CHAPTER IV. WATER DEMAND PROJECTION

CHAPTER IV. WATER DEMAND PROJECTION

4.1. Urban Water Demand

4.1.1. City Plan

In September 1959 the Federal Capital Commission (FCC) was established to prepare a master plan for the new capital of Pakistan, and various sub-committees under the FCC made a study in detail on the possibility to found the new capital from the viewpoints of topography, transport, regional power, welfare, sanitary, water supply, sub-area development plans, and land use, etc. Based on the study results, a Greek consultant prepared the master plan in October 1960. The Capital Development Authority came into being under CDA Ordinance, 1960 with the task of building the capital as one of its functions, then the actual building process started in October 1961.

Since then, the capital city has rapidly developed. The city development has been particularly stead in Islamabad. In parallel with the progress in construction, CDA is presently currently the master plan so as to have another look at the city development.

On the other hand, Rawalpindi functioned as the interim capital of Pakistan from 1959 to 1968. Its development was remarkable specially in the 1960s. As a result, these two cities have come to exercise combined functions as one city in the process of development.

Under the historical background, the City Planning Bureau of Punjab Provincial Government prepared a master plan for the Greater Rawalpindi in 1970 which aims at completing the city development in 1990. Nevertheless a part of the Greater Rawalpindi has already been developed disorderly and unsystematically despite prudent administrative guidance which takes time.

Although Islamabad and Rawalpindi have, of course, different city functions and structures, it can be said that Islamabad is a beautiful artificial city whose construction was based on a city plan, while Rawalpindi is a city developed disorderly and unsystematicly and whose social infrastructure is unfunctionable and insufficient.

Progress in development and the present characteristics of both cities are described in Section C.1.1. of Appendix.

The service areas of urban water in both cities are divided as follows;

City	Sub-Area	Size of Service Area (sq.km)
Islamabad	Proper Area	220
Rawalpindi	Urban Area, RMC	27
	Urban Area, CANTT	61
	Rural Area	171
	Sub-total	259
	Total	479 sq.km

The above sub-areas are shown in Figure IV-1-1.

4.1.2. Population Forecast and Served Population

A. Population Forecast

The population forecast of a city should be made as accurate as possible in formulating an urban water plan. However, the future population could be affected by various factors resulting from its development process of the relevant city and from surrounding areas, and it is difficult to discuss future population unconditionally.

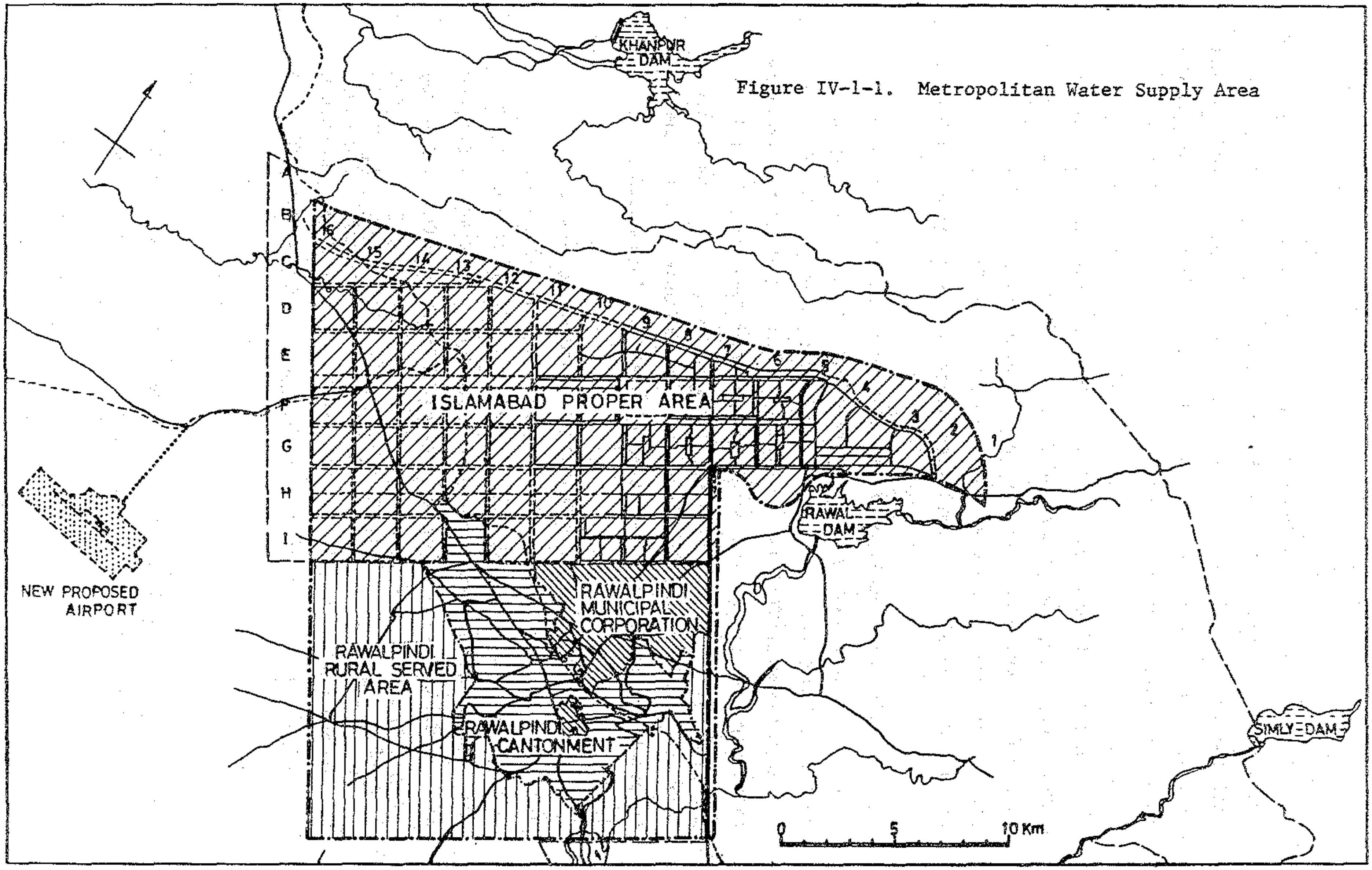


Figure IV-1-1. Metropolitan Water Supply Area

The census is taken every 10 years. The latest available population data were taken at the 1981 census. The population in each target year, which is forecasted from the present land use and from the future land use plans is a basic factor in forecasting the demand for urban water.

In the field survey, a future population has been forecasted after recognizing the types and functions of the two cities each having different background in consideration of a trend of population movement between the two cities and from and to Rawalpindi as well as a population absorbing capacity of each city.

In principle, past population data were plotted on a graph to find out the trend of population movement in each sector so as to calculate the population by sector in the target years. For Rawalpindi RMC, CANTT and rural area were sub-divided into regions similarly. In consideration of a population absorbing capacity of each region, the population was computed with annual population increase rate for each 10 years as shown in Table IV-1-1. The computation methods of population are described in Appendix C.1.2.

Table IV-1-1. Summary of Population Forecast

Sub-area	1987	2000	2010	2030
Islamabad Proper	284,000	621,000	760,000	1,006,000
Rawalpindi Area	1,036,000	1,449,000	1,770,000	2,346,000
Urban Area	945,000	1,327,000	1,631,000	2,150,000
CANTT	409,000	616,000	782,000	1,100,000
RMC	536,000	711,000	849,000	1,050,000
Rural Area	91,000	122,000	146,000	196,000
Total	1,320,000	2,070,000	2,537,000	3,352,000

B. Areal Service Rate

a. Islamabad

Urban water shall be supplied to the whole Islamabad (100%) in the given target years in consideration of the function and type (block-wise) of this city.

b. Rawalpindi

- RMC

Presently urban water is supplied to 75 percent of the population in RMC. It is judged that urban water shall be supplied to the whole city area (100%) in the final target year 2030. The areal service rates in medium target years of 2000 and 2010 are determined at 87.5 percent and 92 percent, respectively.

- Rawalpindi CANTT

Presently urban water is supplied to 67.5 percent of the Rawalpindi CANTT population. This rate is low as compared with that of the RMC. It is attributable to the fact that a high priority in water supply is given to military water and to the fact that villages are scattered at different distances. The target rate in 2030 is determined at 95 percent on condition that some people will continuously use well water. The target rates in 2000 and 2010 are determined at 80 percent and 85 percent, respectively.

- Rawalpindi Rural Area

Groundwater through wells is used in a part of Rawalpindi Rural Area. It is estimated from the data in hand that groundwater is used in about 25 percent of the whole rural area of Rawalpindi. The eastern portion of the area seems to have a high absorbing capacity of population, and will be developed as an urban area whose boundary with CANTT cannot be clearly distinguished. Under the situation, the areal service rates in eastern and western rural portions are determined at 90 percent and 75 percent, respectively, resulting in a rate of 85 percent as a whole. The rates till 2010 are determined at 50 percent for the eastern portion and 30 percent in the western portion, resulting in 45 percent on an average.

The above can be summarized as shown in Table IV-1-2.

Table IV-1-2. Future Service Ratio and Population Served

Sub-areas	Item	1987	2000	2010	2030
Islamabad Proper(I)	Population	284	621	760	1,006
	Service Ratio	100	100	100	100
	Population Served	284	621	760	1,006
Rawalpindi CANTT	Population	409	616	782	1,100
	Service Ratio	67.5	80	85	95
	Population Served	276	492	664	1,045
RMC	Population	536	711	849	1,050
	Service Ratio	75	87.5	92	100
	Population Served	402	622	781	1,050
Sub-Total	Population	945	1,327	1,631	2,150
	Population Served	678	1,114	1,445	2,095
	(Service Ratio)	(71.7)	(83.9)	(88.6)	(97.4)
Rural	Population	91	122	146	196
	Service Ratio	25	30	45	85
	Population Served	22	36	65	166
Total (II)	Population	1,036	1,449	1,777	2,346
	Population Served	700	1,150	1,510	2,261
	(Service Ratio)	(68.5)	(79.4)	(85)	(96.4)
Grand Total (I)+(II)	Population	1,320	2,070	2,537	3,352
	Population Served	984	1,771	2,270	3,267

Note: Population ..... 1,000 persons
Service Ratio .. Percent

4.1.3. Water Demand Projection

A. Definition of Water Demand

Urban water is used for domestic, public, commercial and industrial purposes. Wastage and leakage shall be considered in studying the water demand. In an area like Rawalpindi CANTT, water use for military service should be taken into account in addition to the above categories.

The above-mentioned categories of water use are further divided into the following;

Categories	Water Use in Detail
Domestic Use:	Individual Use: Drinking, Washing, Shower, Shampoo, Prayers' Purification and Flush Lavatory Household Use: Cooking, Laundry, Cleaning, Car Washing, Bathing and Garden Irrigation
Public Use:	Governmental Offices, Educational Institutions, Schools and Universities, Embassies, Hospitals, Railway Stations, Mosques, Churches, and Parks
Commercial/Industrial Use:	Commercial Entities (Retailers, Wholesales), Hotels, Restaurants, Movie Theaters, Factories for Potable Water, Food Processing Industries, Stone Suppliers, and all other Light Industries
Military Use:	The Whole Military Facilities in Rawalpindi Cantonment

Urban water demand in the future could be defined as the sum of the net total demand for water by these consumers plus leakage and wastage. The net total demand for water is estimated from the present water use by consumers in each category in consideration of the future development plan, population increase and other data.

Leakage and wastage from the water supply system are estimated from the present quantities and prospective reduction by the improvement of the supply system in the future.

B. Domestic Water Consumption

The number of family members per household, one of the basic units in determining the water demand, is not much different between Islamabad and Rawalpindi. However, the per capita income, extension rate of flush lavatory, shower & bath and washing machine, and size of irrigated gardens, which reflect the living standard of people, are quite different between the two.

However, in Islamabad where the living standard has been already high, the extension of flush lavatory, shower & bath, and washing machine, etc., will not cause a great increase in domestic water demand. Affected by the decreasing number of family members per household and the upgrading of living standard, the demand for domestic water in Islamabad will continue to increase little by little.

On the other hand, in Rawalpindi the above-mentioned rates are all lower as compared to Islamabad. However, it is noted that these rates are not the major factors of insufficient supply of domestic water in Rawalpindi. The quantity of served water is absolutely short, which has caused a complaint by many citizens. Under these circumstances, it will be an urgent requirement to supply more domestic water in order to cope with rising per capita income in future.

In the study, families in both cities were classified into five categories by their housing lot sizes and the number of rooms per household, and furthermore, divided by sectors and wards. For each category of families, water demand per household was clarified. By combining the above two, 10 units of domestic water demand were determined. From the population distribution to each category of households and the units of domestic water demand, the average daily domestic water demand per capita in 1987 has been computed as follows;

Islamabad Proper Area	Rawalpindi Urban Area
222 lcd (49 gcd)	118 lcd (26 gcd)

From these figures, the daily domestic water demand per capita in the final target year is estimated, in consideration of the annual increase rate of per capita income, as follows;

(unit: lcd (gcd))

Cities	1987	2000	2010	2030
Islamabad Proper Area	222 (49)	236 (52)	245 (54)	257 (56)
Rawalpindi				
Urban Area	118 (26)	151 (33)	177 (39)	227 (50)
Rural Area	-	150 (33)	177 (39)	227 (50)

From the above figures, the daily domestic water demand in the future is computed as follows;

(unit: MLD (MGD))

Cities	1987	2000	2010	2030
Islamabad Proper Area	63.0 (13.9)	146.6 (32.2)	186.2 (41.0)	258.5 (56.9)
Rawalpindi				
Urban Area	80.0 (17.6)	166.9 (36.7)	256.3 (56.4)	475.6 (104.6)
Rural Area	-	5.5 (1.2)	11.5 (2.5)	37.6 (8.3)

C. Public Water Demand

Public facilities have been constructed in Islamabad with priority so as to provide the city with the necessary functions of a new capital of Pakistan. Therefore, the water demand for public facilities is very large although it would decrease in parallel with the increase in population in future.

In the study, military water is excluded in the category of public water. In Rawalpindi the public water consumption per capita is low under the existing condition that water does not appear from water plug, even if the tap is turned on owning to the absolute shortage of water presently supplied. Therefore, it is considered that the public water consumption per capita will increase to a considerable extent, if a sufficient quantity is supplied in future.

The Study Team was furnished by both cities with all data on the public water use at the city public facilities. From these data and a socio-economic study, the following figures have been obtained;

	Islamabad Proper	Rawalpindi Urban
Averaged Daily	42.9 MLD (9.4 MGD)	12.1 MLD (2.7 MGD)
per Capita	151 lcd (33 gcd)	18 lcd (4 gcd)

From the above figures, the daily public water consumption in the future is computed as follows;

(unit: MLD (MGD))

Sub-area	1987	2000	2010	2030
Islamabad Proper Area	42.9 (9.4)	44.0 (9.7)	45.8 (10.1)	65.5 (14.4)
Rawalpindi Urban	12.1 (2.7)	32.3 (7.1)	53.6 (11.8)	93.2 (20.5)
Rawalpindi Rural	- -	0.7 (0.2)	1.7 (0.4)	5.9 (1.3)

D. Commercial and Industrial Water

In Islamabad the store-type shopping areas and industrial zones have been separately developed, while in Rawalpindi commercial and industrial areas which bring residence and place of work close together have appeared almost spontaneously. In Islamabad the commercial and industrial water consumption is large while in Rawalpindi it is small.

In Rawalpindi the commercial and industrial water consumption in general is extremely small. This is attributable to the present insufficient urban water supply, which is far below the demand. In the future, water consumption will increase in parallel with economic growth.

In the study, the demand and supply projection of commercial and industrial water was based on the analysis of the data furnished by both cities regarding their commercial and industrial facilities and those obtained in the socio-economic study. The future economic growth was also taken into consideration.

The future average daily demand is estimated as follows;

(unit: MLD (MGD))

Sub-area	1987	2000	2010	2030
Islamabad Proper Area	34.6 (7.6)	72.4 (15.9)	92.7 (20.4)	126.4 (27.8)
Rawalpindi Urban	20.9 (4.6)	37.9 (8.3)	63.1 (13.9)	118.5 (26.1)
Rawalpindi Rural	- -	0.7 (0.2)	2.0 (0.4)	8.4 (1.8)

E. Military Water

In Rawalpindi CANTT the headquarters of the Pakistani military forces are located. Moreover, the official residences of President and Premier are also placed in this city. Military water is given the highest priority among others, and is the second largest in CANTT after the domestic water.

MES data show that the military water demand in 2000 will be 7 MGD (31,800 cm.d). Based on it, the military water demand in the year 2010 is estimated as follows;

(unit: MLD (MGD))

	1987	2000	2010	2030
Military Use	11.8 (2.6)	31.8 (7.0)	37.8 (8.3)	52.2 (11.5)

Water demand for each category mentioned in above B to E is described in Appendix C.1.3.

F. Leakage and Wastage

The sum of leakage and wastage is larger than any other category of water use in both cities. Reportedly it reaches about 30 percent of the total water production. Water charge is collected without measuring water consumption quantity. Moreover, due to the present intermittent water supply, users forget to turn off a water tap after using water, resulting in a large wastage. People pay no attentions to wastage. Materials being used for the present water system are of poor quality. There are some technical problems which has caused leakage and wastage. These constraints are complicatedly linked, and caused the above-mentioned sum of leakage and wastage.

In formulating a long range plan of water supply in the final target year 2030, it is important to reduce the leakage and wastage as much as possible. It is important for reducing the cost of related water resources facilities.

Therefore, the causes of leakage and wastage have to be clarified in order to reduce the sum of leakage and wastage to some 20 percent in the year 2030 by comprehensive effort for improvement. The figures of leakage and wastage in the target years before the final one are 30 percent in 1987, 27.5 percent in 2000, and 25.0 percent in 2010.

In order to achieve the leakage and wastage rate in the target years, the following countermeasures on the reduction of wastage and leakage losses from water supply systems should be comprehensively taken. (The detail can referred from Appendix C.1.5)

- Modernization of Technical/Institutional Aspects
- Modernization of Moral/Mental Aspects
- Modernization of Financial Aspects

Concretely speaking, great emphasis must be placed on the combined attainment of all day service, perfect metering and quantity tariff systems. These are also the crux of the modernization of the three aspects.

G. Determination of Total Average Daily Demand

The trend of water demand for the years 2010 and 2030 was projected according to the basic unit of water consumption in each category mentioned in the previous paragraph in consideration of an increasing demand due to the improvement of social and environmental circumstances which is anticipated at present, and the result of the projection is shown in Table IV-1-3, IV-1-4 and IV-1-5.

(unit: CMD (MGD))

Sub-area		1987	2010	2030
Islamabad Proper Area		200,800 (44.2)	433,000 (95.3)	563,000 (123.8)
Rawalpindi	RMC	102,000 (22.4)	284,300 (62.5)	446,400 (98.2)
	CANTT	76,300 (16.8)	263,500 (58.0)	477,800 (105.1)
	East Rural Area	500 (0.1)	14,300 (3.1)	47,000 (10.3)
	West Rural Area	300 (0.1)	6,000 (1.3)	17,900 (3.9)
	Sub-total	179,100 (39.4)	568,100 (124.9)	989,100 (217.5)
	Total	379,900 (83.6)	1,001,100 (220.2)	1,552,100 (341.3)

H. Determination of Total Maximum Daily Demand

In a JICA report, 1985, the daily maximum water consumption is estimated at 125 percent of the average daily water consumption (Load rate 0.80) taking into account such factors as temperature, humidity, and scale of the both cities, etc. Apart from it, the load rate of Lahore whose water demand in 1985 is similar to those of the both cities in 2000 was 0.82.

Therefore, the daily maximum water requirement in this study, is determined at 125 percent of the averaged daily water demand. This percentage is nearly equal to those of the similar scale cities in other countries including Japan.

The planned daily maximum water supply for each service area is tabulated below;

(unit: CMD (MGD))

Sub-area		1987	2010	2030
Islamabad Proper Area		251,000 (55.2)	541,300 (119.1)	703,800 (154.8)
Rawalpindi	RMC	127,500 (28.0)	355,400 (78.2)	558,000 (122.7)
	CANTT	95,400 (21.0)	329,400 (72.4)	597,300 (131.4)
	East Rural Area	600 (0.1)	17,900 (3.9)	58,800 (12.9)
	West Rural Area	400 (0.1)	7,500 (1.6)	22,400 (4.9)
	Sub-total	223,900 (49.2)	710,200 (156.1)	1,236,500 (271.9)
	Total	474,900 (104.4)	1,251,200 (275.2)	1,940,300 (426.7)

Planned capacities of intake facilities and main water conveyance pipelines are determined based on the above figures.

Table IV-1-3. Projected Population and Water Demand of Islamabad Proper Area

Item	1987	2000	2010	2030
Total Population	284,000	621,000	760,000	1,006,000
Population Served	284,000	621,000	760,000	1,006,000
Service Ratio (%)	100	100	100	100
Water Demand				unit: MLD (MGD)
Domestic Use	63.0 (13.9)	146.6 (32.2)	186.2 (41.0)	258.5 (56.0)
Public Use	42.9 (9.4)	44.0 (9.7)	45.8 (10.1)	65.5 (14.4)
Commercial/Industrial Use	34.6 (7.6)	72.4 (15.9)	92.7 (20.4)	126.4 (27.8)
Leakage/Wastage Use	60.3 (13.3)	99.7 (21.9)	108.3 (23.8)	112.6 (24.7)
(Percentage of L/W to demand)	(30)	(27.5)	(25)	(20)
Total	200.8 (44.2)	362.7 (79.7)	433.0 (95.3)	563.0(123.8)
Average Daily Demand	200.8 (44.2)	362.7 (79.7)	433.0 (95.3)	563.0(123.8)
Maximum Daily Demand	251.0 (55.2)	453.4 (99.7)	541.3(119.1)	703.8(154.8)
Per Capita Daily Demand				unit: LCD (GCD)
Domestic Use	222 (49)	236 (52)	245 (54)	257 (56)
Total	707 (155)	584 (128)	570 (125)	560 (123)

Note: Average daily demand in 1987 is actual existing value.

Table IV-1-4. Projected Population and Water Demand of Rawalpindi Urban

Item	1987	2000	2010	2030
Total Population	945,000	1,327,000	1,631,000	2,150,000
Population Served	678,000	1,114,000	1,445,000	2,095,000
Service Ratio (%)	71.7	83.9	88.6	97.4
Water Demand				unit: MLD (MGD)
Domestic Use	80.0 (17.6)	166.9 (36.7)	256.3 (56.4)	475.6(104.6)
Public Use	12.1 (2.7)	32.3 (7.1)	53.6 (11.8)	93.2 (20.5)
Commercial/Industrial Use	20.9 (4.6)	37.9 (8.3)	63.1 (13.9)	118.5 (26.1)
Military Use	11.8 (2.6)	31.8 (7.0)	37.8 (8.3)	52.2 (11.5)
Leakage/Wastage Use	53.5 (11.7)	102.0 (22.4)	137.0 (30.1)	184.9 (40.6)
(Percentage of L/W to demand)	(30)	(27.5)	(25)	(20)
Total	178.3 (39.2)	370.9 (81.5)	547.8(120.5)	924.2(203.3)
Average Daily Demand	178.3 (39.2)	370.9 (81.5)	547.8(120.5)	924.2(203.3)
Maximum Daily Demand	222.9 (49.0)	463.6(102.0)	684.8(150.6)	1,155.3(254.1)
Per Capita Daily Demand				unit: LCD (GCD)
Domestic Use	118 (26)	151 (33)	177 (39)	227 (50)
Total	263 (58)	333 (73)	379 (83)	441 (97)

Note: Average daily demand in 1987 is actual existing value.

Table IV-1-5. Projected Population and Water Demand of Rawalpindi Rural

Item	1987 West Area	1987 East Area	1987 Total	2000 West Area	2000 East Area	2000 Total	2010 West Area	2010 East Area	2010 Total	2030 West Area	2030 East Area	2030 Total
Population	29,000	62,000	91,000	37,000	85,000	122,000	44,000	102,000	146,000	63,000	133,000	196,000
Population Served	8,000	14,000	22,000	11,000	35,000	36,000	19,000	46,000	65,000	46,000	120,000	166,000
Service Ratio (%)	27.6	22.6	24.2	30	29.4	30	43	45	44.5	73	90	85
Water Demand (unit: MLD (MGD))												
Domestic Use				1.7	3.8	5.5	3.4	8.1	11.5	10.4	27.2	37.6
Public Use				0.2	0.5	0.7	0.5	1.2	1.7	1.6	4.3	5.9
Commercial/Industrial Use				0.2	0.5	0.7	0.6	1.4	2.0	2.3	6.1	8.4
Leakage/Wastage Use				0.8	1.8	2.6	1.5	3.6	5.1	3.6	9.4	13.0
Total	0.3	0.5	0.8	2.9	6.6	9.5	6.0	14.3	20.3	17.9	47.4	64.9
Average Daily Demand	0.3	0.5	0.8	2.9 (0.6)	6.6 (1.5)	9.5 (2.1)	6.0 (1.3)	14.3 (3.1)	20.3 (4.4)	17.9 (3.9)	47.0 (10.3)	64.9 (14.2)
Maximum Daily Demand							7.5 (1.6)	17.9 (3.9)	25.4 (5.5)	22.4 (4.9)	58.8 (12.9)	81.2 (17.8)
Per Capita Daily Demand												
Domestic Use						150(33)			177(39)			227(50)
Total						264(58)			312(69)			391(86)

Note: Average daily demand in 1987 is actual existing value.

I. Gross Water Requirement

A considerable amount of water losses unavoidably takes place in the course of water intake, treatment, conveyance, and terminal distribution. The majority of water losses occur in or downstream of water conveyance facilities. The above-mentioned water losses are included in the planned daily maximum water supply. However, those water losses appearing in the course of intake facilities to treatment plant are not clear.

As a matter of course, water conveyance distances between intake points and treatment plants different with sites. In this plan, a water quantity for flushing sediment at a desilting basin and water losses in a conveyance canal are designed at 5 percent of the design daily maximum water requirement. In designing the water sources facilities such as dams, the annual water requirement is determined by adding 5 percent of the planned daily water requirement and multiplying by the number of days in a year.

4.2. Irrigation Water Requirement

4.2.1. General

Quite a few factors are intricately related to the computation of the irrigation water requirement. A comprehensive scheme for the computation is described in the flow chart shown in Figure IV-2-1.

4.2.2. Proposed Crops and Their Cropping Pattern

A. Proposed Crops

Crops to be introduced are selected on the basis of the present cropping pattern, soil, farmer's demand, national policy for agriculture and the barani area development plan. Selected crops are summarized as follows. (Refer to Appendix C.2.2.)

- Major Crops
  Wheat for winter crops, maize for summer crops.

- Cash Crops
  Vegetables, fruit, sugarcane and oilseed for cash needs.

- Fodders
  Alfalfa, berseem and stalks of maize for the promotion of dairy farming

- Beans
  Soybeans for the prevention of continuous crop hazards.

B. Cropping Intensity

It is recommended that cropping intensity be raised as high as practicable from the present intensity (100% to 110%) in order to increase yield with irrigation.

Available water resources are so limited that cropping intensity will attain to the maximum 140 percent which is achieved by irrigation in the adjacent area and is adopted by the Irrigation and Power Department of the Punjab. It is considered that the intensity will be lowered according to available water resources in drought years.

Accordingly, the following cropping intensity is adopted and their cropping pattern is described in Figure IV-2-2.

Crops	Intensity (%)
(Winter)	(75)
° Wheat	40
° Oilseed	5
° Fodder	5
° Vegetable	15
° Sugarcane	5
° Orchard	5
(Summer)	(65)
° Maize	30
° Soybean	5
° Fodder	5
° Vegetable	15
° Sugarcane	5
° Orchard	5
Total	140

Figure IV-2-1. Flow Chart for Computation of Irrigation Requirement

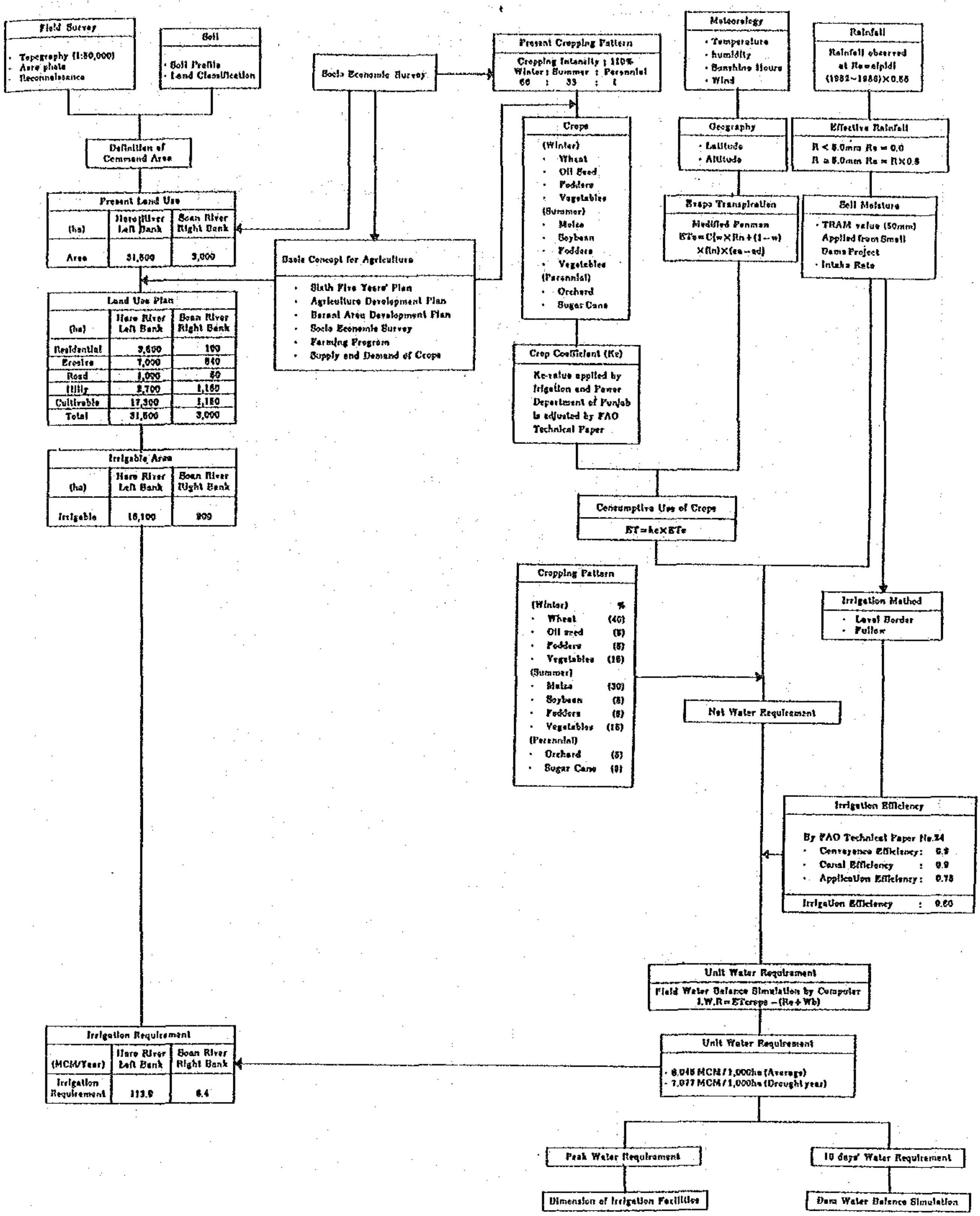
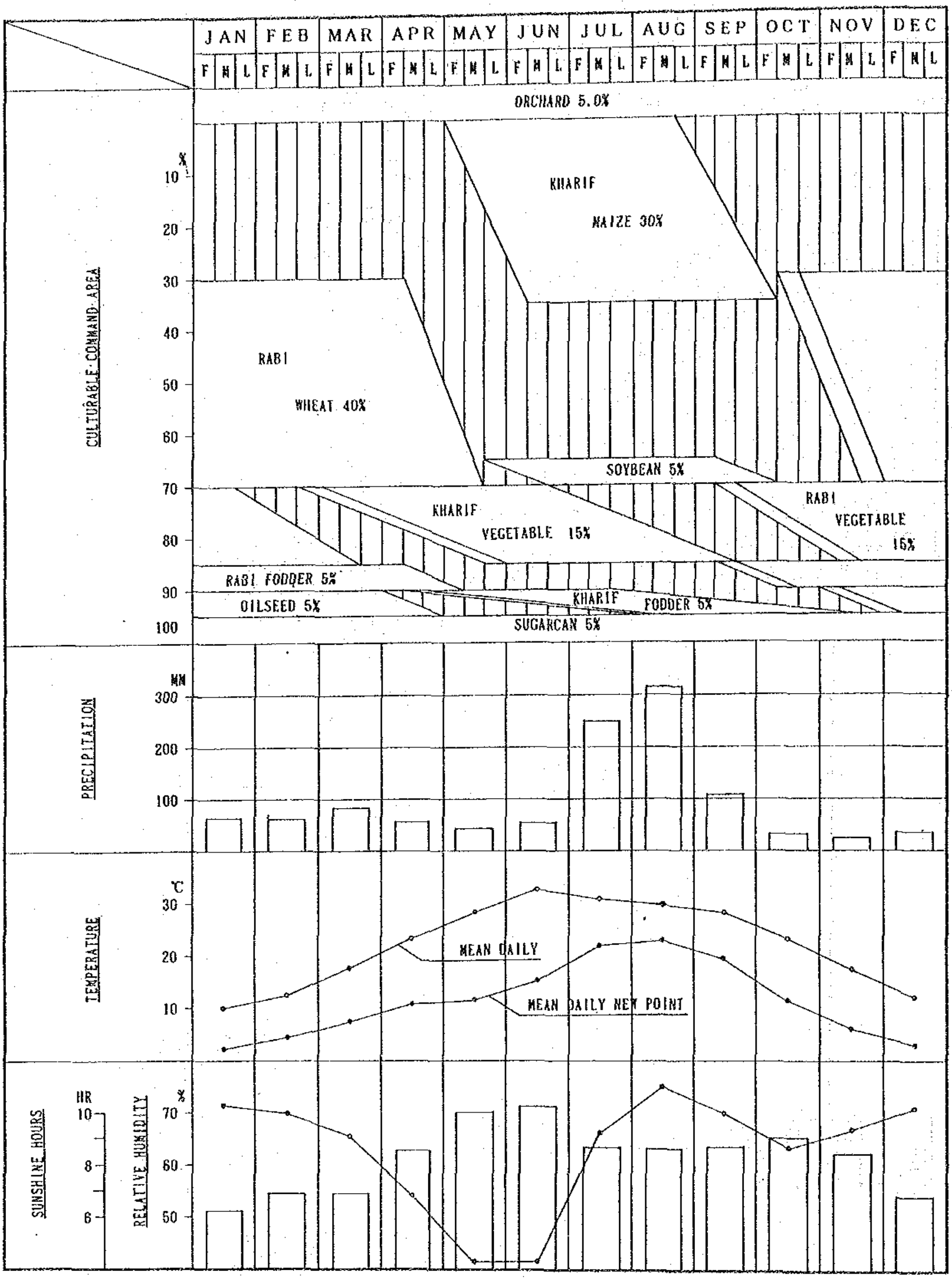


Figure IV-2-2. Proposed Crops and Their Cropping Pattern.



4.2.3. Unit Irrigation Water Requirement

A. Consumptive Use of Crops

a. Reference Crop Evapotranspiration

Reference crop evapotranspiration is generally recognized as a fairly reliable index in calculating consumptive use of crops. This index can be determined by several methods, that is to say, Blaney-Criddle, Radiation, Penman and pan evaporation. Adoption of each of these methods depends on the availability of meteorological data.

The reference crop evapotranspiration estimated by the first three methods is tabulated in Appendix C.2.3. When comparing those estimates with the actual evaporation measured by pan evaporation it can be noticed that the modified Penman method is best suited to the study area because the estimated evapotranspiration equals to actual evaporation x 0.8.

(unit: mm)

Month	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
ETo	1.8	2.5	3.7	5.5	7.4	8.4	6.7	5.7	5.0	3.8	2.4	1.6

b. Crop Coefficient

The crop coefficients to be applied can be adjusted according to "FAO technical Paper No.24" on the basis of those in "Irrigation Requirement of Crops in the Punjab 1980". (Refer to Appendix C.2.3.)

c. Land Preparation

A certain amount of water is required for land soaking. This water should moisture the soil so that the moisture allows smooth land preparation.

This amount is considered 50 mm in depth in reference to the TRAM value which derived from the Small Dams Project studied by the Asian Development Bank.

d. Consumptive Use of Crops

Consumptive use of crops can be calculated in ten days base by the following formula.

$$ETo = c(W \times Rn + (1 - W) \times f(u) \times (ea - cd))$$
$$ET = Kc \times ETo$$

Where;	ETo :	Reference crop evapotranspiration (mm/day)
	W :	Temperature-related weighing factor
	Rn :	Net radiation in equivalent (mm/day)
	f(u):	Wind-related function
	(ea - ed):	Difference between the saturation vapour pressure at the mean air temperature and the mean actual vapour pressure of the air (m bar)
	c :	Adjusting factor
	Kc :	Crop coefficient

Detail calculation is shown in Appendix C.2.3.

B. Unit Irrigation Water Requirement

a. Basic Concept

Regarding a field as a water reservoir, net irrigation water requirement can be obtained by simulating water balance in the field which the components are effective rainfall, consumptive use of crops, soil moisture and TRAM value. Then the gross irrigation water requirement can be obtained by multiplying the net irrigation water requirement by irrigation efficiency.

In addition, the peak irrigation water requirement should be estimated in the drought year appearing once in ten years so that the dimension of irrigation facilities can be determined.

$$N.W.R = (Re + Wb) - ET\ Crops \leq TRAM$$
$$G.W.R = N.W.R/E$$

Where;

ETcrops	:	Consumptive use of crops (mm/day)
Re	:	Effective rainfall (mm/day)
Wb	:	Soil moisture from the previous days (mm)
E	:	Irrigation efficiency
N.W.R	:	Net irrigation water requirement (mm)
G.W.R	:	Gross irrigation water requirement (mm)
TRAM	:	Total readily available moisture (mm)

b. Daily Rainfall

Daily rainfall data are required for field water balance simulation. In adjacent areas, however, Chaklala has the only meteorological station observing daily rainfall. Monthly rainfall data are only available at Bahtar, Saltampur and Songjani near the Study Area. Comparing the Chakalala data with the other data on monthly bases, it has been found that less rainfall than at Chakalala is observed at the three stations. The adjustment of the data is, therefore, necessary for applying them to the study area.

Therefore, the daily rainfall in the Study area is determined by multiplying the Rawalpindi daily rainfall by 0.85.

c. Effective Rainfall

Not all the rainfall is consumed by crops, a fairly large amount of rainfall runs off the surface or percolate into the soil.

Effective rainfall is defired as follows.

° Daily rainfall (R) 5.0 mm < Effective rainfall (Rc) = 0.0 mm

° Daily rainfall (R) 5.0 mm $\geq$ Effective rainfall (Rc) = 0.8 x R no exceed to TRAM Value

d. TRAM Value

Total readily available moisture (TRAM) is the total moisture consumed in effective soil layer when average soil moisture decreases to the wilting point from water holding after 24 hours of soil moisture. TRAM is considered to be 50 mm in depth according to the small dams project studied by the Asian Development Bank.

e. Irrigation Interval

Irrigation interval is obtained by the following formula.

Irrigation Interval = TRAM/ETcrop (day)

Where, TRAM : Total readily available moisture (mm)
ETcrop: Maximum consumptive use (mm)

f. Irrigation Efficiency

i) Irrigation Method

Judging from the topography, soil and irrigation technique, the following irrigation methods can be introduced.

- Wheat, maize and other crop ... Level border
- Vegetable and orchard .... Furrow

ii) Irrigation Efficiency

Irrigation efficiency applied to the area can be determined by the irrigation efficiency table prepared in "FAO Irrigation and Drainage paper No24". (Refer to Appendix C.2.3.)

On the assumption that irrigation canal will be properly lined and well maintained, irrigation efficiency can be calculated in the following way.

Ec (Conveyance efficiency) = 0.9
Eb (Canal efficiency) = 0.9
Ea (Application efficiency) = 0.75

Therefore, E = ec x Eb x Ea = 0.6

g. Unit Irrigation Water Requirement

i) Gross Irrigation Water Requirement

The outcome of the computations is as follows;

Cropping Intensity (%)	Irrigation Water Requirement (1)1/ (MCM/1,000 ha/Year)	(2)2/
110	4.772	5.619
120	5.094	6.014
130	5.570	6.545
140	6.045	7.077

Note: 1/... Average irrigation water requirement in 35 years from 1952 to 1986.

2/... Irrigation water requirement in the drought years appearing once in a ten-year period.

Taking into account the fact that the available water resources are so limited, it is recommended that the optimum irrigation water requirement be determined by changing the cropping intensity or cropping area to meet the available water resources.

ii) Peak Irrigation Water Requirement

The peak irrigation water requirement is computed by the probability analysis of the peak 10 days irrigation requirement. The dimension of irrigation facilities is determined by applying the peak water requirement taking place once a ten-year period. The outcome is as follows;

Qmax = 0.439 MCM/1,000 ha/10days
= 0.508 cu.m/sec/1,000 ha

iii) Yearly 10-Day Irrigation Water Requirement

The yearly 10-day irrigation water requirement during the period of 1960 to 1980 is computed for the dam water balance simulation.

4.2.4. Haro River Left Bank Command Area

A. Land Use Plan

On the basis of the present land use, land use plan can be established in due consideration of the following view points,

- ° Farmer's opinion (Refer to Appendix C.2.1.)
- ° Barani development plan
- ° Land capability
- ° Soil conservation

Future development of the areas will be planned according to the following assumptions.

i) Uncultivable Area

- ° Residential area will expand as population grows.
- ° River including gullied land will be properly protected by various soil conservation schemes.
- ° Road network will be well planned according to communication needs.
- ° Stony hills will be utilized for garden, parks or public community.

ii) Cultivable Area

Great efforts have been made by farmers themselves or authorities concerned in order to promote higher production. Increasing cropping intensity will be more emphasized than extending cultivable area.

iii) Irrigable Area

Irrigable area is determined assuming that canals and levees will occupy seven percent of the total cultivable area.

Therefore, the land use plan will be as follows:

Land Use	Area (ha)
1. Residential Area	3,500
2. Eroded Areas	7,000
3. Roads	1,000
4. Hilly Mountains	2,700
5. Cultivable Area	17,300
Total	31,500
Irrigable Area	16,100

B. Irrigation Water Requirement

The maximum annual irrigation water requirement is computed as follows;

$$Q = 7.077 \text{ MCM/1,000 ha/Year} \times 16,100 \text{ ha}$$
$$= 113.9 \text{ MCM/Year}$$

4.2.5. Soan River Right Bank Suburban Area

A. Land Use Plan

Land use is planned as follows by quoting the rural development plan by ABAD and the assumption previously mentioned.

Land Use	Area (ha)
1. Residential Area	1,000 1/
2. Erode Areas	540
3. Roads	50
4. Hilly Mountains	260 2/
5. Cultivable Area	1,150
Total	3,000
Irrigable Area	900 2/

Note: 1/ ... About 80% of hilly mountains will be consolidated into residential area.
2/ ... Quoted by ABAD's development plan.

B. Irrigation Water Requirement

The maximum annual irrigation water requirement is obtained as follows;

$$Q = 7.077 \text{ MCM}/1{,}000 \text{ ha/Year} \times 900 \text{ ha} = 6.4 \text{ MCM/Year}$$

Accordingly, the total maximum annual irrigation water requirement is summarized as follows:

Project	Command Area (ha)	Total Maximum Annual Irrigation Water Requirement (MCM/Year)
° Haro River Left Bank Command Area	16,100	113.9
° Soan River Right Bank Suburban Area	900	6.4
Total	17,000	120.3

4.3. International Airport Water

4.3.1. Outline of the Development Scheme

A. Background of the New Airport

To cope with increasing air traffic requirements, the Civil Aviation Authority (CAA) decided in 1983 to carry out a feasibility study on the construction of a new international airport for the capital city of Pakistan. The location of this new airport was selected to be near the village Rakh Pind Ranjha in Rawalpindi Tehsil. The master planning and preliminary study for the new airport was completed in June 1986.

B. Location and Main Facilities

The new airport will be located 25 km southwest of Islamabad city center, about 15 km east of Fatehjang town and about 18 km south of the Sang Jani railway station. The present airport will be reserved for the exclusive use of PAF. The total area is about 1,060 hectares, and the land acquisition has been made by CAA. The airport will have a runway of 3,800 m long with 45 m width. In the south of this runway, a land will be reserved for construction of a future second runway and an industrial area of 120 ha. All buildings related to the airport operations are situated north of the runway for the direct access from the city, and the planned buildings are:

- Passenger terminal with a total area of building about 38,600 sq.m;
- Control tower and the operation building;
- Fire station;
- Cargo terminal;
- CAA office building;
- Airport maintenance area with a vehicle maintenance building, facilities maintenance building, warehouse, etc;

- Power plant;
- Flight kitchen;
- Mosque;
- Agricultural - horticultural building; and,
- Various ancillary building.

4.3.2. Traffic and Water Demand Projection

The traffic projections such as passenger traffic, cargo traffic and aircraft traffic recommended in the master plan for the new airport are as follows;

Year	Aircraft	Cargo (metric ton)	Passenger (thousands)	Peak Hour Passenger
1990	16,700	33,100	1,850	1,006
1995	18,900	45,300	2,498	1,159
2000	20,400	60,200	3,049	1,369

The water requirement by the new international airport is about 0.5 MGD (2,300 cmd) in the target year 2000 tentatively. This value came from the new airport master plan report which was issued by CAA. Since this projection is only considered to meet the new airport facilities requirement, the total average daily demand up to the target year 2030 approved by the Coordination Committee is 1.5 MGD (6,800 cmd), including the requirement of airport facilities and relative industry/commerce. From this project, the total average daily demand in 2010 is tentatively determined at 1.0 MGD (4,600 cmd).

The total average annual water demand is summarized as follows:

	2000	2010	2030
Water Demand (MCM/Yr)	0.8	1.7	2.5

4.4. Summary of Water Demand

Water demand as direct targets of the study is summarized below, and the total water demand including indirect targets are shown in Table VII-4-4.

Summary of Water Demand

(Unit: MCM/Year)

- Urban Water

Sub-Area	1987	2010	2030
° Islamabad Proper Area	73.3	165.9	215.8
° Rawalpindi RMC	37.2	109.0	171.1
CANTT	27.8	101.0	183.1
Rural Area	0.3	7.7	24.9
Sub-total	65.3	217.7	379.1
Total	138.6	383.6	594.9

- Irrigation Water

Sub-Project	2000	2010	2030
° Haro River Left Bank Command Area	70.8	88.1	102.3
Small Lift Irrigation Scheme	(11.6)	(11.6)	(11.6)
° Soan River Right Bank Suburban Area	6.4	6.4	6.4
Total	88.8	106.1	120.3

- International Airport Water

Item	2000	2010	2030
° Water Demand of Airport	0.8	1.7	2.5

CHAPTER V. METEOROLOGICAL AND HYDROLOGICAL ANALYSIS

CHAPTER V. METEOROLOGICAL AND HYDROLOGICAL ANALYSIS

5.1. Available Data and Their Reliability

5.1.1. Available Data Collected

Meteorological and hydrological data have been collected from the various agencies. Location of the stations where those data were observed is shown in Figure V-1-1.

A. Meteorological Data

Meteorological data have been collected mainly from RMC and WAPDA, and additionally from the Irrigation Department (NWFP) and the Forest Department (Punjab) on the following six items;

- Temperature
- Relative humidity
- Wind velocity
- Sunshine duration
- Evaporation (Class-A pan)
- Precipitation

Synoptic observation is conducted by four stations at Chaklala (Islamabad Airport), Murree in the mountainous area, Barkot and Rawal Dam. The former two stations belong to RMC, and the latter two to WAPDA. Other stations than above four observe only precipitation.

In the study area, the first meteorological station was established at Rawalpindi in 1866, and operated until 1951 when it was moved to Chaklala. In this study, we could collect the monthly precipitation record from 1890 at this station. Thus, precipitation data are collected at 37 stations in the study area, and at 27 stations in the surrounding area, and at 64 stations in total.

B. Hydrological Data

Hydrological observation is conducted by WAPDA, the Irrigation Department and the Capital Development Authority (CDA) in the study area. Although the observation accuracy and period vary at the stream gauging stations, all of the rivers have or had been observed in the study area. All of these observation records have been obtained in this study, at 32 stations in the study rivers and 4 stations in the adjacent rivers. The WAPDA stations are observing not only water level and discharge but also suspended sediment and water quality.

The following table shows the summary of the collected hydrological data:

Table V-1-1. Stream Gauging Stations

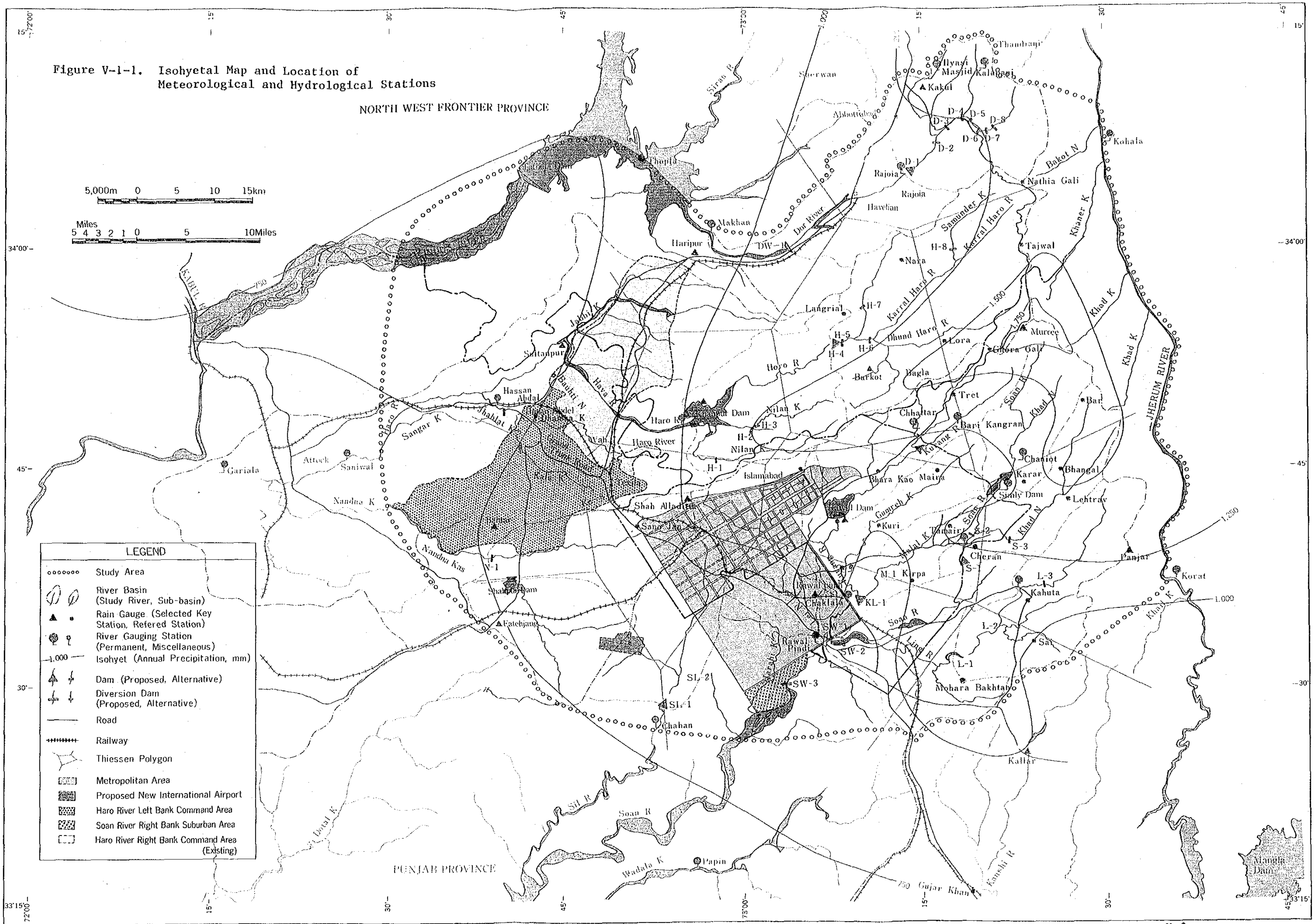
Observation Institute	Study Rivers: Indus	Dor	Haro	Soan	Jhelum	Total	Adjacent Rivers: Siran	Kanshi	Total	Grand Total
WAPDA	1	1	7	9	3	21	2	2	4	25
Irri.Dept(F)	-	4	-	-	-	4	-	-	-	4
" " (P)	-	-	-	5	-	5	-	-	-	5
CDA	-	-	-	2	-	2	-	-	-	2
Total	1	5	7	16	3	32	2	2	4	36

Note: (F): Frontier Province, (P): Punjab Province

5.1.2. Reliability of Data

Reliability has been examined on the collected meteorological and hydrological data through discussions with the concerned agencies on their observation methods, inspection of the observation

Figure V-1-1. Isohyetal Map and Location of Meteorological and Hydrological Stations



sites and data analysis. As a result, the following inference has been made on the reliability of the data:

A. Reliability of Meteorological Data

The reliability is judged to be sufficiently high on data at the synoptic stations belonging to RMC and WAPDA (Chaklala, Murree, Barkot, Rawal dam). For this study, the data at Chaklala and Murree have been used considering the length of observation period and the representativeness of areas for the alluvial plain and for the mountains. The observation period is for 33 years from 1954 to 1986 excepting few observation items at the selected two representative stations.

Table V-1-2. Meteorological Stations and Data for the Study

Observation Items	Station and Observation Period: Chaklala	Murree
(Altitude) MSL	510 m(1,670 ft)	2,205 m(7,236 ft)
Mean Temperature	1954 - 86	1954 - 86
Relative Humidity	1954 - 86	1954 - 86
Wind Velocity	1954 - 86	1954 - 86
Sunshine Duration	1957 - 86	not observed
Pan Evaporation	1966 - 73	not observed

On the other hand, precipitation data are judged to contain less reliable data at some stations. For evaluating the reliability of data, isohyet has been prepared using annual precipitation data as shown in Figure V-1-1. Omitting such stations which differ from other stations on their annual precipitation amount in the Figure, the following thirteen (13) key rain gauge stations have been selected to represent the study area. The study estimates areal precipitation using those 13 key rain gauge stations.

Table V-1-3. Key Rain Gauge Stations

Rain Gauge Station	Data Period	Basis	Data Missing	Annual Precipitation (mm)	Agency
Bahtar	Sep.1952 - 68	Monthly	some	759	RMC
Barkot *	Oct.1962 - 79	Daily	no	1,364	WAPDA
Chaklala *	1952 - 86	Daily	no	1,086	RMC
Fateh Jang	1948 - Apr.86	Monthly	1980-83	743	Forest(P)
Haripur	1952 - 83	Monthly	some	820	RMC
Kakul *	1953 - 86	Daily	no	1,039	RMC
Kallar *	Jul.1960 - 80	Daily	no	938	WAPDA
Khanpur	Aug.1954 - 86	Monthly	some	1,064	RMC
Murree *	1952 - 86	Daily	some	1,750	RMC
Panjar	Aug.1952 - 68	Monthly	some	1,300	RMC
Rawal Dam *	1963 - 79	Daily	some	1,231	WAPDA
Shah Alladitta	1953 - 68	Monthly	many	1,036	RMC
Sultanpur	Sep.1952 - 68	Monthly	some	699	RMC

Among above 13 key rain gauge stations, six (6) stations marked with * have been used for runoff analysis of the study rivers, because daily precipitation records are available at these six stations.

B. Reliability of Hydrological Data

The data of WAPDA is judged to be the most reliable among available hydrological data. WAPDA has set own criteria as shown in the table below for evaluating the accuracy of discharge measurement, and discharge measurement is evaluated at each WAPDA station every year.

Table V-1-4. Accuracy of Discharge Measurement by WAPDA

Accuracy Evaluation	Measurement Error
Excellent	Within ± 2%
Good	± 2% - ± 5%
Fair	± 5% - ± 8%
Poor	more than ± 8%

Accuracy of discharge measurement is not known at other stations than WAPDA, because they do not employ such criteria. The accuracy is judged, however, to be fairly low, because of inexpedience in the measurement and no rating curves at those stations.

Consequently, this study is using only the data of WAPDA in hydrological study.

5.1.3. Interpolation of Data

Some of collected data are missing due to troubles on equipment, or their observation period is too short. Such data can not be sufficiently used for the study without interpolation of data. The following methods have been applied to interpolate the missing data. Details on interpolation are described in Appendix A.

A. Interpolation of Daily Precipitation Data

Interpolation has been made for the daily precipitation data of Barkot, Murree and Rawal dam using adjacent stations such as Kakul and Chaklala.

B. Interpolation of Discharge Data

Discharge data have been interpolated by the runoff analysis as mentioned later for the periods missing data or for the period when discharge is not observed.

5.2. Meteorological Analysis

5.2.1. Climate

Climate can be classified into the following major four seasons in the study area.

Table V-2-1. Climate Season

Season	Period
Cold Season	December - March
Hot Season	April - June
Monsoon	July - August some times mid September (fluctuating by year)
Post Monsoon	Mid September - November

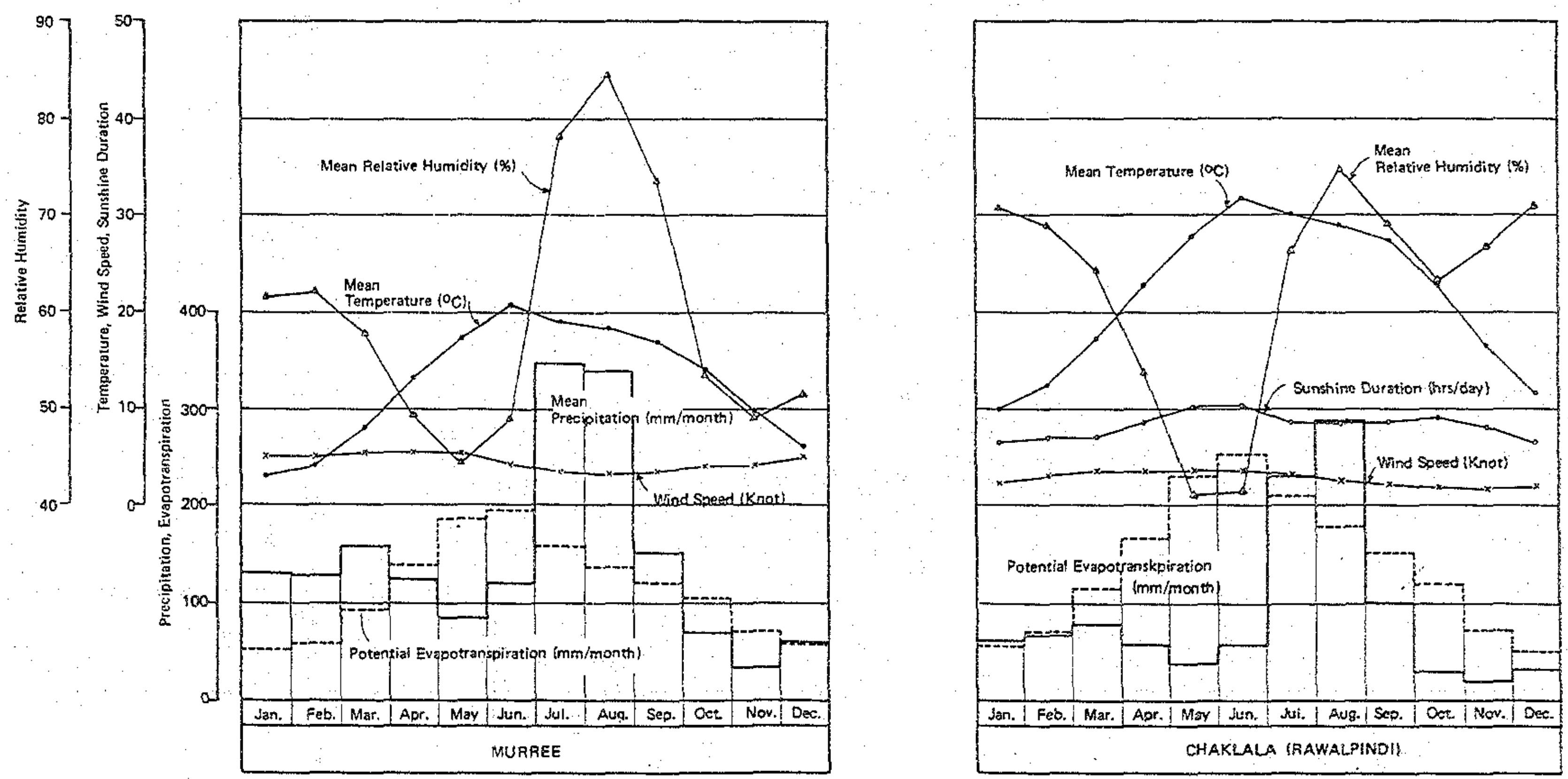
The cold season lasts from December to March, and is characterized by moderate temperature and fine weather. However, depressions pass the area periodically from west, and bring widespread rains and low temperature meanwhile. Consequently, the diurnal range of temperature becomes large in this season. Although snow falls in the Murree and the Margala hills, most of it melts soon except for the high ranges around Murree because temperature rises after depression passing. The amount of rainfall, however, is not much as compared with that during the monsoon. In March and April the weather becomes progressively warmer with scarce spring time.

The hot season lasts from April to June, and is characterized by the continentality with hot and dry climate. May and June are usually the hotest and dusty with the maximum temperature rising up to 45°C, and mean relative humidity falling below 50 percent during these two months.

Table V-2-2. Climatic Elements at Murree and Chaklala (Rawalpindi)

Climatic Element		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Annual	Period
Murree (El. 7,236 ft = 2,205m)															
Precipitation	(mm)														
Max		257.1	60.5	15.7	209.0	160.9	200.6	523.5	263.4	221.5	254.1	57.5	65.9	2,289.7	1977
Mean		131.7	128.9	157.9	124.5	85.7	120.7	347.8	338.3	150.1	69.3	34.3	60.1	1,749.5	1952-86
Min		42.9	103.4	113.5	116.9	110.9	55.1	219.8	220.0	95.7	193.3	23.6	0.0	1,295.1	1969
Mean Temperature	(°C)	3.0	4.0	8.0	13.2	17.4	20.7	19.0	18.3	16.9	14.1	9.7	6.0	12.5	1954-86
Mean Relative Humidity	(%)	61.4	61.9	57.8	49.4	44.4	48.9	78.2	84.7	73.3	53.4	49.0	51.5	59.5	1954-86
Mean Wind Speed	(knot)	5.0	4.9	5.4	5.4	5.3	4.2	3.4	3.2	3.4	3.9	4.0	4.9	4.4	1954-86
Potential Evapotranspiration															
	(mm/day)	1.7	2.1	3.0	4.6	6.0	6.5	5.1	4.4	4.0	3.4	2.4	1.9	3.8	computed
	(mm)	53	59	93	138	186	195	158	136	120	105	72	59	1,374	computed
Chaklala (El. 1,670ft = 510m)															
Precipitation	(mm)														
Max		159.8	73.4	176.5	131.6	109.6	19.4	580.2	338.2	131.3	10.0	5.0	0.0	1,735.1	1981
Mean		62.3	67.8	78.2	57.2	37.8	57.3	254.0	287.6	101.5	30.3	19.6	32.2	1,085.8	1952-86
Min		143.3	13.6	20.6	44.1	45.7	24.7	263.7	91.9	47.0	0.0	1.8	12.5	708.9	1964
Mean Temperature	(°C)	10.0	12.3	17.2	22.7	27.7	31.6	29.9	28.8	27.3	22.6	16.5	11.6	21.5	1954-86
Mean Relative Humidity	(%)	70.7	68.8	64.1	53.6	41.0	41.3	66.1	74.7	68.9	63.0	66.5	70.8	62.5	1954-86
Mean Wind Speed	(knot)	2.2	3.0	3.4	3.4	3.5	3.4	3.1	2.4	2.0	1.7	1.6	1.7	2.6	1954-86
Mean Sunshine duration	(hrs/day)	6.4	6.8	6.8	8.5	10.1	10.2	8.5	8.5	8.6	9.0	8.1	6.5	8.2	1957-86
Pan-Evaporation	(mm)	61.0	82.6	152.4	208.8	309.1	347.7	268.7	208.8	171.2	139.7	85.9	58.9	2,093.7	1966-73
Potential Evapotranspiration															
	(mm/day)	1.8	2.5	3.7	5.5	7.4	8.4	6.7	5.7	5.0	3.8	2.4	1.6	4.6	computed
	(mm)	56	70	115	165	229	252	208	177	150	118	72	50	1,662	computed

Figure V-2-1. Monthly Climatic Elements at Murree and Chaklala (Rawalpindi)



The monsoon (southwest monsoon) generally reaches the area towards the beginning of July and lasts up to the end of August or the mid September. During this period, the monsoon brings much rainfall, about 60 percent of annual rainfall, and high floods are occasionally caused by a series of tropical depressions. The beginning and termination of monsoon varies by year. Severe drought is caused when the monsoon reaches the area late. In case of such drought, serious difficulties are brought on water supply and irrigation.

The post monsoon lasts from mid September to November, and it is the most pleasant season in a year. During this season, it is fine weather and the temperature goes down toward the cold season.

Table V-2-2 and Figure V-2-1 show the major climatic elements at Chaklala and Murree which are the representative meteorological stations in the study area. There is much difference between the climates at Chaklala and Murree, as mentioned in Section 2.3.

5.2.2. Precipitation

A. General

As mentioned above, about 60 percent of the annual rainfall is brought by the southwest monsoon from July to the mid September with the highest amount of rainfall in August. (see Table V-2-2) On the other hand, the depressions also bring rainfall during the cold season, but the amount of rainfall is only about 20 percent of the annual rainfall, less as compared with that during the monsoon.

Consequently, the amount of rainfall varies by season, and such variation of rainfall is one of the major obstructions in agricultural development of the area. Furthermore, the amount of rainfall also greatly varies by year, and droughts occasionally bring difficulties on water supply and damages on agricultural products.

B. Distribution of Annual Precipitation

As seeing the isohyetal map in Figure V-1-1, annual amount of rainfall remarkably differs by area. Annual rainfall ranges from 700 mm to 1,000 mm in the western alluvial plain, and increases toward the eastern mountainous area and reaches 1,750 mm at Murree. Abundant rainfall is observed in the Margala and the Murree hills which are the uppermost river basins of the Haro, the Kurang and the Soan rivers. On the other hand, the river basins of the Dor and the Ling rivers receive relatively less rainfall as compared with above three river basins.

Mean annual rainfall of each river basin has been estimated by Thiessen method, and it may be highest as 1,413 mm in the Soan river basin and least as 837 mm in the Nandna Kas basin. Mean annual rainfall of the whole study area is estimated around at 1,000 mm. Table V-2-3 shows mean annual rainfalls of various river basins estimated by Thiessen method.

Table V-2-3. Mean Annual Areal Rainfall

River Basin	Mean Annual Rainfall (mm)	Point	Catchment Area (sq.km)	Remarks
Dor River	1,269	Dw-1 Site	517.7	
Haro River	1,403	Khanpur Dam	778.0	
Kurang River	1,309	Soan Junction	580.3	
Soan River	1,413	Sw-1 Site	487.9	Excluding the Kurang, the Ling
Ling River	1,129	Soan Junction	404.6	
Jabbi Kas	(820)	Haro Junction	304.0	
Bauhti Nala	(820)	Bauhti Village	12.8	
Jhablat Kas	857	Hassan Abdal SGS	248.6	
Nandna Kas	837	N-1 Site	462.0	
Lei Nala	1,009	Soan Junction	211.2	
Sil River	1,009	SL-1 Site	237.6	
Whole Study Basin	(1,000)		6,800	

C. Long Term Fluctuation of Annual Precipitation

Figure V-2-2 shows the long term fluctuation of annual rainfall by 5 years moving average at Chaklala and Murree where rainfall is observed for a long period. As seen in the Figure, two stations show similar pattern of fluctuation, the long term fluctuation of annual rainfall of the study area can be studied based on the record at Chaklala where the longest observation record is available. According to the record at Chaklala, the following can be stated;

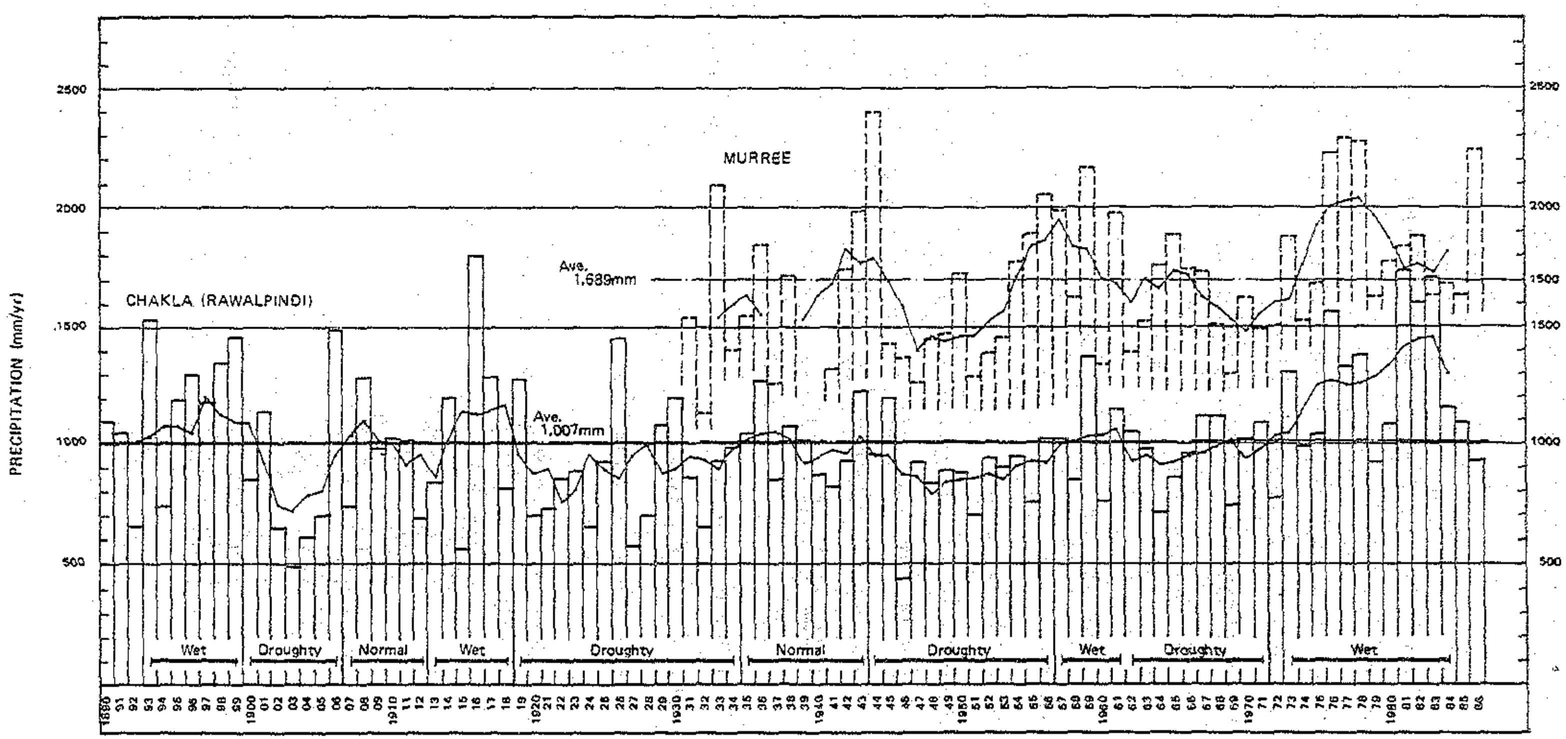
Although the cycle of wet and droughty periods is not clear, there are clear periods of wet and drought, and they last for certain years. This fact suggests the necessity to consider not only the particular drought year while planning the water resources development but also certain years centering the particular drought year in case of the development by dams. The following table shows the distinguishable periods of wet and drought years.

Although there were a few short wet periods in the years from the 1920s to the 1960s, these 50 years was a drought term with less rainfall. On the other hand, it was a wet term after 1973.

Table V-2-4. Wet and Droughty Periods by Annual Rainfall

Period	Duration	Wet/Droughty	
1893 - 1899	7 years	Wet	Period
1900 - 1906	7	Droughty	"
1913 - 1918	6	Wet	"
1919 - 1934	16	Droughty	"
1944 - 1956	13	Droughty	"
1957 - 1961	5	Wet	"
1962 - 1971	10	Droughty	"
1973 - 1986	14	Wet	"

Figure V-2-2. Moving Averaged Annual Precipitation



D. Probable Drought and Wet Years by Annual Precipitation

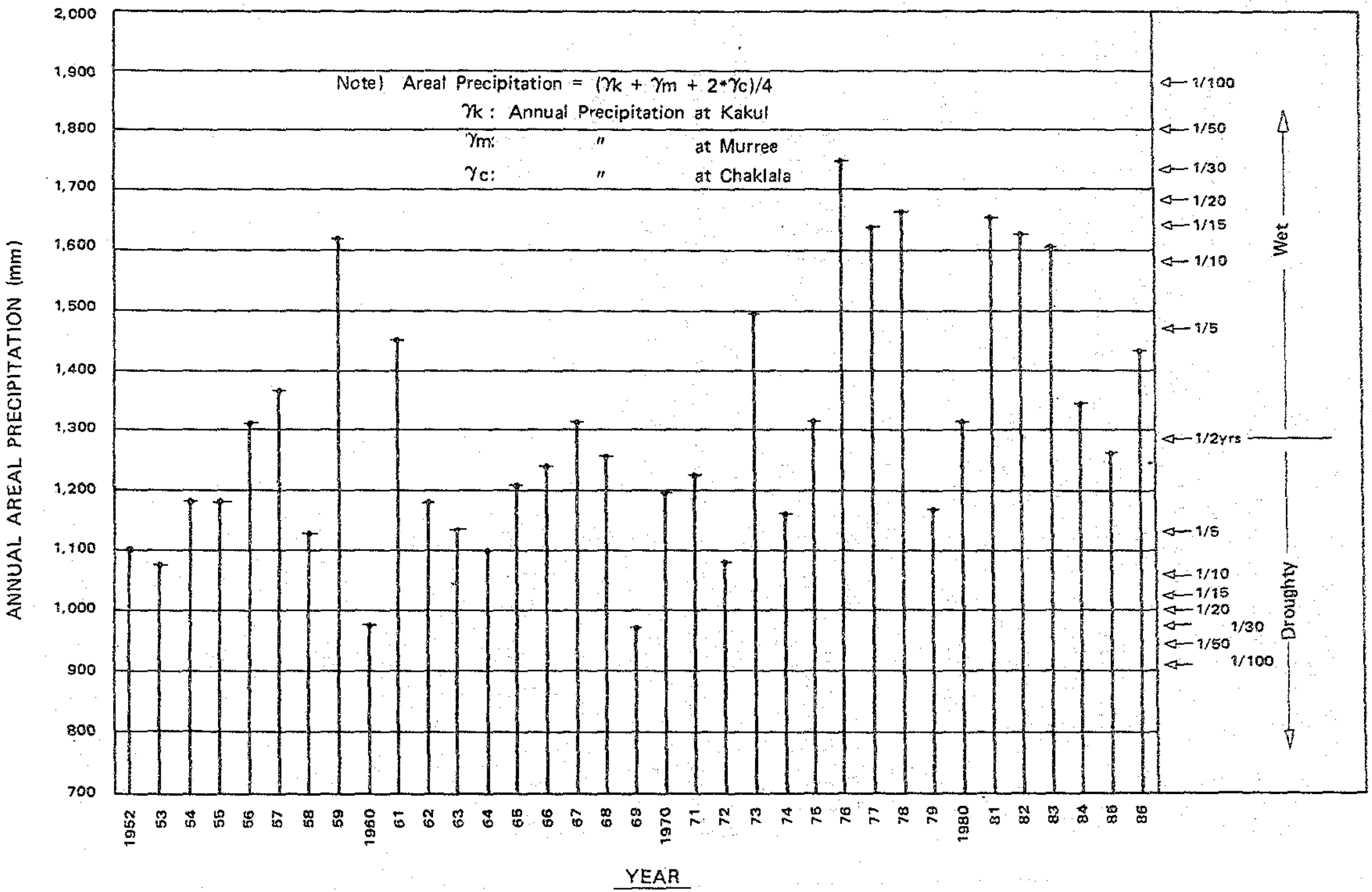
The drought years and the wet years have been studied using the weighted annual rainfall by the data at Chaklala, Kakul and Murree where reliable long term observation records are available. (weight 2:1:1, period 1952-86, see Figure V-2-3) The weighted annual rainfall can be considered to be approximate annual areal rainfall in the Margala and the Murree hills. Probable 1/10 drought year (once in ten years) is close to the years of 1953 and 1972. And, the years of 1960 and 1969 were considerably droughty and equivalent to 1/30 drought year approximately.

Table V-2-5. Probable Drought and Wet Years by Precipitation

Mean Annual Weighted Rainfall	1,306 mm (100%)	1952-86
Maximum Annual Rainfall	1,748 mm (134%)	1976
Minimum Annual Rainfall	971 mm (74%)	1969

Return Period	Drought Year: Probable Rainfall (mm)	Drought Year: Years Corresponding	Wet Year: Probable Rainfall (mm)	Wet Year: Years Corresponding
2 yrs	1,287	1956, 67, 68, 75, 80, 85	1,287	1956, 67, 68, 75, 80, 85
5 "	1,129	1958, 63, 64, 74, 79	1,472	1961, 73, 86
10 "	1,057	1953, 72	1,581	1959, 83
15 "	1,023	-	1,640	1977, 78, 81, 82
20 "	1,002	-	1,679	-
30 "	974	1960, 69	1,733	1976

Figure V-2-3. Annual Areal Precipitation from 1952 to 1986



5.3. Hydrological Analysis

5.3.1. River Basin and River Regime

A. River

a. Dor River

The profile of the Dor river and its important tributaries on the left and right banks are shown in Figure V-3-1.

The Dor river is a left bank tributary of the Siran river and it joins the Indus river also. The confluence with the Siran river has been submerged in the Tarbela reservoir since construction of Tarbela dam in 1974. The characteristics of the Dor river, as may be seen from the above figure, are similar to those of the Haro river in that it falls rapidly in the first 10 km and then runs with a gentle slope. The left bank tributaries are longer and steeper than those on the right.

b. Haro River

The profile of the Haro river and all the important tributaries on its left and right banks important tributaries on its left and right banks are shown in Figure V-3-2.

The Haro river is a left bank tributary of the Indus river. As may be seen from the above figure, the left bank tributaries are longer and steeper than the right bank tributaries. It may also be seen that the Haro river, like the Dor river, falls very rapidly in the first 10 km or so and then runs with a gentle slope.

- Nandna Kas

The Nandna Kas is a major tributary of the Haro river and its length is about 68 km. It joins the Haro river on left bank at 10 km upstream from the confluence of the Indus. It drains the area of 912 sq.km and its slope is 1/70 in upstream and 1/140 in downstream at the confluence of the Bahudra Kas.

- Jhablat Kas

The Jhablat Kas is a tributary flowing into the left bank of the Haro river. It originates from Pind Bahadur Khan Village located between Fatehjang-Hassanabdal road and joins the Haro river at a point some 5 km of the downstream of Hassanabdal. The Kas runs through the high ridge near Hassanabdal.

c. Soan river

The profile of the Soan river and the major tributaries on its left and right banks are shown in Figure V-3-3.

The Soan river is a left bank tributary of the Indus river. It rises in the south-western range of Murree hills, and after flowing through various hills and gorges enters the plains near Cherah.

Flowing in a south-western direction a distance of about 240 km through the plains, it joins the Indus river about 16 km upstream of Kalabagh.

- Malal Kas

The Malal Kas is a tributary with length of 24.8 km joining the Kurang river which drains into the Soan river at 216 km upstream from the Indus. It drains the area of 92.3 km^2 with a bed slope of 1/90.

- Ling River

The Ling river is a major tributary of the Soan river on left bank. It originates in the Lehtrar hills with altitude of 1,850 m and drains the area of 427 km^2 into the Soan river at 225 km upstream from the Indus at Kahuta where altitude is 470 m. It flows in the mountainous area and its bed slope is about 1/40.

- Sil River

The Sil river is a major tributary of the Soan river and it is also called as the Fateh Jang Kas. It originates near Bodia Rustam Khan with altitude of 560 m and drains the area of 595 sq.km into the Soan river at altitude of 338 m near Balawal. The length of the Sil river is 71 km and the bed slope is about 1/300.

Table V-3-1 shows the summary of channel morphology of each river.

Table V-3-1. Channel Morphology

Name of River	Drainage Area	Length of River	Average Width of Basin	Coefficient of Shape	Average Slope	Elevation
	A (km^2)	L (km)	W (km)	F		(m)
Dor R.	608	64.4	9.4	0.147	1:30	2,690-390
Haro R.	3,095	144.8	21.4	0.148	1:60	2,740-270
Nandna Kas	912	68	13.4	0.197	1:320	550-335
Soan R.	11,228	273.5	41.1	0.150	1:120	2,440-240
Malal Kas	92.3	24.8	3.7	0.150	1:90	760-470
Ling R.	427	58	7.4	0.127	1:40	1,850-470
Sil. R.	596	71	8.4	0.118	1:300	560-338

Figure V-3-1. Profile of the Dor Basin

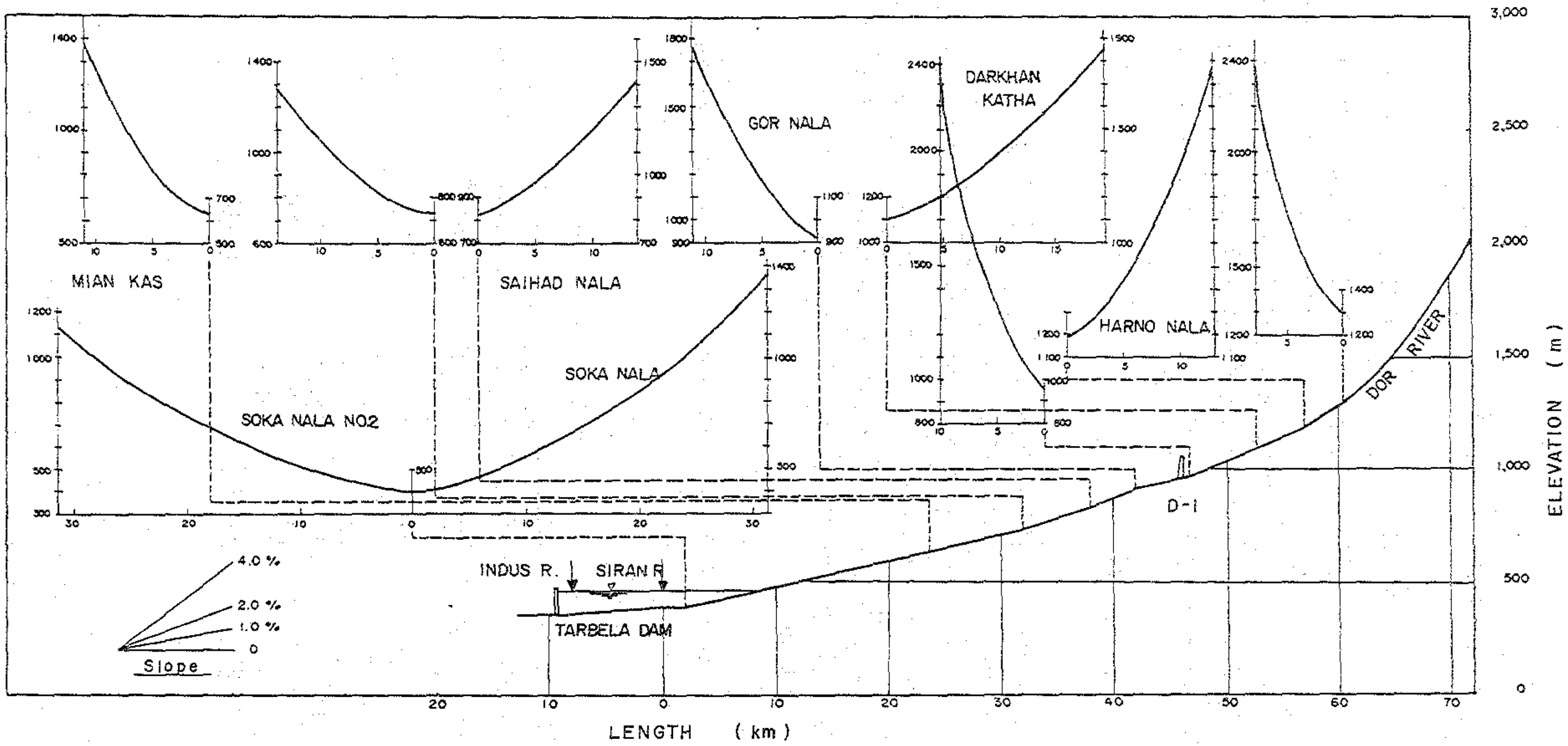


Figure V-3-2. Profile of the Haro Basin

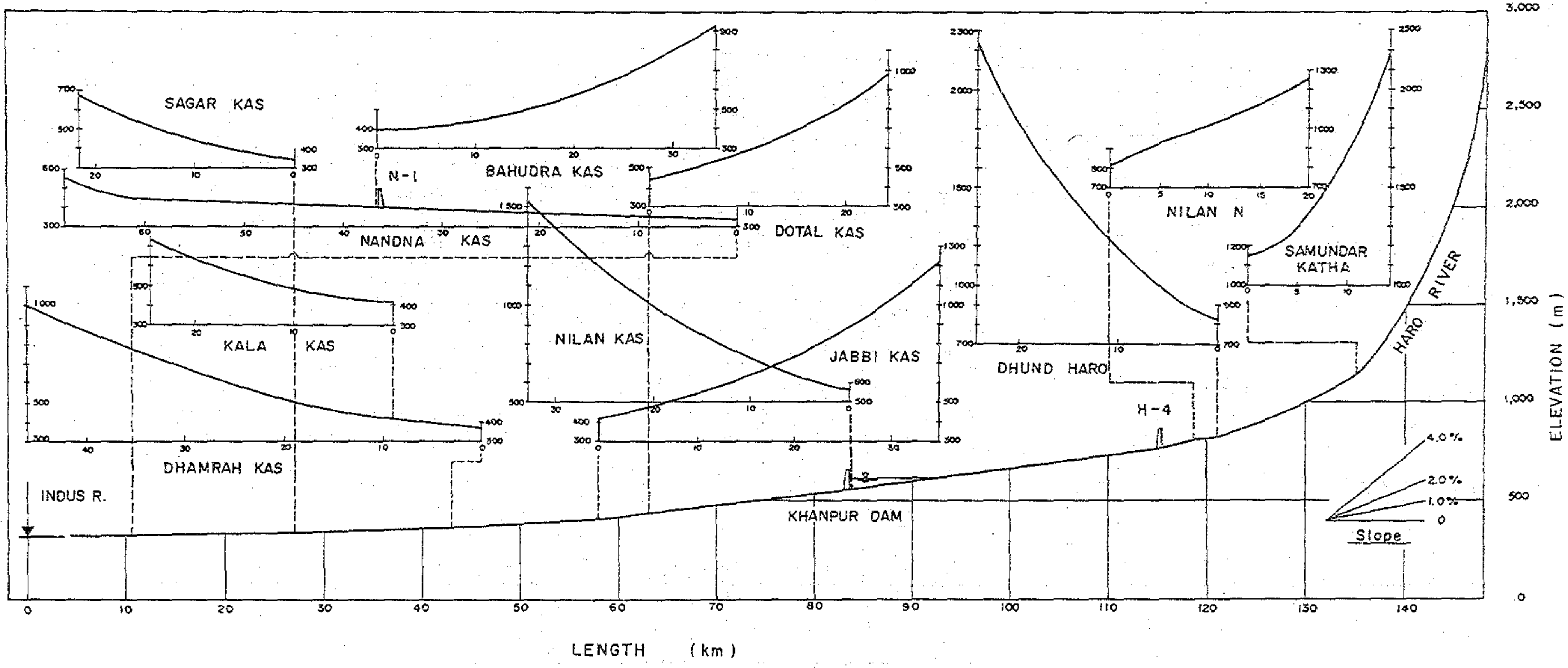
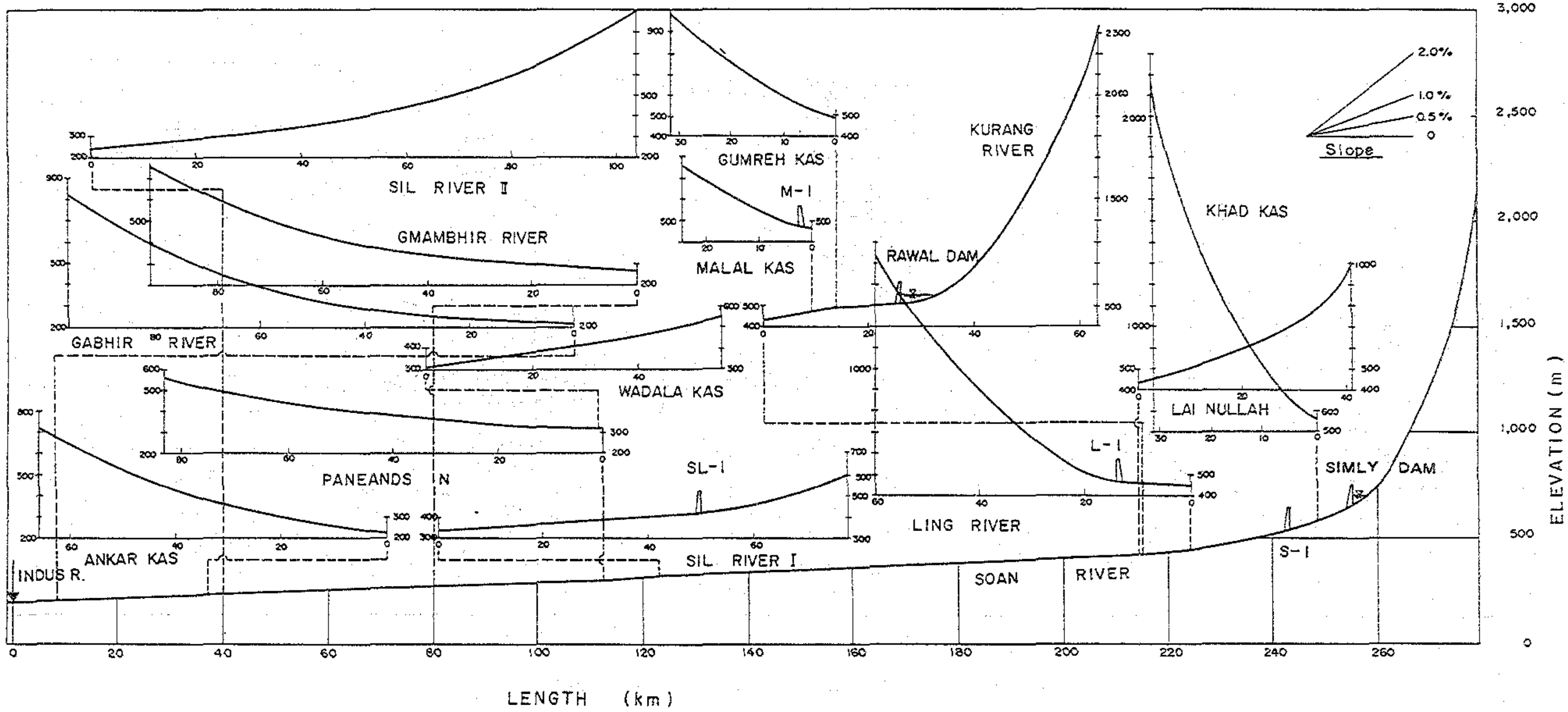


Figure V-3-3. Profile of the Soan Basin



B. River Regime

a. Coefficient of River Regime

As seeing in Table V-3-2, the coefficient of river regime differs by rivers. And the Figure V-3-4 shows the discharge-duration curves at the several key sites in the study area.

1) The Dor and the Haro Rivers

The coefficients of river regime of the Dor and the Haro rivers are so small approx. 100 that the flow of these rivers is relatively steady. The geology of these two river basins is composed of calcareous layers. As many cavities can be seen, their permeability might be judged to be considerably high. In these rivers, the base flow is fed mainly with groundwater, consequently the river flow becomes so steady. Furthermore, hydrographs of these rivers are relatively gentle, because the interflow has a large portion in total runoff and the groundwater recharge is also large. The Jhablat Kas, a tributary of the Haro river, which flows through the alluvial plain has a particularly small coefficient of river regime. This river has an extremely steady flow. In the Thablat Kas basin, there exist many springs where abundant groundwater outflow erupts. As the Jhablat Kas is gaining the groundwater outflow in its base flow, its river flow is so steady. The Bauhti Nala, adjacent to the Jhablat Kas on north, has also steady flow due to the same natural conditions as the Jhablat Kas.

ii) The Soan River

On the other hand, the river basin of the Soan river and its tributaries, the Kurang river and the Ling river, is composed of alternation of sandstone and shale. Consequently, the base flow of these rivers is scarcely fed with the groundwater. Therefore, the river flow of these rivers is not steady and their coefficients of river regime are so large. Groundwater recharge is relatively small in the basin as comparing in the Dor and the Haro river basins. And the surface runoff has a large portion in total runoff. As a result of these facts, the runoff increases sharply when raining and decreases also sharply after raining in the Soan river and its tributaries. The hydrograph of these rivers, therefore, is very sharp and the discharge sharply drops to the level close to the base flow after rain. However, the lower reaches of the Kurang river below the Rawal dam has a good quantity of base flow, because it flows through the groundwater basin which has higher groundwater table than the river bed of the Kurang. The Sohan Nala, a small stream, flowing through the National Park and draining into the Kurang river, has also a good quantity of base flow due to same reason. The Soan river at the GT road bridge nearby Rawalpindi after confluence with the Kurang river gains much increase of base flow mainly from the Kurang river and the coefficient of its river regime becomes small.

iii) Tributaries in the Alluvial Plain

The Haro river and the Soan river at their downstream have many tributaries flowing through the alluvial

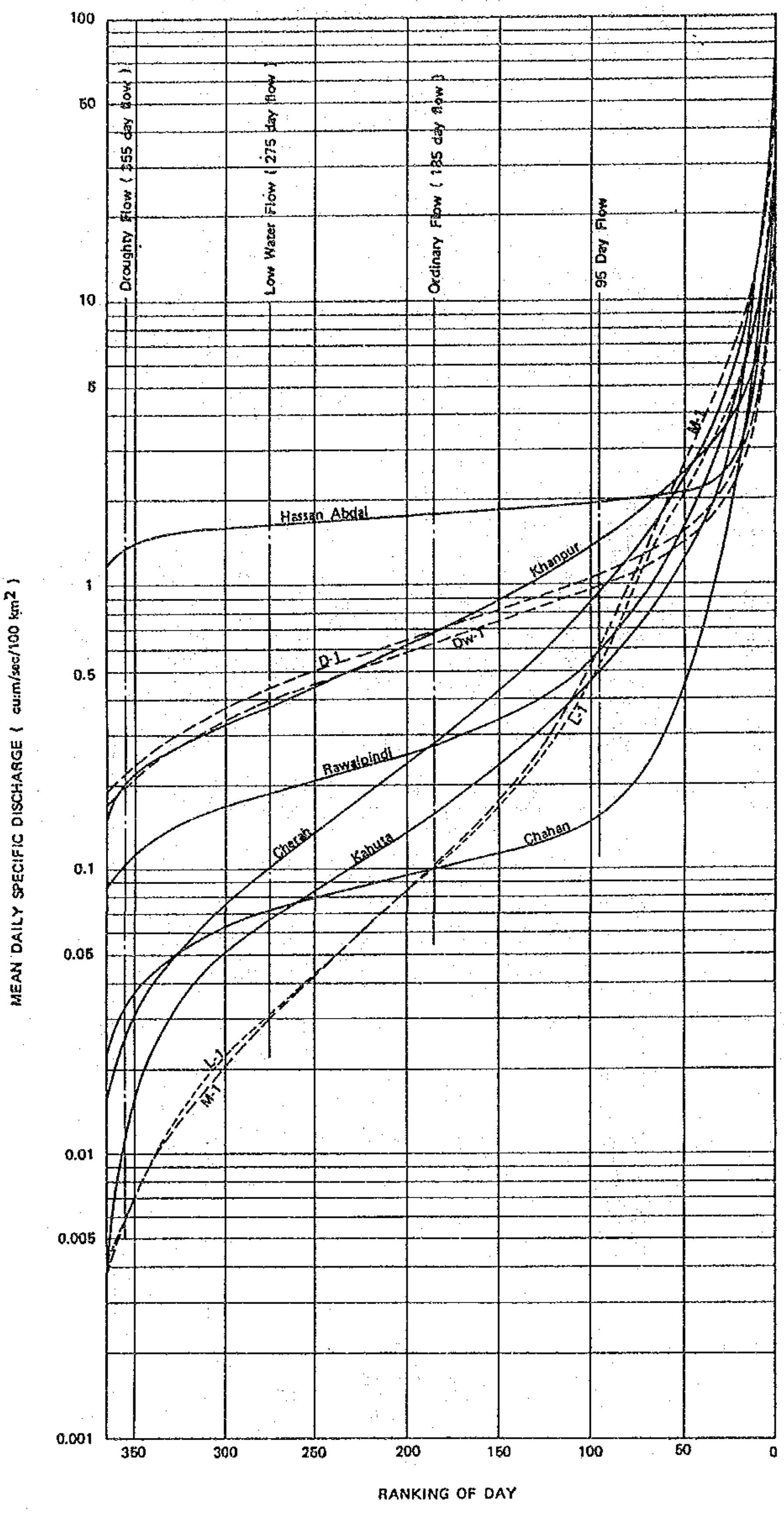
Table V-3-2. River Regime

River	Gauging Station or Development Site	Catchment Area	Discharge (cu.m/sec) Minimum	Droughty	Low	Ordinary	95 Days	High	Coefficient of River Regime
		(km^2)		(355 days)	(275 days)	(185 days)	(95 days)		
Dor	D-1	292.3	0.561	0.65	1.30	2.03	3.27	46.6	83.1
	Dw-1	517.7	0.893	1.03	2.10	3.28	5.30	75.8	84.9
Haro	Khanpur	778.0	1.19	1.57	2.95	5.29	11.17	156.1	131.0
Jhablat Kas	Hassanabdal	248.6	2.90	3.39	4.08	4.41	4.89	61.7	21.3
Soan	Cherah	326.3	0.051	0.083	0.33	0.91	3.04	235.0	4,626.0
	Rawalpindi	1,684	1.46	1.78	3.15	4.71	10.3	667.0	457.0
Ling	Kahuta	145.0	0.005	0.018	0.095	0.225	0.72	93.4	18,680.0
	L-1	285.0	0.012	0.017	0.090	0.297	1.45	87.9	7,325.0
Malal Kas	M-1	82.8	0.003	0.004	0.024	0.084	0.48	37.1	12,367.0
Sil	Chahan	241.0	0.055	0.084	0.17	0.24	0.38	80.5	1,464.0

Remarks 1) Average observed discharge at gauging stations, Average computed discharge at prospect dams.

2) Coefficient of river regime = High flow/Minimum flow

Figure V-3-4. Discharge-Duration Curve



plain. These tributaries except the Jhablat Kas and the Bauhti Nala have relatively large coefficients of river regime but smaller than those in the upstream basin of the Soan river. The hydrographs of the tributaries are also sharp. However, there exists perennial flow in the tributaries which have a certain extent of basin area (e.g. the Sil river at the Chahan Station: 241 sq.km, the Nandna Kas at the Shahpur dam: 203.9 sq.km), but their base flow is relatively small, about 35 litres/sec/100 sq.km.

b. Runoff

i) Runoff Pattern

As mentioned in Section 2.5, the rivers in the study area have two peaks of runoff in March and August. However, the runoff in March is smaller than that in August. (see Figure II-5-2)

ii) Annual Runoff

Table V-3-3 shows the annual runoff at the key sites from 1960 to 1980.

As seeing in the above table, the rivers in the study area generally have a large fluctuation of annual runoff by year. On the other hand, the fluctuation of that of the gigantic rivers as the Indus and the Jhelum is relatively small. In the study area, only the Jhablat Kas has a steady flow with less fluctuation due to abundant outflow of groundwater in its basin.

Table V-3-3. Annual Runoff at Key Sites

River	Site	Drainage Area (sq.km)	Annual Runoff (MCM)			Runoff Height (mm)			Ratio to Mean	
			Mean	Max.	Min.	Mean	Max.	Min.	Max.	Min.
Dor	Dw-1	517.7	155	240	112	299	464	216	1.55	0.72
Haro	Khanpur	778.0	327	629	127	420	808	163	1.92	0.39
Soan	Cherah	326.3	175	361	51.7	536	1,106	158	2.06	0.30
	Rawalpindi	1,684	639	1,301	291	379	773	173	2.04	0.46
Kurang	Rawal Dam	275.1	(100	191	56	364	694	204	1.90	0.56)*5
Ling	Kahuta	145.0	74.6	154	31.3	514	1,062	216	2.06	0.42
Jhablat	Hassan Abdal*1	248.6	170	187	152	686	752	611	1.10	0.89
Sil	Chahan	241.0	50.3	99.5	20.0	209	413	83	1.98	0.40
Indus	Darband*2	166,019	76,700	101,600	64,400	462	612	389	1.32	0.84
	Tarbela Dam*3	168,350	69,200	84,500	57,400	411	502	341	1.22	0.83
Jhelum	Kohala*4	24,890	24,900	34,500	13,000	1,000	1,386	522	1.39	0.52

Note) *1: September 1961 - June '65 *2: 1961 - April 1974

*3: October 1973 - September 1986 *4: 1965 - 1980 ..... 1960 - 1980 other than *1 - *4

*5: Data Source: Upper Kurang Study

On the other hand, the mean runoff height of the Jhelum reaches 1,000 mm in a year which is much higher than other rivers. This is due to the existence of a heavy rain zone reaching the maximum annual precipitation higher than 2,000 mm in Kashimir which covers almost of the uppermost river basin of the Jhelum. The Indus river, the other gigantic river, has less runoff (411 mm in a year) than the Jhelum river, but the Indus river has an extremely larger runoff amount than the Jhelum because of a huge river basin extending to the central area of the Himalayas.

In the study area, the Jhablat Kas has the highest runoff height reaching 686 mm in a year on average. As mentioned above, this river is a peculiar river receiving abundant outflow of the groundwater in its river basin and is much different from other rivers in the runoff mechanism. Among rivers other than the Jhablat Kas, mean annual runoff is the highest, 536 mm at Cherah upstream the Soan river and the least in the Sil river in the alluvial plain (209 mm). Mean annual runoff height of the rivers in the study area, except the rivers having abundant outflow of the groundwater, ranges from 209 mm in the alluvial plain and to 536 mm in the uppermost of the study basin where much precipitation is received.

ii) Probable Drought and Wet Years by Annual Runoff Amount

Based on the probability analysis of annual runoff data for 21 years from 1960 to 1980, the wet year and droughty year differ by the rivers.

Accordingly, for the whole study area these years have been analyzed using the synthetic runoff estimated as shown in Table V-3-5. As the result of the probability analysis, the years corresponding to a return period of 10 years are estimated at the year of 1977 as the wet year and at the year of 1974 as the drought year. The year of 1968 was considered as the normal year.

Table V-3-4. Probable Drought and Wet Years by the Synthetic Annual Runoff

Wet/Droughty	The Year Equivalent to the Return Period 10 Years	The Years exceeding the Period 10 Years
Wet Year	1977	1976, 78
Normal Year	1968 (note: Return Period 2 years)	
Drought Year	1974	1969, 72

Table V-3-5. The Synthetic Annual Runoff in the Study Area

River	D.A. (sq.km)	Mean Annual Runoff (MCM)	Point
Dor	517.7	155	Dw-1 Site
Haro	778.0	327	Khanpur Station
Soan	1,680.0	639	Rawalpindi Station
Sil	441.5	92	Total basin area at SL-1 (237.6 sq.km) and Shahpur dam (203.9 sq.km)
Total	3,417.2	1,213	

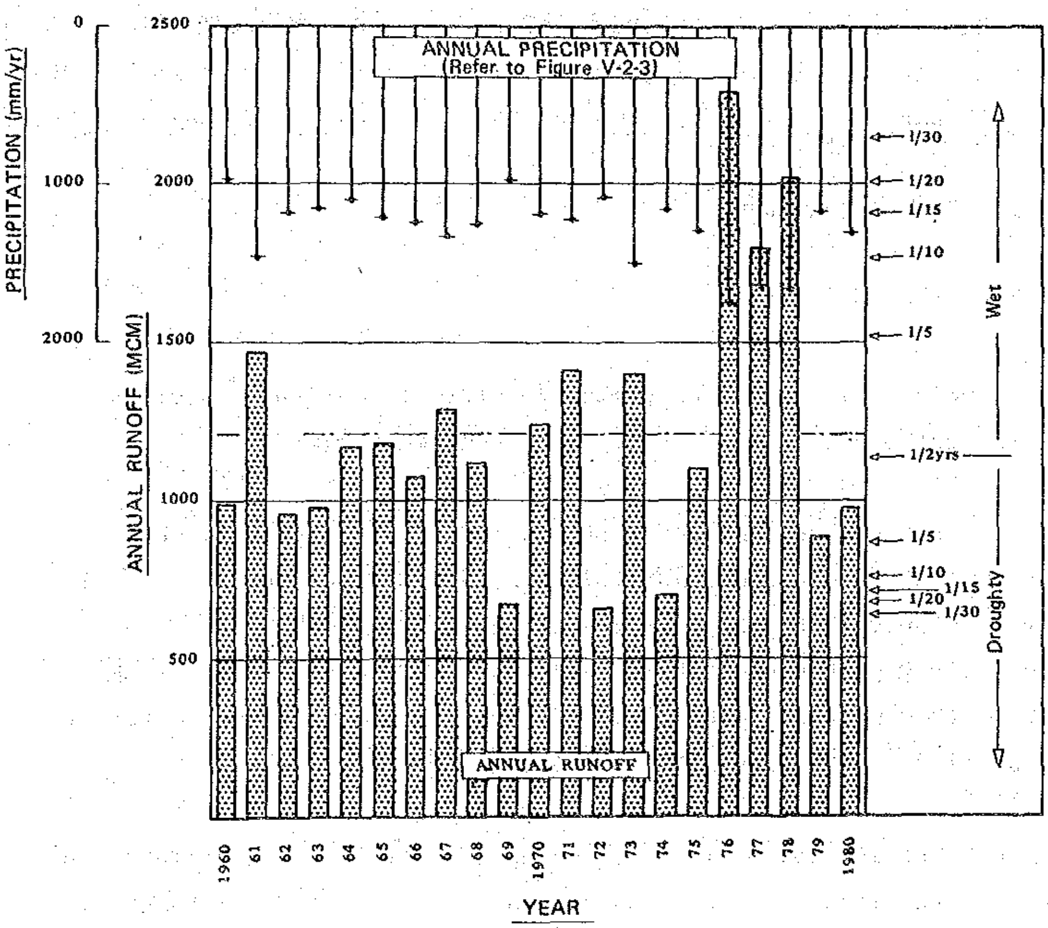
Return Period	Probable Annual Runoff (MCM) Wet	Normal	Droughty
2 years		1,139	
5 "	1,515		869
10 "	1,767		759
15 "	1,911		711
20 "	2,011		682
30 "	2,153		646

Note: Drainage area has been considered at the potential water resources development sites.

Figure V-3-5 shows a series of the synthetic annual runoff. The following facts can be recognized in this figure.

- Wet years continuing from 1976 to 1978.
- Droughty years frequently occurring from 1969 to 1974.

Figure V-3-5. Annual Runoff and Its Probability



c. Floods

According the flood records in the study area, high floods generally occur during the monsoon centering in August. The specific runoff of the maximum recorded flood reached 8.35

cu.m/sec/sq.km in the study area. This specific runoff is equivalent approximately to c = 75 in the Creager equation. Table V-3-6 shows the maximum floods recorded at major gauging stations in the study area.

Table V-3-6. The Maximum Recorded Floods in the Study Area

River	Station	D.A. (sq.km)	Discharge (cusecs)	Discharge (m^3/s)	Discharge ($m^3/s/km^2$)	Recorded Date	Observation Period
Haro	Khanpur	778.0	28,800	816	1.05	23 Jun'71	1960 - 75
Soan	Cherah	326.3	96,200	2,724	8.35	20 Aug'75	1960 - 75
	Rawalpindi	1,684	107,000	3,030	1.80	13 Aug'70	1953 - 75
Ling	Kahuta	145.0	39,800	1,127	7.77	31 Aug'70	1961 - 70
Sil	Chahan	241.0	31,400	889	3.69	20 Aug'75	1962 - 75

5.3.2. Runoff Analysis

A. Introduction

Runoff analysis has been made in order to estimate runoff at the the prospective water resources development sites.

Taking into account the fact that the drought year is set in once in ten years, runoff analysis must be returned for at least 20 years. Observation period of runoff gauging differs by stations. Since fairly good rating daily basis data during the period from 1960 to 1980 were collected, runoff analysis can be returned for the period of 21 years from 1960 to 1980.

Considering the minimum regulated river flow to be released to maintain the rivers healthy and the water resources development by diversion dams, runoff analysis has been made in principle on a daily basis, but on 10 days' basis in some rivers.

The runoff analysis has been made by correlation analysis between gauging stations (10 days' basis), the tank model analysis (daily basis) and the conversion by catchment area ratio. Runoff analysis has been proceeded in two steps. In the first step, gauging records have been interpolated to prepare complete data for 21 years from 1960 to 1980 at the Khanpur, Cherah, Kahuta and Chahan gauging station. However, interpolation of the data at Hassan Abdal gauging station was not possible because of no available daily rainfall record nearby this station. And, using complete data at gauging stations, runoff analysis has been made to estimate runoff at the prospective development sites in the second step.

B. Runoff Analysis at the Gauging Stations

At the gauging stations where data are not available for a long period of time and are considered to be applied to other river basins, runoff analysis has been returned by the tank model or the correlation from 1960 to 1980.

Daily rainfall data applicable for the tank model analysis are available only at the following six stations, so that the areal rainfall has been modified, referring to the isohyetal map when needed. Computed runoff can not be exactly adjusted to the observed runoff especially in case of flood, because the precise areal rainfall can not be computed by the data only at 6 stations.

Therefore, the structure of tank has been determined in consideration of the fact that computed runoff is adjusted to the total amount of observed runoff in long term basis and to the observed recession.

<u>Rainfall Stations used in the Tank Model</u>

- Rainfall stations where the gauging period is long enough
  Kakul, Murree, Chaklala (1960 - 80)

° Rainfall stations where gauging period is not long enough

Barkot (Oct. 1962 - 79)

Kallar (Jul. 1960 - 80)

Rawal Dam (1963 - 79)

Following Table V-3-7 showing the summary of result of runoff analysis at the gauging stations.

Table V-3-7. Runoff Analysis at Gauging Stations

(unit: mm)

Station	Analized Period	Annual Average Areal Rainfall	Analysis by Tank Model: Actual Runoff	Computed: Runoff	Computed: Evapotrans.	Computed: Grand Water	Analysis by Correlation	Interpolation
Khanpur	'63-'75	1,318	363 (0.275)	381 (0.289)	711 (0.540)	226 (0.171)		
Cherah	'63-'75	1,318	453 (0.344)	459 (0.348)	704 (0.534)	156 (0.118)	Rawalpindi Correlation Coefficient r = 0.887 (Apr.1960-1980)	by correlation Jan.-Mar. 1960
Kahuta	'62-'70	1,258	391 (0.311)	401 (0.319)	714 (0.567)	143 (0.114)	Cherah Correlation Coefficient r = 0.911 (20 Jul.1960-10 Jul.'71)	by correlation 1960-20 Jul. 1961 11 Jul. 1971-1980
Chahan	'63-'80	981	204 (0.208)	211 (0.215)	605 (0.617)	159 (0.162)		by tank model 1960-2 Mar. 1962

Remarks: ( ) ratio to rainfall

C. Runoff Analysis at the Prospective Development Sites

Considering the analogy of river basins, runoff at the prospective development sites has been computed with the interpolated gauging data by applying the catchment area ratio and the tank models developed at the gauging stations.

Runoff analysis of the Dor river basin has been made by applying the tank model at Khanpur station because the Dor river basin is similar to the Haro river basin in geology and morphology.

The Soan river basin has been analyzed with the tank model developed at Cherah and Kahuta. The tributaries in the alluvial plain have been analyzed using the tank model developed at Chahan in the Sil river.

The results of the above analysis are shown in Table V-3-8.

Table V-3-8. Runoff Analysis at the Prospective Development Sites

River	Develop-ment Site	Catchment Area (km^2)	Runoff Analysis	Annual Average Runoff (MCM)	Annual Average Height (mm)
Dor	D-1	292.3	Tank model at Khanpur Station	96.24	329
	Dw-1	517.7	- ditto -	154.72	299
Haro	H-4	498.5	Multiplied Khanpur runoff by catchment area ratio	209.50	420
	Khanpur Dam*	778.0	Applied Khanpur runoff	326.95	420
Nandna Kas	Shahpur Dam*	203.9	Multiplied Chahan runoff by catchment area ratio	42.54	209
	N-1	462.0	- ditto -	96.35	209
Soan	Simly Dam*	152.8	Multiplied Cherah runoff by catchment area ratio, and adding CDA intake after 1969 (Same procedure as Khanpur Conduction F/S)	83.45	546
	S-1	341.1	Multiplied Cherah runoff by catchment area ratio	182.85	536
	Sw-1	1,472.8	Multiplied Rawalpindi runoff by catchment area ratio	558.70	379
Kurang	KL-1	283.7**	Using three tank models at Khanpur, Cherah and Chahan, and applying C.A.4.0, 157.7, 122.0km^2 considering geological formation.	102.66	362
Malal Kas	M-1	82.8	Applying tank model at Cherah and Kahuta	38.03	459
Ling	L-1	285.0	Multiplied Kahuta runoff by catchment area ratio, then adjusted the runoff by reduction of runoff coefficient estimated by tank model	107.10	376
Sil	SL-1	237.6	Multiplied Chahan runoff by catchment area ratio	49.57	209

Note * ... Existing Dam, ** ... excluding the catchment area of the Rawal dam

5.3.3. Sediment Analysis

A. Sediment Condition

The Dor and the Haro rivers rise and flow mostly in the Siwalik Range, carrying heavy sediment loads in silty area. The sediment loads come mainly from land slides, bed and bank erosion, gully erosion due to broken mountain side terraces and overgrazing in the catchments.

The Soan river and its tributaries are recent drainage system, with typical a feather-like basin, cutting deep channels and eroding gullies in the Potwar Plateau. Geologic erosion is the dominate source of the sediment and it is increased by the weathering action of wind and rain on the weak sandy shales and loose clays of the Siwalik Range.

B. Sedimention

Estimation of the rate of reservoir depletion by sediment accumulation involves following basic concepts;

(i) Sediment flow
(ii) Trap efficiency of the reservoir
(iii) Density of the sediments

a. Sediment Inflow

The inflow of sediment to the reservoir consists of the "suspended load" and "bed load".

The suspended load can be estimated from the available data of sediment records at the gauging stations installed in the vicinity of the proposed damsite.

There is no easy practical method for measuring the bed load discharge transported by streams. In general, bed load estimates range from about 10 percent to 50 percent of the total sediment load.

For this study the bed load is assumed to be 10 percent of the estimated suspended sediment load, i.e., 9.1 percent of the total sediment load.

b. Trap Efficiency

Trap efficiency represents the percentage of sediment inflow which is deposited in the reservoir. The trap efficiency is calculated with following equation;

$$Er = 100 \left(\frac{Qs}{Q_I} \right)$$

where

- Er : Trap efficiency (%)
- Qs : Annual sediment deposition in the reservoir
- Q_I : Total annual sediment inflow to the reservoir

The trap efficiency is assumed at 60 percent for the damsites with catchment area less than 100 km^2 and at 40 percent for those more than 100 km^2, from the actual observation on relation between the trap efficiency and the catchment area in the existing reservoirs.

c. Density of Sedimentation

The specific weight of sediment is predicted in order to estimate the storage space which is displaced by sediment in a given period of time.

The density of sediment adopted is the unit weight of fresh deposits of the sediment, that is, equivalent to the density of suspended sediment.

C. Estimation Methods

Several methods for estimation of sediment yield are listed below.

(i) KHOSLA's empirical method.
(ii) Irrigation Research Institute formula.
(iii) Estimation from the generalized sediment rate and catchment area curve.
(iv) Estimation from the sediment records.

The results of the examples of design and the actual sediment lead to a relationship between specific sediment versus catchment area, as shown in Figure V-3-6. From Figure V-3-6, the erodibility can be interpolated at about 400 m^3/km^2 in the Haro basin and about 900 m^3/km^2 in the Soan basin, within the cope of 10 - 10^3 km^2 catchment area.

D. Specific Sediment

Estimation of the specific sediment of the proposed reservoir are calculated by the following methods.

(1) Estimation by measuring water usage through the reservoir.
(2) Estimation from using the trap efficiency of existing reservoir.

The results of the calculation are shown in Table V-3-9.

Table V-3-9. Design Specific Sediment

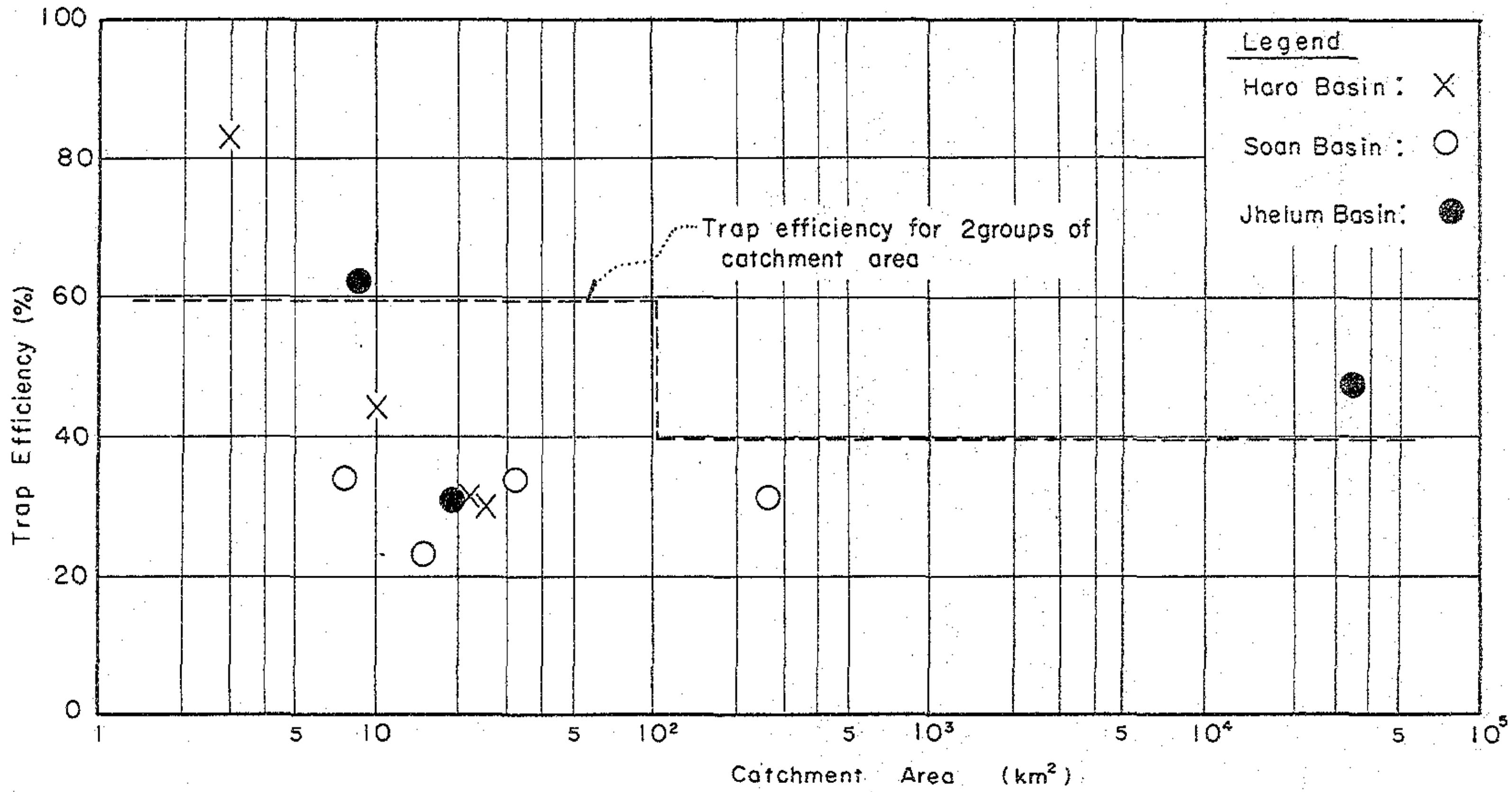
Damsite	Name of River	Catchment Area (km^2)	Specific Sediment m^3/km^2 Method (1)	Method (2)	Adopted m^3/km^2
D-1	Dor R.	292.3	760	464	500
H-4	Haro R.	498.5	532	326	400
N-1	"	462.0	467	330	"
S-1	Soan R.	341.1	1,854	1,716	900
L-1	"	285.0	1,030	799	"
SL-1	"	237.6	1,375	777	"
M-1	"	82.8	983	886	"

The Values of method (1) are generally seen to be larger than the values of method (2). For the decision of the specific sediment to be applied to the facility design in this study, it is desirable that the specific sediment should be determined based on the actual measurement records of the sedimentation in the reservoir and suspended sediments in order that the values will present the specific characteristics of the basins.

The specific sediment are adopted the results of method (2) for the above-mentioned reasons, however various experienced-based methods are of course valuable in their own way.

The locations of the gauging stations are far away from the proposed damsite, so that in this study we can identify the specific sediments as belonging to the same river. The value for the S-1 site is determined to be 900 m^3/km^2 in spite of its being about two times as large as those of other sites in the same river.

Figure V-3-6. Trap Efficiency of the Reservoir



CHAPTER VI. EVALUATION OF WATER RESOURCES DEVELOPMENT POTENTIAL

CHAPTER VI. EVALUATION OF WATER RESOURCES DEVELOPMENT POTENTIAL

6.1. Selection of Suitable Water Resources

6.1.1. Basic Concept of the Water Resources Development

A. Water Users of the Study Area

Direct and indirect water users of the study are classified as follows:

(1) <u>Water Users as Direct Targets of the Study</u>

- Islamabad Urban, Proper Area (Urban water)
- Rawalpindi Urban and Rural (ditto)
- Haro River Left Bank Commanded Area (irrigation)
- Soan River Right Bank Suburban Area (ditto)
- New International Airport (Urban water)

(2) <u>Indirect Water Users considered in the Study</u>

- CCA of Right Bank Canal of Khanpur Dam Project (Irrigation)
- CCA of Left Bank Canal of Khanpur Dam Project (ditto)
- Pakistan Industrial Development Corporation, Taxila (Urban)
- Pakistan Ordnance Factories, Wah (Urban)
- CCA of Left Bank Canal of Rawal Dam (Irrigation)
- CCA of Shahpur Dam Project (ditto)
- CCA of Jhablat Kas Lift Irrigation Project (ditto)

(3) <u>Other Users Concerned</u>

- Present and future users in the downstream of the proposed sides of water resources development (not particular)

- Present and future users, in the upstream of proposed points of water resources development, who should be considered in the water utilization plan in the study (Upper Kurang Irrigation Project).

B. Basic Concept of the Water Resources Development

Thus, so various direct or indirect water users are in the study area that rather complicated consideration is necessary to make appropriate water demand-supply balance in both the regional plan and time schedule.

Therefore, for its optimization, it is essential to establish a basic concept of the water resources development. The following are basic concepts of the water resources development in the study.

(1) <u>Point of Development Potential</u>

With the exception of points which have been already developed or designed, economical and steady water resources will be studied and selected from the whole study area. Those evaluations should be based on synthetic consideration of both natural and social conditions and, especially on the ratio of development cost to developed water production. The ratio is defined as "unit water cost" in the study.

(2) <u>Surface Water and Groundwater</u>

No previous precedence will be taken into consideration while deciding on surface water development and groundwater development. For final selection of them, above "unit water cost" will be referred to.

(3) <u>Conduction</u>

In principle, developed water resources will be connected to the nearest users. However, "unit water cost" for conveyance is prior to the actual distance between two points on selection of conduction line.

For such schemes as trans-basins, trans-provinces or involving complicate water allocation problems on water right, final adoption on these schemes (*1) will be left to the authorities concerned. In this study, only technical proposals will be submitted.

Note(*1): Schemes in such situations are the Dor Conduction (Transbasins) Scheme and the Upper Kurang River Development Scheme. (Refer to Chapter 7.4.3.)

(4) Form and Size of Development

On surface water development, the development method either by a diversion dam or by a storage dam will be decided considering runoff condition of the stream. In case of a storage dam, the location and dam type will be assumed first technically and the size-cost curve and water balance simulation study will be given to the dam. The dam size will be optimized on its study and finally the amount of water production by the dam will be set.

In case of other development forms, the size of a facility will correspond to the reasonable maximum potential of water resources computed on collected hydrological data.

(5) Water Resources Potential

It will be computed in trials of water balance with applied runoff data and projected water demand. The projected water demand consists of not only the principal water usage but also the reasonable release to downstream.

Even though there is no evident water right downstream, in general, certain amount of water release will be considered for uncertain use. It is named as "Minimum Regulated River Flow" in the study.

6.1.2. Major Water Resources of the Study Area

According to the basic concept of water resources development mentioned above, numbers of possible water resources have been pointed out and their locations are listed up with those of the users (beneficiaries) in Table D-4-4 of Appendix D.

Their schematic profiles by elevations are shown in Figure D-4-1 of Appendix D.

6.1.3. Selection of Proposed Facility Sites

A. Selection of Proposed Storage Damsites

The study area has been basically restricted to the Dor river, Haro and Soan river basins. In the first steps, suitable damsites were found using 1/50,000 topographic maps keeping in mind the runoff records in the river basin and operation records for the existing dams. In the second step, dam axises were considered based on geological and topographical reconnaissance in the field.

The following procedures and methodology were considered preliminary to the determination of suitable damsites and sizings.

i) Reservoir Size

The optimum storage capacity of respective dams has been studied on three or four sizes to find the optimum dam size in consideration of annual runoff at the damsite.

ii) Dambody and Volume of Dam

The shape of dams, including dam top elevation, slope and top width of dambodies, is determined using the design criteria described in 6.1.4.

The volume of a dambody is calculated roughly using the values for dam height, dam top length and river bed excavation width.

iii) Construction Costs

The total cost of a proposed dam can be divided into dam construction cost and compensation cost.

Dam construction costs are estimated using the specific unit cost of the dambody. In this preliminary study, specific unit costs per dam volume are assumed based on those of recently constructed dam projects in Pakistan.

iv) Water Costs Curve

At the first stage of the study, "construction costs/effective storage capacity curves" are worked out for the comparison of proposed damsites.

B. Selection of Diversion Damsites and Conduction Route

The selection of the proposed diversion dam sites and the conduction routing have been made in following the procedures in which one or two candidates are selected on the topo-maps in the scale at 1:50,000 as the base map, and the field investigation is carried out for the candidates to finalize the selection and routing through studying their economy, technological easiness for implementation and easiness in operation and maintenance of the facilities. And the following prerequisites to selection were taken into consideration.

a. The diversion dam site must be located as closely as possible to the service areas and secure the necessary and sufficient water head for the purpose. Furthermore, the river should have possibly narrow width at the intake point and have the stable flow center as close as possible to the bank proposed for the intake site.

b. The proposed conduction routes should be taken in the areas with topographical undulation as little as possible so that the expensive structures like aqueducts, syphons, tunnels can be decreased in number to provide or those structures can be shortened in their length even when constructed. In addition, the construction works and operation and maintenance of the facilities should be easy as much as possible.

C. Intake Tower

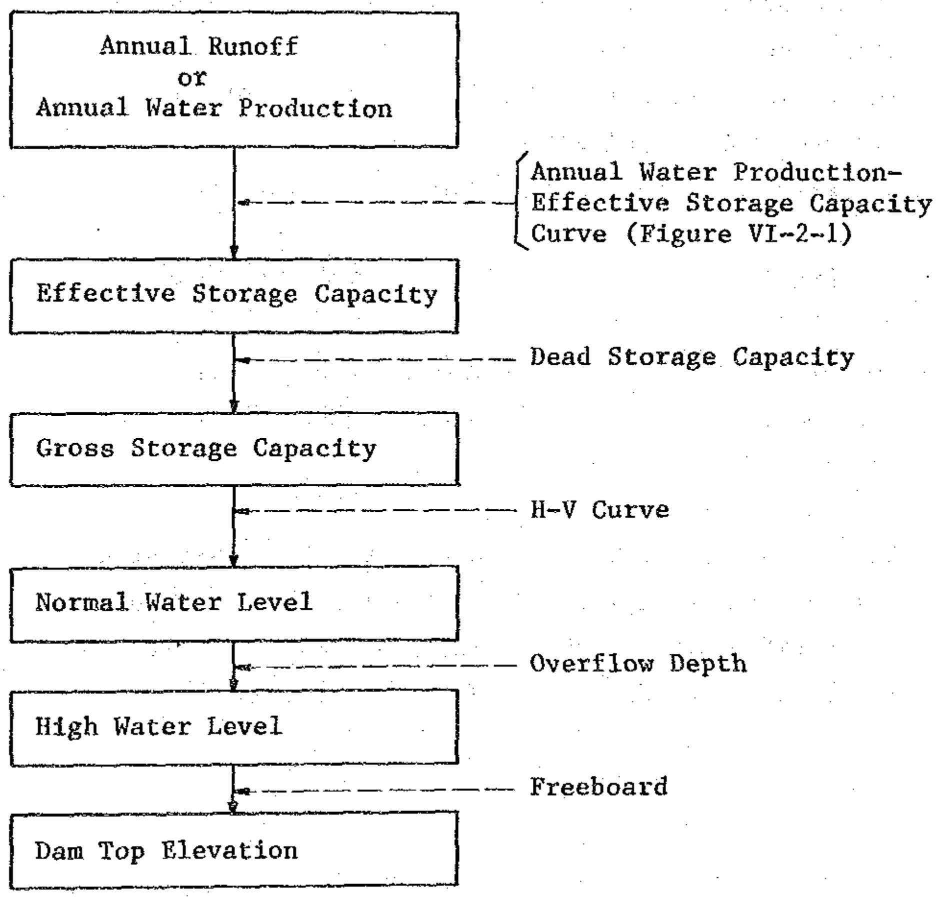
The existing intake facility available for water intake from the Khanpur reservoir is only that located at downstream of the saddle embankment at the right bank. Since, however, the capacity of the said intake facility is only the total of the demand of the both canals at the left and right banks, new intake tower will have to be constructed to meet the amount of water resources to be developed by H-4 and D-1 dams. And the new site deemed most suitable is proposed only at the inlet located about 500 m east of the saddle embankment on the left bank.

6.1.4. Design Criteria of Facilities

A. Storage Dam

a. Determination of Dam Top Elevation

The following procedures and methodology are applied to determine reservoir water levels and dam top elevations.



b. Dambody

i) Type of Dam

The type of dam is selected according to the topographical, geologic and hydrological conditions of the damsites. From the topographical point of view, the shape of the valley is the most important factor in the selection.

On the other hand, geological conditions such as the thickness of river deposits and weathered rock foundation, the strength and permeability of the bedrock, the distribution of construction materials are also very important factors in the selection.

ii) Freeboard

In determining the dam top elevation a certain freeboard is necessary above the High Water Level of the reservoir for safety purposes.

In this preliminary study, approximately 4 percent of the dam height is adopted for the freeboard of a dam which differs 2.0 to 3.0 m for a concrete dam and 2.0 to 4.0 m for an embankment dam respectively.

iii) Dam Slopes and Top Widths

The slopes of dambodies are one of the most important factors influencing not only safety of dams but also volume of dambodies. The dam shape should be determined after thoroughly studying the topographical conditions of the damsites, the nature of the bed rock, construction materials to be used and the type of spillway to be adopted.

The slopes of dambodies are tentatively determined according to the following criteria.

Type of Dam	Slope (Horizontal Ratio) Upstream	Downstream
Concrete Gravity Dam	1:0.1	1:0.8
Embankment Dam	1:3.0	1:2.5

Top width of dambodies is adopted 8.0 to 20.0 m depending upon the height of dam. The width of a concrete dam is 4.0 m constantly.

c. Spillway

i) Design Flood Discharge

Specific runoff of peak flood discharges can be studied using such empirical formula as Creager's Formula, Logarithmic Formula, Ingli's Formula, Dicken's Formula, Rational Method, and so on.

In this study, Creager's Formula (C=75) is adopted in consideration of the fact that Creager's Formula (c=75) is close to the maximum recorded value of peak flood discharge. The design flood discharge of the Khanpur dam (Q=130,000 cusec) is very similar to that of Creager's Formula, (C=70).

ii) Overflow Water Depth of Spillway

In this study, the overflow water depth is calculated using the following simple equation, taking no account of reservoir surcharge effects which are usually considered in the design of spillways in Pakistan.

$$Q = C \times L \times H^{1.5}$$

Where,

- Q : Design flood discharge (cu.m/sec.)
- C : Coefficient of overflow = 2.1
- L : Length of overflow weir (m)
- H : Overflow water depth (m)

iii) Type of Spillway

The type of spillway studied preliminarily would be the ungated type. The major reasons for this are summarized as follows;

- Ungated spillways do not suffer from any operation and maintenance problems.
- The only demerit an ungated spillway would have is that it requires an additional dam height to keep overflow water depth. This is not so serious at this stage.

B. Diversion Dam

The proposed diversion dams will be designed for the site with 200 - 300 m river width and with intake capacity by 0.5 - 5.0 m^3/sec (20 - 180 cusec), and the foundation rocks of the structure are found either considerably deep or rather shallow. In view of construction cost economy, a floating type structure will be adopted on sufficiently bearable sand and gravel foundation provided that suitable foundation rocks are found deep from the river bed surface, while the fixed type with concrete structures directly constructed on foundation rocks shall be adopted if sufficiently bearable foundation rocks are found shallow. The designed flood discharge is estimated by Creager's Formula. A coefficient of 50 shall be applied as that for the Punjab province according to the case of the Small Dam Project.

Since the weir type with flood gates on the crest is expensive in construction coat, the weirs are designed with a table concrete body providing the flushing gate only.

C. Conduction Main

A detail field investigation should be carried out after deliberate consideration of conduction main route on the topographic map on a scale of 1 to 50,000. To propose an alignment of the conduction main, the following factors should be fully considered.

- Traditionally open canal systems have been mostly used for irrigation networks, while for conduction mains of urban water supply, closed systems in combined system with pipeline, tunnels and syphons have been used. In case of open channels involved some parts in conduction mains, sandy flows during heavy rain would cause sediment deposits on the sills of structures to result in serious difficulty in operation and maintenance of the facilities.

- A conduction main should be so economically designed as to minimize its total length and construction cost.

- The design should be made so as to simplify construction works.

- The center line of canal/pipeline alignments should be provided along the topographical contour lines for minimizing the construction cost and facilitating operation and maintenance works in avoiding heavy cutting works and high embankment works as much as possible.

- Tunnel should be as straight as possible.

- To minimize construction period the vertical or inclined shafts are proposed for tunneling 4 km or more.

As a basic conception of water head allocation, it is required that an overall construction cost should be minimized by means of allocating more water head available to such structures that require much construction costs (steep) and allotting less head to the structures that require low costs (gentle). As a consequence, design velocity for each structure is determined as acceptable from the viewpoint of operation and maintenance within the allowable maximum velocity.

D. Water Supply to the New International Airport

As groundwater exploitation by tubewells is considered the most economical for water supply to the new international airport in the amount of 6,800 m^3/day (1.5 MGD).

In view of the existing results available in a range from 0.06 m^3/s to 0.10 m^3/s by tubewell pump-up in the area, the tubewells for the new air port should be increased in number according to the development stages of the air port facilities, and consequently, at the very beginning, four tubewells should be provided for the purpose and then increased to six by 2030 to meet the increasing demand.

6.1.5. Selected Damsites

The preliminary study was carried out for twenty-five proposed damsites in the three river basins. Figure VI-1-1 shows the relation between raw water costs and effective storage capacities for each damsite.

Selection of optimal dams would be confirmed after future study in which water demand patterns, relations to existing storage dams and intake facilities (diversion dams and pumping stations), and the operation plans of a series of dams in the same river basin were investigated.

The results of the study mentioned above give seven suitable damsites, namely D-1, H-4, N-1, S-1, M-1, L-1 and SL-1. These will be selected tentatively for study work and field surveys.

Proposed Damsites

River Basin	1st Stage	2nd Stage
Dor River	8 damsites (D-1..D-8)	1 damsite (D-1)
Haro River	9 damsites (H-1..H-8, N-1)	2 damsites (H-4, N-1)
Soan River	8 damsites (S-1..S-3, M-1) (L-1..L-3, SL-1)	4 damsites (S-1, M-1) (L-1, SL-1)
Total	25 damsites	7 damsites

On the second phase of the field survey and study, two new damsites were proposed as alternatives to N-1 and M-1.

i) Suspension of N-1 Dam Scheme

After careful survey N-1 Dam on the Nandna Kas proved to have serious demerits. These included:

- Submergence of a part of the Shahpur Dam and CCA of the project.

- Removal of 500 KV extra high voltage transmission line.

- High dam construction costs.

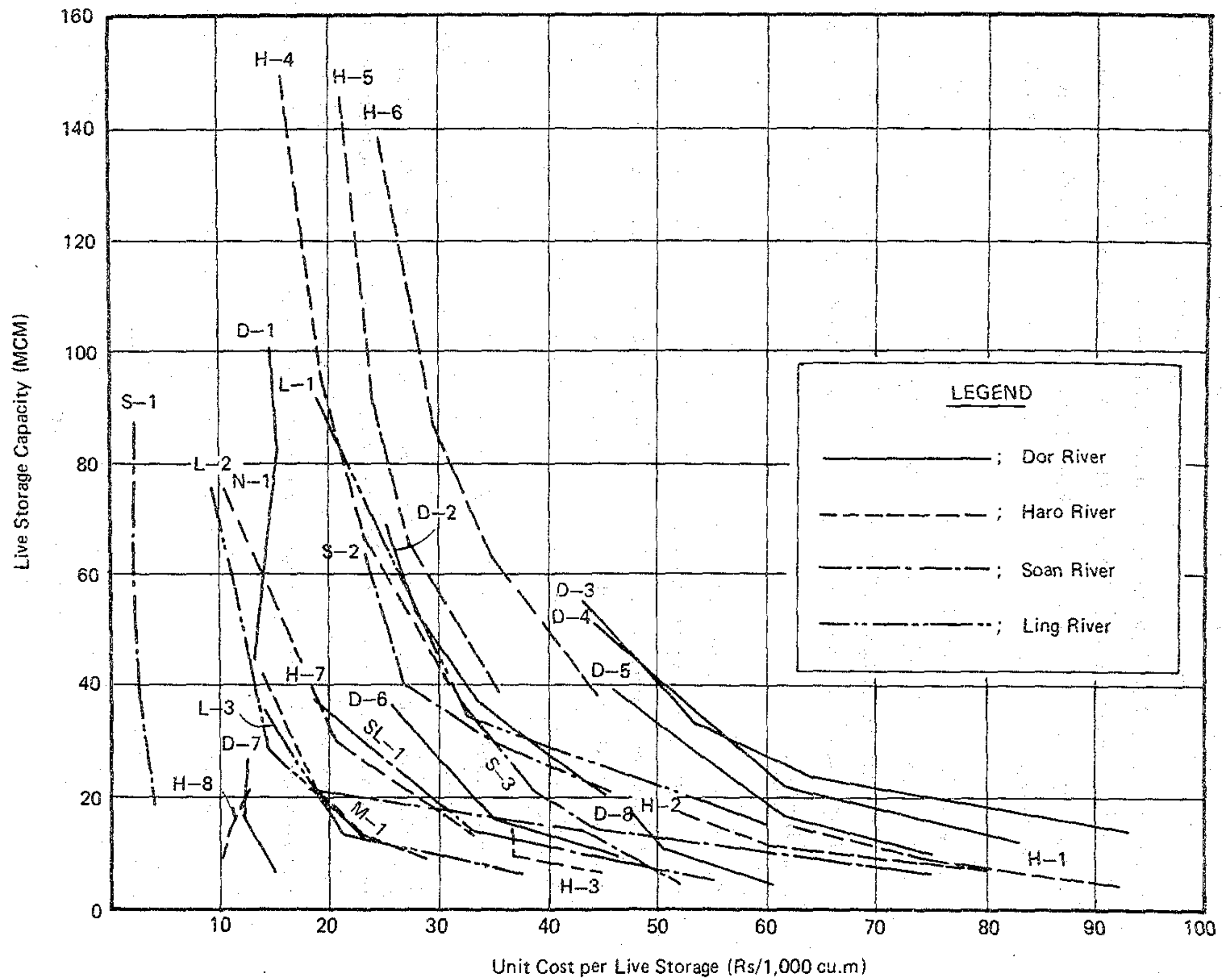
In consequence, as an alternative to the development of Nandna Kas, the heightening of the Shahpur Dam has been studied and proposed.

ii) Removal of M-1 Damsite

Among these proposed damsites, site M-1 on the Malal Kas does not have favourable topographical and geological conditions, which makes it costly. An alternative damsite was searched for along the Malal Kas and it was finally found at the former gauging station of Lohi Bher, near Panwal village on the Kurang river.

The site has been named KL-1 (Kurang Lower Site No.1)

Figure VI-1-1. Unit Cost and Storage Capacity



6.2. Surface Water

6.2.1. Basic Criteria for the Evaluation of Development Potential

For evaluation of the development potential of surface water, water balance has been studied on various scales and sizes of the reservoir capacities and the water demands. For this study, the following basic criteria have been premised in the water balance simulation.

A. Runoff

Runoff is computed based on the result of runoff analysis, and 10 day's basis runoff for 21 years from 1960 to 1980 is used for the simulation.

B. Lake Evaporation Loss

Lake evaporation has been estimated at 80 percent of potential evapotranspiration at Chaklala, and water loss by lake evaporation is balanced with precipitation at each development site.

C. Sediment

Sediment at each water resources development site is estimated on the basis of specific sediment and project life of dam (100 years). Only the direct sediment discharge, the catchment area is considered if other dams are located in upstream.

Specific sediment in the river basins is estimated as follows.

Dor river basin	500 $m^3/km^2/year$
Haro river basin	400 "
Soan river basin	900 "

D. Seepage Loss

Seepage loss is composed of losses from a reservoir bed and through a dam body. This study, however, does not count such seepage loss in water balance, provided that most of seepage loss appears in downstream and feeds minimum regulated river flow to be released from the development sites.

E. Minimum Regulated River Flow

Minimum regulated river flow (MRRF) is released from a dam or a diversion dam to ensure 355 days flow (droughty flow) for downstream water utilization or for maintaining rivers in healthy condition at the downstream fixed gauging station.

F. H-A, H-V Curve

H-A, H-V curves of proposed damsites are prepared by survey or using topographical maps of 1:50,000 scale.

G. Initial Storage Capacity

Simulation of water balance has been made for the period from January 1960 to December 1980. The average storage capacity in the end of December from 1960 to 1980 is considered to be the initial storage capacity in January 1960. Initial storage capacity is at about 70 percent of effective storage capacity in case that effective storage capacity is small as compared to annual runoff.

H. Water Demand

In simulation of water balance, water demand is represented by urban water and irrigation water. Urban water has less fluctuation in monthly water demand ranging from the lowest at 0.80 in January to the highest at 1.25 in June.

Monthly Fluctuation of Urban Water Demand

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Average
0.80	0.85	0.90	1.00	1.15	1.25	1.05	1.00	1.05	1.05	1.00	0.90	1.00

On the other hand, irrigation water has large fluctuation of water demand comparing with urban water. Annual average irrigation requirement is 6.158 MCM per 1,000 ha with cropping intensity of 140 percent.

Monthly Fluctuation of Irrigation Water Requirement

(MCM/1,000 ha/yr.)

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.
0.349	0.401	0.579	0.611	0.757	0.961	0.456	0.285	0.474	0.478

Nov.	Dec.	Total
0.477	0.330	6.158

I. Safe Water Utilization Ratio

Safe water utilization ratio should be designed to allow water shortage to occur once in 10 years. In the simulation of water balance, therefore, water shortage is allowed to occur twice in 21 years. Water shortages less than 5 percent of water demand are not considered as water shortage, because such minor water shortages are to be tolerated.

6.2.2. Development Potential of the Dor River Basin

A. General

The Dor river is not yet fully developed, however some of its flow has been utilized by farmers for irrigation below Havelian. The irrigation area is estimated at 3,800 ha in this river basin. Therefore, many water resources remain for further water use in this basin and it can be conducted to the metropolitan area through the Khanpur reservoir.

On above considerations, the D-1 dam is proposed at Rajoia in the upstream and the Dw-1 diversion dam in the downstream.

B. Water Balance Simulation

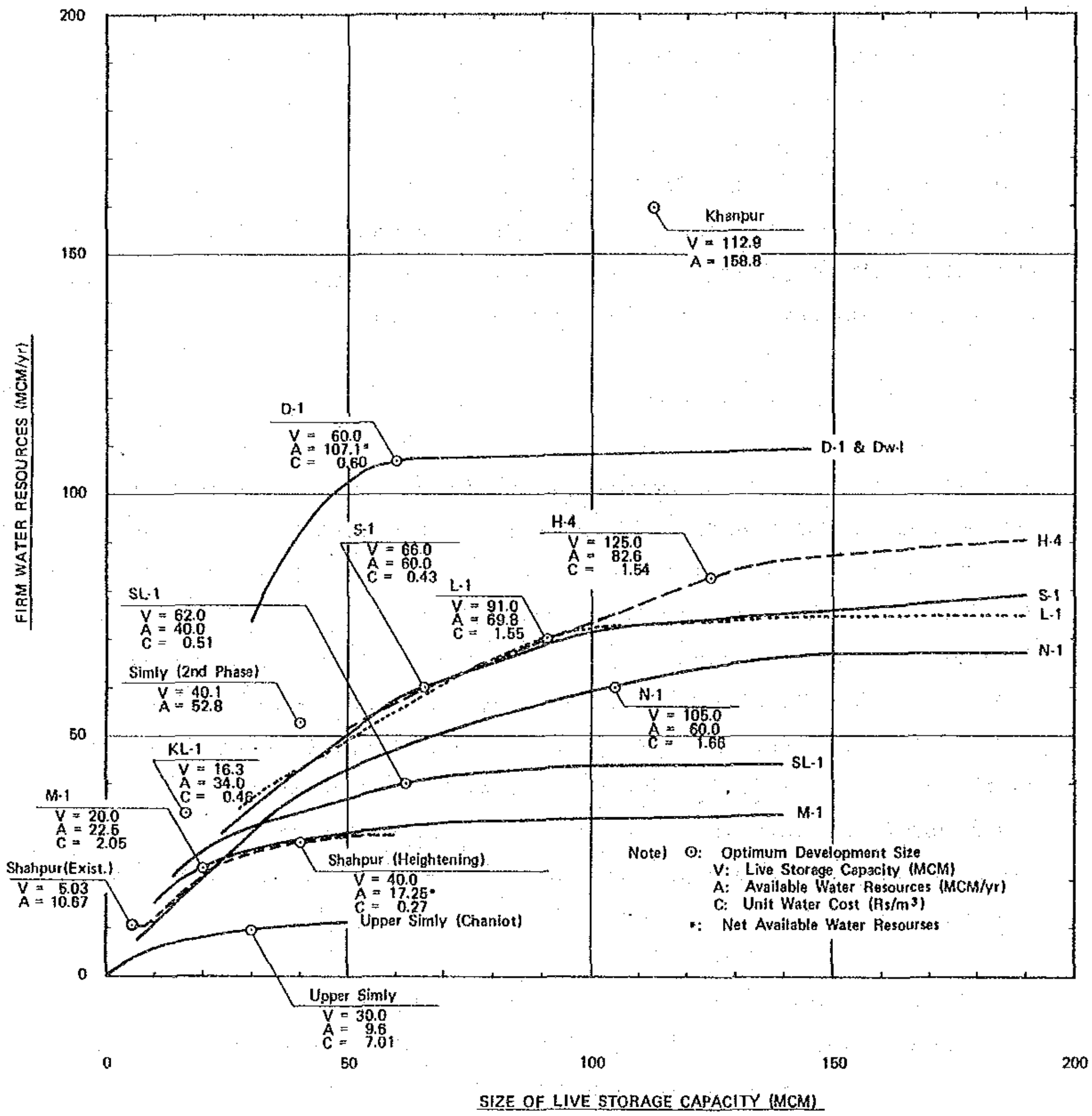
Water balance has been simulated on various sizes of the D-1 dam to estimate the available water resources in the Dor river basin. The Figure VI-2-1 shows the available water resources at the Dw-1 diversion damsite. Conduction capacity from the Dw-1 to the Khanpur reservoir is set at 5 m^3/sec by the study on optimizing the conduction capacity. However it will be necessary to re-examine the conduction capacity in the feasibility study.

C. Available Water Resources

From the results of water balance simulation in the Dor river basin shown in Figure VI-2-1, water resources will be developed up to 109 MCM annually out of a total runoff of 155 MCM at the Dw-1 diversion damsite when large storage capacity is given to the D-1. However, water resources will not increase so efficiently beyond 100 MCM/year. The optimum development size of the D-1 dam has been studied from the economic viewpoint for unit cost of water developed by the D-1 and the Dw-1. And the optimum water resources development has been finalized at 107.1 MCM in a year on average with the live storage capacity of 60 MCM (dam height H = 85 m) for the D-1 dam as shown in Figure VII-3-1.

Besides, some amount of water can be expected from extra water resources. This extra water will be borne of flood water from the residual basin below the D-1 damsite, and it is estimated at 7.8 MCM/year in a year on average when conduction capacity is set at 5 m^3/sec. However, for utilizing this extra water effectively, it is

Figure VI-2-1. Relation Between the Firm Water Resources and the Size of Live Storage Capacity



necessary to control it in the Khanpur reservoir without spilling. This fact has been examined in a simulation combining the Dor river basin and the Haro river basin. However, none of the extra water can be controlled and utilized due to less capacity of the Khanpur reservoir, so most of the extra water will spill from the Khanpur dam.

Table VI-2-1. Available Water Resources in the Dor River Basin

Site	D.A.	Average Annual Runoff	Live Storage Cap.	Annual Firm Water Resources	Unit Water Cost
	(km^2)	(MCM)	(MCM)	(MCM)	(Rs/m^3)
D-1	292.3	96.2	60.0		
Dw-1	517.7	154.7	0	107.1	0.60
Total	517.7	154.7	60.0	107.1	0.60

6.2.3. Development Potential of the Haro River Basin

A. General

The Haro river and its tributaries such as the Jhablat Kas have been utilized for agriculture in a traditional way since ancient time because of relatively steady flow through a year. The large scale development of water resources in this basin, however, has been started recently by constructing the Khanpur dam on the Haro river and the Shahpur dam on the Nandna Kas. On the other hand, the Jhablat Kas is utilized by many agencies for domestic water and agriculture water pumping from the outflows of the springs or from the Jhablat Kas. In the present utilization of surface water including the springs is estimated in total at 31 MCM/year or equivalent to 0.98 cu.m/sec (see Table D-4-3, Appendix D) in the Jhablat Kas basin.

However, the present water resources development does not utilize the whole potential of the Haro river basin. To evaluate the remaining potential, the following four possible sites have been studied in this basin.

H-4 dam: proposed upstream the Khanpur dam on the Haro river to control the spill of the Khanpur dam.

N-1 dam: proposed downstream the Shahpur dam on the Nandna Kas, but canceled due to submergence of the irrigation area of the Shahpur dam.

Shahpur dam heightening: proposed instead of the N-1 dam to fully utilize the runoff of the Nandna Kas.

Jw-1 diversion dam: proposed at the lower reaches of the Jhablat Kas.

Besides the above potential sites, the Khanpur reservoir will be linked with the Dor river to introduce water from the Dor river after completion of the D-1 dam and the Dw-1 diversion dam.

B. Water Balance Simulation

Water balance simulation has been made in the following manner to estimate the available water resources in the Haro river basin.

a. Haro River Main Stream Development

Case 1(H) ... Khanpur dam only

This case assumes examining the firm water resources developed by the Khanpur dam under prospective safe water utilization to allow water shortage once in ten years.

The MRRF is also given to the Khanpur dam in the same way as in other cases below. The MRRF is estimated at 49.5 MCM/yr equivalent to 1.57 cu.m/sec. In this simulation, the firm water resources by the Khanpur dam is estimated at 158.8 MCM/yr out of the total runoff of 327 MCM/yr.

Case 2 (H) ... Khanpur dam + H-4 dam

In this case, the H-4 dam is proposed in the upstream of the Khanpur dam to minimize the spill of the Khanpur dam. And available water resources have been examined on various sizes of the H-4 dam. The water resources can be developed upto around 90 MCM/year by H-4 dam in this case. However, the annual water resources will not increase so efficiently beyond 85 MCM as shown in Figure VI-2-1. From the study of unit water cost, the optimum size of the H-4 dam has been set at 125 MCM of live storage (H = 117.5 m) as shown in Figure VII-3-1. In case of this size, the total available water resources will be 82.6 MCM in a year.

Case 3 (H) ... Khanpur dam + H-4 dam + Dor Link

In this simulation, the Dor river has been linked with the Khanpur reservoir by the conduction from the Dw-1 diversion dam. A possibility of extra utilization of water from the Dor river has been examined in this simulation, but no extra water can be utilized because of the small capacity of the Khanpur reservoir, as explained in 6.2.2.

b. Nandna Kas Development

Case 1 (N) ... N-1 dam

The Nandna Kas is one of major left bank tributaries of the Haro river, and flows beside the Haro River Left

Bank Command Area. The N-1 dam is proposed on the Nandna Kas below the Shahpur dam, which was completed construction recently. Though the direct catchment area of the N-1 dam is reduced to 258.1 km^2 from 462 km^2 because of the Shahpur dam, the N-1 dam has still large catchment area enough for development and some of runoff to the Shahpur dam can be expected as spill from the Shahpur dam. However the N-1 dam has been canceled due to submergence of the irrigation area of the Shahpur dam.

The N-1 dam has been simulated under the condition of spill from the Shahpur dam. Spill from the Shahpur dam has been simulated, and it has been assigned to the N-1 dam as inflow combining with direct runoff into the N-1 dam. The MRRF has been given to the N-1 dam by 5.05 MCM/yr equivalent to 0.160 m^3/sec.

In simulation of the Shahpur dam, conditions of water demand and capacity of the Shahpur dam have been taken from the PCI report on the Shahpur dam. They can be summarized as follows:

- Water Demand 10.67 MCM/year

Irrigation Water	7.35 MCM/year
	(5,959 Aft/year for 4,308 acres)
Domestic Water	3.32 MCM/year (2.0 MGD)

- Storage Capacity

Full Water Capacity	17.66 MCM
	(14,320 Aft at 1,458.25 ft)
Dead Water Capacity	12.63 MCM
	(10,241 Aft at 1,452.5 ft)
Effective Capacity	5.03 MCM

Mean annual inflow is estimated at 20.6 MCM (16,700 Aft) at the Shahpur damsite in the PCI report, however it seems too small in our study. Therefore, mean annual inflow is revised to 43.0 MCM averaged from 1960 to 1980 in this study and simulation of the Shahpur dam has been made using the revised inflow. According to the result of Shahpur dam simulation, water shortage will occur five times in 21 years, that is once in four years.

Under the above conditions, water balance simulation of the N-1 dam has been made and its result is presented in Figure VI-2-1. As seen in the figure, the N-1 dam will allow developing the available water resources ultimately up to about 67 MCM/yr covering the irrigation area of 10,900 ha. However, this amount of water resources can not be expected because of cancellation of the N-1 dam.

Case 2 (N) ... Shahpur Dam Hightening

Instead of the N-1 dam, heightening of the Shahpur dam has been considered to fully utilize water at the Shahpur dam site. In this case, modification of the Shahpur dam is considered as follows;

Shahpur Dam Modification

(1) Lowering the DWL from 442.72 m (1,452.5 ft) to 440.44 m.
Dead Water Capacity = 7.18 MCM

(2) Raising the FWL from 444.47 m (1,458.25 ft) to 449.58 m.
Full water capacity = 47.26 MCM

(3) Effective Storage Capacity increased from 5.03 MCM to 40.08 MCM.

(4) MRRF is newly given to the Shahpur dam by 2.24 MCM/yr equivalent to 0.071 cu.m/sec.

Under the above conditions, available amount of water resources has been estimated at 27.92 MCM/yr by heightening the Shahpur dam. However, net available amount is only 17.25 MCM, because prospective water demand to the existing Shahpur is 10.67 MCM/yr out of 27.92 MCM/yr.

c. Jhablat Kas Development

The Jw-1 diversion dam has been proposed at the lower reaches of the Jhablat Kas for producing the irrigation water for the Haro River Left Bank Command Area. Water balance simulation of the Jw-1 has been made using the runoff data only limited to a short period from September 1961 to July 1965 because of difficulty in extending the runoff data in hydrological analysis. However, in this period, the considerable droughty year 1964 in the alluvial plain is included, accordingly this simulation will be acceptable to estimate the available amount of water resources in the Jhablat Kas. The droughty year is estimated in the year 1964 according to the records at Chaklala, which say that the droughty year occurs once in 20 years.

In simulation of the Jw-1 diversion dam, the existing water right has been estimated at 2.0 cu.m/sec including the MRRF at the Jw-1 sites.

POF (Operation Capacity)	1.00 cu.m/sec	
Jhablat Kas Lift Irrigation	0.28 "	
MRRF and Other Water Use	0.72 "	(22.7 MCM/yr)
Total Water Right	2.00 cu.m/sec	

As a result of simulation, lifting capacity of pumps is set at 2.7 cu.m/sec at the Jw-1 site including regulating ponds in the irrigation area whose total capacity is 8.0 MCM. The Jw-1 diversion dam will be able to produce irrigation water of 70.8 MCM per year which will cover about 10,000 ha even in the droughty year.

C. Available Water Resources

In the Haro river basin, total net water resources will reach 170.65 MCM per year by the development.

Table VI-2-2. Available Water Resources in the Haro River Basin

Site	Prospective Water Resources					Annual Available Water Resources	
	D.A.	Ave. Annual Runoff	Live Storage Cap.	Annual Firm Water Resources	Unit Water Cost	Developed by Exist. Facil.	Net Production
	(km^2)	(MCM)	(MCM)	(MCM)	(Rs/m^3)	(MCM)	(MCM)
H-4	498.5	209.5	125.0	82.6	1.54	-	82.6
Khanpur	778.0	327.0	112.9	158.8	-	158.8	-
Shahpur	203.9	42.5	5.03	10.67	-	10.67	-
Heightening	203.9	42.5	40.0	17.25	0.27	-	17.25
N-1	462.0	84.3	105	60.0	1.66	-	-
Jw-1	248.6	170.4	0	70.8	0.77*	-	70.8
Total						169.47	170.65

Note: * cost including the regulating Ponds. (see Table VII-3-6)

6.2.4. Development Potential of the Soan River Basin

A. General

The Soan river is composed of three major rivers in its upstream, one is the main stream of the Soan river and the others are the Kurang river and Ling river which are tributaries of the Soan river. Of these three rivers, the Soan and the Kurang have been developed by the Simly dam and the Rawal dam respectively. On the other hand, the Ling river remains without any development of water resources. However, the Soan and the Kurang have not yet reached the ultimate development on water resources by said two dams. Furthermore, other tributaries such as the Sil river still remains without development in the alluvial plain.

According to the above conditions, six dams and one diversion dam have been considered in this basin.

Upper Simly dam: proposed at Chaniot on the Soan river upstream the Simly dam to minimize spill of the Simly dam and to increase the availability of water resources in the uppermost of the basin. However this dam has been canceled because of extremely high water cost as compared to that of the S-1 dam (see Table D-3-4, Appendix D). The S-1 dam will develop the water resources including these by the Upper Simly dam with cheaper water cost.

S-1 dam: proposed on the Soan river downstream the Simly dam to develop the water resources in the residual area below the Simly dam and its spill.

M-1 dam: proposed on the Malal Kas which is a lowest tributary of the Kurang river in elevation, but canceled because of higher water cost as compared to the KL-1 dam which is proposed below the M-1 dam and the dam construction is not feasible technically.

KL-1 dam: proposed the most downstream the Kurang river to develop the residual area below the Rawal dam.

L-1 dam: proposed in the Ling river

SL-1 dam: proposed in the Sil river

Sw-1 diversion dam: proposed just upstream the GT road bridge in the Soan river, but canceled for the following reasons clarified by the hydrological study;

- The Soan river is relatively unsteady in its flow because of a diversion dam.

- The base flow at the Sw-1 site is almost fed by the Kurang river. This base flow will be developed in the Kurang river basin in the future.

B. Water Balance Simulation

In the Soan river basin, water balance simulation has been made individually on each development site, and the results are shown in Figure V-2-1. The simulation was done on the following conditions:

Conditions on Simulation

Development Site	Related Existing Structure	MRRF MCM/yr (cu.m/sec)	D.A. (km^2)	Remarks
Upper Simly dam	Simly Dam	-	126.4	MRRF is not considered for this dam.
Simly dam	-	-	152.8	Simulated under Second Phase Development.
S-1 dam	Simly Dam	11.42 (0.362)*1	341.1	
M-1 dam	-	2.78 (0.088)*1	82.8	
KL-1 dam	Rawal Dam	6.00 (0.190)*2	283.7	Considering only the residual area below Rawal dam.
L-1 dam	-	9.52 (0.302)*1	285.0	
SL-1 dam	-	2.60 (0.082)	237.6	

Note: *1: To feed 1.78 cu.m/sec of flow at the Rawalpindi Station in accordance with the drainage area.
*2: This flow will be utilized by development of the Kurang river below Rawal dam.

In connection with simulation of the S-1 dam and the Upper Simly dam, the Simly dam has been also examined for its water resources availability and spill from the Simly dam, because spill from the Simly dam will flow into the S-1 reservoir. Water balance of the Simly dam has been examined in its second phase development which is under execution to increase water supply by raising storage water level. In second phase, it is proposed to increase water supply to 49 MGD, however, it is known from the simulation that only

65 percent of 49 MGD (equivalent to 31.85 MGD = 52.8 MCM/yr) can be developed with safe water utilization ratio once in 10 years in the second phase. The simulation of the S-1 dam has been made under water supply of 31.85 MGD by the Simly dam.

The Upper Simly dam has been analyzed in combined operation with the Simly dam. The effectiveness of the Upper Simly dam is to minimize the spill of the Simly dam, accordingly the available amount of water resources of the Upper Simly dam is only the amount increased from 52.8 MCM/yr which is already developed by the Simly dam.

C. Available Water Resources

Table VI-2-3 shows the optimum development scale at each prospective site from a economic point of view taking unit water cost into consideration. The Upper Simly dam will produce the most expensive water and the M-1 will follow it in this basin. The water resources produced by these two dams can be also developed with lower cost by other dams such as the S-1 dam and the KL-1 dam in the lower reaches. Therefore, it is recommended to develop four dams, namely S-1, KL-1, L-1 and SL-1, and which would produce 203.8 MCM of water per year in this basin.

Table VI-2-3. Available Water Resources in the Soan River

Site	Prospective Water Resources					Annual Available Water Resrouces	
	D.A.	Ave. Annual Runoff	Live Storage Cap.	Annual Firm Water Resources	Unit Water Cost	Developed by Exist. Facil.	Net Production
	(km^2)	(MCM)	(MCM)	(MCM)	(Rs/m^3)	(MCM)	(MCM)
Upper Simly	126.4	69.0	30.0	9.6	7.01	-	-
Simly dam	152.8	83.4	40.09	52.8	-	52.8	-
S-1 dam	341.1	130.9	66.0	60.0	0.43	-	60.0
M-1 dam	82.8	38.0	20.0	22.5	2.05	-	-
KL-1 dam	283.7	102.7	16.3	34.0	0.46	-	34.0
L-1 dam	285.0	107.1	91.0	69.8	1.55	-	69.8
SL-1 dam	237.6	50.1	62.0	40.0	0.51	-	40.0
Total						52.8	203.8

6.2.5. Comprehensive Evaluation of Development Potential

Simulation on available water resources under respective safe water utilization ratio at the prospective development sites is summarized in Figure VI-2-1. The figure shows the relation of the live storage capacity to the available water resources.

In the figure, by setting the live storage capacity on abscissa and water resources on ordinate, the relation between available water resources and live storage is indicated and safe water utilization is set in the year when shortage occurs twice in 21 years from 1960 to 1980. Therefore this curve indicates the firm water resources under the safe water utilization ratio. Within the range where the curve has steep gradient, available water resources effectively increase in the size of storage capacity of dam, however, they will will not increase as much as storage capacity increases in the range with a gentle gradient. The marked point on the curve means a point of the lowest unit water cost of developed water.

Table VI-2-4 shows the potential development selected from the economic viewpoint. And, the table reveals the following facts.

(1) Total live storage capacity of dams (including Khanpur dam) will be 573 MCM, and, in this case, average utility ratio will be 1.89 to the total inflow 1,085 MCM.

(2) Total net available water resources are prospected at 482 MCM, and the development ratio at 0.444 to the total inflow 1,085 MCM.

(3) Total MRRF will be 133 MCM, which will be 12.3 percent of the total inflow 1,085 MCM.

(4) The total of the net available water resources development and the MRRF will be 614 MCM, that is 56.6 percent of the total inflow 1,085 MCM.

Table VI-2-4. Potential Water Resources in the Study Area

Development Site	D.A. (km^2)	(1) Ave. Annual Runoff (MCM/yr)	(2) Live Storage (MCM)	Utility Ratio (1)/(2)	(3) Potential Water Resources (MCM/yr)	(4) Water Resources developed by Existing Facil. (MCM/yr)	(5) Net Water Resources Development (3) - (4) (MCM/yr)	Development Ratio (5)/(1)	Evp. Loss (MCM/yr)	M.R.R.F. (MCM/yr)	Spill (MCM/yr)	Years in Shortage
Dor River												
D-1	292.3	96.2	60.0	1.60		0						
Dw-1	517.7	58.5	0	-		0				32.5	10.9	
Sub-Total	517.7	154.7	60.0	-	107.1	0	107.1	0.692	1.6	32.5	10.9	1966, 70
Haro River												
H-4	498.5	209.5	125.0	1.68		0						
Khanpur Dam	778.0	117.5	112.9	1.04		158.8				49.5	36.6	
Total	778.0	327.0	237.9	1.37	241.4	158.8	82.6	0.253	5.3	49.5	36.6	1970, 75
Dor-Haro Link (Conduction Capacity 5.0 cu.m/sec.)												
H-4		209.5	125.0	1.68		0						
Khanpur Dam		228.2	112.9	2.02		158.8				49.5		
Total		437.2	237.9	1.84	348.5	158.8	189.7	0.434	5.3	49.5	40.4	1970, 75
Shahpur Dam	203.9	42.5	40.0	1.06	27.92	10.67	17.25	0.406	6.6	2.2	5.9	1973, 75
Jw-1	248.6	170.4	0	-	101.80	31.0	70.8	0.415	-	22.7	45.9	
Sub-Total	1230.5	539.9	277.9	1.94	371.12	200.47	170.65	0.316	11.9	74.4	88.4	1970,73,75
Soan River												
S-1	341.1	130.9	66.0	1.98	60.0	0	60.0	0.458	4.5	11.4	55.0	1970, 75
L-1	285.0	107.1	91.0	1.18	69.8	0	69.8	0.652	5.5	9.5	22.3	1970, 75
KL-1	283.7	102.7	16.3	6.30	34.0	0	34.0	0.331	6.8	2.3	59.6	1962, 74
SL-1	237.6	50.1	62.0	0.81	40.0	0	40.0	0.798	4.4	2.6	3.1	1962, 75
Sub-Total	1147.4	390.8	235.3	1.66	203.8	0	203.8	0.521	21.2	25.8	140.0	1962,70,74,75
Total	2895.6	1085.4	573.2	1.89	682.02	200.47	481.55	0.444	34.7	132.7	239.3	1962,70,73,74,75

6.3. Groundwater

6.3.1. General

The Potwar Plateau in the eastern part of the study area was formed through the filling-up of the ancient mountains and basins with Quaternary deposits composed of unconsolidated clay, silt, sand and gravel layers.

The aquifer in the study area is mainly borne in the unconsolidated Quaternary deposits and the bedrock forms the unpervious groundwater containers (groundwater basins).

As it is seemed that the Quaternary deposits get coarser toward the foot of the mountains, high yield aquifers distribute mainly in the eastern part of Potwar Plateau.

Six main groundwater basins can be detected in the study area as shown in Figure VI-3-1. As shown in this figure, the groundwater basins are elongated in WSW-ENE direction controlled by the main trend of the geological structure. The outline of the groundwater basins is listed in Table VI-3-1.

The storage volume of groundwater is calculated based on the assumption that the effective porosity of the deposits is 10 percent and the depth of the groundwater basins is around 300 m.

Table VI-3-1. Features of the Groundwater Basins in the Study Area

Groundwater Basin	Depth to Groundwater Surface (m)	Capability of Aquifers	Degree of Groundwater Development	Assumed Volume of Groundwater ($\times 10^9$ m^3)
A	5 - 70	high to medium	medium	4
B	5 - 30	high to medium	medium	3
C	5 - 20	high to medium	high	2
D	10 - 30	low to medium	low	1
E	10 - 30	low	low	1
F	5 - 20	low to medium	low	3

Source: GE-1, GE-4

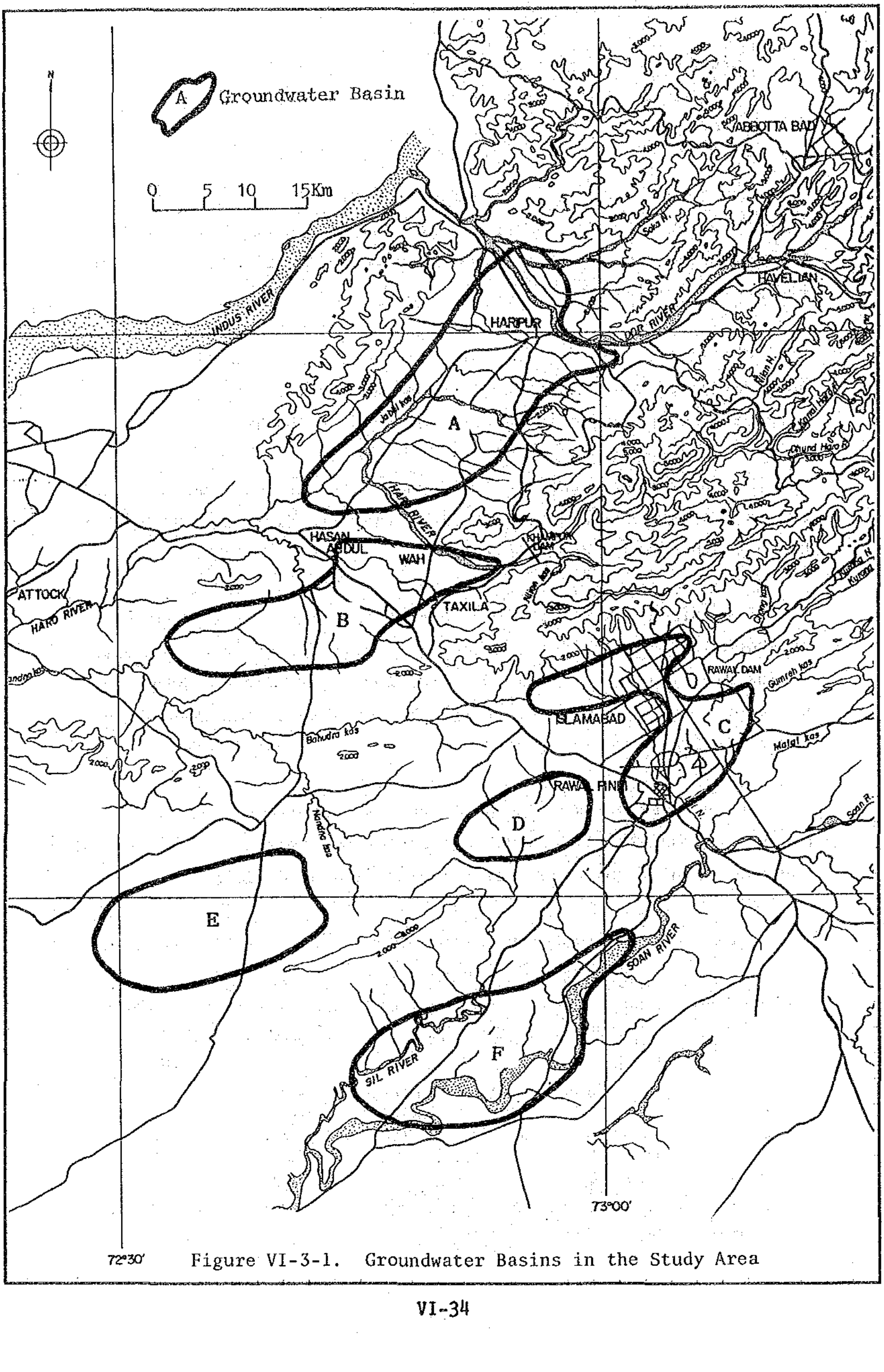


Figure VI-3-1. Groundwater Basins in the Study Area

Therefore, it can be concluded that the huge amount of groundwater resources are borne in the vicinity of the Study Area.

For the selection of the subject areas of groundwater development for the metropolitan area of Islamabad - Rawalpindi, the following conditions should be required;

- ° The existence of high yield aquifers is anticipated
- ° They have huge groundwater basins
- ° They are situated near by the metropolitan area

Consequently, the three groundwater basins of B, C and D are selected as the subject areas of groundwater development that they would meet the above conditions.

6.3.2. Subject Areas of Groundwater Development

A. Hydrogeological Structure

a. Islamabad and Rawalpindi Area (Groundwater Basins C and D)

The groundwater basins in the capital area have the characteristic of being able to be further subdivided into six groundwater basins. These are shown in Figure VI-3-2.

The highest yield aquifers are distributed in the National Park groundwater basin (C-II) and there are many artesian wells in this area.

The transmissivity of the aquifers is assumed to be 500 to 1,000 m^2/day in the National Park basin and 100 to 500 m^2/day in other basins.

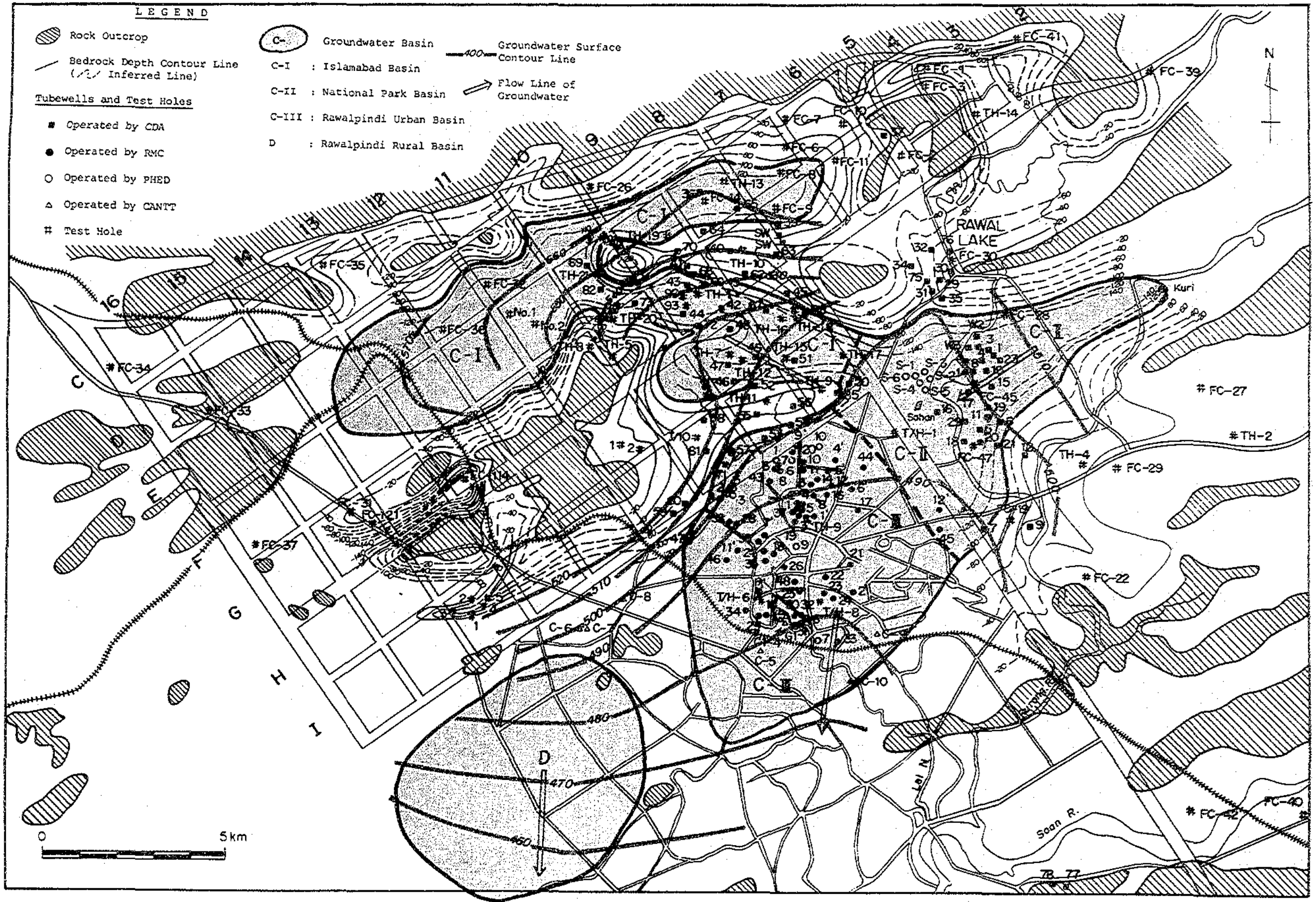


Figure VI-3-2. Hydrogeological Map of Islamabad and Rawalpindi Area

b. Wah and Taxila Area (Groundwater Basin B)

The groundwater basin in the Wah and Taxila area shows a belt shaped feature elongating in the ENE to WSW direction with a width of about 10 km.

As shown in Figure VI-3-3 the groundwater flows in northwest direction and the flow beyond the northern boundary of the groundwater basin is interrupted by impervious bedrock ridges.

Three groundwater channels can be inferred from the concaves in the groundwater surface contour lines, also from the existence of springs and the fact that their transmissivity reaches to several thousand cubic meters per day per meter. It is thought that such highly permeable zones might be ancient river courses in which coarser materials accumulated.

B. Groundwater Abstraction

a. Islamabad, Rawalpindi Area

More than 150 tubewells have been installed in this area. Groundwater development has been highly concentrated in the central to western part of the capital area, as shown in Figure VI-3-2.

The total annual discharge amount has reached 59 MCM by the year of 1987. Discharge amount of each groundwater basins is as follows;

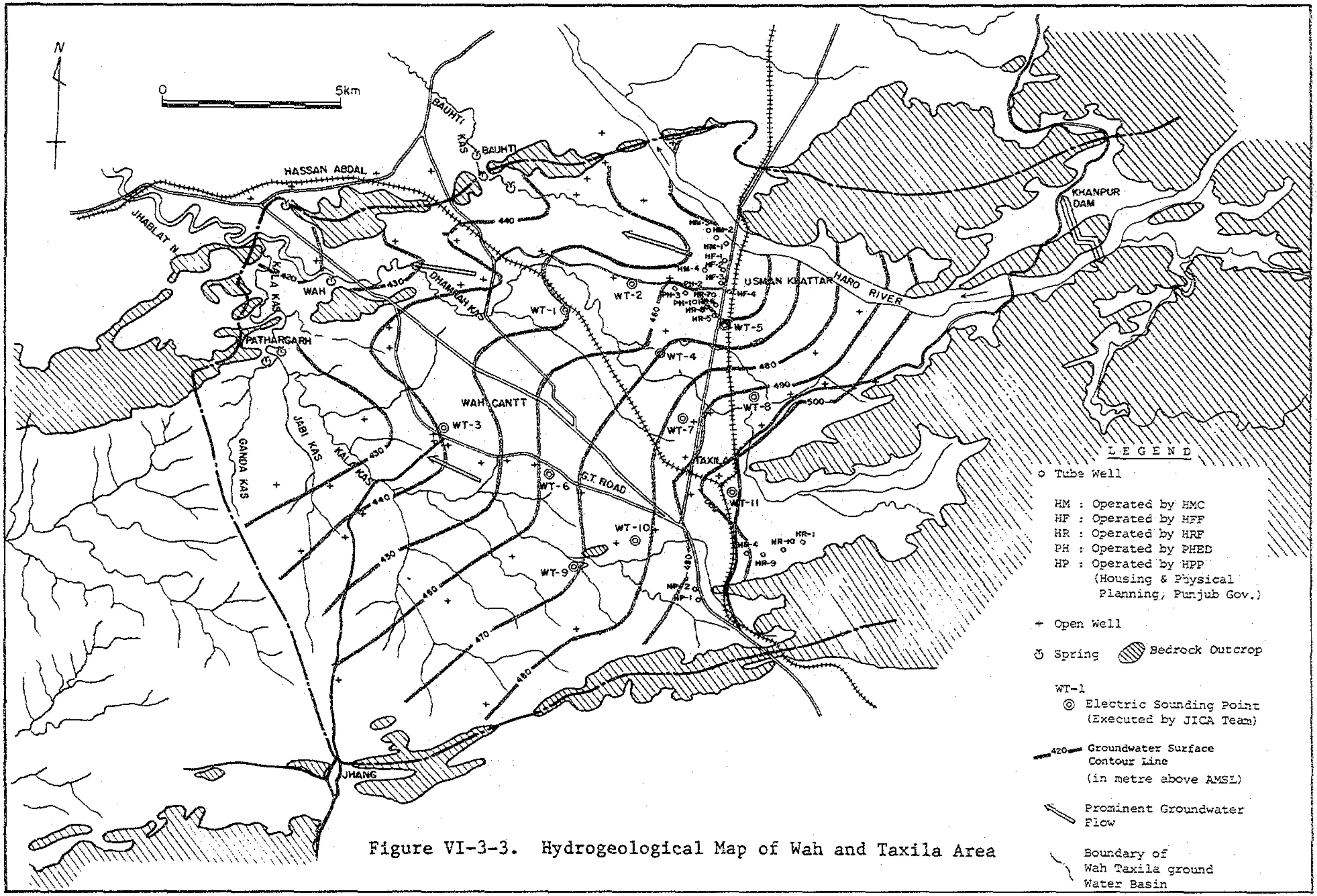


Figure VI-3-3. Hydrogeological Map of Wah and Taxila Area

Islamabad (C-I)	13 MCM/year
National Park .... (Including Golf Course Area) (C-II)	22 MCM/year
Rawalpindi Urban (C-III)...........	24 MCM/year
Rawalpindi Rural (D)	Negligible
Total	59 MCM/Year

In the left bank area of Lei Nala in Rawalpindi urban area decrease of groundwater table has been prevailing on the existing wells especially in summer season as the results of excessive discharging from the highly concentrated tubewells.

b. Wah and Taxila Area

The present groundwater discharge through some 50 tubewells from the Wah and Taxila groundwater basin is around 21 MCM/year as shown below;

Taxila Area	11 MCM/year
Wah Area	10 "
Total	21 MCM/year

The decrease of groundwater table caused by excessive discharging has not recognized so far in this area, because large amount of discharge from springs through the said aquifer is observed on the Jhablat Kas and other streams.

C. Potential Areas for Groundwater Development

Areas in which large groundwater basins are located and the groundwater resources have not yet been highly developed are selected as potential areas for groundwater development.

a. Islamabad and Rawalpindi Area

The followings have been selected as potential areas for groundwater development in the capital area.

- ° The western part of the Islamabad sector,
- ° The right bank area of the Kurang river in the National Park,
- ° The western park of Rawalpindi City (Cantonment area),
- ° The western rural area of Rawalpindi City.

The areas listed above are shown in Figure VI-3-4.

b. Wah and Taxila Area

Though large scale groundwater development has not been conducted in this enormous groundwater basin, the Wah and Taxila area as a whole is judged to have significant potential for future groundwater development.

6.3.3. Geo-electric Sounding

Geo-electric sounding has been carried out in the selected potential areas in order to clarify hydrogeological conditions.

Evaluated resistivity values have been interpreted according to the four layer model. By comparing these with the sub-surface materials described in the geological logs of existing tubewells and test holes resistivity layers over 50 .m, might be correlated to aquifers.

The points of geo-electric sounding are shown in Figure VI-3-3 and VI-3-4.

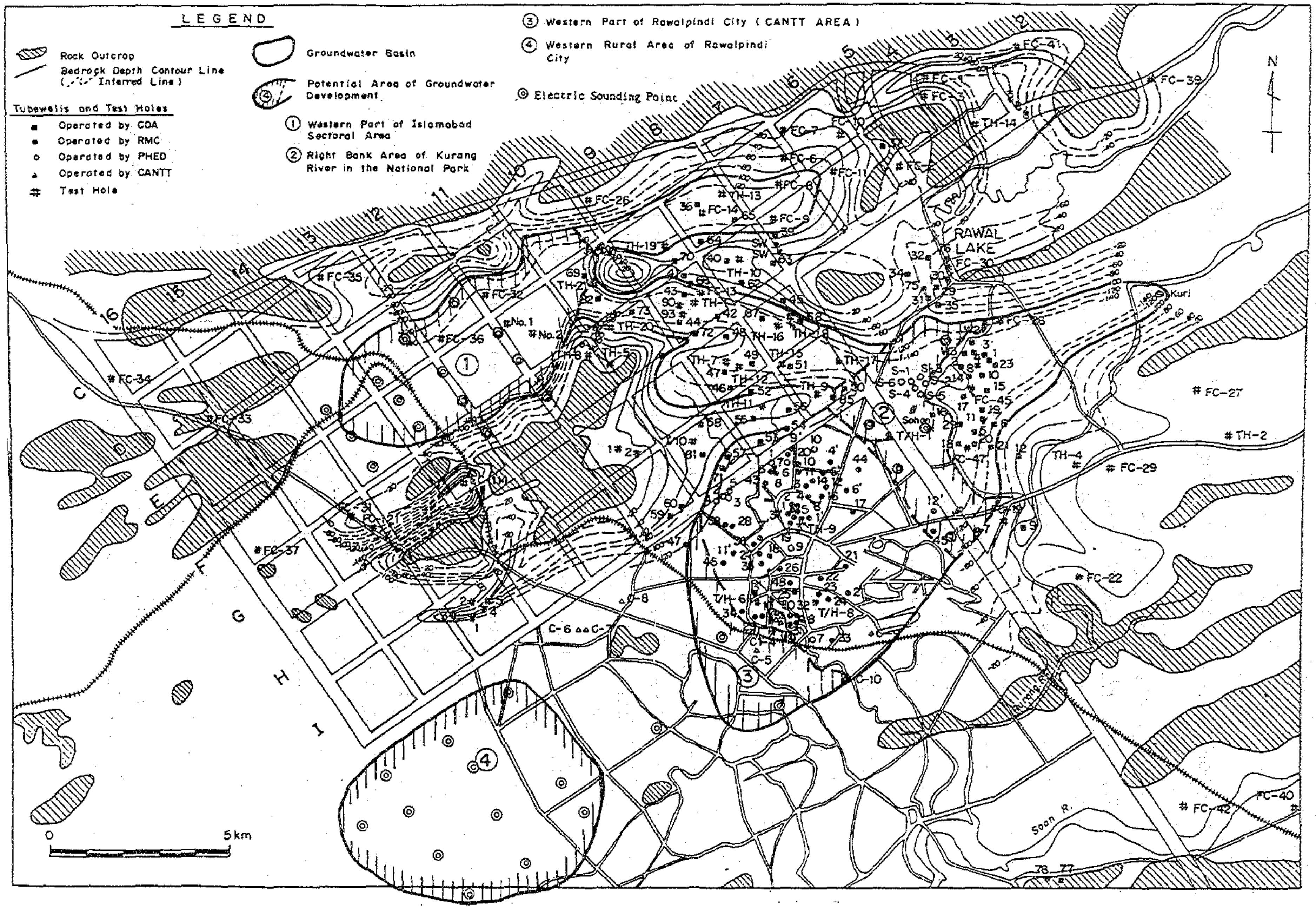


Figure VI-3-4. Potential Areas of Groundwater Development in the Capital Area

A. Results of Geo-electric Sounding

Based on the distribution of the resistivity layers, the characteristics of each sounding area are described below.

a. Islamabad Area .... 15 probe points

The resistivity value in this area is generally low, less than 50 .m. Although resistivity layers of about 50 .m are also present, these layers are discontinuous and localized.

b. National Park Area .... 5 probe points

Some few layers with resistivity of about 50 .m to 100 are interlayered in alluvial deposits over the whole of this area. Therefore the sand and gravel layers which form excellent aquifers are continuously distributed in this area.

c. Rawalpindi Urban Area .... 4 probe points

The resistivity layers in this area generally show lower resistivity values, less than 20 .m. Although some resistivity layers of 50 to 70 .m are occasionally detected, they are very thin and their continuity is very poor.

d. Rawalpindi Rural Area .... 12 probe points

The resistivity values in this area are generally low except for the thin high resistivity layers about 50 .m to 140 .m distributed at a depth of over 50 m. These high resistivity layers are 10 m to 20 m in thickness, and discontinuous and localized.

e. Wah and Taxila Area .... 11 probe points

A high resistivity layer zone is distributed in the northeast area of the Wah Cantonment, that is, the north area of HMC and HFF. This high resistivity zone is almost coincident with one of the assumed groundwater flow channels described in Chapter 6.3.2.

The western and southern areas of the Wah Cantonment are occupied with clayey layers. The resistivity in these areas is less than 50 .m.

B. Estimations of the Potential Areas

a. Capital Area

According to the results of the geo-electric sounding and evaluation of existing tubewell production, the right bank area of Kurang river in the National Park area is the sole area where high yield aquifers are thought to be continuously distributed. In the other areas promising aquifers might be seldom borne.

b. Wah and Taxila Area

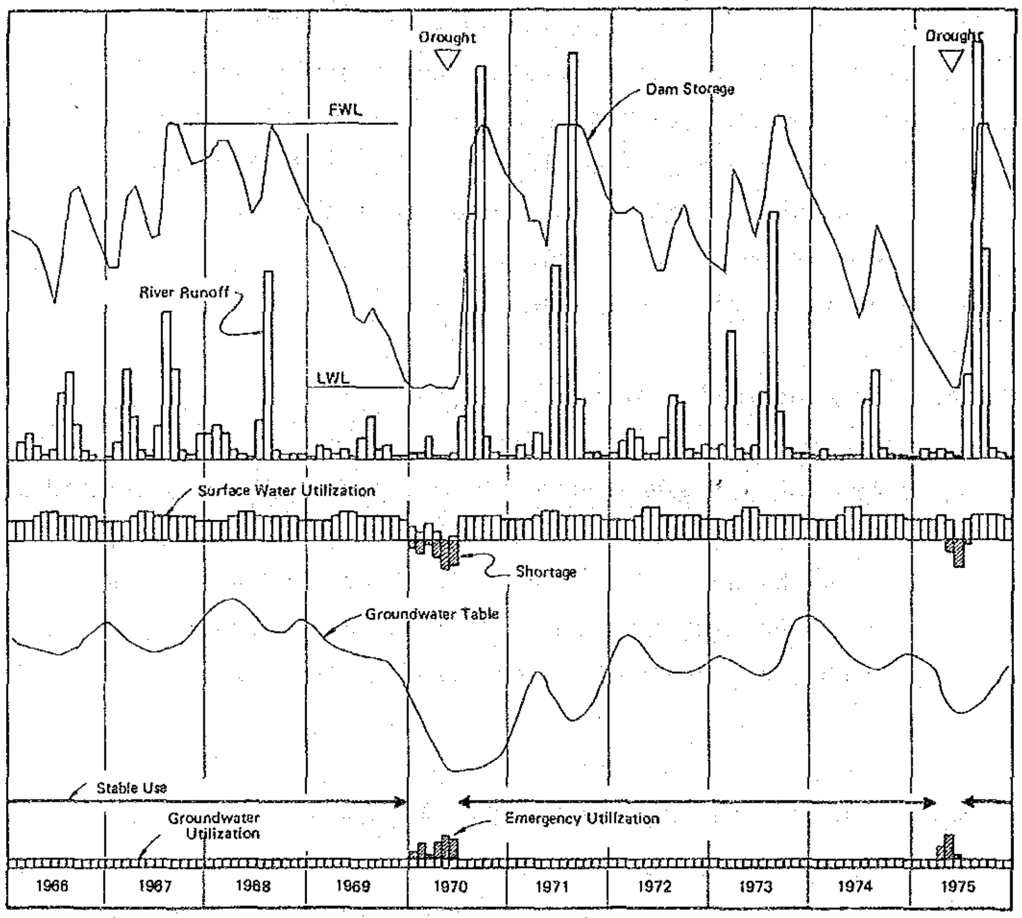
As high permeable zones are inferred to be distributed in the shallower portion along the three assumed groundwater flow channels, the potentials for groundwater development could be high in these channels. In other areas, such a high yield aquifer may be rarely intercalated.

6.3.4. Potentials for Groundwater Development

There are two conceptions for the groundwater development scheme. One is to use the groundwater under stable conditions over a years time. The other is to use only in the drought year.

For a stable utilization, the discharge to be used in a year should be restricted to an amount within the annual amount recharged to the groundwater basins. Otherwise, many detrimental effects including ground subsidence will arise.

For an emergency utilization in the drought year, a discharge may be permitted to exceed the annual recharge amount. The amount of utilization can be recharged in several years after the drought year. The schematic drawing indicating the concepts of the stable use and emergency use is shown below.



A. Islamabad and Rawalpindi Area

a. Stable Utilization

Based on the results of the hydrological analysis in the study area, the amount of infiltration into the catchment area of the three eastern groundwater basins reaches 140 MCM/year.

It is, however, impossible to estimate the amount recharged to the groundwater basins without information concerning the seepage into the bedrock and other basins. Long-term fluctuation data on the groundwater table can also give important suggestions for estimating the recharge amount to the basins. As such information is not yet available, an accurate estimation of the stable utilization amount is impossible.

Meanwhile, current measurements make it clear that about 30 MCM/year of groundwater is flowing into the Kurang River in the National Park groundwater basin. Full exploitation of the effluent amount, however, will cause considerable decrease in the runoff of the Kurang river. Therefore the amount exploited will be restricted to 50 percent of the effluent amount. In this case, 15 MCM/year can be exploited in the National Park groundwater basin.

The groundwater basin in the Rawalpindi rural area is separated from other groundwater basins and groundwater has hardly been exploited in this basin. The infiltration amount from the precipitation in the catchment area is assumed to be around 22 MCM/year, based on hydrological analysis. Because the infiltration mainly take place on the surface of the groundwater basin, the great part of the infiltration amount might be exploited as groundwater resources.

On the other hand, the strata in this basin are assumed to be abundant in finer materials, making it impossible to successfully exploit any more than 50 percent of the total recharge amount (infiltration amount). 50 percent of the annual recharge amount in the Rawalpindi rural area is some 11 MCM/year.

As mentioned above, groundwater of 26 MCM/year might be developed at the least in the capital area including the Rawalpindi rural area. But the actual potential for groundwater development should be assessed based on further studies. Such studies would include the measurement of long-term groundwater table fluctuations and so on.

b. Emergency Utilization

The groundwater basins in the capital area have enormous capacities and the reserved groundwater is up to 3 billion tons in total.

Dropping the entire groundwater table by 10 m, 140 MCM of water could be gained, assuming that the effective porosity of the deposits is 10 percent.

Therefore, it is the conclusion of this report that sufficient groundwater resources have been preserved in the capital area against a severe drought year.

For emergency groundwater utilization, a total scheme combining the groundwater and surface water should be employed.

B. Wah and Taxila Area

a. Stable Utilization

According to the hydrological analysis the amount for the entire catchment area which infiltrates to the subsurface is assumed to be around 210 MCM/year and a large amount of the infiltration flows into the groundwater basin as recharge. Hydrological analysis suggests that the effluent flow to the rivers (Jhablat Kas and Bauhti Nala) was 160 MCM/year during the period from 1961 to 1965. This means that some 80 percent of the annual amount of groundwater recharge was flowing out to the other basins in the form of surface runoff. It can be that this phenomenon is due to the special hydrogeological structure that the groundwater is dammed up by the unpervious ridges.

The assumed present groundwater balance is shown next and it takes into account well discharges and recharge from the Khanpur dam reservoir.

1. Rainfall Infiltration in Catchment Area	210 MCM/year
2. Recharge from Khanpur Dam	20 "
Total	230 MCM/year
3. Discharge by Tubewells	21 "
4. Effluent through Springs	160 "
5. Leakage and Outflow	49 "
Total	230 MCM/Year

As shown above, the amount of effluent groundwater is 160 MCM/year and this figure is thought to be almost coincident with the maximum value of groundwater development potential.

Because the surface runoff and the groundwater flow are closely related, groundwater abstraction of large quantities by tubewells will immediately cause the runoff in the rivers to decrease.

Accordingly, there are two methods by which to develop groundwater in this area. One would be to exploit the groundwater in the basin as groundwater. The second would be to develop the surface water in the northern part of the groundwater basin which had its origin in the groundwater.

b. Emergency Utilization

The amount of groundwater reserved in the Wah and Taxila groundwater basin is estimated to be around 3 billion tons. If the entire groundwater table in the basin is dropped 10 m by discarding, the water discharged is calculated to be around 140 MCM on the assumption that the effective porosity is 10 percent.

If the discharging area is set 5 km apart from the springs in order to prevent them from being effected by the drawdown, about 10 MCM may be exploited with a 10 m drawdown.

As mentioned above, it is the consideration of this report that there are enough groundwater resources available in this area to meet the drought year. For emergency groundwater development the total water resources development plan recommended in the capital area should be considered for this area too.

6.3.5. Further Investigation

The groundwater investigations performed in this study only delineate an outline of the hydrogeological conditions of the study area. It has not been made clear in this study that even the present groundwater abstraction amount is optimum or not because of the lack of data. For the reasons mentioned above, further studies should proceed for more accurate estimation of the area's groundwater development potential based on more detailed and reliable data. Such data would include long-term hydrographs of the groundwater tables.

The Landsat imagery techniques is the one of the effective method for groundwater investigation. Nevertheless it is judged that the landsat data is insufficient in this study because the Landsat imagery techniques contribute for the groundwater investigation in the vast and undeveloped area such as a desert and would not be so helpful to the highly developed areas such as the study area where many and detailed data can be available.

Other investigations for which further studies are necessary are listed below;

a. Investigations for the Aquifer

1. Simultaneous well observation covering the entire study area (twice a year, using the existing wells).
2. Study at the aquifer with drilling surveys and geo-electric soundings.
3. Study of permeability and storage coefficient of the aquifers with aquifer tests.
4. Chemical analysis of groundwater.

b. Investigation for the Groundwater Flow

1. Long-term observation of groundwater table (observation wells and existing wells)

2. Observation of precipitation.

3. Current measurement of the rivers.

4. Precise measurement of discharge amount of groundwater.

5. Assumption of recharge amount of groundwater.

6. Study of groundwater balance.

c. Planning of the Groundwater Utilization

1. Prospect of groundwater balance of the each groundwater basin.

2. Consideration of the possibility of urgent groundwater utilization for the each groundwater basin.

3. Comparison and examination of the measures for water shortage of the rivers.

4. Planning of groundwater utilization.