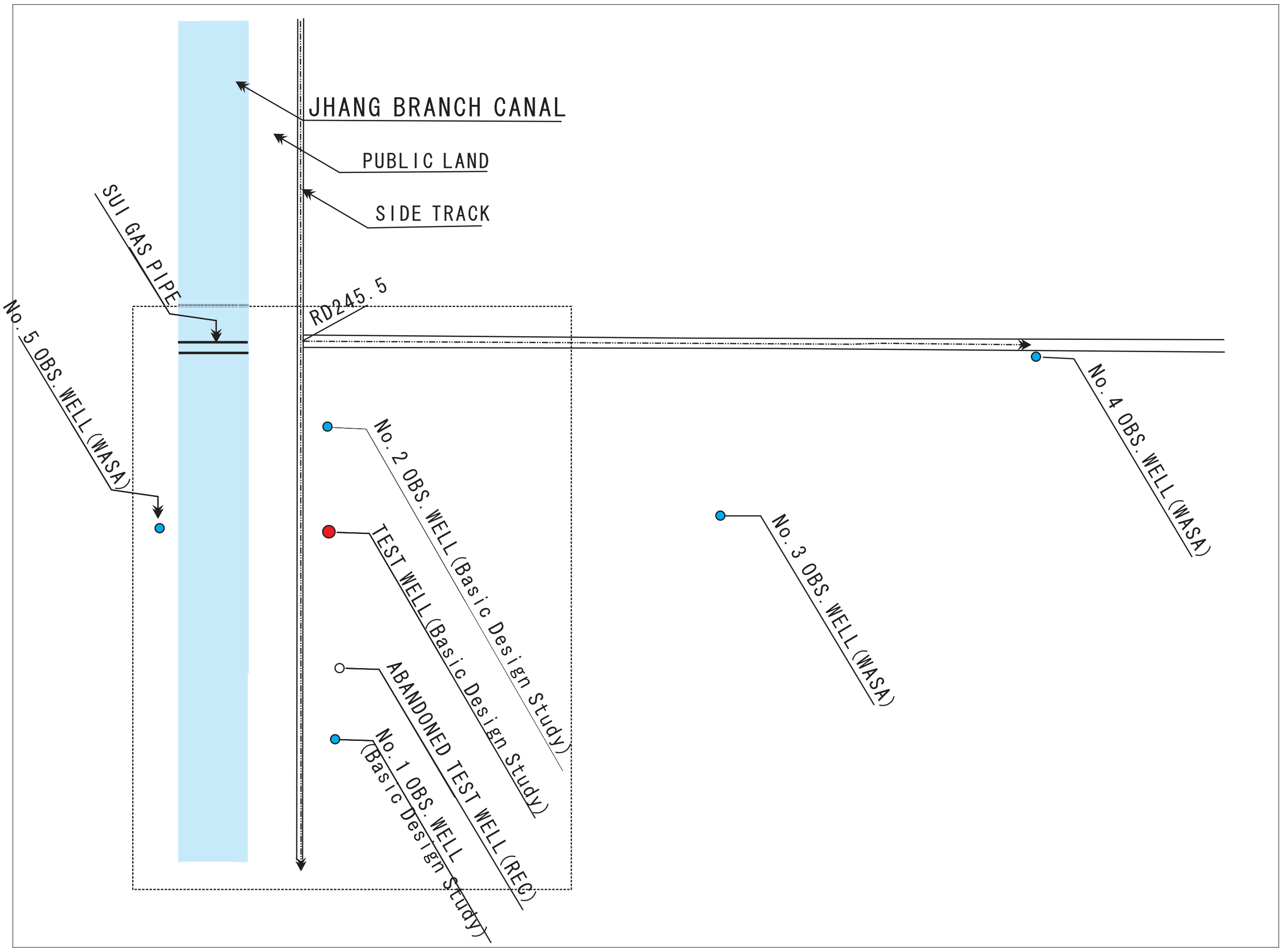
Fig. 2-3.

c. Aquifer test

* The water levels at the test well and observation wells were stabilized in about 360 minutes ater pumping started during testing due to apparently forced recharge from the Jhang Branch Canal.
* The analysis of the test results indicates that pumping at the planned tubewells would hardly affect groundwater level in the surroundings thanks to constant recharge from the canal, provided that an appropriate unit discharge rate is employed, with adequate spacing set between tubewells. During the closure of the canal in winter season when no recharge occurs, however, the level tends to gradually fall down, and eventually is likely to affect the surroundings on the medium to long term basis.

d. The test results are described in detail in the following section 2-2-1-2).

# FIG. 2-2 TEST DRILLING SITE ALONG JHANG BRANCH CANAL



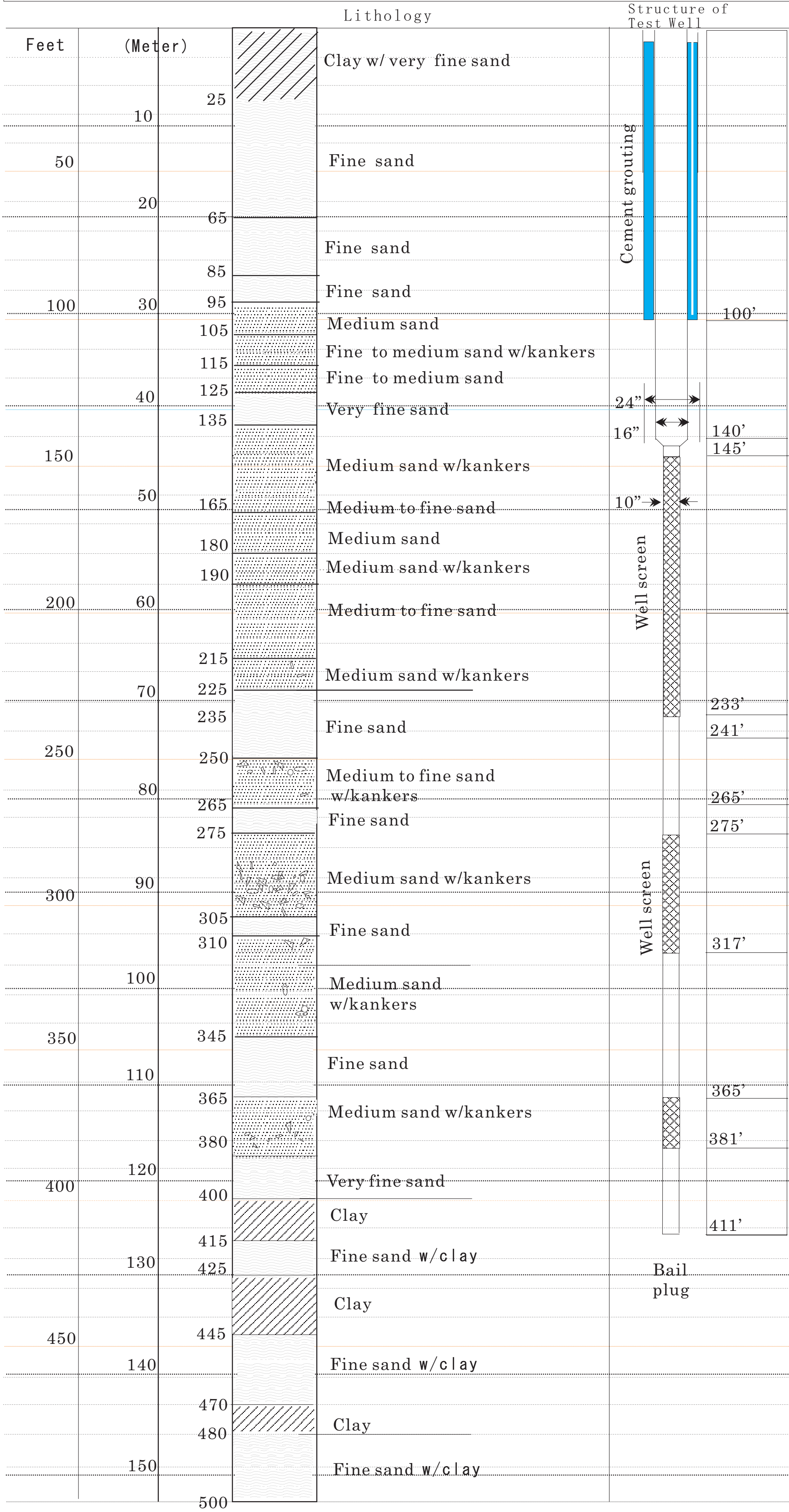

DRILLING SITE FOR BASIC DESIGN STUDY

Drilling machine is working for test well drilling. The tripod behind is for observation well No. 2.

The green belt on the left bank of the canal.

# Fig. 2-3 STRUCTURE OF THE TEST WELL FOR B/D STUDY

Well No. 22 Construction Date: 4 - 09 - 03 ～ 07 - 09 - 03
S. W. L.= 5.74 m(1988) Q . = 3.75cusec ( 375 m3/hr)




Characteristics of Test well

Main aquifer is sand divided into 3 sections by interbeds of clay. The drilling point of the test well had a thick sand layer corresponding to the second group, but lacks the third group. Accordingly although drilling reached a depth of 153m, casing and screen were installed to 125m, with the lower part left untapped.

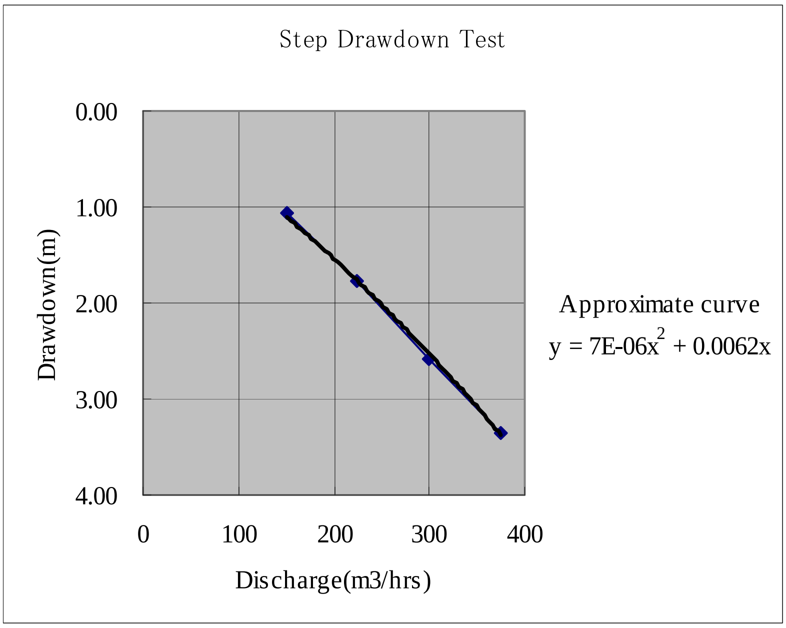
Each drilling point is planned to be confirmed for the depth by geophysical survey during the detailed design.

2) Water sources planning

(1) Discharge

REC, the local consultant, which conducted an extensive survey for groundwater development from the Jhang Branch Canal to the city, recommended a discharge of one tubewell to be within a range of 2 to 4 cusec. During the step drawdown test in this study, 4 incremental discharges within the said range were employed to examine an optimum rate. The results indicating the respective discharge rates versus drawdown are plotted in the following graph (Fig.2-4):

Fig. 2-4 Step drawdown graph




A plotted curve on log-log graph is almost straight without any abrupt change in its gradient, indicating that the ratio of drawdown to discharge is nearly proportional to each other. While any discharge keeps such a proportional relation with drawdown, it is defined as “safe yield”, and can be pumped without risking eventual depletion of the source. The results of the step drawdown test justifies REC’s former estimate of a unit discharge rate, ranging from 2 cusec to 4 cusec.

For the determination of a unit rate of discharge per well, the following conditions were taken into account:

a. The pumping water level at a tubewell gradually recovers to its initial static water level after pumping is stopped, depending upon the condition of recharge to the aquifer. In this study, it took about 6 hours for the test well pumped at a constant rate of 3 cusec to restore its initial level. It is not economical to idle 20 or more wells for 6 hours a day.

b. The existing tubewells in the Chenab wellfield discharge as much as 4 cusec from one well. As their daily idling hours are limited to 4 hours, the water levels at them do not return to their initial state by the time pumping re-starts, and the continuous operation of group wells in the wellfield has resulted in ever-increasing fall of groundwater level in and around the wellfield.

c. From such experiences at the site, the unit pumping rate for the project is proposed to be limited to less than 3 cusec to ensure water level recovery in an idling period as short as practicable, which will be 4 hours at maximum, as ongoing practice of WASA. An economic pumping rate to meet such requirements is 2 cusec (approximately 200$m^3$/hr). At this rate, the water levels of wells can be restored in about 4 hours after the pump stopped.

(2) Number of tubewells

a. Calculation of the number of tubewells

The factors to determine the required number of tubewells are as follows:

| | |
|---|---|
| * Targeted total discharge | 91,000$m^3$/day |
| * Unit rate of discharge from one well | 200 $m^3$/hr/well (approx. 2 cusec) |
| * Duration of pumping a day | 20 hours/well (4-hour idling) |

Based upon these factors, the required number of tubewells is:

91,000 ($m^3$/day) / ［200 ($m^3$/hr) x 20 (hrs)］ = <u>22.75 (say 23) wells</u>

In addition, standby tubewells are required for maintenance and repair. If 10% of the total number is assumed, it is 2 wells. As a result, the required number of tubewells totals “25”.

(3) Distance between adjacent tubewells

i. Estimate of seepage

The tubewells for the project are to be installed in a belt-like wellfield along the left side of the Jhang Branch Canal where artificial recharge from seepage of the canal can be anticipated. If their total discharge is less than an amount of estimated seepage, their water level can recover to the initial one during a proper idling period after it once lowered during pumping. A cycle of pumping and idling can maintain the regional groundwater level.

For the project WASA examined a range of seepage to the proposed wellfield along the canal in November 2003. The past study by REC for the area also included this type of test. The results are shown in the following table 2-3:

Table 2-3 Results of Seepage Tests along the Jhang Branch Canal

| | Client | Tested by: | Year | Estimated seepage rate |
|---|---|---|---|---|
| 1 | FDA/REC | Irrigation Research Institute | 1981 | 633 $m^3$/hr/km (of channel) =15,192 $m^3$/day/km |
| 2 | WASA | | 2003 | 437$m^3$/hr/km (of channel) =10,488 $m^3$/day/km |
| | | | Average | 535$m^3$/hr/km (of channel) =12,840 $m^3$/day/km |

Seepage tests are likely to result in a wide range of differences in the estimated rates, depending upon methodologies employed, physical properties of tested channels, etc., therefore, it should only be treated as an estimate. Nevertheless, it will still be needed as a reference to prevent excessive withdrawal from the canal, since the tubewells in this project will be constructed along the canal and will be recharged directly from it. Therefore, an average rate of the two tests is employed for this project. The rate can function as an index for the discharge of groundwater along the canal.

ii. Discharge of tubewells for irrigation

The existing tubewells for irrigation in and around the wellfield have been utilizing groundwater deriving from seepage. The amount of their discharge is estimated as follows:

a. According to the field survey by WASA for the project, there are 108 tubewells existing in an area about 20 km long and 3 km wide along the canal including the site for the proposed wellfield.
b. An average discharge from a tubewell is estimated as 1 cusec (100$m^3$/hr), based upon the practice of drilling for irrigation wells and capacities of pumps.
c. Their running hours are normally 10 hours daily. In addition, during two months in the rainy season every year, most of them are closed. This practice leads to an average operating ratio of 40%.
d. An approximate total discharge of 108 tubewells are estimated as follows:
   - Daily discharge along a section of 1 km of the canal
   = ［(108 wells) x (100 $m^3$/hr/well) x 24 hours x 0.4］ / (20km) =5,184 $m^3$/day/km

The total estimated discharge of irrigation tubewells accounts for about 40% of an average seepage rate of 12,840 $m^3$/day/km.

iii. Distances of proposed tubewells

Based upon the results of the foregoing estimate, the spacing between 2 tubewells for 23 regularly in operation is calculated as follows:

a. Total amount of seepage a day available for groundwater discharge in the area
   (It is assumed 90% of the estimated seepage can be utilized for the purpose.)

= 12,840 x 0.9 =11,556 $m^3$/day/km

b. Total amount of seepage a day allowable for groundwater discharge by the proposed tubewells
= (a) - (discharge by irrigation wells) = 11,556 - 5,184 = 6,372 $m^3$/day/km

c. Daily discharge rate of one tubewell for the project
= 200 $m^3$/hr x 20 hrs/day = 4,000 $m^3$/day

d. Length of the canal that can recharge the discharge rate (c)
= (c) / (d) = 627m

As a result of this estimate, a distance between two wells should be <u>600 m</u> as a minimum requirement. The total distance of the wellfield with 25 tubewells along the canal is, therefore,

(25 - 1) x 0.6km ≒ 14.5 km.

The major parameters for planning water sources for the project, defined in the foregoing estimates, are summarized in the following table 2-4.

Table 2-4 Major Parameters for Water Source Planning

| | Parameters | Target |
|---|---|---|
| 1. | Planned daily maximum discharge | 91,000 $m^3$/day |
| 2. | Planned daily discharge / one well | 200 $m^3$/hr =4,000 $m^3$/day |
| 3. | Total number of tubewells | 23 + 2 as standby |
| 4. | Daily running hours | 20 hrs/day |
| 5. | Distance between 2 wells in the well-field along the canal | 600 m |

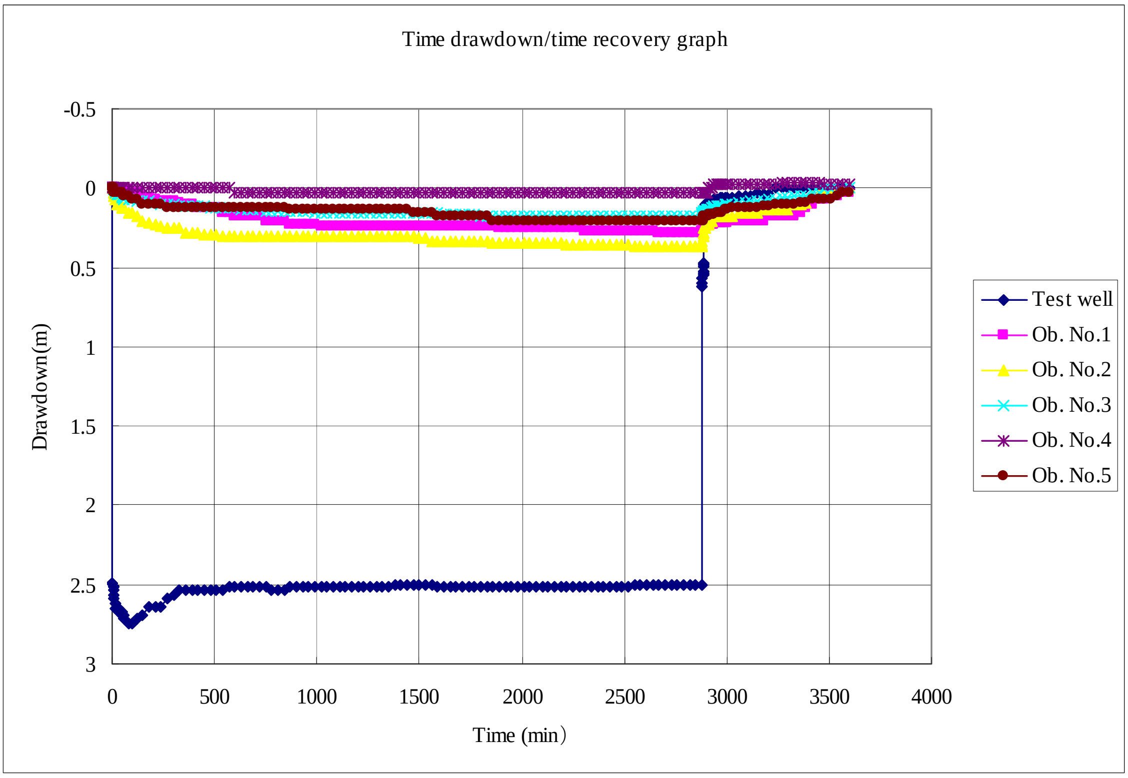
(4) Examination of Level Lowering and Extent of Influence

An extensive farming area spreads south of the Jhang Branch Canal, growing mainly sugar cane with delivery of canal water, together with groundwater discharge from private tubewells for irrigation. Since the operation of proposed tubewells on completion of the project has been feared by the local communities to affect their tubewells, lowering regional groundwater level, the extent of their influence is examined as follows:

i. Results of the time drawdown test

Drawdown at tubewells by pumping and its influence on their vicinity can be estimated by hydraulic calculation and graphical analyses, employing the values of “T” and “S” produced through the analysis of the results of time drawdown test, plotted in Fig.2-5.

Fig. 2-5 Time drawdown/time recovery graph




The time drawdown test for this study was carried out at a constant discharge rate of 3 cusec (about 300m$^3$/hr) for successive 48 hours, and was followed by the time recovery test after pumping stopped to confirm the recovery of levels. The members for the test consisted of one test well and five observation wells where the changes of water levels were measured simultaneously at predetermined intervals of time. The plotted graph shows specific features of the test as follows:

a. The static water level at the test well was 5.334m before starting pumping. In about 360 minutes after pumping started, the level stabilized at 7.83m, without any further drawdown until the end of pumping in 2,880 minutes. The thus stabilized level indicates the occurrence of direct recharge from the nearby canal, in this case the Jhang Branch Canal.

b. The water levels at 5 observation wells lowered little by little as pumping proceeded. All of them reached stabilization in 300 minutes after pumping at the test well started.

c. The drawdown at No. 4 observation well at a maximum distance of 359m from the test well was recorded as 20mm at the end of 48-hour successive pumping at the test well. The location of this well seemed to undergo no substantial influence by pumping at the test well.

d. The recovery of the level at the test well to the static water level took 6 hours after pumping stopped, while those at 5 observation wells, more or less 10 to 12 hours.

ii. Calculation of Coefficients of Aquifer

As described in the foregoing section, the test well seems to have received recharge from the canal during a greater part of its pumping period. The data consisting of stabilized level for most part offers no significant information on the features of aquifer function. The analysis, therefore, mainly depended upon the data before substantial recharge occurred, using the relation of drawdown to time and distances of the test well and 5 observation wells. The process of calculation for the coefficients of the aquifer "T" and "S" are referred to Appendix. The results are summarized in the following table 2-5, together with reference to the data obtained through the past studies covering the same area and its vicinity.

Table 2-5 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 12,000$m^3$ /day.

The calculated values of T and S in this study differ widely, probably due to intensive influence of direct recharge from the canal. However, in comparison to those in the previous studies, they are interpreted to fall within an acceptable range.

iii. Prediction of extent of influence

Using the values of "T" and "S" thus obtained and the relation of distance from the test well to the observation wells with drawdown, the extent of influence of pumping at the proposed tubewells was examined. The detailed process of calculation is shown in Appendix. For this project, the unit rate of discharge from one well is to be 200$m^3$/hour at successive pumping of 20 hours/day, with the idling time of 4 hours. The summary of the analysis is described as follows:

a. The tubebwells are to be installed at intervals of 600m along the canal. Interference among the tubewells occur in case the influence of pumping at one well extends beyond a radius of 300m. The hydraulic analysis predicts that the radius of influence remains within a radius of that limit during 20 hours of daily pumping at a well, thanks to constant recharge from the canal which equals discharge. Accordingly pumping at the proposed tubewells will not exert any adverse influence on surrounding tubewells for irrigation.

b. All canals of the Jhang Branch Canal are closed every year for about one month for their maintenance and repairing works. During this period, they have no delivery from the headwork, and there is no recharge from them. As a result, water level at pumping wells continues to lower with a larger rate of drawdown than during the normal period. The influence of pumping can extend beyond the radius of 300m. At the end of closure in about one month, the radius of influence is likely to extend to 2.5 km at the minimum from pumping wells and as far as 5 km in the worst case, depending upon the values of T and S. Influence of a middle range between the minimum and the maximum is likely to occur in the actual scene.

c. The amounts of lowering of level are predicted, based upon the assumption described in the foregoing section as follows:

* In case of the minimum influence:
  The radius of influence reaches 2.5 km from pumping wells.
  Groundwater level at a distance of 730m from wells is calculated to lower by 0.25m after 30 days of canal closure.
* In the worst case
  The radius of influence will enlarge to a distance of 5km from pumping wells.
  Groundwater level at a distance of 1,900m from wells is to lower by 0.25m after 30 days of canal closure.

d. To make the matter worse, lowering of level is likely to be almost doubled due to interference between two adjacent pumping wells, since the influence from each well extends beyond the limit of 300m.
If influence of a middle range of two extremes is assumed, such effect of influence will result in lowering of groundwater level by 0.5m at a distance of 1.3 km from the wells.

e. Once groundwater level is thus lowered, it will not be able to return to the former one even after the canals restart delivery, since it was assumed to utilize seepage to its full extent for discharge of existing tubewells for irrigation as well as for the project. The lowered level will probably persist, and the closure of canals each year will further deepen the level. This assumption leads to a prediction that within 5 years after the tubewells for the project starts the operation, groundwater level at a distance of 1 km from them will be 1 m deeper than the initial one.

f. The discharge of this project relies on the recharge of the canals, and the lowering of the groundwater level also depends on the extent of recharge from the canals. Although the level of recharge has been clarified to a certain extent by the data of WASA seepage tests, the influence from the wider area is still uncertain. In the worst case scenario, the lowering of groundwater level will gradually progress during the closure of the canals.

g. There is another risk of influence from 28 tubewells in the Chenab wellfield, where groundwater level has continuously been lowering for the past 10 years. This study included the estimate of their influence to the wellfield along the Jhang Branch Canal. The analysis indicates the influence soon reaches this area and that within 10 years will lower the level in this area by 2 to 3 meters.

iv. Groundwater Monitoring Program

The hydraulic analysis indicates regional groundwater level will probably be affected by the operation of the tubewells for the project, and within about 5 years after the commencement of operation a part of tubewells for irrigation located near the project wells may face difficulty in pumping.

However, since the prediction of this study is based upon the results of aquifer test at a single tubewell, it goes without saying its accuracy is in a limited extent. It is recommended, therefore, that WASA establish a properly constructed monitoring system and launch measurements of level at its own tubewells as well as at monitoring stations with reference to the following remarks.

a. Monitoring wells should comply with WASA's standard design.

b. Monitoring wells should be installed along three lines, one at a distance of 500m from the canal, the other at 1,000m, and another 3000m. Each line should have 5 monitoring wells at the minimum.

c. The tubewells for the project should be subject to daily measurements of static water level and the dynamic water level at the respective wells just before pumps are stopped. Discharges should also be recorded by flow meters installed in the respective stations.

d. Measurements of levels at the monitoring wells should be undertaken by the responsible operators at the nearby tubewell stations. During one month after pumping from the tubewells starts, levels at the monitoring wells along a line of 500 m distance should be measured in particular as frequently as practicable, at the intervals of 30 minutes to 1 hour or the like. If influence tends to expand more than expected, such measurements should be extended to those along a line of 1,000m. After one month, measurements at monitoring wells may be taken at a less frequency, say several times a day.

If laborious monitoring is continued for one year after commissioning, the data for predicting precisely the extent and size of influence of pumping wells could be collected, at least for the period when canals continues to deliver water.
During the period of canal closure, the extent of measurements at monitoring wells may expand to a line of 3,000m.

It is further recommended that WASA should develop friendly relations with local communities for risk management concerning the security of regional groundwater through the establishment of a liaison committee consisting of representatives of stakeholders in the area including those from WASA and local administration. Representatives from concerned villages should be responsible for preparing the inventory of tubewells in their areas and regularly reporting their conditions. The committee should constantly be aware of the general opinion and the movements of the society, and intervene actively and practically when problems arise.

(5) Water Quality

Water quality in the study area extending about 30 km long from the left bank of the Chenab to Faisalabad was examined in detail by the previous studies. Its features are reported to be as follows:

a. Groundwater quality is good in the area along the Chenab where fresh deposits continues to the bedrock at a depth of about 200m, with its content of TDS ranging from 500 to 1,000 mg/lit.

b. Discharge from WASA's existing tubewells in the Chenab wellfield has maintained good quality with its TDS at a level of more or less 500 mg/lit since their commissioning in 1992. The Chenab wellfield is located where one of old channels of the Chenab runs with fresh deposits continuing towards the present course of the stream.

c. On the contrary, quality tends to get deteriorated in the direction towards the city, where older deposits with an increasing content of salt become predominant. Moreover shallow groundwater there has entirely been contaminated artificially.

d. The city keeps one exceptional wellfield along the Rakh Branch Canal where groundwater with good quality can be secured. Fresh water with its TDS ranging from 500 to 1,000 mg/lit occurs in lenses in a belt-like zone along both sides of the canal, thanks to seepage of the canal. WASA presently operates about 20 tubewells in this wellfield. There were 50 wells in the past and excessive pumping had resulted in degradation of water quality and lowering of water level. Based on the evaluation of seepage volume conducted for the formulation of World Bank’s 1992 Master Plan, the number and the distribution of wells were reorganized to the current state in 2001. WASA continues to conduct water quality tests for Rakh well field.

e. The isograms of TDS in the study area as the results of the previous study are shown in Fig.2-6 for reference.

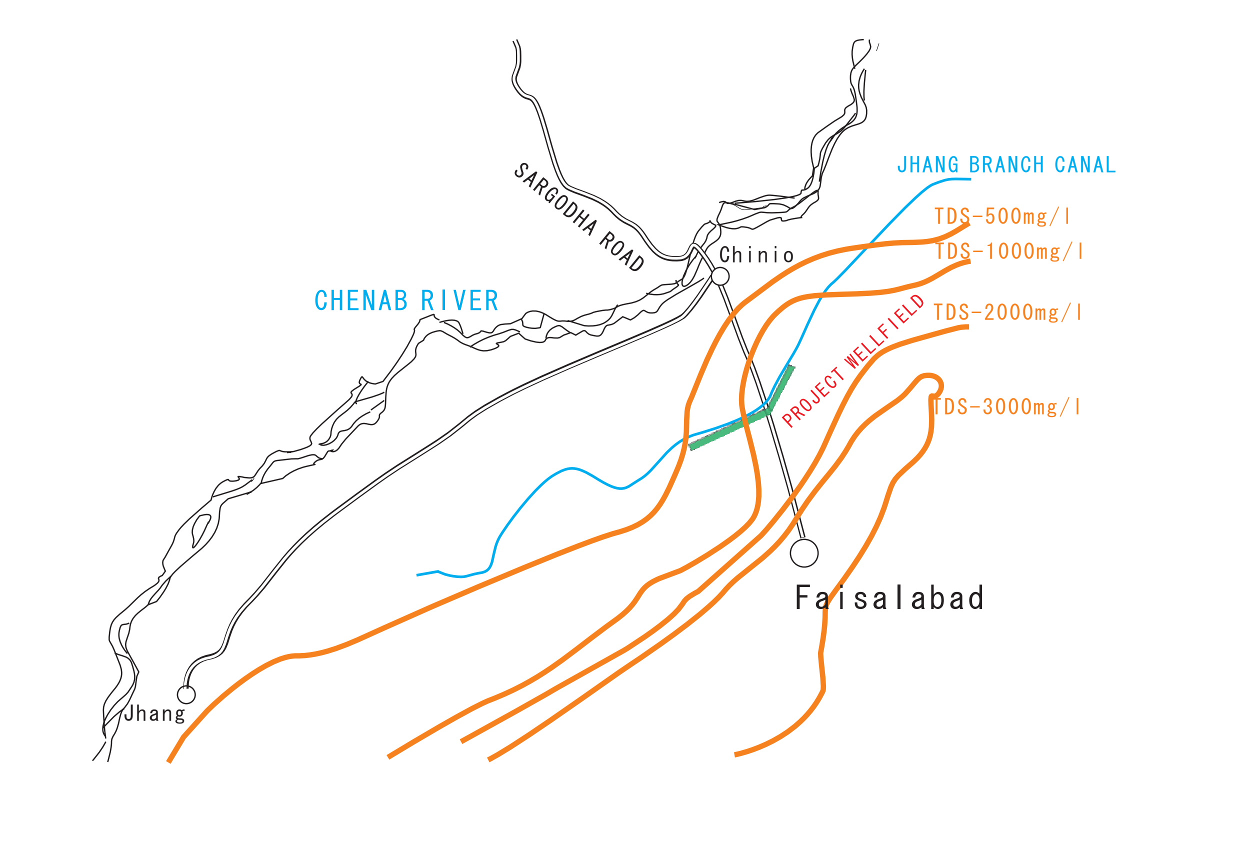
During the first and the second stages of this study, water quality of existing tubewells was extensively examined. A part of samples were analyzed by WASA Laboratory. Appendix shows the results of water analysis on the field as well as by the laboratory of tubewells for irrigation, together with data for WASA's Chenab tubewells.

To examine water quality at the tubewells for the project, samples were taken from the test well at the end of time drawdown test during the second stage of the study. The results analyzed by WASA laboratory is shown in Table 2-6, together with data of tubewells for irrigation existing within a distance of 100m from the wellfield. All the data indicates the occurrence of groundwater with good quality in the proposed wellfield.

Good quality can be guaranteed through ample seepage from the canal as is the case with the city's wellfield along the Rakh Branch Canal. The area around the Jhang Branch Canal is considered to have been a periphery zone of the old river course of the Chenab, where fresh deposits prevails, leading to good quality of groundwater occurring in them. This geological feature of this wellfield differs from that along the Rakh Branch Canal where old deposits with deteriorated quality of groundwater dominates. However, in order to maintain good quality, periodical monitoring will be required for quality as well as groundwater level.

WASA has been taking TDS as the standard indicator of water quality, with 500mg/lit as the target value. In the preceding project implemented in Chenab wellfield(approximately 5km to 10km north of this Project’s target wellfield), this value had been maintained. However, since the target area in this Project is near the area of poor water quality, WASA considers 1,000mg/lit (WHO recommendation) to be within an acceptable level.

Fig. 2-7 shows the isograms of TDS in the study area, based upon the results of water analysis on site and by WASA laboratory through the first and second stages of the study. Compared to the previous maps in 1970s and 1980s shown in Fig.2-6, the deterioration of groundwater quality is now in progress.



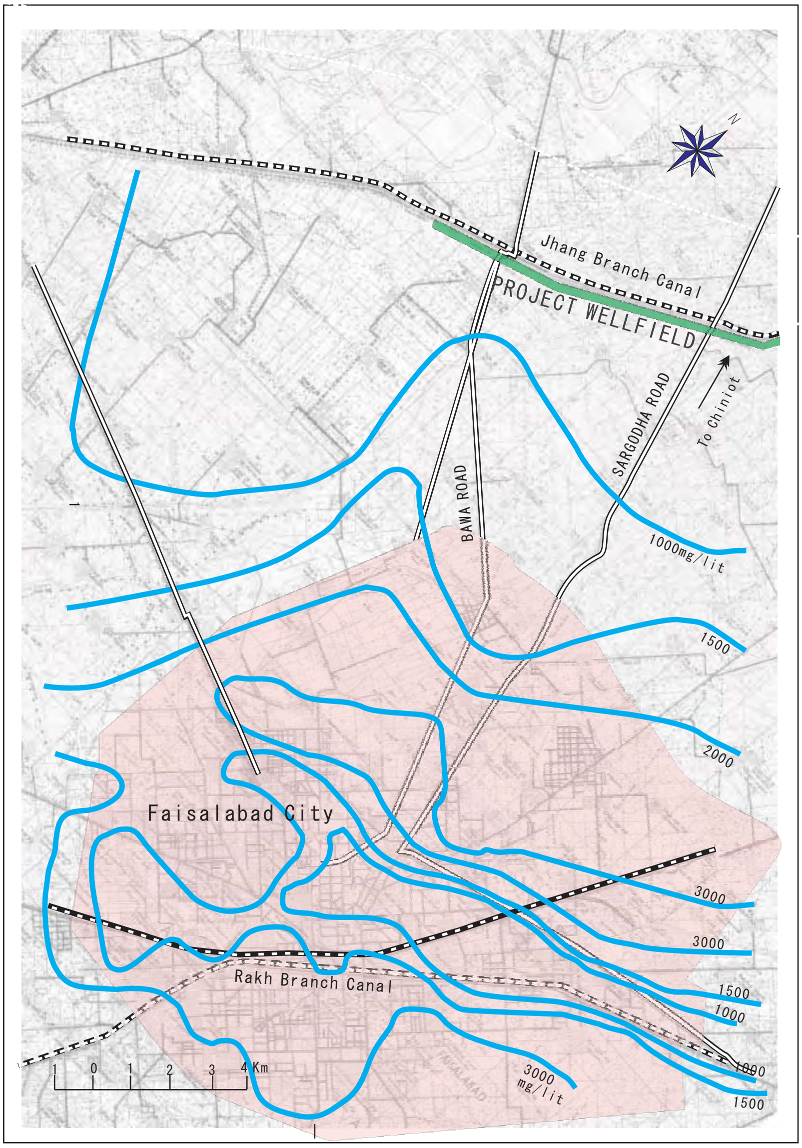




FIG. 2-6

TDS ISOGRAMS
BY THE PAST STUDIES

TOP:
TDS Isograms in 1993
by the World Bank' s
Master Plan

LEFT:
TDS Isograms in 1981
by the Republic
Engineering Corporation

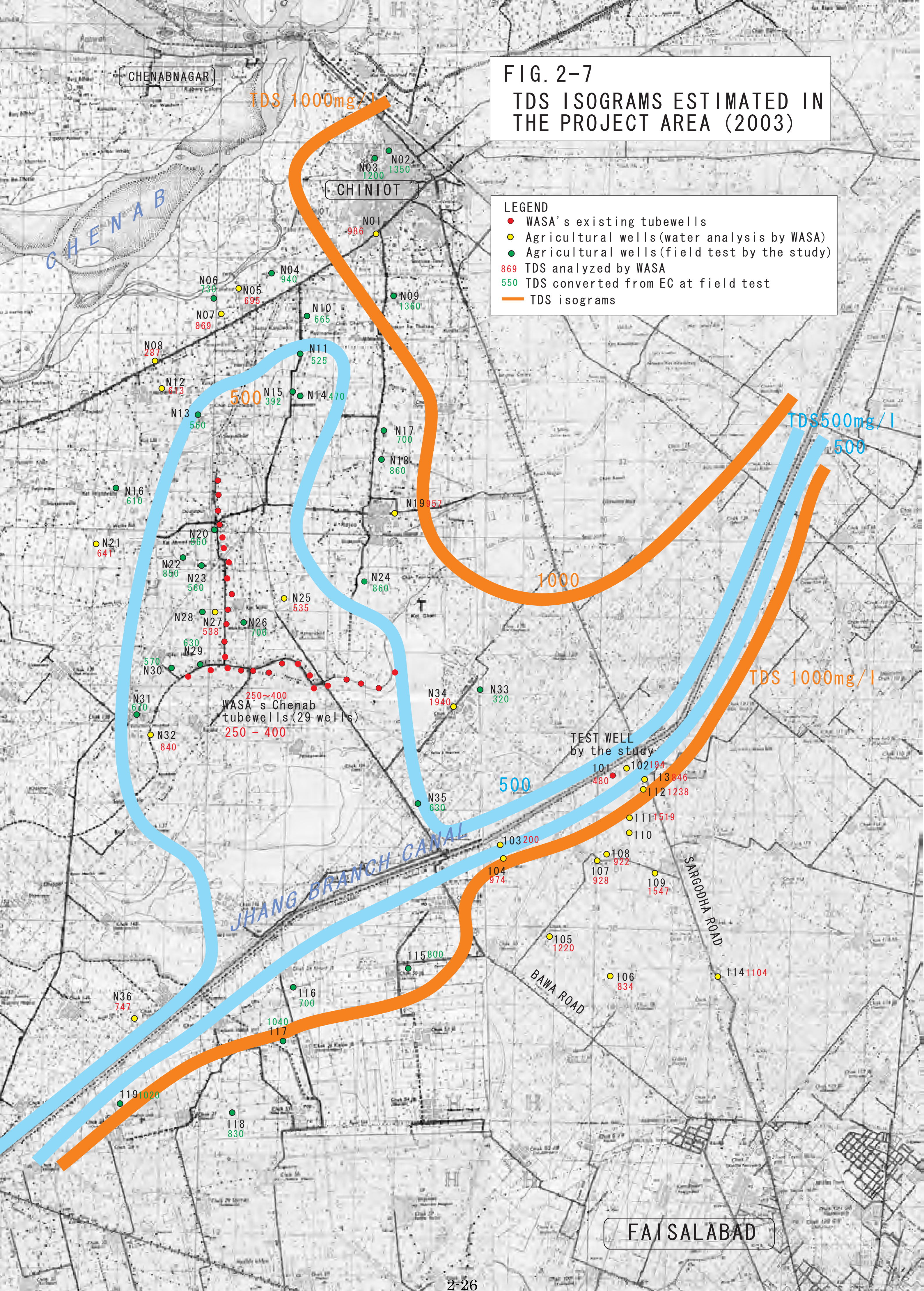



FIG. 2-7
TDS ISOGRAMS ESTIMATED IN THE PROJECT AREA (2003)

The World Bank's Master Plan anticipated good quality could be preserved in the reach of the Chenab, and assigned the area for another wellfield under the assumption that the degradation of water quality in the urban area will not spread further, with the Jhang Branch Canal acting as a buffer zone. However, the result of this current study suggests a progression of artificial pollution in the lower region of the Chenab River beyond Chiniot City (population 250,000), in the north-western part of the project wellfield. Degradation of water quality in villages near the canal is becoming apparent, probably due to raw sewage from the city being discharged into the streams along the Chenab River.

To the south of the proposed wellfield are many villages of large scale, each with a population close to 10,000. Daily life and active farming in these villages have resulted in artificial contamination of shallow groundwater in the surrounding areas through discharge of large quantities of waste water. Particularly all domestic sewage and waste water flow into lower land within villages, creating a large pond of waste storage. Such a situation seems to highly contribute to the progressive deterioration of groundwater quality, with its TDS rising to an extraordinary higher range than that in the surrounding area. (In Fig.2-7, tubewells No. 2, No.3, No. 34 and No. 112 are typical examples.)

Being an agricultural area, there was a concern of agrochemical contamination. However, from the studies conducted in 1998 and in the first Basic Design Study, the influence of agrochemicals was not detected. This may be due to the fact that the majority of the farmers are poor and the absolute quantity of fertilizers used tends to be extremely low. Recently there is a move towards establishing an agrochemical testing system in the City, in collaboration with WASA Water Quality Laboratory. A development of a comprehensive water quality monitoring system which integrates testing in agrochemical contamination will be recommended.

Although a progressive artificial pollution in the surrounding areas may be predicted, it can be concluded, based on the past experience with the tubewells around the Rakh Branch Canal, that the possibility of the Jhang Branch Canal Project wellfield being affected is minimal as long as the intake is balanced with the recharge from the Canal.

Meanwhile, the increasing contamination of groundwater from the shallower level, utilized by the villagers as domestic water, is a source of concern for the villagers and appropriate measures for environmental protection is urgently needed. Positive efforts are required from the Administration at this stage.

Table2-6 Water Analysis for the Test Well and Tubewells in its Vicinity

| | | Period of analysis | T ℃ | pH | EC μM/cm | Turbi-dity NTU | TDS mg/ℓ | Ca mg/ℓ | Mg mg/ℓ | Total hardness mg/ℓ | Cl mg/ℓ | Total iron mg/ℓ | Total Nitrogen mg/ℓ | $NO_3$ mg/ℓ | $NO_4$ mg/ℓ | P mg/ℓ | $SO_4$ mg/ℓ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Test well(101) | Sep. 2003 | 24.0 | 7.80 | 230 | 1.5 | 480 | 24 | 10 | 100 | 36.0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 2 | Exist. Tubewell (102) | Dec. 2002 | 22.3 | 7.91 | 260 | 0 | 194 | 32 | 41 | 244 | 29.7 | - | 0 | 0 | 0 | 0 | - |
| 3 | Exist. Tubewell (103) | Aug. 2003 | 23.2 | 8.30 | 247 | 0 | 200 | 54 | 26 | 240 | 50 | - | 0 | 0 | 0 | 0.04 | - |
| 4 | Canalwater (for reference) | Aug. 2003 at time of sampling at No.3 well | 27.9 | | 160 | | | | | | | | | | | | |

Remarks

1) For the locations of the wells with a number, refer to the map in Fig.2-7.

2) Temperature and EC were measured on site at time of sampling.

3) Samples from the test well were analyzed at an official laboratory in Lahore.

4) Samples from irrigation wells (102 &103) were analyzed by WASA laboratory.

Table 2-7 Water Analysis Results of WASA's Existing Tubewells in Chenab Wellfield

(Samples collected and analyzed in September 2009)

| Tubewell No | Color | Ordor | Taste | pH | EC | TDS | Ca | Mg | Total Hardness | Carbo-nates | Bicar-bonates | Cl |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $\mu$ s/cm | mg/ℓ | mg/ℓ | mg/ℓ | mg/ℓ | mg/ℓ | mg/ℓ | mg/ℓ |
| No. 3 | None | None | Good | 7.5 | 758 | 531 | 50 | 37 | 275 | Nil | 160 | 100 |
| No. 4 | None | None | Good | 7.5 | 625 | 435 | 48 | 31 | 250 | Nil | 250 | 92 |
| No. 5 | None | None | Good | 7.6 | 634 | 446 | 51 | 31 | 280 | Nil | 180 | 70 |
| No. 6 | None | None | Good | 7.5 | 493 | 346 | 52 | 38 | 230 | Nil | 170 | 78 |
| No. 7 | None | None | Good | 7.5 | 482 | 339 | 48 | 27 | 200 | Nil | 170 | 100 |
| No. 8 | None | None | Good | 7.5 | 467 | 329 | 40 | 25 | 250 | Nil | 180 | 40 |
| No. 18 | None | None | Good | 7.5 | 570 | 400 | 40 | 25 | 200 | Nil | 150 | 55 |
| No. 19 | None | None | Good | 7.5 | 638 | 447 | 48 | 52 | 250 | Nil | 200 | 42 |

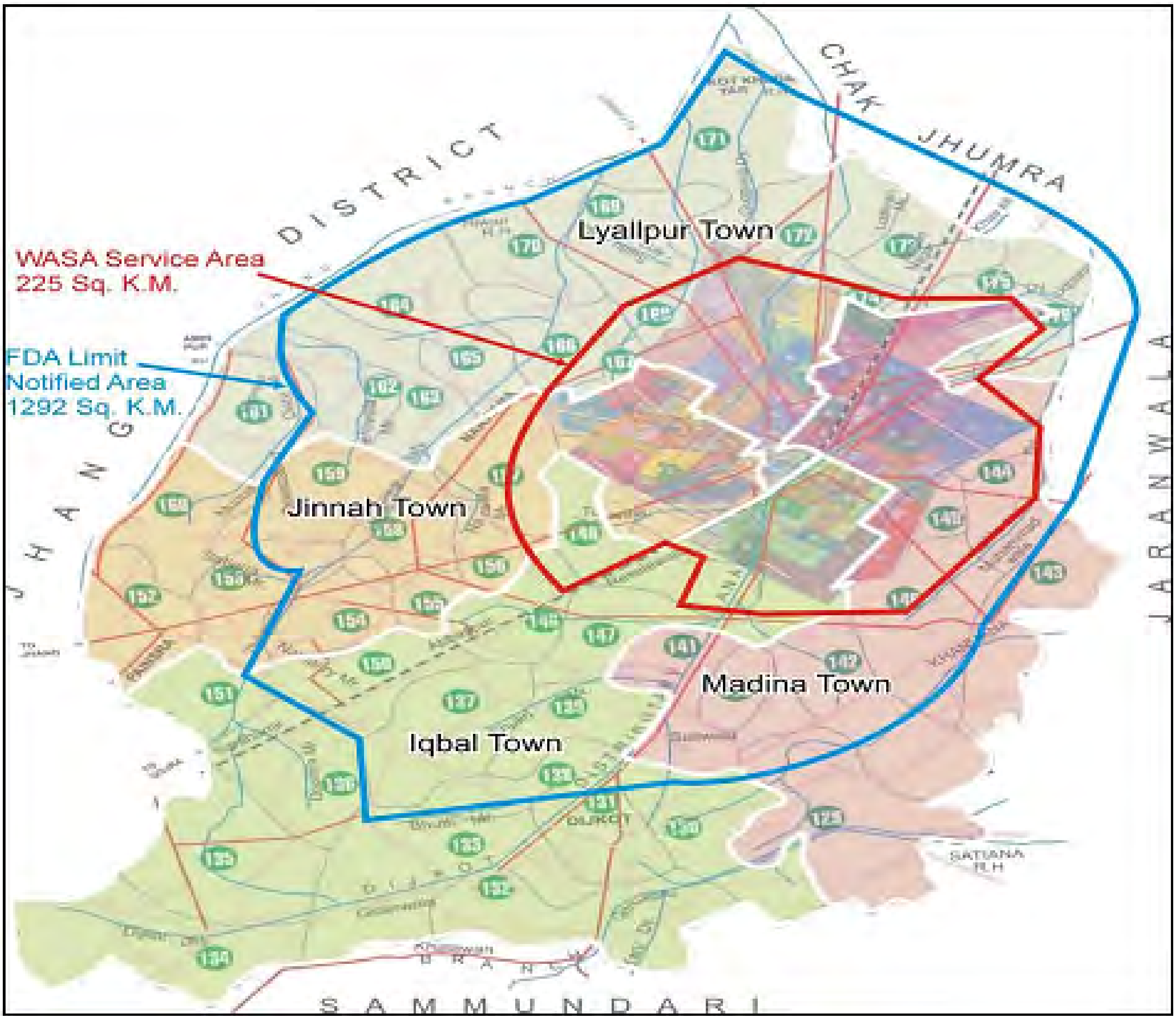
### 2-2-2-2 Water supply planning

1) Service area of the project

Faisalabad city in Punjab Province is the third largest city in the country with an estimated population of 2.8 million in 2010. In the past it served as a regional agricultural center of Punjab, but in recent years textile industry has flourished and it has now grown to be one of major industrial cities in Pakistan.

In June 2005 the District of Faisalabad with the city serving as its center was promoted to the City District with all its administrative area divided into 8 Towns, . As a result the former municipal zone of the capital now consists of 4 Towns.

Fig 2-8 Administrative Area of Faisalabad City (2005)



The ongoing service area of WASA is limited to the so-called peri-center area fter the announcement of the City District, however, it has been expanded to the Metropolitan Area (WASA Service Area in Fig. 2-8) Accordingly the area outside of the peri-center within the Metropolitan area will become the target of future development by WASA.

The peri-center area having WASA water service in place has long been suffering from the acute shortfall of water supply. The main sources of the existing water system for the city are the tubewells installed in the Chenab wellfield about 20km northwest of the city. The water from the wellfield is transmitted via a booster pumping station to the Terminal Reservoir (hereinafter called as T/R), situated in the northwestern suburbs of the city. The water is then supplied to the service area in the city, largely divided into two zones in the east and the west, through arterial mains extending toward the east to southeast side of the city. The network remains yet to cover the whole area, particularly at the end of existing trunk main in the east side, where the shortage of water supply has long been acute due to lower pressure. During the preceding process of implementation for Phase I of the project, the construction work for reinforcing the existing network was executed as the first phase for the purpose of improving water supply in this area. The effect of this work will be verified as water increases upon completion of the second phase and this project, Project II of Phase 2 for the construction of main water facilities in the future.

2) Population

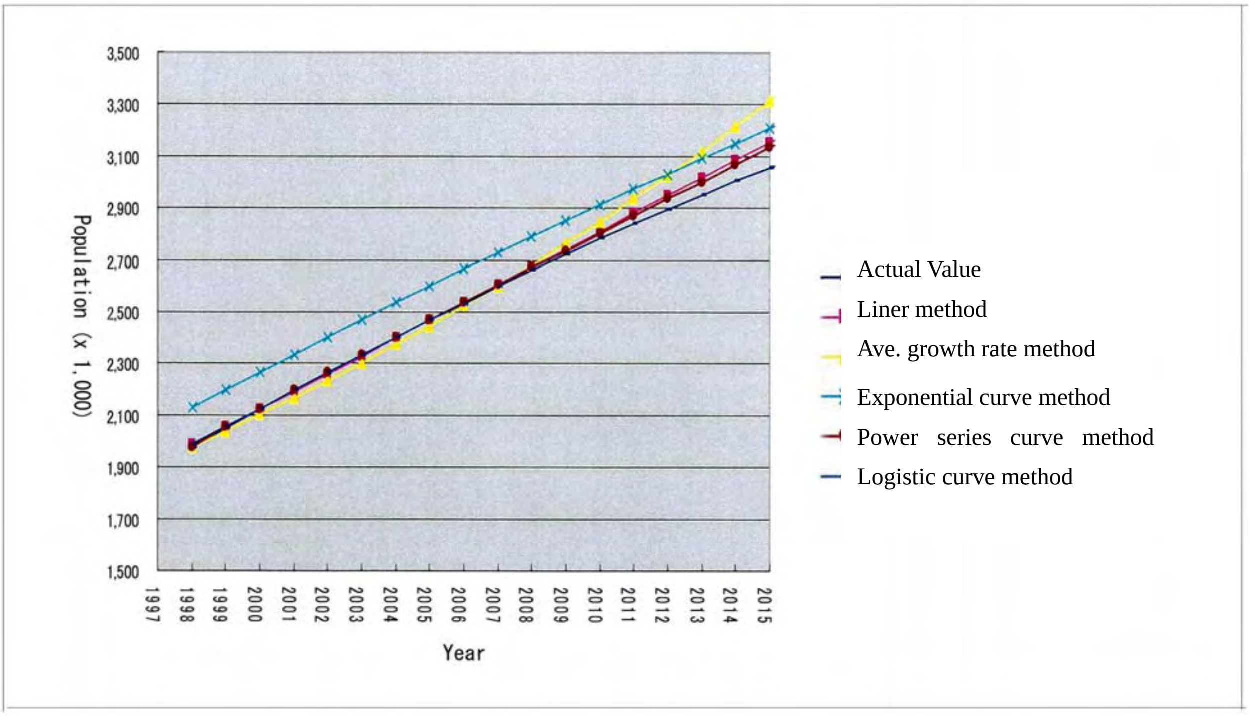
The population of the city is now estimated at 2.8 million based upon the national census in 1998. It was 2.2 million in 2001. The growth rate ranged from 2.6 to 3.6 %. Comparing it with the statistical data in 1981, the present population increased by about 2.5 times in 25 years.

Based on the past statistical data, the future population of Faisalabad was projected for the study using four estimation formulas. The data of four estimates are plotted in Fig.2-9 for graphical comparison. Among them, the values calculated with the "mean increase number method" (linear method) show an intermediate range of population increase, and is considered to provide a suitable basis for the study. Assuming this forecast, the population of the city is estimated to reach 2.95million in 2012, when this project is to be completed.

Table 2-8 Change of the Population of Faisalabad (x 1,000 people)

| year / item | Actual | | | | | | Estimated | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1981 (census) | 1991 (census) | 1995 | 1998 (census) | 2000 | 2001 | 2007 | 2010 | 2012 | (2,013) |
| Projected Population | 1,104 | 1,583 | 1,875 | 1,997 | 2,020 | 2,200 | 2,600 | 2,811 | 2,948 | 3.054 |
| Data | WB | WB | Statis-tics | Census | WASA | WASA | WASA | | Completion of the project | |

Fig. 2-9　Projection of the Population of the City (thousands)




3) Examination of basic elements for supply planning

(1) Water sources

According to the preceding basic design study for the project in 2003, WASA depended mainly upon tubewells in two major wellfields for water sources. In addition, WASA had two slow sand filtration plants within the service area, functioning as supplementary water service. This implementing review study in 2009 confirmed the composition of sources for WASA system had no substantial change.

The present state of each water source is as follows:

1)Tubewells

a. Chenab Wellfield

In the ADB project, completed in 1992, 25 tubewells were installed in the Chenab wellfield as the main source of water supply for the city. In 2000, WASA added four tubewells in its own effort to increase supply.

The design discharge rate from 25 tubewells in this wellfield was initially 225,000m$^3$/day. The basic design study in 2003 saw that the actual production rate those days had ranged from an average rage of 160,000m$^3$/day to a maximum of 180,000m$^3$/day, although the record marked a few cases of a maximum production of 200,000m$^3$/day in the first quarter of 2001, when the number of operating wells had increased to 29.

According to WASA's operation record, the maximum supply from the wellfield was recently 200,000 m$^3$/day, although the total capacity of 29 sources in the wellfield was previously raised to about 260,000 m$^3$/day. Full operation of the facilities there has now become difficult since a part of them need overhauling and rehabilitation from time to time after continued operation for nearly 18 years. WASA now plans to rehabilitate the whole of the tubewells and pumping equipment in the wellfield under a development programme with fund from the provincial government. This program will help it to maintain at least maintain the current water supply rate from this source in the foreseeable future.

b. Rakh Branch Canal wellfield

Until the completion of the Chenab wellfield, the entire water supply for the city used to rely on the tubewells installed alongside the Rakh Branch Canal running across the east side of the city. In the early 1990s the number of wells alongside totaled more than 50. Tubewells used to be concentrated in this zone since the quality of groundwater along the narrow strips on the both sides of the canal was acceptable, contrary to those in other areas of the city where TDS was well over 2,000mg/lit, unacceptable for drinking mainly due to salinity. The wellfield located in the downtown area had thus been overly developed, and resulted in shutdown of many of the tubewells due to lowering of groundwater level and degradation of water quality.

The World Bank’s master plan suggested to limit the total discharge in this strip to a range of the seepage amount from the canal to preserve proper quality and quantity. In compliance with this recommendation, WASA rehabilitated the existing tubewells by 2002. As a result, old and new tubewells of 16 in total are presently in operation along the canal side. The design discharge of this source is 1 cusec (=102m$^3$/hour) for each tubewell and the total supply rate from 16 tubewells in this wellfield is currently 30,000 m3/day on the average.

This implementing review study confirmed that WASA completed the development of new sources along the Rakh Branch Canal with installation of 12 tubewells in 2008. The pumping equipment is yet to be installed. However, this new project with budget from the provincial government can soon add 23,000 m$^3$/day to the current water supply rate.

In addition, further development of the reach of this canal is now under planning with French assistance. The project aims to add a rate of 23,000 m3/day with new tubewells of 10 in number. Since PC-I of this project was recently approved by ECNEC, the implementation is expected to start soon, targeting to complete it within 3 years, including another component as will be explained in the following section.

2) Surface water treatment facilities

a. Jhal Khanuana water treatment plant

The Jhal Khanuana water treatment plant, situated in the central part downtown, on the left bank of the Rakh Branch Canal, used to withdraw the canal water, and to feed it to the arterial mains after processing it through slow sand filtration. Although the design capacity was 17,000m$^3$/day, it continuously decreased to about one-third 5,000m$^3$/day due to deterioration of facilities during the basic design study in 2003. The previous implementing review study in 2007 confirmed the final shutdown of the filtration plant due to deterioration. However, WASA now has a plan to rehabilitate it

As the latest move relating to this source, the French project proposed to install a rapid sand filter with a capacity of 45,000 m$^3$/day in the premises of Jhal Khanuwana plant. The completion of the project is targeted in 2013.

b. Millat Town water treatment plant

The treatment method is slow filtration just like Jhal Khanuwana plant, and withdraws surface water from one of the distributaries of Jhan Branch Canal located away from the city. It produces about 5,000m$^3$/day. This treatment plant independently supplies water to the new residential area of Millat Town.

The total production from the existing water sources was about 230,000m$^3$/day by the previous basic design study in 2003. Summing up their examination under this study, the anticipated supply rate in the future is estimated in Table 2-9:

Table 2-9 Maximum Production Capacity of Water Sources

| Water source | Existing water sources | | | | | WASA | JICA | France |
|---|---|---|---|---|---|---|---|---|
| Year | 1993 (WB:MP) | 1998 (JICA:B/D) | 2001-2 (JICA:B/D ) | 2007 JICA-IRS | 2009 This study | 2011 | 2012 | 2013 |
| Chenab wellfield | 225,071 | 204,750 | 200,000 | 200,000 | 200,000 | 200,000 | 200,000 | 200,000 |
| Rakh Branch Canal wellfield | 83,178 | 20,200 | 20,000 | 20000 | 30,000 | 58,000 (WASA) | 58,000 | 76,000 (France) |
| Jhang Branch Canal wellfield | | | | | | | 91,000 (JICA) | 91,000 |
| Jhal Kanuana water treatment plant | 17,125 | 6,825 | 5,000 | | Aban-doned | | | 50,000 (France) |
| Milat town treatment plant | | | | | 4,550 | 4,550 | 4,550 | 4,550 |
| Total Production | 325,374 | 231,775 | 232,000 | | 234,550 | 262,550 | 353,550 | 398,550 |

(2) Water service rate and served population

WASA charges for its service on a fixed tariff system based on the sizes of the properties in a similar manner to those in other major cities of the country. Most of all the households in the city are without water meters. Accurate water consumption, therefore, remains unknown. Furthermore accurate data for served population is not available. In 1992 the World Bank conducted a household survey of 5,400 for its task of revising the master plan for the city's water service, and estimated the water coverage at 60%. On the other hand, the consumer survey conducted about the same time by FDA for 16,000 households reported it was about 50%.

To examine the coverage rate during the basic design study in 2003, WASA recommended to estimate it by calculating the extent of its service area where secondary distribution lines and house connections had already been installed. The calculated service area based upon the data provided by WASA turned out to account for about 50% of the city's entire administrative area. Since pipelines were installed mainly in the populous area, the ratio of the citizens living in the service area to the entire population of the city was likely to be more than the ratio of the two areas thus compared. This study assumed 10% for a rate of additional population, taking into account various factors affecting the estimate such as the ongoing daily water supply rate, average per capita daily consumption, etc. Consequently the estimated coverage of WASA's water service was proposed to be at a rate of 55%.

The same service rate employed by the basic design study is proposed for this study in 2007, since there was no substantial increase in the number of new contracts for supply during the period between the two studies, mainly due to shortages of water. Since the estimated population of Faisalabad is now 2.6million (2007), the current served population is 1.43 million, taking a rate thus estimated. In future the prospect of enhanced water supply through this project is likely to invite more new customers than in the past at a rate of approximately 1 % annually. According to this assumption, at the time of completion of this project in 2010, the water coverage rate could rise to 60%, with the served population increasing to 1.69 million among the total projected population of 2.81 million.

According to the results of 1998 national census, the number of households in the city was 279 thousand with the average number of family members at 7.1. On the other hand the number of the households connected to the water service in 2002 was 98 thousand, indicating the water service coverage by connections accounted for only 35% of the total households. The reasons for the difference are not clear, but one of guesses is that more than one household are illegally connected to single service lines. Connections of this type are reportedly widespread, since unregistered households need not pay the fixed rate of water tariff to be charged upon their property sizes. Although the number of house connections can provide useful information for estimating the water coverage rate, the current figure of registered connections cannot tell the real situation in Faisalabad. It is strongly recommended to WASA, therefore, to survey the conditions of current service connections for collecting the exact data on the actual situation of its water service. When WASA moves to shift its fixed water tariff to the metered one, such data is indispensable for planning an effective method for execution.

The number of water supply connections is shown as a reference in the table 2-9:

Table 2-10 Number of Connections (December 2009)

<table>
<tr><td colspan="2">Type</td><td colspan="4">Number of connections</td></tr>
<tr><td colspan="2">Year</td><td colspan="2">2003</td><td>2007</td><td>2009</td></tr>
<tr><td colspan="2">Households</td><td colspan="2">98,000</td><td>99,300</td><td>104,891</td></tr>
<tr><td rowspan="2">Industrial</td><td>Industrial</td><td rowspan="2">1,790</td><td>90</td><td rowspan="2">1,810</td><td rowspan="2">2,431</td></tr>
<tr><td>Commercial</td><td>1,700</td></tr>
<tr><td colspan="2">Total</td><td colspan="2">99,790</td><td>101,110</td><td>107,322</td></tr>
</table>

WASA classifies the categories of consumer’s usage into (a) Household use, (b) Industrial use and (c) Commercial use. The industrial use is the one by large consumers such as factories, public institutions, banks, hospitals and schools. The number of consumers in this category was 78 in December 2002. The commercial use is for consumers with 1/4" diameter pipes (same as households), and the number of connections was about 850 during the same period. WASA estimates that the consumption of these two categories accounts for about 15 % of the total. The confirmation was made in 2003 during the basic design study to examine the actual situation, based upon WASA’s computerized billing log as follows:

a. Industrial use

According to the billing record of December 2002, the monthly consumption of this type was estimated at 250,000$m^3$/month. The number of connections was 78, all of them large consumers such as top textile plants, public institutions, banks, hospitals and hotels. The 21 users among them were paying the bills for metered volumes, which accounted for a greater part of charges in this category. The remaining users were charged on the assumed volumes of consumption determined by WASA in its tariff system, depending upon the diameters of their connections. They were paying for the consumed amounts of water, but the rates of consumption are fixed.

b. Commercial use

All the users in this category are served with unmetered connections of the same diameter, 1/4”as domestic users, but their tariff rate is about 4 times higher than that for the latter, since they are assumed to consume more. There are 850 users in this category, but without meters, the amount of consumption remained only a guess. Assuming the daily consumption of one connection to be 2 $m^3$, the monthly consumption from 850 connections was 51,000 $m^3$ in total.

According to this study, the total consumption in the two categories amounted to 310,000 $m^3$ per month, indicating that their average daily consumptions are around 10,000 $m^3$. The estimated consumption rate corresponded to around 6% of the total water supply rate, but this share needed some adjustment because the data based for this estimation was of December 2002, namely the increase on demand in summer time and data correction. Thus it was assumed that about 10% of the total effective water supply was consumed for the industrial and commercial usage.

As WASA gives priority to the general public in its inadequate supply condition, most of the industrial users installed their own tubewells on their premises to secure necessary water. (The World Bank estimated in the Master Plan report, 1992, that 95% of industrial users had their own sources). However, the withdrawal of groundwater even from their own sources is not free. It is charged by WASA as "Aquifer Charges” set in its tariff system, and it has been one of important sources of income for WASA (although increasingly uncollected). Although WASA has been anxious to respond to the increasing demand in the industry as it could contribute to WASA’s revenue with a tariff rate 3 times higher than that for domestic use, it has been highly difficult to increase the share of this sector under the current level of water supply. As for the commercial use, since a great majority of users in this category are engaged in family business of small scale, the average consumption rate is supposed to be in a similar level as that of domestic usage, both categories using the same connection size of 1/4”. Accordingly the consumers in the commercial use may be treated together with ordinary domestic users in respect of demand.

As far as the present water supply continues, the increase of supply for the industrial sector will be frozen. Even after the supply rate increases in 2010, much cannot be expected for the sector if the public demand is prioritized. Under the circumstances, dependency on private tubewells will have to be maintained.

(3) Examination of the supply rate

a. Ongoing daily supply rate

The present study examined the latest records of WASA operation for supply. The supply rate from the Chenab source accounts for about 90% of total supply rate for the city. According to the latest records the maximum and the average daily supplies from this source were respectively 200,000 and 174,800$m^3$/day.

Accordingly, the maximum daily supply for the city is currently estimated as follows:

| | | |
|---|---|---|
| * Chenab source | | 200,000$m^3$/day |
| * Rakh Branch Canal source | | 20,000$m^3$/day |
| * Millat Town treatment plant | | 5,000 $m^3$/day |
| | Total | 225,000$m^3$/day |

This average daily supply is more than that identified at the time of Basic Design (160,000$m^3$/day, June 2002), indicating the efforts made by WASA to respond to the ever growing demand. However, there is a possibility that such increase is causing the continuous lowering of water level due to excessive withdrawal. Therefore, it will be appropriate to set the maximum daily supply at 180,000m$^3$/day, as it had been before.

b. Unaccounted-for water

The past studies such as the World Bank's Master Plan estimated that there was about 30% loss of water supply in the system due to leakage, and based upon this estimate, WASA currently assumes the effective rate of water supply at 75%. The inspection of the existing pipelines within the city during the previous studies under the project tells that leakage rarely occurs in the arterial mains thanks to their relatively young age and good quality in material as well as workmanship and to utilize the ductile cast iron pipe manufactured in Japan. However, the distribution pipes made from asbestos from the secondary mains to service lines, which are a combination of recent and older installations, seemed to be leaky. Interviews to citizens in the east side of the city during the social survey for the previous basic design study revealed that they had experiences with drinking water contaminated with sewage. In May 2005 a case of a large scale contamination incident took casualties of 11 citizens, mostly infants. In an aftermath of this incident, the provincial government intervened and directed WASA to replace old connections, not only the affected area but also other parts of the city. The replacement work has since been in progress, and this situation is likely to have good effects in improving unaccounted-for water ratio due to leaks. This project expects the work now in progress could contribute to the improvement of leaks by 5% in 2010 when the operation of the new facilities under the project will be commenced.

c. Daily maximum supply rate

The water supply to the city has been restricted to 3 cycles of pump operation a day totaling 6 hours at the maximum so that the actual demand of consumers remains unknown. This situation makes it difficult to determine various factors for planning including a ratio for determining the daily maximum supply rate. As a measure to address the task, this study takes the present consumption pattern for reference, since the target of the project is now to urgently alleviate the current shortage of water in the city rather than undertake a plan targeted by the World Bank's master plan. Assuming this policy, the load factor, which is the ratio of the daily maximum supply to the daily average supply, is obtained, based upon the recent performance in supply from T/R, based upon the record.

The calculation is presented as follows:

(Daily maximum supply rate =200,000m$^3$/day)

/ (Daily average supply rate=174,800 m$^3$/day) = 1.144

d. Peak hour factor

The peak hour factor determines the design hourly maximum supply rate. It is again difficult to obtain the reliable range of this value as is the case with the daily maximum supply rate. Although the World Bank employed a ratio of 1.9 to the daily maximum supply rate, based upon the demand projection, this project prefers a ratio of 1.6, which is the standard level for cities of the same size as Faisalabad.

e. Actual per capita daily average supply rate

The actual average daily supply rate per capita including industrial and commercial uses is calculated as follows, using the current actual supply, served population and the effective ratio described in the foregoing sections.

(Daily maximum water supply x effective ratio) / (current population x water coverage ratio)

= Per capita daily maximum water supply rate

(234,550$m^3$/day x 0.75) / (2.742 million x 0.58)) = <u>117 lit/capita/day</u>

(Note: The estimated water coverage ratio is 58% in 2009, due to increase of supply contracts)

Subtracting the estimated 10% of the industrial and commercial use and dividing by maximum load factor (=1.14) results in a per capita daily average supply rate for household, as follows:

［(234,550 $m^3$/day x 0.75) x0.9)］ / (1.15) / (2.742 million x 0.58)) = <u>87 lit/capita/day</u>

The calculation indicates the current per capita average supply is in the level of less than 100 liters per capita per day. The estimated level is quite low among the standards of other cities in Pakistan, which is 30gal (=135 liters) and more. This situation shows the acute shortage of water in the city.

f. Design per capita average water supply rate

In the early stage of water supply planning for the city, the ADB’s master plan in 1970s determined on 135 lit/capita/day as the average water supply rate in 2000 and the World Bank’s master plan, based on a consumption pattern survey conducted as part of the household survey, set the rate by income levels and proposed 135, 180, 320 lit/capita/day for low, middle and high income families respectively. The average rate across different income levels is 170 lit/capita/day. For the third largest city in the country, the supply standard given by the World Bank may be preferable. However, the real situation in supply is that only demand has been soaring with constantly increasing population as the future plan to augment water for the year 2000, proposed by the World Bank’s master plan, was not realized. As a result, the shortage of water is now so acute that even the amount of supply augmented through the implementation of this project cannot meet the rates formerly proposed by the World Bank.

Table 2-11 shows the result of the projection of the water supply after the completion of this project, based upon the factors previously examined. According to this forecast, the per capita daily average supply rate will be 130 lit. in 2012, provided the industrial and commercial demand is kept at the present level, and presuming the water coverage rate will increase from the present 58% to 64%.

Table 2-11 Water Supply Projection

| Year | 1.Estimated population | Daily maximum supply rate ($m^3$/day) | | | | Daily ave. supply($m^3$/d) | 7. Coverage rate | 8.served population | Per capita daily supply (lcpd) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 2.intake rate | 3.effective water | 4.industrial /commercial | 5.household | 6.household | | | 9.Ave. | 10. Max. |
| 2007 | 2,600,000 | 224,550 | 168,340 | 16,830 | 151,510 | 131,750 | 0.550 | 1,430,000 | 92 | 106 |
| 2008 | 2,674,000 | 224,550 | 170,660 | 16,830 | 153,830 | 131,750 | 0.560 | 1,497,440 | 87 | 103 |
| 2009 | 2,742,000 | 234,550 | 175,150 | 16,930 | 159,080 | 138,330 | 0.580 | 1,590,000 | 87 | 100 |
| 2010 | 2,811,000 | 234,550 | 187,640 | 16,830 | 167,810 | 145,920 | 0.600 | 1,687,000 | 86 | 90 |
| 2011 | 2,880,000 | 262,550 | 215,290 | 16,830 | 198,460 | 172,570 | 0.620 | 1,785,600 | 97 | 111 |
| 2012 | 2,948,000 | 353,550 | 296,982 | 16,830 | 280,180 | 245,060 | 0.640 | 1,886,720 | 130 | 150 |
| | | | | | | | | | | |
| 2013 | 3,054,000 | 398,550 | 342,753 | 16,830 | 325,923 | 283,410 | 0.714 | 2,180,000 | 130 | 150 |
| | | Supply increased in 2010 by WASA increase 2012 by this project<br><br>2013 by French project | The present leakage at a rate of 25% is improved to 20% by 2010, and an improvement of 2% per year after 2010. | (4) =10% of (3) Assumed to keep the same level after 2007 | (5) =(3)-(4) | (6)=(5)/1.15 (load factor for max. supply). | | (8) =(1)x(7) | (6)/(8) | (5)/(8) |

### 2-2-2-3 Facility planning

1) Components of Planned Facilities

Major facilities planned in this project (Project II of Phase II or Project for Expansion of Water Supply System in Faisalabad) under the responsibility of the Japanese side are composed of the following.

a. Tubewells
b. Intake pumps
c. Tubewell pump stations
d.Plumbing and electrical works in the pump stations

2) List of Facilities

Table 2-12 List of Intake Facilities

| Classification | Facility | Quantity | Specifications | | |
|---|---|---|---|---|---|
| a. Water Source Facility | Tubewell | 25 | Design discharge (a) | Total 91,000m$^3$/day<br>Discharge rate of one tubewelll<br>= 200 m$^3$/hr (2cusec) | |
| | | | Basic Depth (b) | 160m | |
| | | | Diameter (c) | Pump housing section | 16"、0～45m |
| | | | | Water intake section | 10", 45～160m |
| | | | Screen (d) | Structure | Wire wound type |
| | | | | Material | Stainless Steel |
| | | | | Basic length | 30m |
| b. Intake Facility | Intake Pump | 25 | Type | Vertical-motor driven, vertical shaft turbine pump | |
| | | | Pumping Rate | 200m$^3$/hr | |
| | | | Total Head | 70m～40m (of which, 20m is under ground) | |
| | | | Riser Pipe | 200mm (@3m) | |
| | | | Motor | Vertical shaft, totally enclosed fan cooled type<br>30～80HP, 1,450 rpm, 50Hz, 400V | |
| | Tubewell station | 25 | Structure | Reinforced concrete structure with water-proof mortar finish for outside wall and roof) | |
| | | | Dimensions | 7.000 x 6,500 (45.4m$^2$) | |

Remarks: The tubewell drilled for testing during the basic design study will be used as one of permanent production wells. Accordingly 24 new tubewells will be drilled through the implementation of the project.)

3) Specifications of tubewells

a. Design discharge

- With the total discharge from the planned wellfield targeted at 91,000$m^3$/day, the unit pumping rate of one tubewell is determined at 200$m^3$/hr, based upon the analysis in Section 2-2-1 Water Sources Plan.
- The duration of daily pumping is proposed to be 20 hours for the purpose of ensuring the water level recovery while the pump is idle.
- The number of tubewells necessary for ensuring the targeted discharge is therefore 23 ((91,000$m^3$/day) / (200$m^3$/hr x 20 hours) =22.75).
- Two (2) standby tubewells corresponding to 10% of the calculated requirement for the number of tubewells should be added in case of shutdown or repairing of working ones. As a result, the planned tubewells totals 25.

b. Basic drilling depth

According to the results of the geophysical survey for this study, the second and third aquifers occurring from about 40m to 170m in depth below ground surface across the planned wellfield will be targeted as the zones for the intake of groundwater. The aquifer conditions and depths can vary, and during the detailed design study it is proposed to carry out the geophysical survey once more to make final confirmation of the drilling depths, after the extent of the wellfield is determined and exact drilling points of 25 tubewells are pinpointed. Since the average depth to the lower horizon of the third aquifer is assumed to be 160m, this will be set as the basic drilling depth.

c. Casing

- The structure of the tubewell is basically divided into the upper portion of a large diameter to accommodate a pump and the lower one with a smaller diameter for the intake of water where screen is installed.
- The pump capable of delivering the discharge rate of 200 $m^3$/hr requires more or less 16" diameter for housing, and this size is set as a standard casing diameter for the upper portion of the tubewell. Since the upper horizon of the aquifer is estimated to occur 40 meters or deeper below the surface, the 16" diameter upper casing shall end at 45 meters at the deepest. A water level measuring tube (25mm steel pipe) will be installed down to the pump section.
- The lower water intake portion where screen is installed can have a reduced diameter to cut off the cost of the structure, while the design should be based upon such factors as the discharge rate, aquifer thickness, and uphole velocity (1.5 m/sec). These factors dictate the size should be 8" or larger. Taking an additional factor into account for minimizing the drawdown, one size larger casing and screen, 10", is proposed for the lower portion of the tubewell. The upper casing will be joined with the lower one with a reducer as local practice employed for the existing tubewells in Chenab wellfield.

d. Screen

A stainless steel wire-wound type screen with a maximum water intake surface area will be adopted. A larger water intake area means a smaller inflow velocity, effectively suppressing drawdown and preventing sand incoming. The length should be determined basically according to the thickness of the aquifer, but the economic length can be calculated by selecting the appropriate length so that the velocity becomes less than 15mm/sec using the following equation.

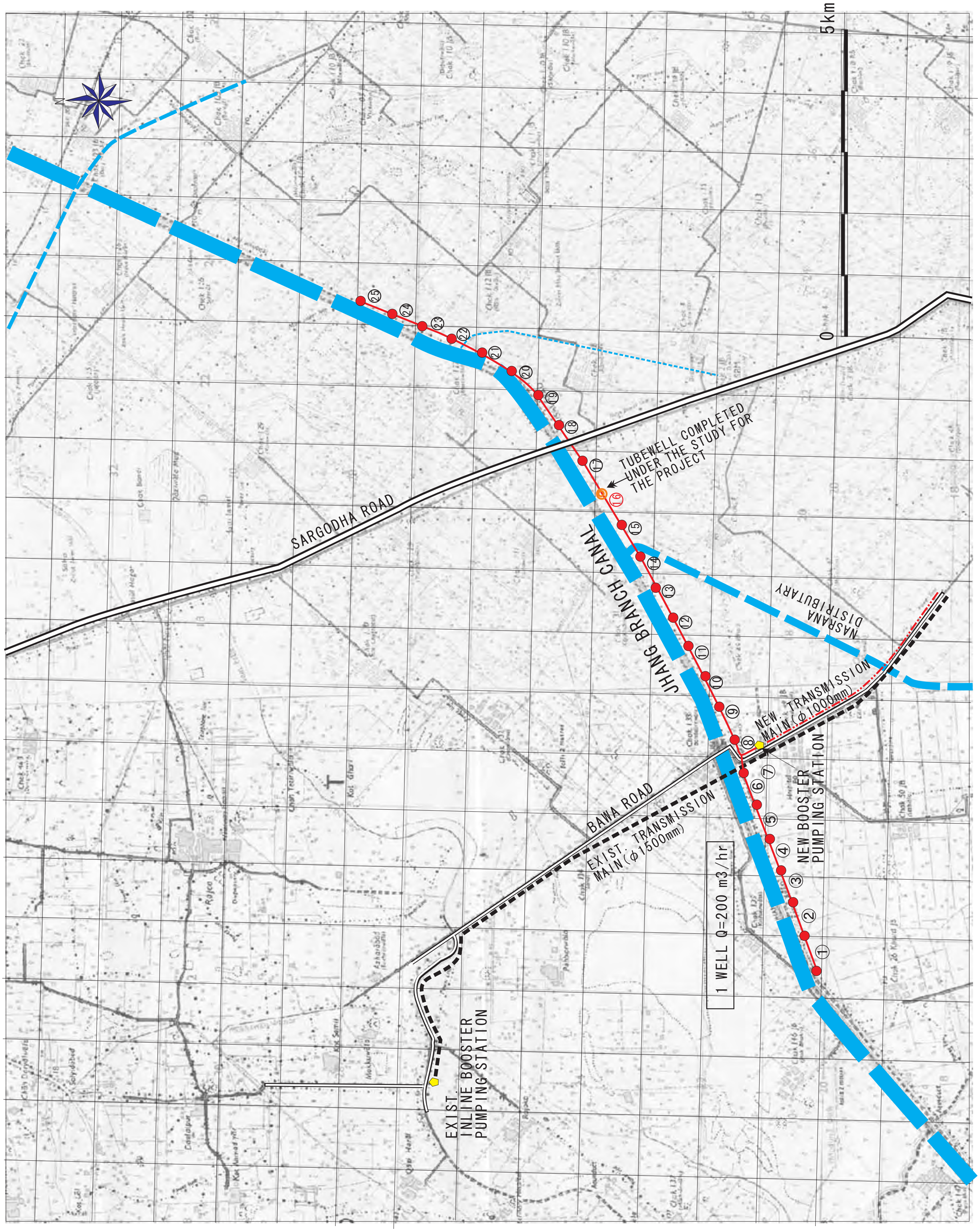
A (water intake area) x V (incoming velocity) = Q (design pumping rate) (V<15mm/sec)

The standard screen length designed for the existing tubewells constructed by the previous ADB project was 30m. This length gives a velocity about 8mm/sec, which is sufficiently below 15mm/sec, and this design will be employed for this project as well.

e. Layout of the wellfield

- The wellfield for this project is located in public land of narrow strip about 20m wide along the left bank of the Jhang Branch Canal under management of the Electricity and Irrigation Department of the Punjab government. The basic design for the wellfield is to align 25 tubewells in parallel with the channel of the canal. In the existing wellfield along the Rakh Branch Canal within Faisalabad, WASA adopted a similar design for the alignment of tubewells, resulting in stable discharges with a good quality, thanks to the constant seepage from the canal.
- For this project, the tubewells are designed to be installed at intervals of 600m, in accordance with the concept of minimizing the lowering of water level as a result of the pumping tests.
- One test well was drilled at the center of the planned wellfield during the field survey. Since its discharge and quality satisfy the design criteria, it will be used as one of the production wells for this project. Using the location of this well as the reference point, the other wells will be located at 600m intervals to the up- and downstream of this well (See Fig.2-10).

FIG. 2-10
LAYOUT OF TUBEWELLS ALONG JHANG BRANCH CANAL



4) Tubewell pumps

a. Type

As intake pumps for the project, types of those submerged in tubewells are required for the longstanding stable operation of the intake system. There are two types available: one is vertical turbine pumps; and the other, submersible motor pumps. The former is of Pakistan make and was employed for the previous project. In view of advantages and merits in this equipment described hereunder, it is recommended to adopt it for this project as well:

- A submersible motor pump is suitable for systems requiring high heads for lifting water from the tubewell having low water level. The pump runs at as high a speed as 3,000 rpm. On the contrary, tubewells in the project area are retaining high water levels close to the ground surface thanks to the constant recharge from the canal, and the range of water level lowering in the future will be limited. This situation allows to employ the vertical turbine pump operating at 1,500 rpm, 1/2 speed of the corresponding submersible pump. It goes without saying that this much lower rate of revolution is more advantageous in terms of operation and maintenance of equipment.
- Since the submersible motor pump is a combined unit consisting of a submersible pump and motor to be installed inside the tubewell, the whole unit must be raised out of it for inspection and maintenance whenever repair is needed. Repairs of motors are more frequent than pumps themselves. On the other hand, for the vertical turbine pump, only the pump is installed inside the tubewell and the drive unit is installed above ground. For both types, the units are connected by a transmission drive axis. As the drive unit, other than a motor, an engine is also used particularly in areas without electric power, although motors are more economical. The existing pumps are driven with vertical electric motors. If a motor needs troubleshooting, it can be inspected and repaired above ground without pulling the pump up, giving advantage for operation and maintenance. Also, the motor is not a special submersible type, but a normal above ground type, and this is easier to repair than the submersible type.
- WASA is familiar with operation and maintenance of this type since vertical -motor- driven, vertical turbine pumps have been working continuously for 10 years in the previous ADB project. During this period, electrical problems with motors and power distribution panels were encountered, but problems with mechanical parts of the pump itself were not experienced.

b. Pump capacity

- The unit discharge rate is set at 200$m^3$/hr.
- The total head is determined as the total of the pumping water level (including projected water level lowering in the future) of the well, the losses in the effluent pipeline and valves in the tubewell station, friction losses in the collector mains, and height difference between

the ground levels of the tubewell and the booster pumping station.

- Pumping water level in a tubewell

  The pumping water level is calculated from analysis of the pumping test results of the test well with certain assumptions.

  = (Average static water level of the wellfield=6-9m) + (Drawdown when pumping at 200$m^3$/hr =2m) + (further water level lowering during canal distribution shutoff periods and measure for influence on water level lowering from the existing Chenab wellfield =5m) = about 14m

  Moreover, as allowance for characteristics of the unconfined aquifer which is directly influenced by rainfall, a maximum pumping water level of 20m is assumed (the existing tubewells of Chenab area initially had water levels similar to that of the test well of the present study, but after 10 years, their pumping water level lowered to 25m at the maximum)

- Other losses

  The 25 wells of the wellfield will be aligned alongside the approximately 14km belt of the Jhang Branch Canal and water pumped from them will pass through the collector mains to the pump well in the Jhang Brunch Canal booster pumping station. The loss in water flow varies at the respective locations of the tubewells with about 40m at the farthest upstream of the wellfield down to less than 10m at the well field near the pumping station. The ground level at the booster pumping station as the delivery point is about 2 m lower than the upstream of the canal, but most of the course of collector main along the canal is nearly flat.

The calculation of required heads of the pumps for 25 tubewells involves all the above components. The maximum head is 60 m for the pumps in the upstream tubewells far from the booster pumping station, and the minimum, 30 m for those close to the pumping station. As a result, the pumps are classified into 4 groups differing in the head requirement.

5) Tubewell pump station

The main structure of the tubewell station is of reinforced concrete, following the design for the existing tubewell stations. The roof will be structured with reinforced concrete and waterproof mortar finish to allow the work for lifting and lowering the pump with a heavy-duty chain block.

As ancillary facilities to the pumps, effluent pipe of the tubewell station, air release valve, gate valve, check valve, water meter and others will be installed for flow control and measurements.

6) Pipeline connection

The discharge pipes from the tubewell pumps to be installed in this project are planned to be connected to the collector main running along the tubewell pump staitons, which is now under construction by another contractor in the preceding phase of the project. The demarcation of works in this project and the preceding one is as follows:

a. The contractor for this project shall extend the pump discharge pipeline to the outside of pump station for a basic length of 18m with its end closed with a blind flange.

| | |
|---|---|
| - Pipe material | Galvanized steel pipe (GIS G3442 or equivalent) |
| - Method of connection | Threaded connection |
| - Pipe diameter | 300 mm (for a part of extension) |
| - Length of outside pipeline | 18m (basic) |

b. The contractor for the preceding phase (Phase II) is required to connect the collector main of 400 to 900 mm in diameter to the extension of pump discharge line (300mm). Fittings necessary for connection work shall be prepared by this contractor.

7) Secondary electrical work

Primary electric supply work is to be undertaken by WASA. Electrical work for which the contractor is responsible is as follows:

a. Installation of control panel for the intake pump at each pump station
b. Wiring work for power supply to the pump at each pump station from the power meter to be installed by the Pakistani side on the outside wall of the building to the control panel inside
c. Lighting for the pump station

8) List of other facilities under construction in Phase -2 of the project

The entire framework of this project is comprised of the water source facitilites (this project), collecting, transmission and distribution facilities. The lists of these related facilities are shown in the following tables:

a. Collecting facility

| Facility Classification | Type | Quantity | Type of Pipe | Diameter | Length | |
|---|---|---|---|---|---|---|
| Collecting Facility | Collector Main | Total Length 14,620.4m | Ductile Cast Iron Pipe | 400 mm | 3,704.2m | Total 11,431.0m |
| | | | | 500 mm | 3,396.3m | |
| | | | | 600 mm | 2,515.9m | |
| | | | | 700 mm | 1,814.6m | |
| | | | Steel Pipe | 900 mm | 3,189.4m | Total 3,189.4m |

b. Transmission facility

| Facility Classification | Facility or Equipment | Quantity | Specifications | |
|---|---|---|---|---|
| Transmission Facility | Pump Well | 1 no | Structure | Reinforced Concrete Structure, Effective Depth 4m |
| | | | | Flat stab, bent structure |
| | | | Capacity | 4,000m$^3$、Single compartment |
| | Pump Station | 1 no. | Structure | Reinforced Concrete Structure |
| | | | Dimensions | 26,000 x 14,000 |
| | | | Ancillary equipment | Mobile crane, 5,000kg capacity |
| | | | Annex buildings | Chlorinator building: 13,000 x 6000 (1 no.) |
| | | | | Operation control building and Operator's quarters *to be constructed by the Pakistani side |
| | Booster Pump | 3 nos. + 1 standby | Discharge rate | 91,000m$^3$/day (daily operating hours: 20 hours) |
| | | | | @25.3m$^3$/min x 33m x 980 rpm x 190kW |
| | | | Diameter | 450 mm (suction) x350mm (discharge) |
| | | | Type | Double suction volute pump |
| | | | Motor | High tension 3 phase cage type induction current |
| | | | | 190 kW x 6p x AC 3.3 kV x 50 Hz |
| | | | Ancillaries | 450mm butterfly valve (suction side, 5 bar manual) |
| | | | | As above (discharge side, 10 bar automatic) |
| | | | | 450mm swing check valve |
| | | | | 1,000mm ultrasonic flow meter |
| | Chlorina-tion system | Automatic 2 nos. manual 1 no. | Chlorinator | Injector Type: Dosing Rate 0-6,000 g/hr |
| | | | | Measurement range 20：1 |
| | | | Chlorine cylinder | 6 nos. ＋2 standbys, 1 ton volume |

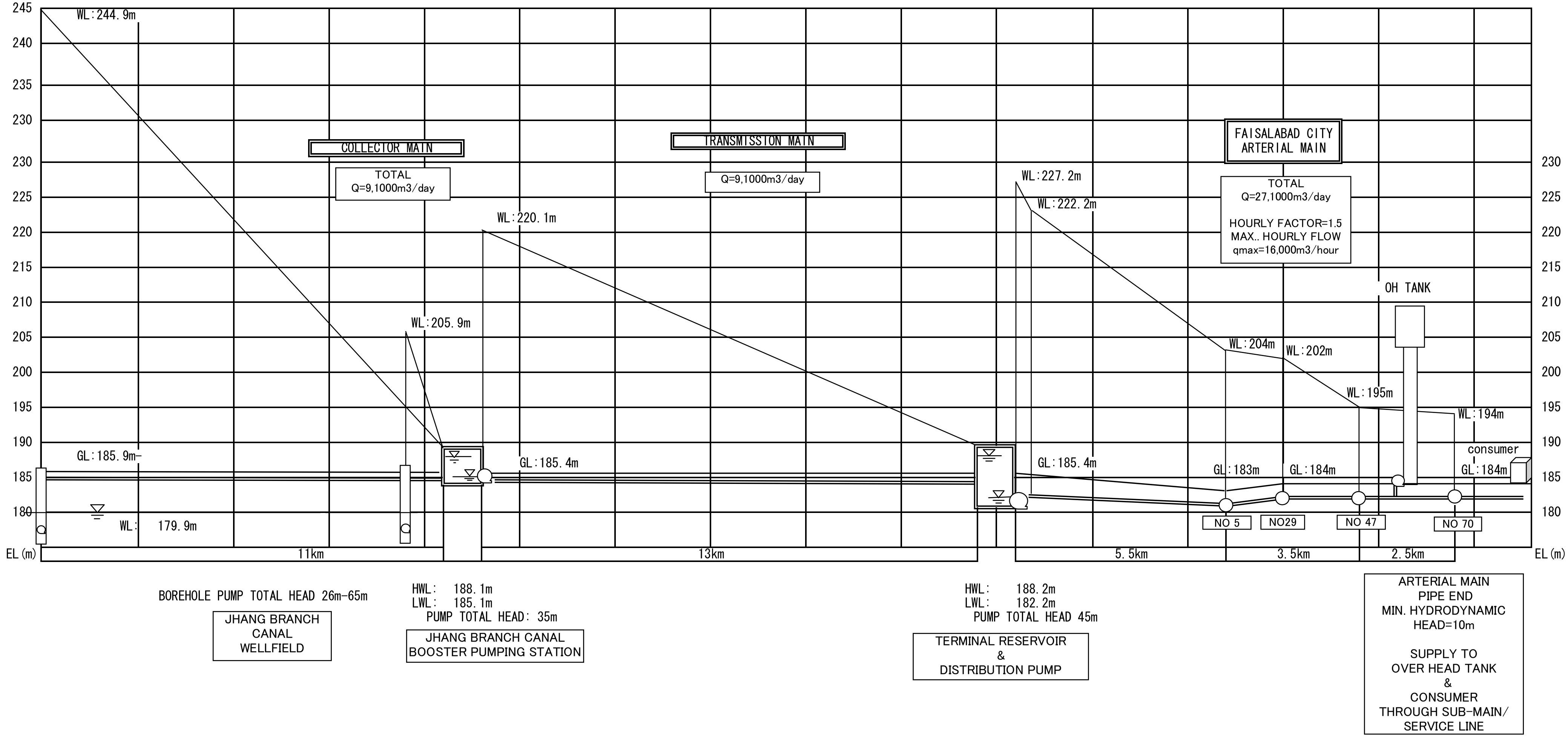
| | | | | |
|---|---|---|---|---|
| | | | Weighing scale | 1 set, 0-4,000kg |
| | | | Leakage detector | 1 set, 0-5ppm |
| | | | Protective equipment | Chlorine neutralizer spray unit (3 units) |
| | | | | Self-priming oxygen respirator (2 units) |
| | | | Decontamination equipment | Neutralizing tower (FRP/PVC), Neutralizer storage tank (FRP) |
| | | | | Neutralization blower, pump |
| | Transmis-sion main | 13,000 m | Steel pipe | Dia. 1,000mm (Exterior: polyethylene coating, interior :epoxy resin paint) |

c. Distribution facility

| Classification | Facility or Equipment | Quantity | Specifications | |
|---|---|---|---|---|
| Distribution Facility | Terminal reservoir | 1 no. | Structure | Semi underground reinforced concrete structure, effective depth 6 m |
| | | | | Flat slab, bent structure |
| | | | Reservoir capacity | 36,000m$^3$ |
| | Pump station | 1 no. | Structure | Reinforced concrete |
| | | | Ancillary equipment | Mobile crane facility 7,500 kg |
| | | | Dimensions | 36,500 x 15,000 |
| | Distribution pumps, Type (1) | 2 nos. | Discharge rate | @31.6m$^3$/min x 45m x 980 rpm x 330kW |
| | | | Diameter | 500mm(suction side) x 350mm (discharge side) |
| | | | Type | Double suction volute pump |
| | | | Motor | High tension 3 phase wound induction motor |
| | | | | 330 kW, 6P, 3.3 kV, 50 Hz |
| | | | | Liquid resistor |
| | Distribution pumps Type (2) | 3 nos. | Discharge rate | @63.2$^3$/min x 45m x 980 rpm x 620kW |
| | | | Diameter | 600mm (suction side) x450mm (discharge side) |
| | | | Motor | 620 kW, 6P, 3.3 kV, 50 Hz |
| | | | Valves for 5 pumps | 500/600 mm butterfly valve (suction side, 5 bar, manual) |
| | | | | 500/600 mm swing check valve (10 bar) |
| | | | | 500 mm cone valve (discharge side 10 bar, electrically operated) |
| | | | | 1,200 mm ultrasonic flow meter |

For reference in the planning of water facilities in this project, the hydraulic grade line from the water sources to the terminal reservoir is shown in Fig. 2-11.

Fig.2-11
HYDRAULIC PROFILE OF JHANG BRANCH CANAL
WATER SYSTEM TO FAISARABAD CITY



### 2-2-2-4 Procurement plan of equipment for operation and maintenance

1) Examination of the requested contents and the procurement plan

The request from the WASA for the project included the procurement of the operation and maintenance equipment necessary for proper management of the facilities by the WASA after the completion of the project. The contents of the request for the procurement were examined through the discussions with WASA during the preceding basic design study. Based upon the conclusion of the study, the equipment was supplied to WASA in the previous implementation period of Phase 1.

Further the conditions of equipment thus procured were confirmed through the follow-up inspection of the completed works after one year from the date of completion (April, 2007). The inspection team reported all the procured equipment and materials were being effectively utilized by WASA.

The procured equipment and materials are listed in the following table 2-12:

Table2-13 List of Equipment Procured under the Project

| | Equipment | Q'ty | Specifications | Purpose |
|---|---|---|---|---|
| 1 | Water level meter | 12 | Battery driven, potable type<br>Measurement depth: 50m | For monitoring plan of groundwater level |
| 2 | Water analysis equipment | 1 | a. Photo-spectral type including reagents for analysis | For testing use by WASA's laboratory |
| | | 2 | b. Potable TDS meter | For monitoring of quality |
| | | 2 | c. Potable EC/pH meter | For monitoring of quality |
| 3 | Equipment to monitor distribution | 1 | a. Ultra-sonic flow meter | For monitoring distribution system |
| | | 2 | b. Automatic pressure recorder | |
| | | 2 | c. Leakage sound detector | For detecting leakage |
| 4 | Voice communi-cation system | 1 | Frequency zone VHF<br>Effective distance 5～30km<br>Power source: commercial power<br>Locations for installation<br>a. WASA headquarters: fixed type x 1 unit<br>b. Terminal reservoir: fixed type x 1 unit<br>c. Jhang B. Canal booster pumping station: fixed type x 1 unit<br>d. Tubewell Operators: potable type x 15 nos.<br>e. Existing inline booster pumping station: fixed type | For communication between the respective facilities and the headquarters and for directions from headquarters |

### 2-2-3 Outline Design Drawing

The basic design drawings are shown in the following pages.