increase of earth pressure especially when excavation works come across fractured fault zones. Even in this case the minimum cross-section is requested to be kept after lining. A diameter of 2.40 m is, therefore, adopted for a long tunnel. According to design criteria prepared by United States Bureau of Reclamation as well as by Japanese Ministry of Agriculture, Fishery and Forestry, three types of standard cross sections of tunnel are established. (Refer to Table B.III-1)

3.2.3. Tunnel Geology

A. General Geology

Geological formations exposed in the Project Area, in and around the Margala hills, as investigated by Geological Survey of Pakistan, are presented in Table B.III-2. Among these, such formations belonging to Jurassic to Eocene age are, as they had been called "Hill Limestone", distributed in the hilly areas of Margala, Kala Chitta and Kalri Murat, and are marine deposits composed mainly of limestone. Strata of Miocene age and onward, on the contrary, are all nonmarine deposits and are spread in the flat areas which lie between the hilly areas. Outline of the geology in the Project Area is shown in geological maps as attached Appendix D (Drawing).

Among Jurassic to Eocene formations, Patala, Chor Gali and Kuldana formations contain a considerable amount of relatively soft argillaceous rocks as represented by shale and marl, while most of the other formations are composed mainly of limestone. Exceptionally in the area, Chichali formation comprises sandstone and, therefore, this can be an effective Key bed. With exception of Chichali formation, it is difficult in the field site to classify geological features.

Among formations belonging to Miocene to Holocene age, Murree formation of Miocene age and so-called "Alluvium" of Pleistocene to Holocene are distributed widely in the Project Area. Murree formation consists of alternating beds of sandstone and mudstone.

This sandstone looks gray to reddish brown in colour and is classified into two types, namely very hard type and rather soft one. Mudstone is generally soft and reddish brown in colour. In the area where Murree formation dominates, there exist some strata which contain limestone. Formerly this strata were considered to be a part of Fathehjang member which formed the base of Murree formation and were considered as the secondary deposits derived from the older marine formations, but it has become considered recently that the strata are constituents of Kuldana formation or Chor Gali formation of Eocene age.

Alluvium consists of alternating beds of silty clay and gravel. Silty clay is composed mainly of secondary deposited loess and sometimes called as "Potwar Loessic Silt" due to the fact that it is distributed widely in the Potwar basin. With exception of small amount of concretions of carbonates, this clay little contains coarse grains but consists mainly of silt particles showing uniform facies. The gravel layers, which exist not only intercalated in clay layers but also spread on a terrace, consist mainly of pebbles of limestone. Such layers which are relatively consolidated and found nearby Lei Nalla are called "Lei Conglomerate". There exist a number of terraces with various elevation on the foot of the Margala hills and along Nilan Kas, Haro and Soan rivers. Generally speaking, stratigraphy of Quaternary strata in this area has not yet been established, and so, it is difficult to classify alluvium into further sub-members.

Kamlial formation to Soan formation overlying Murree formation are predominant nearby and south of the Soan river, and contains soft sandstone, mudstone and Conglomerates.

Major geological structures in this area have a trend of ENE to WSW. Among these structures, a thrust which lies on the southern foot of the Margala hills and on which formations of Jurassic to Eocene age are in contact with Murree formation (partly with Kuldana formation) are called as"Margala Fault". From structural and geological point of view, this fault is classified as one of the greater faults which are situated in the thrust zone spread in the front of Himalayan mountains. At present, there is no reliable evidence showing that the fault is active. Except alluvium, all strata in the area are folded. Accompanied by overturned folds and thrusts, strata of Jurassic to Eocene age are tightly folded with an interval of several hundred meters, while rather open folds are found on the strata of Murree formation with an interval of the order of several kilometer.

B. Geology around Tunnel Routes

Investigated routes of tunnels pass through the range of Margala hills between Khanpur dam and the points of Shah Allahditta and/or Golra. Geologic sections and conditions along and around the selected routes of tunnels were investigated employing the following procedures;

- Review of NESPAK (1980) report

- Study on a geological map (1/50,000 scaled) compiled by Geological Survey of Pakistan

- Collection of informations mainly provided by Geological Survey of Pakistan

- Analyses based on aerophotographs (1/20,000 scaled)

- Surface geological reconnaissance survey mainly along Libana-Khurram Gujar route, Shah Allahditta - Sabra - Tarmakki - Choi route and around Portal and Shaft sites

Geology of Margala hill consists mainly of limestone subordinated by marl and shale with a trace of sandstone. These strata are folded and faulted, and divided into zones by three major thrusts which extend from east to west or northeast to southwest. In such zones in general, the older layer is found to the south while relatively new layer exists to the north of the area, and as a whole the layers are folded southerly overturned.

Three major thrusts run along the southern foot of Margala hills, Khurram Gujar - Bol - Sabra Valley, Nilan Kas Valley near Gramthun (Tarmakki). These thrusts are named tentatively in this report as Margala, Sabra and Gramthun faults. Between Gramthun and Sabra faults, there may exist one more small scaled thrust. Margala fault may not be simple and single fault, but may be estimated to be a group of faults aligned in echelon. Thickness of fault has not been investigated clearly. However, it is estimated on a basis of outcrops to be more or less 20 to 30 meters.

As concerns geological distribution, Margala hill limestone, Chor Gali formation and Kuldana formation belonging to Eocene age are distributed in the area between Khanpur dam and Gramthun fault, composed mainly of limestone with an unnegligible amount of argillaceous rocks. Between Gramthun and Sabra faults, Samana Suk, Chichali, Lumshiwal formations and Lockhart limestone of Jurassic to Palaeocene age are dominant and their main constituent is limestone. In the area between Sabra and Margala faults, all strata formed in Jurassic to Eocene age are distributed and the greater part is occupied by strata belonging to Palaeocene to Eocene age. They are mainly composed of limestone and marl, and shale is also distributed around a valley along Sabra fault, where Kuldona and Chor Gali formations are found, and on the flat middle ridge where Patala formation dominates. In the southern flank of Margala hills and between Gramthun and Sabra faults, sandstone layers of several ten meters wide, belonging to Chichali formation, are found. To the south of Margala hills, Murree formation consisting of alternating beds of sandstone and mudstone is predominant.

Limestone in this area is generally hard and gray in colour, and contains abundant fossiles of foraminifers and, sometimes, of vivalves. Based on the condition of crack, limestone is classified into three types; namely massive limestone, cracked limestone and nodular limestone. Massive limestone has sparse cracks and used to form a cliff. Nodular limestone is composed of alternating beds of limestone and subordinate shale, and because that limestone is more competent than shale when folded, limestone layers are deformed and subdivided into small nodules with diameters of five to 20 cm.

Marl is gray in colour and likely appears soft rock, containing abundant foraminifer fissiles. This is classified into two types, namely massive type and fissile type. Shale is brown to greenish brown in colour, fissile and easily eroded. Sandstone contained in Murree formation is gray to reddish brown in colour and classified into two types, very hard type and relatively soft type. Mudstone is generally reddish brown in colour and soft. Dips of strata varies from 20 to 90 degrees, and in general, they are steeper in the north side of the fault and gentle in the south side.

C. Groundwater Condition around Tunnel Routes

Along the valleys in the area of Margala hills, with exception of a few rivers, there exists no stream which keeps flow during the period of no rain. Groundwater flows are appeared as spring waters at the limited points spread along valleys and on the foot of Margala hills. This means that rain water is concentrated deep inside the hill body, and would suggest the fact that some water ways exist to convey groundwater to the above springs. The most important springs are found at Khurram Gujar and Shah Allahditta, and the former is proud of the spring water discharge of several thousand cubic meters per day, especially during rainy season. Accordingly, there are some possibilities of happenings that the tunnel comes across massive volume of groundwater during the course of excavation work and/or spring water discharge is influenced by that.

D. Engineering Consideration

Rock Type

From viewpoint of tunnel engineering, rocks are classified into four types;

Type I	:	Massive limestone; almost no support needed.
Type II	:	Cracked limestone, alternation of limestone and marl/shale, massive marl, sandstone (cretaceous); supports partly needed.
Type III	:	Shale, alternation of shale and marl, fissile marl, alternation of sandstone and mudstone (Murree formation); supports needed.
Type IV	:	Overburden (clay and gravel); supports heavily needed.

Special Problems

As mentioned previously, there exist three major faults along the routes of tunnel. Dimensions and conditions of these faults are not clear, but much care should be taken in excavating the faults because they are rather big scale from every aspect of structural geology and much earth pressure may occur. It is also probable that the tunnel may come across a big scale ground aquifer and, to cope with such situation, a countermeasure should be worked out in advance. Problem would be less against such an accident in the case when the tunnel is excavating the type I or II rocks and, on the other hand in cases of type III or IV rocks, the tunnel wall is quite dangerous against a breakdown. Especially when excavation work is progressed through fault cray or shale which could be impermissible layer, much attention should be paid.

TABLE B.III-1 CLASSIFICATION OF TUNNEL TYPE

Tunnel Type	Supports	Geological Condition	Rock Type 1/	Rock Load from Karl Terzaghi Hp(m) 2/
A	No Supporting or Rock Bolt	Little cracked solid rock mass, moderately jointed	All of Type I Rock Part of Type II Rock	0 to 0.25 x B
B	Steel Support H-100x100x6/8 @ 1.50 m	Cracked and slightly weathered rock mass, or very consolidated soil	Part of Type II Rock Part of Type III Rock	0.25 x (B+H) to 0.5 x (B+H)
C	Steel Support H-100x100x6/8 @ 0.90-0.60 m	Weathered rock, fault zone or consolidated soil	Most of Type III Rock All of fractured fault zone and unconsolidated sediment	0.6 x (B+H) to 1.1 x (B+H)

Note:

1/ Rock Type I : Massive limestone
II : Cracked limestone, alt. of limestone & marl, massive marl, sandstone.
III : Shale, alt. of shale & marl, fissile marl, Murree Fm (sandstone & madstone) fault zone.
IV : Unconsolidated Sediment, Overburden (silty clay, gravel)

2/ Height (H) and width (B) of tunnel excavation.

3/ If type C with rock load greater than 0.8 x (B+H), then interval of steel supporting should be shortened.

4/ When rock load is much greater than 1.1 x (B+H) on passing fractured fault zones, a particular type should be established with increased thickness of lining and supplemental reinforcement.

Table B.III-2 GELOGIC FORMATIONS IN AND AROUND MARGALA HILL

Geochronology	Name of Formation	Thickness(m)	Principal Geology	Obsolete Name
Quaternary				
Holocene	So-called "Alluvium"	150 ∿ 900	Silty clay	Boulder Czl.
Pleistocene	Lei Conglomerate		Gravel or Conglomerate	
Tertiary				
Pliocene	Siwalik Group			
	Soan Fm.	120 ∿ 145	Caarse facies	
	Dhok Pathan Fm.	1,800	Alternation facies	
	Nagri Fm.	300 ∿ 2,000	Arenaceous facies	
	Chinji Fm.	750	Argillaceous facies	
Miocene	Rawalpindi Group			
	Camlial Fm.	90 ∿ 650	Sandstone with shale & Conglomerate	
	Murree Fm.	3,000	Alternation of Sandstone & Mudstone	
Oligocene				
Eocene	Kuldana Fm.	150	Shale & Marl.	
	Chor Gali Fm.	45	Limestone & Shale with marl.	
	Margala Hill Limestone	105	Limestone with Shale & marl.	Hill Limestone Nummulitic Fm.
Palaeocene	Patala Fm.	60 ∿ 180	Marly Limestone & Shale	
	Lockhart Fm.	90 ∿ 240	Limestone with marl & Shale	
Cretaceous & Jurassic	Lumshiwal Fm.	35	Sandy Shelly Limestone	
	Chichali Fm.	18 ∿ 65	Sandstone	Giumal Sandstone
	Samana Suk Fm.	180 ∿ 360	Limestone	

3.3. Raw Water Reservoir

3.3.1. Necessity of Raw Water Reservoir

The Left Bank Canal has been constructed for most of its length along the foot hills of Margala range. There are a number of cross drainage structures and small tunnels including Haro river syphon along the canal alignment. Although the canal itself is lined for its entire length, but closure of the canal for the periodic maintenance and possible repair of various structures must be envisaged. In addition to this possibility of blockage due to rock slides caused by adjacent hill torrents cannot be ignored in an open canal. Therefore, adequate provision for the raw water reservoir during canal closure shall be required.

3.3.2. Design Capacity

Under the major premise of additional constructions of culvert at the portions of deep-cut, slope protection works, spillway or wasteway and other overall improvement works for the existing Left Bank Canal, design capacity of raw water reservoir was determined as 2 days capacity, which is reserved for general works of operation and maintenance, rehabilitation works and disaster rehabilitation works.

3.3.3. Structural Design

Geological foundation at and around the proposed sites of raw water reservoir is found to be alternation of silty clay layer and sand gravel layer, of which silty clay layer is usable as impervious materials for embankment. However this material is easily erosive and thus protection works on the surface of embankment was designed to be accompanied. In addition, to prevent water leakages through sand gravel layers, earth blanket was designed on the bottom surface of reservoir.

3.4. Pumping Station

3.4.1. Selection of Pump

High pumping heads of more than 30 m are mostly required for the project. In view of required discharge and head, double suction volute pump was selected to be used.

3.4.2. Control Method

Considering operational and economic advantages as well as operation and maintenance, flow control is primarily based on the simplest method of control by charge of operating pump number and on-off control in terms of water level control in the discharge pool. Accordingly HWL and LWL are to be established in both suction and discharge pools, providing proper capacities which are determined as 30 minutes capacity of the maximum design discharge.

3.4.3. Number of Pumps

Number of pumps are determined in consideration of the following factors;

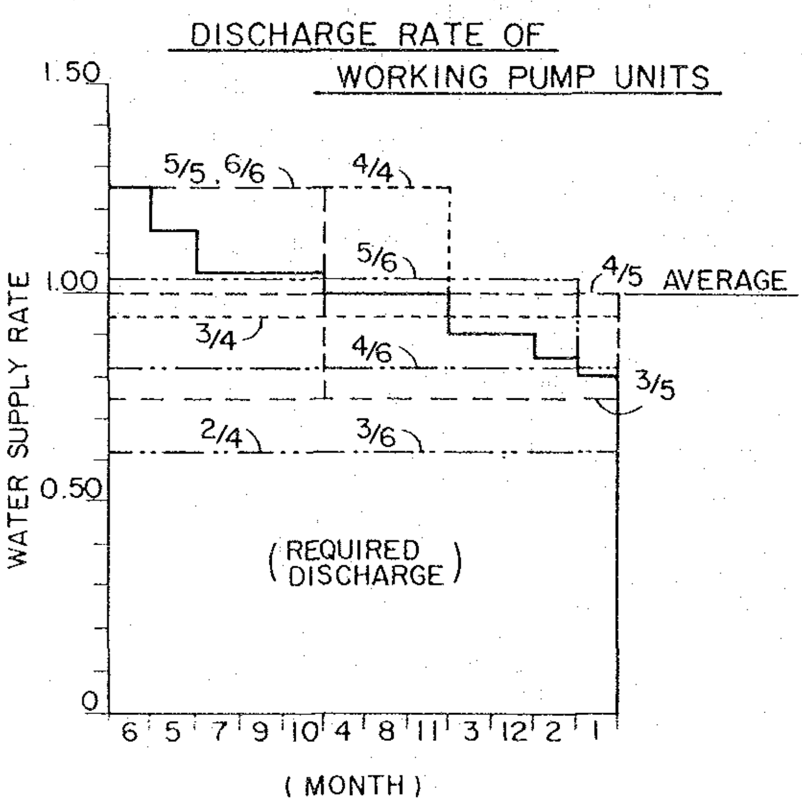
A. For easy operation and maintenance, the number of pump units should be as small as possible and ones of equal capacity should be adopted.

B. The larger is the delivery, the higher the pump efficiency, so the ones with the largest delivery should be used.

C. At least one spare unit must be provided. If the total number of pump units is small, spare unit will be costly.

D. The number of units should be determined so as to be operated effectively corresponding with water-load varieties of seasonal changes. (Varieties range from 1.25 to 0.8 of average)

E. Number of units should be decided conforming to the phased plan of facility installation. Rate of required capacity is approximately 2:1:1 corresponding to phase I, II, and III respectively.



The required monthly average discharges are plotted on the above figure in order from the maximum in june to the minimum in January. The figure indicates relation between the required discharges and controlled discharges for cases that the number of pump units, excluding spare one, is given as 4, 5 and 6. From the operational point of view, installation of five pumps will be most suitable. Although a plan with four units of pump is most acceptable when phased schedule of facility installation is taken into consideration, but this plan is inferior when necessary cost allocation for spare unit is accounted. Consequently, including one spare unit of pump, number of pumps are determined at six. As concerns booster pumps required for distribution systems, since the design capacity is small, numbers are determined as two units for ordinary uses plus one unit for spare.

3.5. Water Treatment Plant

3.5.1. Raw Water Quality

Raw water was taken three times during field survey period from August to October, 1984 at four points as shown in Figure B.II-8. The sampled water are from surface water of the Kampur Reservoir, canal water at the branch to the Right Bank Canal and the division works for irrigation and the Haro river water at about one kilometer upstream of the dam. Sampled water was analized at the laboratory of CDA as shown in Tables B.II-2. In addition, water of the reservoir and canal (at point 4) was brought to Japan to check the existence of algae (see Table B.II-3).

The result of water quality analysis shows the following characteristics:

A. Values of turbidity and color were low at every four sampling points.
B. Iron and manganese were not found at every sampling point.
C. Every sampled water shows high values of pH and alkalinity which is considered to be caused by soil conditions of limestone strata of mountain behind the dam.
D. A few numbers of bacteria and coliform groups were found in every sampled water. Pollution of water is considered as low which is supported by the low contents of chloride and nitrogen.
E. In surface water of the reservoir 25 nos/ml of algae was found. This content is considered small.

From the above, water quality at four points is considered to have similar characteristics and no special treatment will be necessary.

3.5.2. Water Treatment Process

On the basis of the water quality analysis, the treatment process is examined. However, considering insufficient period and number of sampling and the facts that the Khanpur dam was constructed recently, and water quality might be changed in the future, the water quality observed at the Simly and Rawal Lake filtration plants are taken into account for the determination of the treatment process in this study.

Considering the turbidity will increase during wet season as shown in the water quality of the existing treatment plant, the rapid filtration method is employed as the general concept. The chemicals applied at the plant are planned to be Alum for coagulation and chlorine for disinfection as used in the existing plant. Pre-chlorination will be applied considering that algae growth might be occur in future due to big volume of the reservoir resulting long retention time and higher temperature in dry season.

The following treatment process is proposed considering the characteristics of raw water and practice of the existing treatment plant:

<u>Pre-chlorination</u>: Pre-chlorination is empolyed for the purpose of destruction of algae and oxidation of dissolved organic matters. Color of raw water is considered to be increased according to algae growth. Dosage of chlorine is at receiving well.

<u>Coagulation</u> : Alum is used for coagulation and dosed at mixing well.

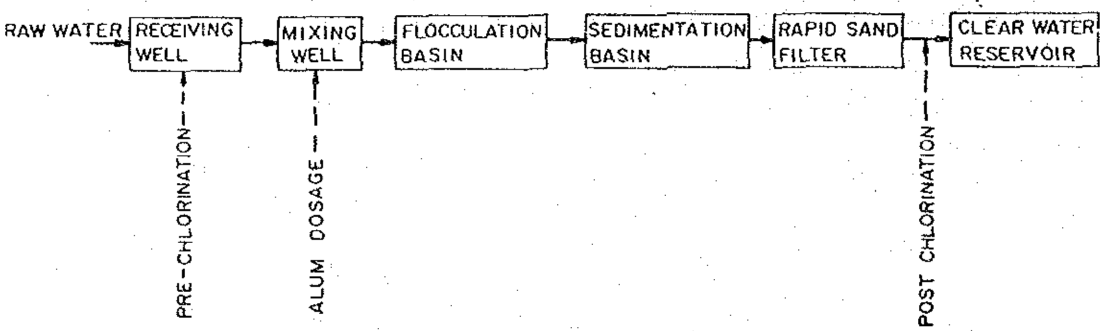
<u>Flocculation</u> : Suspended solid is flocculated by slow mixing.

Sedimentation : Most suspended solid is removed in the sedimentation basin with the optimum retention time.

Filtration : Remained suspended solid in the clarified water is removed by the allowable range of less than 5 ppm.

Post-chlorination: Filtered water is disinfected by chlorine dosage.

The chart for the proposed water treatment process is shown below.



PROPOSED WATER TREATMENT PROCESS

3.5.3. Design Capacity

Design capacity of the plant is planned at 121.6 MGD (553,000 m^3/d). Five percent of water loss will be allowed for the treatment including washing water of filter bed, desludging in sedimentation basin, chemical solution and other usages within the plant.

3.5.4. Facilities

The design criteria on the major Facilities of the treatment plant is examined and proposed as follows:

Mixing Well

Energy dissipation	:	about 300 sec^{-1}
Mixing method	:	mechanical mixing
Retention time	:	2 min.

Flocculation Basin

Energy dissipation	:	70 - 20 sec^{-1}, more than three stage
Flocculation	:	by over-and-under baffles
Retention time	:	30 min.

Sedimentation Basin

Type	:	rectangular horizontal flow
Overflow rate	:	1.0 m^3/m^2 hr
Mean velocity	:	less than 0.4 m/min
Retention time	:	around 4 hrs
Desluding	:	by scraper
Effluent	:	effluent trough at basin end, weir load is less than 300 m^3/m day

Rapid Sand Filter

Type	:	declining rate filtration
Filtration rate	:	120 m^3/m^2 day
Filter media	:	thickness 70 cm
Supporting gravel	:	thickness 25 cm
Water depth above sand	:	1.5 m
Washing	:	backwashing 0.6 m^3/m^2.min. surface washing 0.2 m^3/m^2.min.

Chemical Dosage

Pre-chlorine	:	average dosage rate	2	ppm
		maximum dosage rate	5	ppm
Alum	:	average dosage rate	20	ppm
		maximum dosage rate	60	ppm
Post-chlorine	:	average dosage rate	1	ppm
		maximum dosage rate	2	ppm

Where average turbidity of raw water is assumed as 20 units and that of maximum is 200 units for planning of Alum dosage rate.

3.6. Service Reservoir

In principle, service reservoirs are of gravity flow type.

3.6.1. Service Reservoir Site

In principle in order to expect uniform rate of water head in the service area, the service reservoir is desirable to be situated near the center of the service area. However for the project, since the service areas are located on the sloping surface extending from north to south, the proposed site of service reservoir is selected as near from the service area as possible in consideration of the following conditions;

A. Conduction pipeline up to the service reservoir is designed with design capacity equivalent to the maximum daily distribution requirement.

B. Downwards, distribution facilities are designed with the maximum hourly requirement, which is equivalent to 1.5 times of daily maximum.

3.6.2. Capacity

In general, filtration facilities are operated on the standard of planned maximum daily supply, and so in each hour, a fixed amount of purified water is sent to the service reservoir. On the other hand, as there is hourly change in the distributed amount, such amount of the water as exceeds that of the hourly delivery is stored during the night hours when consumption is low and made to meet the demand during the day hours when the consumption rises for balance of demand and supply. The effective capacity of the reservoir, therefore, must be enough to maintain such balance. The effective capacity can be obtained by totalling such hourly maximum amount plus hourly margins.

The proposed hourly change in the water supply has been assumed as illustrated in Figure B.III-2, after consideration of water use, leakage and etc. in the area, because the actual condition is hardly known as mentioned above. The capacity of a service reservoir shall be based on 6 hours supply of planned maximum daily water supply, making provision for regulation of purified water and supplied water, fire fighting, power stoppage, unforeseen accident, and so on (refer to Figure B.III-2).

<u>Estimation of Capacity of Service Reservoir</u>

Capacity of Service Reservoir
= answering the hourly variation
(Balance of Demand and Supply)
+ Fire fighting water
+ Capacity for power suspension
= 4.05 hr + 0.3 hr + 1.0 to 2.0 hr
≑ 6 hr

Fire fighting water shall become relatively small in quantity as the scale of communities becomes large. The water required for fire fighting in the Islamabad Low Zone that shall need relatively large amount of water for fire fighting against the total water demand, has been assumed as summarized below;

A. The proposed population in 2000 in the Low zone is 220,000, while 620,000 in Islamabad.

B. Fire fighting water vs. population is given in Figure B.III -2, from which the fire fighting water is derived at 13 cu.m/min.

C. The hours required for fire fighting is assumed to be one hour, thus, the required water comes to 780 cu.m (13 cu.m/min x 60 min).

D. It needs 0.3 hours to store the proposed fire fighting water (780/39 cu.m/min = 0.3 hours).

3.6.3. Effective Depth

Effective depth of service reservoir is defined as a distance between HWL and LWL. There is an economic depth of service reservoir for any given site. For a given quantity of water, either a shallow reservoir having long walls and a large floor area may be constructed or alternatively a deep reservoir constructed with high retaining walls and a smaller floor area, resulting in structual and engineering problems of less waterproof and less quakeproof. Since the service reservoirs are of gravity style and it is difficult for dynamic water pressures in pipes distributed in the service area to be kept within a certain acceptable range, in general, effective depth is desirable to be three to six meters. Every service reservoir for the project is of a large scale and therefore the effective depth is taken at six meters in consideration of structure and its floor area.

3.6.4. Water Level

Basic concepts for determination of optimum water level for each service reservoirs are summarized as follows:

- Estimation of projected population and unit water demand in target year of 2000 are worked out based on available census and data collected. Distribution of population and total water demand within each sector and major water supply networks are followed to urban development plans and future water requirements tendency.

- Major water supply pipelines are aligned based on the topographic conditions of each distribution block, water demand and distance from supply pipeline up to the distribution points.

- Treated water supply from the service reservoir to consumers shall be of gravity flow as much as possible.

3.6.5. Structure

In view of shape and structure, service reservoirs are classified into the following three types;

A. Flat slab type of reinforced concrete
B. Overhead type of reinforced concrete
C. Ground type of prestressed concrete

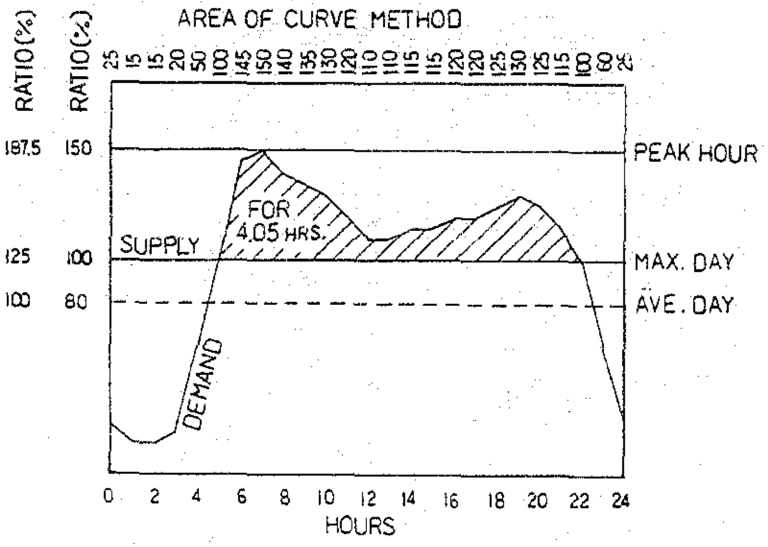
The flat slab type is in common use for this purpose and the most economic in cost in case that suitable construction site is found. Overhead type is also commonly used in Pakistan. The allowable maximum capacity of this type is, however, restricted from technical point of view within the limit of about 5,000 cu.m (0.1 million gallon).

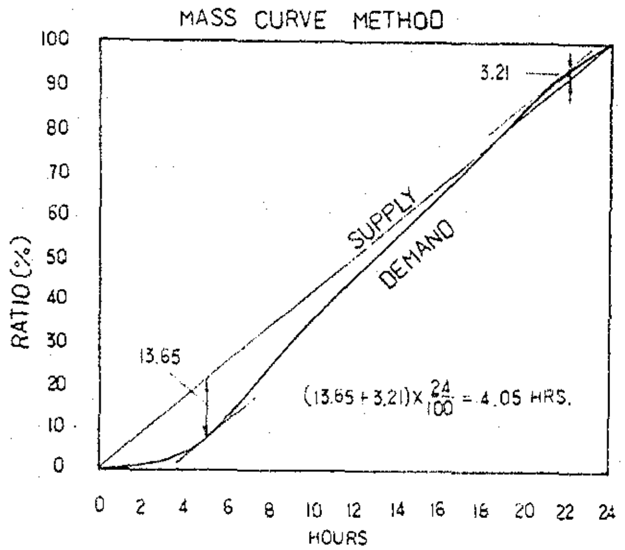
Ground type PC tank is very popular in the world but construction cost is slightly higher than that of flat slab type.

Consequently, the flat slab type is adopted under the study in principle except for the case that topographic and water level condition is not suitable for this type. Thus, ground type PC tank is applied for the specified construction sites.

FIGURE B.III-2 DESIGN CAPACITY OF SERVICE RESERVOIR

ANSWERING THE HOURLY VARIATION

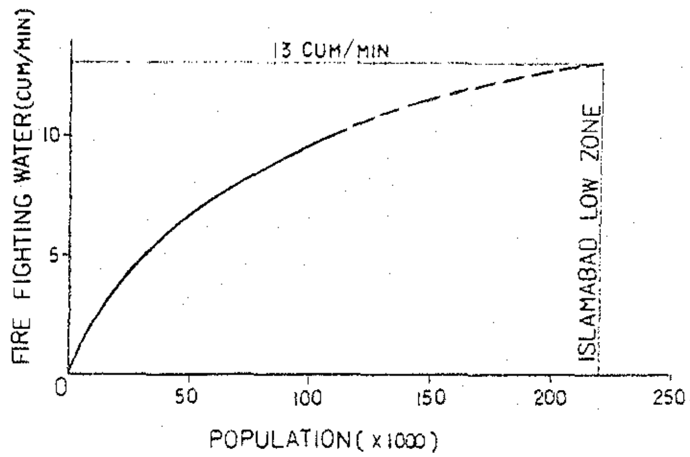




FIRE FIGHTING WATER

Fire Fighting Water to be Included in the Daily Maximum Supply according to Population

Population ($\times 10^4$)	Fire fighting water amount (m^3/min.)
less than 1	more than 2
2	4
3	5
4	6
5	7
6	8
7	8
8	9
9	9
10	10

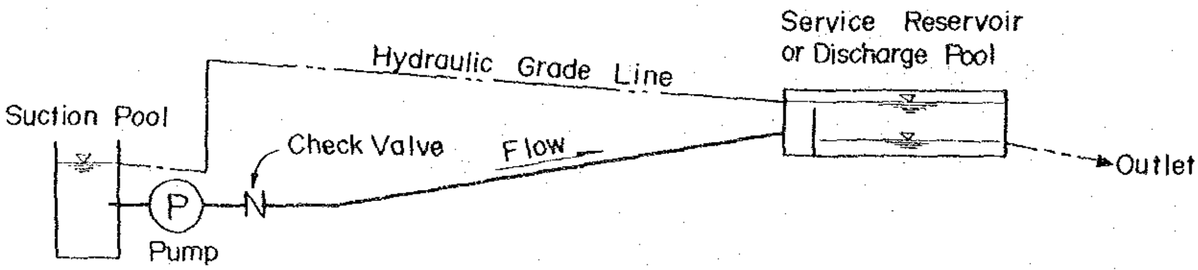


3.7. Distribution Main

3.7.1. Pipelines up to Service Reservoir

Water is supplied to pipelines by gravity or by pumping, categorizing them gravity mains and force mains respectively. The force mains is a pipeline to be laid between a suction pool of a pumping station and a delivery pool or a service reservoir, while the gravity mains is that to connect a service reservoir with a clear water reservoir at a water treatment plant, or a delivery pool at a pumping station.

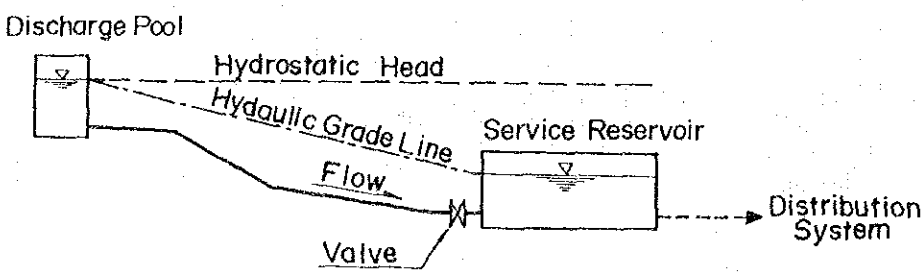
(i) Force Mains



Outline of force main pipeline system is illustrated as above. According to water levels in a service reservoir or a discharge pool, pumps are controlled by means of operating pump number and on-off control. Considering the emergent case of sudden stop of operation due to power stoppage, dynamic water pressure should be taken into account for design of pipeline material.

(ii) Gravity Pipeline

Gravity pipeline system is illustrated as below:



This system may permits easy operation and prevention of oversupply of water by closing a value when the water level in the service reservoir reaches a designed full water level.

Pipeline is subject to transient pressures when valves are alternated open or close. Such transient phenomena must be considered in designing and inner pressure of pipes.

3.7.2. Pipeline from Service Reservoir

In principle pipeline which connect the service reservoir with a unit of distribution network is of gravity flow type. Water level in the service reservoir depends upon status of water consumption in the distribution unit. Diameter of pipe is determined on a basis of the maximum hourly water consumption.

Effective water head at the site of water treatment varies depending on respective alternatives. Accordingly, distribution system up to the ending point of service area is needed to be differentiated, including routing of distribution main and necessity for installation of booster pumping station. Aiming at economic comparison study, it is therefore needed that the cost evaluation for construction as well as for operation and maintenance for distribution facilities between the service reservoir and the ending distribution network be involved in the study, even though it is excluded in the "Scope of Working". However the same system of the smallest unit of distribution networks can be applicable for every alternative plan for the purpose of comparative study, and so it is excluded from the study.

3.7.3. Pipe Material

Diameter of pipe for the on-going project varies from 300 mm to 1,650 mm. Design internal water pressure, inclusive of water hammer pressure, varies with a wide range between about 3 kg/sq.cm and 18 kg/sq.cm. In consideration of diameter as well as water pressure, pipe materials applicable to the Project are as under:

- Prestressed Concrete Pipe	(PRCC)
- Prestressed with Steel Sylinder	(PRCC)
- Asbestos Cement Pipe	(ACP)
- Steel Pipe	(SP)
- Ductile Iron Pipe	(DIP)

A. Prestressed Concrete Pipe

In general considering joint structure, PRCC is usable within a limit of design pressure of 6 kg/sq.cm and static pressure of 4 kg/sq.cm. This type is durable and nonerosive, and flexible at a joint. Weight is heavy. This type of pipe has been manufactured at two factories around Islamabad, however, leakage may occur because of inferior quality control. This type is therefore not usable.

B. Prestressed Concrete Pipe with Steel Sylinder

This type of pipe is manufactured according to AWWA Standard, and utilized for the Simly conduction main pipelines. Diameter of pipe is available between 400 mm and 1,800 mm. Spigot-type joint is mostly used and allowable maximum static and design pressures are 10 kg/sq.cm and 12 kg/sq.cm, respectively. This type is usable for gravity flow pipeline.

C. Asbestos Cement Pipe (ACP)

Diameters up to 400 mm are available. ACP has excellent durability but has little resistance to impact. Many accidents have been reported during construction works as well as during operation and maintenance, and therefore ACP is not recommendable in use.

D. Steel Pipe (SP)

SP has excellent strength, durability, flexibility and elongation, and also has rich impact strength. Weight is relatively light. The joints of SP have excellent water tightness and strength equal to or more than that of pipe body, if welding work is carried out properly. This type is suitable for the Project, especially for pressure pipeline.

E. Ductile Iron Pipe (DIP)

DIP is one of the strongest pipes against external load and is durable. Jointing works are simple and easy as compared with that for steel pipe. The gap between socket and spigot is sealed with a special shaped rubber gasket and, as a result, the joint has excellent water tightness and high resistance to both shearing and bending loads.

F. Selection of Pipe Material

In consideration of economy, strength, durability and workability, pipe materials are selected for the Project as under:

Diameter and Design Pressure	Pipe Material
For Pressure Pipe	
Diameter > 1,350 mm	Steel Pipe
$\leqq$ 1,350 mm	Ductile Iron Pipe
For Gravity Flow Pipe	
Design Pressure $\leqq$ 12 kg/sq.cm	PRCC (with steel sylinder)
>12 kg/sq.cm	Ductile Iron Pipe

3.7.4. Alignment

Pipeline is aligned on the route which is most economic, safe and easy in operation and maintenance. Based on the above conception, pipelines downstream of treatment plant have been aligned on the routes along main roads, which have been planned in the city development plan.

3.7.5. Earth Covering

Since groundwater levels are relatively low around the construction sites, it is not needed to consider additional earth covering against lifting pressures. However, considerable portions of pipeline are placed beneath roads and thus the minimum earth covering has been taken as 1.2 m in the design.

3.7.6. Design Pressure

Water hammer pressures vary depending on pipe length, velocity of water, hydrostatic pressure, pipe material and others. However the following empirical values have been adopted in the study.

(i) Pressure Pipe

Water hammer pressure is taken as 100% of dynamic water pressure when it is less than 4.5 kg/sq.cm, and when dynamic pressure exceeds 4.5 kg/sq.cm the bigger value of either 60% of dynamic pressure or 4.5 kg/sq.cm is taken.

(ii) Gravity Flow Pipe

Water hammer pressure is taken as 100% of hydrostatic pressure when it is below 3.5 kg/sq.cm, and as the bigger value of either 40% or 3.5 kg/sq.cm when hydrostatic pressure exceeds 3.5 kg/sq.cm.

3.7.7. Distribution Unit

A. Structure and Elevation of Smallest Distribution Unit

The smallest distribution units are partitioned according to divisions on which estimates of population as well as water demands have been based. With exception of some local part of high elevation for which waters are boosted by additional booster pumps, ground elevation at each partition is set at such elevation that about 80% of service area is situated below that elevation. (Refer to Figure B.III-3).

B. Required Effective Head

After considering size of distribution unit and pressure required at each house, the following water head has been given in the study at the inter-connection point of each distribution unit. (Refer to Figure B.III-3).

Islamabad	:	120 ft
Rawalpindi	:	100 ft

C. Maximum Hourly Water Supply

A ratio of maximum hourly water supply to average hourly water supply at the time of maximum daily water supply generally tends to decrease as maximum daily water supply increases (refer to Figure below). Due to habitual water shortage in the Project Area, no record on the maximum hourly water supply is available. Hence, proposed maximum hourly water supply has been estimated basing on the proposed maximum daily water supply in 2000 and the ratio of maximum hourly water supply to the average daily water supply illustrated in Figure below, in which Amount the ratio was assumed to be 1.5 for conservative estimation. Amount of water distributed to each unit is shown in Figures B.III-3 as well as in Tables B.III-3 and B.III-4.

Maximum hourly water supply is expressed by the following equation:

$$q = K \times Q/24$$

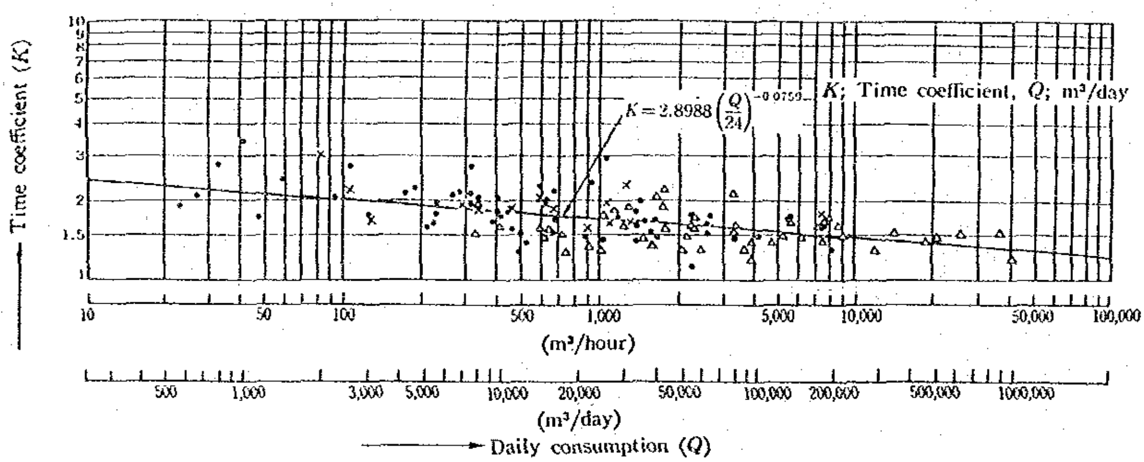
Where, q = maximum hourly water supply (cu.m/hour)

Q = planned maximum daily water supply (cu.m/day)

K = hourly coefficient

= the ratio of maximum hourly water supply to average hourly water supply

K value might be obtained to be 1.363 from Figure. For the project planning purpose, K value of 1.5 has been proposed.



[Notes] ● Residencial area
× All or partly high and middle residencial area
△ All or partly commercial area

Relations between Daily Consumption and Time Coefficient in 19 Cities 129 Service Areas

D. Zoning of Service Area

Ground elevations in the service area vary with a relatively wide range of 400 ft from 1,600 ft to 2,000 ft above mean sea level. In this case it is advantageous to divide the distribution area into two zones, namely high zone and low zone, in order that running cost could be reduced in case of boosting up, pipeline structure would be more economic and safe due to deduction of water pressure, and that leakage losses of water would be less and operation & maintenance works would become easy as the fluctuation of distribution pressure decreases.

Proposed sites of service reservoirs for Islamabad are shown in Figure B.III-4, marked as (A), (B) and (C). Giving a hydraulic gradient of 1/500 to the main distribution lines, designed water levels at the respective service reservoirs are indicated also in Figure B.III-4. The site (A) corresponds to Shah Allahditta treatment plant, and the designed water level at (A) shall be around EL. 2,050 ft depending on the topographic condition of the site, while EL. 2,020 ft at the site (B). For the Sectors where need designed water levels of more than EL. 2,020 or 2,050 ft lifting of water is necessitated. The low zone that can receive water by gravity from the respective treatment plants includes sectors of F-11, G-11, H-11, I-11, H-12, and I-12, each of which is about 50 ft lower in elevation than that of other sectors. So far as elevation is concerned, both sectors of G-14 and G-15 can be classified into low zone, however, these sectors are classified into high zone because they are located far from the service reservoirs, and hence they need high construction cost and complicated distribution networks.

Topography of Rawalpindi is generally flat, excepting for some elevated area. Zoning was made depending on the available water head at service reservoirs concerned that is determined by topographic conditions.

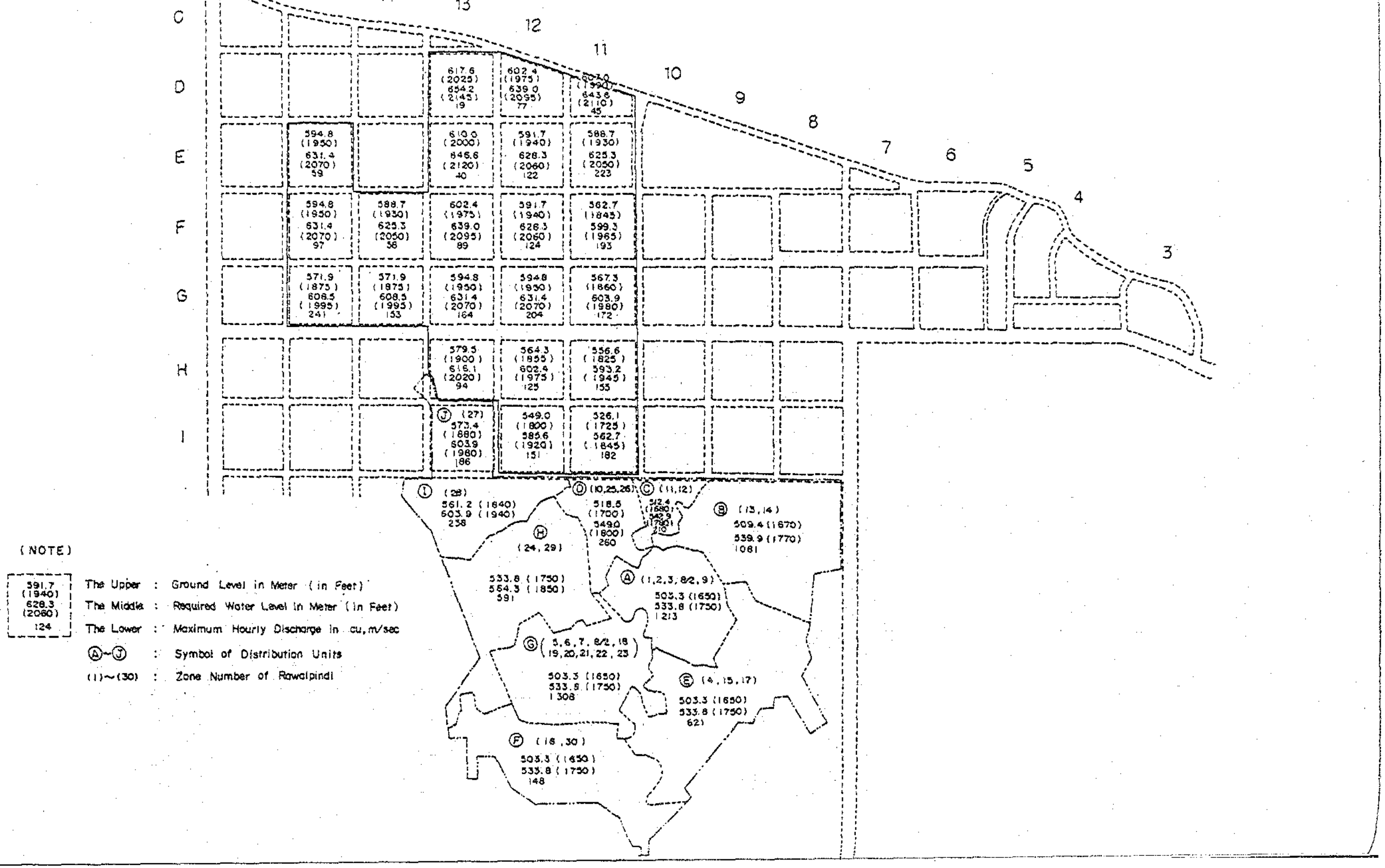
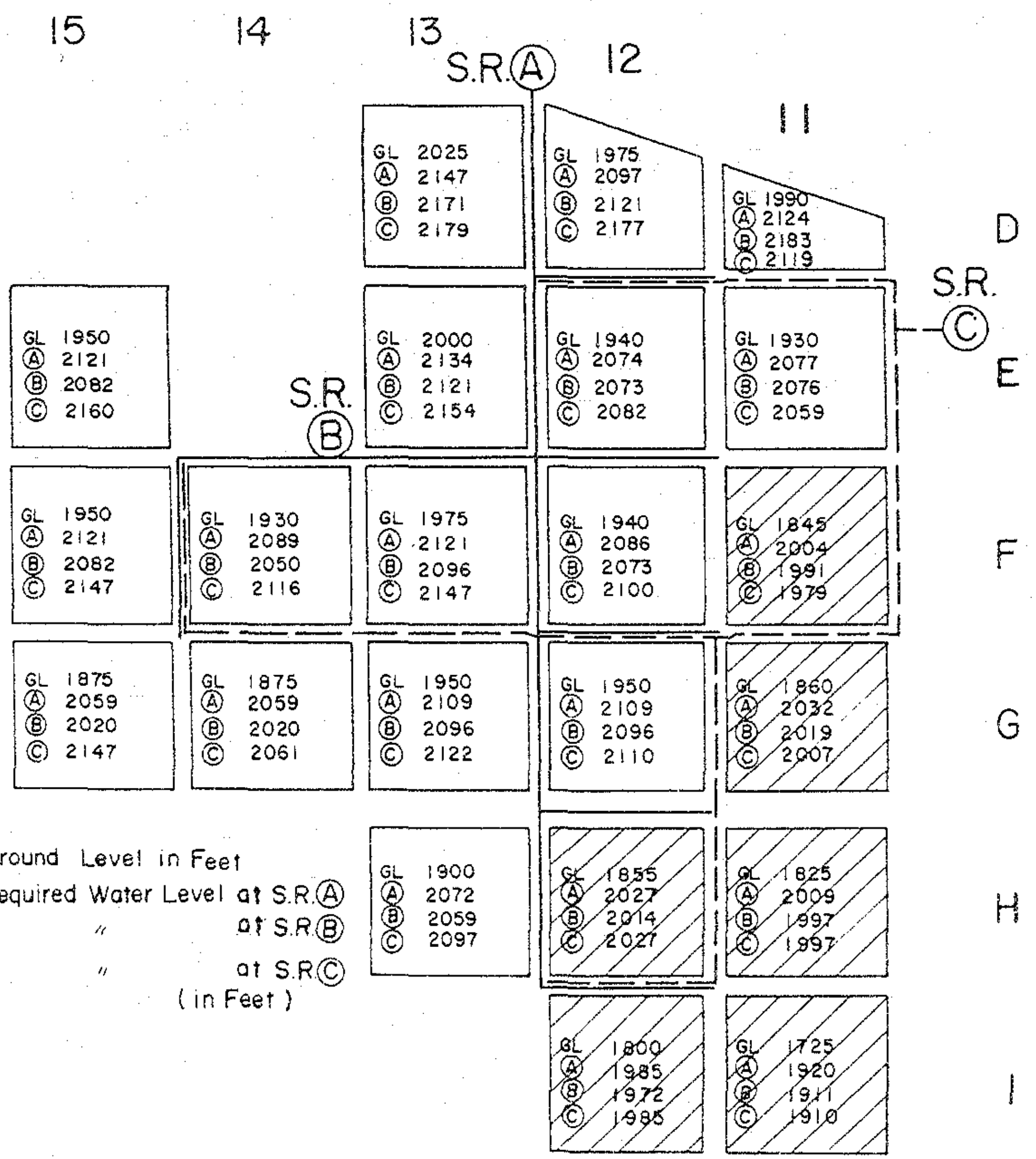


FIGURE B.III-3 GROUND LEVEL, REQUIRED WATER LEVEL AND MAXIMUM HOURLY DISCHARGE OF DISTRIBUTION UNITS

FIGURE B.III-4 **ZONING OF ISLAMABAD SERVICE AREA**



LEGEND

GL : Ground Level in Feet
(A) : Required Water Level at S.R.(A)
(B) : " at S.R.(B)
(C) : " at S.R.(C)
(in Feet)

Table B.III-3 ISLAMABAD WATER SUPPLY IN THE YEAR OF 2000
(Distribution of Khanpur Dam Water)

Sector	(1) Max. Daily Demand (cu.m/D)	(2) Percent	(3) Max. Daily Discharge (cu.m/D)	(4) Max. Hourly Discharge (cu.m/D)
D-11	2,345	1.62	2,600	0.045
E-11	11,631	8.02	12,870	0.223
F-11	10,064	6.94	11,137	0.193
G-11	8,953	6.17	9,901	0.172
H-11	8,054	5.55	8,906	0.155
I-11	9,474	6.53	10,479	0.182
D-12	4,003	2.76	4,429	0.077
E-12	6,359	4.39	7,045	0.122
F-12	6,443	4.44	7,125	0.124
G-12	10,615	7.32	11,747	0.204
H-12	6,512	4.49	7,205	0.125
I-12	7,855	5.42	8,698	0.151
D-13	972	0.67	1,075	0.019
E-13	2,091	1.44	2,311	0.040
F-13	4,643	3.20	5,135	0.089
G-13	8,530	5.88	9,436	0.164
H-13	4,887	3.37	5,408	0.094
E-14	3,002	2.07	3,322	0.058
G-14	7,951	5.48	8,794	0.153
E-15	3,054	2.11	3,386	0.059
F-15	5,025	3.47	5,568	0.097
G-15	12,550	8.66	13,896	0.241
Total	145,013	100.00	160,473	2.787

$$(2) = \frac{(1)}{\text{Total } (1)} \times 100$$

$$(3) = 33 \text{ MGD} \times \frac{4.55}{1,000} \times 0.95 \times 0.90 \times 1.25 \times \frac{2}{100} = 1,604.73 \times (2)$$

$$(4) = (3) \times 1.5/86,400$$

Table B.III-4 RAWALPINDI WATER SUPPLY IN THE YEAR OF 2000
(Distribution of Khanpur Dam Water)

	Zone	Sector	(1) Max.Daily Demand M.D.D. (cu.m/D)	(1) Max.Daily Demand Total (cu.m/D)	(2) Percent	(3) Max.Daily Discharge (cu.m/D)	(4) Max.Hourly Discharge (cu.m/S)
R	A	1	28,938	101,347	43.9	69,898	1.213
		2	32,804				
		3	22,656				
		8/2	6,663				
		9	10,286				
M	B	13	79,220	90,274	39.1	62,255	1.081
		14	11,054				
C	C	11	9,694	17,373	7.6	12,101	0.210
		12	7,679				
	D	10	18,226	21,676	9.4	14,967	0.260
		25	931				
	Sub-total		230,670 (47.2%)	230,670	100.0	159,221	2.764
C	E	4	9,434	51,078	20.1	35,800	0.621
		15	14,788				
		17	27,656				
B	F	16	807	12,357	4.8	8,549	0.148
		30	11,550				
	G	5	9,879	109,277	42.3	75,341	1.308
		6	3,246				
		7	8,353				
		8/2	6,662				
		18	3,250				
		19	66,870				
		20	3,759				
		21	654				
		22	5,234				
		23	1,370				
	H	24	44,645	49,493	19.1	34,019	0.591
		290	4,848				
	I	28	19,821	19,821	7.7	13,715	0.238
	J	27	15,609	15,609	6.0	10,687	0.186
	Sub-total		258,435 (52.8%)	258,435	100.0	178,111	3.092
	Total		489,105 (100%)	489,105		337,333*	5.856

* 69.37 MGD x 1.25 x 0.95 x 0.90 x 4.55 x 1,000 = 337,333 cu.m/D
(4) = (3) x 1.5/86,400

CHAPTER IV. ALTERNATIVE STUDY

CHAPTER IV. ALTERNATIVE STUDY

4.1. General Description

4.1.1. Given Conditions and Study Procedures

A. General

The comparative study of Khanpur water supply systems was carried out based on review of existing report concerning project, data and information collected and following conditions/procedures in order to verify least cost water conveyance system, water treatment plant and other appurtenant structures related including operation and maintenance cost.

B. Given Conditions

a. Available Maps

Design capacities are 150 MLD (33 MDG) for Islamabad and 316 MLD (69.37 MGD) for Rawalpindi as annual averaged water demands, respectively. Possible conduction main routes are existing Left Bank Canal up to the Nicholson Monument and new construction of tunnels and pipe lines.

Topographic maps so far collected are as follows:

Scale 1:50,000 :	Entire project are concerned
Scale 1:21,120 :	Majority of project area except north-eastern part of Margala hill.
Scale 1: 4,000 :	Mainly Islamabad city development area
Scale 1: 6,000 :	In and around Khanpur dam site

In addition to the above, aerophotograph with a scale of 1:20,000 in the project area are available.

b. Basic design discharge

Basic design discharges releasing from the Khanpur reservoir are defined as below:

Islamabad	:	33 MGD x 1.25* = 41.25 MGD
		≒ 2.17 cu.m/sec
Rawalpindi:	:	69.37 MGD x 1.25 ≒ 86.71 MGD
		≒ 4.57 cu.m/sec
Total discharge	:	102.37 MGD x 1.25 ≒ 127.96 MGD
		≒ 6.74 cu.m/sec

Note: * Design discharge in summer peak is assumed to be 1.25 time of annual average water demand

c. Design discharge of major facilities

(1) From Khanpur dam up to the receiving well of water treatment plant.

Islamabad :	2.17	cu.m/sec
Rawalpindi:	4.57	"
Total :	6.74	"

(2) At receiving well of water treatment plant

Canal, conduit and tunnel conveyance water losses are assumed to be about 5 percent of total discharge.

Islamabad :	2.17 x 0.95 ≒	2.06	cu.m/sec
Rawalpindi:	4.57 x 0.95 ≒	4.34	"
Total		6.40	"

(3) Treated water from the clear water reservoir at the plant

Water losses at water treatment plant as washing sand filter and wastage are assumed to be about 10 percent of inflow discharge at receiving well.

Islamabad :	2.06 x 0.90 ≑ 1.85	cu.m/sec
Rawalpindi:	4.34 x 0.90 ≑ 3.91	"
Total :	5.76	"

C. Study Procedures

The study has been conducted based on the following procedures.

1st Step: Data collections and review of existing reports were conducted in initial stage of the survey period.

2nd Step: Field reconnaissance was carried out to get general information of the project area and existing facilities concerned.

3rd Step: Paper location layout was also prepared based on the results of review of existing report, field reconnaissance and collected data/information.

4th Step: Detailed field survey and inspection was carried out to confirm actual field conditions and geological constraint and to re-examine technical consideration as well as economic aspects.

5th Step: Most suitable alternative plans were set up based on paper location and field survey taking into consideration topographic, geological, hydraulic conditions, construction manners and economic point of view.

4.1.2. Approach to Selection of Conduction Route

A. General

The conduction mains proposed for the urban water supply systems would be of a closed type such as pipe conduits, tunnels, syphons and so on, because it prevents the supply system from dust fall, proliferations of water weeds and injury by cattle, and is easy to operate and maintain. Most of conduction mains will be of low pressure pipes for its low cost. Tunnels will be constructed to penetrate Margala hills to shorten the route and to conserve the head. Syphons will be proposed to cross river channels.

In selecting the routes of conduction mains every endeavour has been made to shorten the length of route as much as possible, taking into consideration the interrelation of tunnel length and pumping lift that lift required for pumping decreases as length of tunnels increases, as well as topography and geology.

B. Distribution of available head

In designing hydraulic gradient of the conduction mains being composed of three elements of low pressure pipes, tunnels and syphons, careful examination was made in order to minimize the cost of the conduction mains by optimizing the distribution of the available head among component portions. The higher the drop, the smaller the conduit, and the lower the cost, however, velocity under as given head must fall within a technically reasonable range. Pumping of water is inevitably required for the water supply system due to limited available head.

Standard velocity for conduit component is given as follows;

Conduit		Velocity (m/s)	Hydraulic Gradient
Pipe,	@1,650 mm	Approx. 1.60	1:800
	@1,500 mm	1.90	1:500
Tunnel,	D2,100 mm	1.80	1:650
	D2,400 mm	1.50	1:1,500
Siphon,	D2,000 mm	2.20	1:600

C. Tunnel

a. Water depth

For a tunnel to connect directly Khanpur reservoir with a water treatment plant, because there are no dust fall, nor inflow of rainstorm, water depth is designed to be 0.9 x D (tunnel diameter) with a slight allowance against the depth of 0.94 x D that gives maximum discharge.

b. Length

As the length of tunnel increases, the time of construction becomes long and the construction cost per unit length becomes expensive. Furthermore, in case of accident, restoration works shall need not only time but also difficult countermeasures, and resulting in high costs, when compared to short tunnels. Although it differs depending upon the size of tunnel, geological conditions, and volume of groundwater, optimum length of a tunnel, when drived from one side, may generally falls within 1.0 to 1.5 km. When the length is more than 1.0 - 1.5 km, careful studies shall be made whether provision of an inclined or vertical shaft is advantageous to the project economy, or not.

When proposed tunnels are connected with an intake tower to be constructed in Khanpur reservoir, inner pressure of tunnels will be as high as 3.1 kg/sq.cm, which shall cause increase in construction cost to provide high pressure grouting and reinforcement. Therefore, in order to save the construction costs of tunnels, generally, structure of tunnel is proposed to be of a free flow type.

D. Pipeline

Conduction mains are composed of a head regulator, short length culverts, tunnels, syphons and a pumping station, and long pipelines after lifting. The conduction mains of a closed type are designed for pressure flow that has the following advantage

i) It permits quick delivery of water on demand because there is no time lag in conveyance for on-off operation of closed pressure conduits, while there is a long time lag in free flow conduits.

ii) Easy operation and maintenance and less trouble could be expected because they will not need any gates nor will they need a structure to be overflowed.

iii) Given about 1.0 kg/sq.cm of maximum inner pressure, the construction cost of closed pressure conduits shall be almost same as the costs of free flow conduits.

Constituent units are steel, ductile, PRCC with steel core. Tunnels are best designed to the full projected capacity of the system. This is not necessarily so for pipelines. Parallel lines will be built according to the time phase of the Project. Twin lines will be laid for large capacity pipelines in order to overcome possible failure of pipelines.

E. Pumping station

The volute type of horizontal or vertical shaft is adopted for pumping stations. Number of units of pumps is decided in consideration of characteristics of urban water supply, changes in water demand, interchangeability of spare parts, operation and maintenance, technical and economical advantage, and supply of spare parts. For large scale pumping stations, six units of pumps will be installed including one stand-by unit.

4.2. Alternative Plan and Preliminary Design

4.2.1. Alternative Plan

A. General

Based on the design criteria and general description mentioned in Chapter III and IV of this appendix, three alternative plans on the water conduction main and appurtenant facilities have been basically proposed as follows:

Alternative I

i. The major purpose of this plan is to make the fullest use of the existing Left Bank Canals; multipurpose water conveyance canals for irrigation, industry and urban water, the downstream portion of which was constructed for exclusive use for urban water.

ii. The project could be completed in a short period so as to meet the urgent water needs in Rawalpindi.

iii. The proposed water conduction system starts from the end of the existing canals near Nicolson Monument, lifts water at Sang Jani, and terminates at the beneficiary areas.

Alternative II

i. The major purpose of this plan is to make the better use of the existing multipurpose canals.

ii. As a result of the study, the route that takes water just downstream of Khanpur reservoir has been selected, which has a shorter length among several possible plans in the Alternative II.

iii. The conduction main conveys water by means of tunnel and pipeline, after lifting it at the left bank of Haro river, to the beneficiary areas via Shah Allah Ditta.

Alternative III

i. The major purpose of this plan is to connect the beneficiary areas with Khanpur reservoir by the shortest route of tunnel and pipeline.

ii. Potential water head can be conserved, and as a result water can be conveyed by gravity to the beneficiary areas to a maximum extent.

iii. The conduction main directly diverts water at Khanpur reservoir to convey it to the beneficiary areas via Golra.

Each of the alternatives mentioned above was sub-divided into two to four, and eight alternatives in total were finally warranted for further studies as discussed in the succeeding sections.

B. Alternative I

Water supply system arrangement of the Alternative I starts from the end point of existing Left Bank Canal near Nicholson Monument, reaching the twin cities of Islamabad and Rawalpindi through water treatment plant and pumping station.

Special consideration shall be made on the improvement of existing Left Bank Canal for the proposed alternative I.

The Left Bank Canal was constructed under the Khanpur Project for multiple use of urban water supply, industrial water supply and irrigation. Hugging the foot of Margala hills, the Left Bank Canal is subjected to inflow of rainstorms and soils resulting from rainstorms.

When the existing Left Bank Canal is used for urban water supply, every effort must be made to reduce outbreaks of accident in conveyance systems, for which there are two possible countermeasures. One is to prevent the systems from accidents through the rehabilitation and improvement of the system, together with careful operation and maintenance of the systems. Another countermeasure is to provide at the end of the Left Bank Canal a raw water reservoir with a storage capacity enough to meet requirements for several days.

In this project, the following works for the rehabilitation and improvement of the Left Bank Canal is proposed;

(1) Canals that were constructed in deep cut of more than 10 m height near a tunnel, will be remodeled into a culvert or be covered with reinforced cement concrete plates.

(2) Side slope in cut of more than 4 m height will be protected with stone masonry or cement concrete blocks for the height equivalent to 1/3 to 2/3 of the cut height.

(3) Mortar spraying will be made to such side slopes of rock as are exposed to weathering.

(4) Shoulder ditches will be provided for the reaches where rain water flows in, and lined with cement concrete blocks in case of sharp slopes.

(5) Berm ditches in long reaches in cut will be lined with cement concrete.

(6) Sand traps will be constructed at the berm inlets of the existing lined canals and the canals proposed to be improved under item(5).

(7) Vegetation will be made to side slopes of soils.

In addition to the rehabilitation and improvement of the Left Bank Canal, construction of a raw water reservoir is proposed, in order to make provision for unforeseen trouble in the canal systems, because complete prevention of accidents is impossible, even though the improvement works are undertaken. Furthermore, for convenience of operation and maintenance and to assure the safety of canals, cross regulators will be provided as follows;

(1) An over flow type by pass will be annexed to the existing regulating works at the diversion site for the Left Bank Canal.

(2) Gates will be installed in the escape fall of the Haro river syphon.

(3) An over flow type bypass and a spillway for release of flood water caused by rainstorm will be added to the regulating gate of Mohra Morado tunnel.

(4) An over flow type spillway will be added to the outlet works of Karam Whal tunnel.

Those works are involved in the rehabilitation and improvement of the existing canal system for urban water supply purposes, and are applied to the reaches between Khanpur dam and a proposed diversion site to Islamabad and Rawalpindi. Details are shown in Table B.IV-1.

a. Sub-alternative plan

Three sub-alternatives for this category were set up based on the topographic conditions and location of beneficiaries area concerned. Brief descriptions for this are as belows:

Alternative I-A: (Sang Jani-Shah Allah Ditta-Islamabad and Rawalpindi)

Distribution point: Near Shah Allah Ditta
Facility: Raw water reservoir, water treatment plant, pumping station, pipeline, discharge pool and service reservoir

Alternative I-B: (Sang Jani-Tirnaul-Islamabad and Rawalpindi)

Distribution point: Near Tirnaul
Facility: Same as Alt. I-A.

Alternative I-C: (Sang Jani-Shah Allah Ditta and or Tomar-Islamabad and Rawalpindi)
Distribution point: Sang Jani
Facility: Same as Alt. I-A.

b. Optimum water level for service reservoir

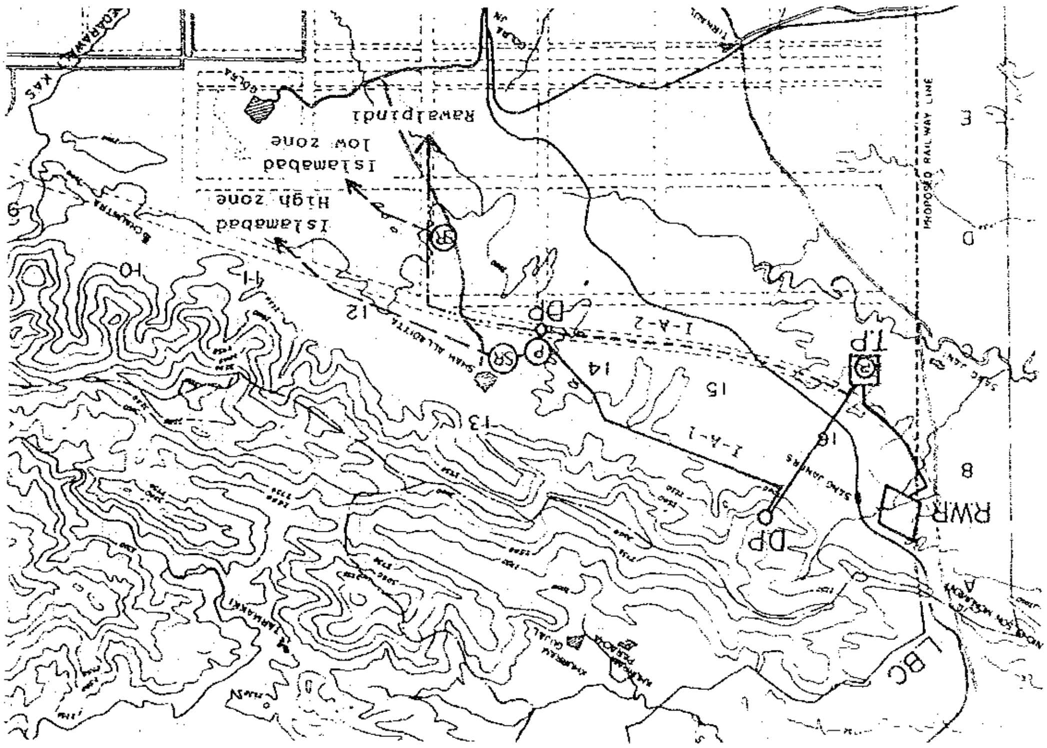
As a result of hydraulic computation and supply pipeline alignment depending upon the basic concepts the following could be obtained as optimum water levels of respective reservoirs:

Unit: Meter (feet)

Alternative:	High Water Level	Low Water Level
Alt. I-A		
- Islamabad		
High zone	664.3 (2178)	658.3 (2158)
Low zone	620.2 (2034)	614.2 (2014)
- Rawalpindi G-13	605.5 (1986)	599.5 (1966)
Alt. I-B		
- Islamabad		
High zone	664.3 (2178)	658.3 (2158)
Low zone	624.1 (2046)	618.1 (2026)
- Rawalpindi Tomar	581.8 (1908)	575.8 (1888)
Alt. I-C		
- Islamabad	664.3 (2178)	658.3 (2158)
High zone	620.2 (2034)	614.2 (2014)
- Rawalpindi Tomar	581.8 (1908)	575.8 (1888)

c. Alternative I-A.

Water supply pipelines along Margala hills foot which convey water to Islamabad low zone and Rawalpindi by gravity flow have two possible routes as illustrated in the following figure.



- Alternative I-A-1:

The pipeline route will pass through hilly side from discharge pool to the service reservoir of Islamabad low zone and Rawalpindi after pumping up to discharge pool with EL 640 m line from Sang Jani Pumping Station.

- Alternative I-A-2:

Pipeline route for this alternative will be aligned along with Capital Boundary road of Islamabad from Sang Jani Pumping Station up to service reservoirs of Islamabad low zone and Rawalpindi directly.

Results of comprehensive comparison

- Advantages and disadvantages of Case I-A-1

(1) It requires slightly large number of appurtenant structures such as bend pipes, air valves and pipe anchor concrete due to complicated topo-conditions and rock bed foundation of pipes.

(2) Construction works, especially rock excavation and pipe installation, will be rather difficult than Alt. I-A-2. Furthermore, temporary access road and operation and maintenance road will be required during and after project.

(3) Total length and weight of supply pipeline are slightly larger than Alt. I-A-2.

Alternative	Pipe Length	Weight
I-A-1	9.0 km	7,900 ton
I-A-2	6.4 km	6,100 ton

- Advantages of Case I-A-2

  (1) Pipeline alignment along with Capital boundary road is most suitable for future operation and maintenance and easy construction works in case that these road construction is completed in advance.

Therefore, case I-A-2 is recommendable for Alternative I-A.

Layout of respective alternatives are illustrated in Figure B.IV-1, B.IV-2 and B.IV-3. Flow diagrams of I-A, I-B and I-C attached in Figure B.IV-4.

C. Alternative II

a. General description

Water supply systems arrangement of the Alternative II is basically from the intake points of Existing Left Bank Canal and or Khanpur Reservoir to twin cities of Islamabad and Rawalpindi through Shah Allah Ditta water treatment plant.

Four sub-alternatives for this category are set up as follows:

- Alternative II-A: (L.B.C - Khuram Paracha-Shah Allah Ditta)
  Intake Point: L.B.C. at RD 40,800
  Raw water reservoir: Approx. 1.1 MCM
  Facilities: Feeder conduit, pumping station, pipeline, tunnel, water treatment plant and service reservoir

- Alternative II-B: (L.B.C. - Julian-Shah Allah Ditta)
  Intake Point: L.B.C. at RD 13,500
  Raw water reservoir: Approx. 0.55 MCM
  Facilities: Feeder canal, pumping station, pipeline, tunnel, conduit, water treatment plant and service reservoir

- Alternative II-C: (Head regulator R & L.B.C or L.B.C. - Tarmakki-Shah Allah Ditta)

  Intake Point: Head regulator R & L.B.c. or L.B.C. at RD 9,100

  Facilities: Feeder conduit, tunnel, syphon, pumping station, pipeline, diversion dam, water treatment plant and service reservoir

- Alternative II-D: (Khanpur dam-Tarmakki-Shah Allah Ditta)

  Intake Point: Khanpur reservoir

  Facilities: Intake pit and pumping station, pipeline, tunnel, diversion dam, water treatment plant and service reservoir

Special consideration shall be made on the construction of Tarmakki intake works for this alternative. In Alternative II-C and II-D, as will be discussed in this section, the proposed route of conduction mains closely approaches to Nilan Kas which is one of the tributaries to drain into Khanpur reservoir. The proposed water level of the conduction mains at this point is EL. 642.0 m (2,105 ft), while the elevation of river bed of Nilan Kas is EL. 640.5 m (2,100 ft). Alternative II-C and II-D convey water to Shah Allah Ditta through Tarmakki, after lifting water at Khanpur reservoir. Tarmakki intake works that divert the river flow of Nilan Kas, being located at elevated point higher than Khanpur reservoir, will save electric costs required to operate proposed pumping plants at Khanpur.

The catchment area of Nilan Kas at Tarmakki is 53 sq.km or corresponding to 7% of the catchment area of Khanpur reservoir of 798 sq.km (308 sq.mile). The intake weir will be seven meters high and 80 m long. the construction costs are Rs. 8.34 million, including canals and a silt basin, while electric costs to be saved will amount to Rs. 2.33 million per year. This may proves that the construction of Tarmakki intake works is economically feasible.

b. Alternative II-A

This alternative is further sub-divided into two plans. Summary of facilities concerned and water level for both plans are shown in Figure B.IV-5 and Table B.IV-2.

Results of comprehensive comparison:

- Disadvantages of cases II-A-2 are;

(1) Pump lifting head is about 18 m higher than cases II-A-1.

(2) Pump delivery pipeline is about 150 m longer than II-A-1.

(3) Inlet and outlet of tunnel are not necessarily suitable for construction works due to flow down of spring water near inlet and extremely narrow construction space at outlet tunnel.

- Disadvantage of Case II-A-1 is that the length of pipelines is 100 m longer than Case II-A-2 of tunnel construction.

Therefore, Case II-A-1 is selected as a suitable plan of Case II-A.

c. Alternative II-C

This alternative is sub-divided into three plans depending upon intake method and construction of diversion dam at Tarmakki as supplemental water supply to conduction main. Summary of facilities and water level for respective plans are shown in Figure B.IV-5 and Table B.IV-3.

Result of comprehensive comparison:

- Disadvantages of Case II-C-1 are;

    (1) Intake point from L.B.C. is just downstream of outlet of Haro river syphon. Total head losses from dam intake up to the proposed intake point are some 19.7 m with 2.9 m length due to topo-condition and drop structures constructed.

    (2) Total lifting head of pumps is about 17 m higher than Alt. II-C-3.

- Disadvantages of Case II-C-2 are;

    (1) Same as item (1) of Alt. II-C-1.

    (2) Total lifting head of pumps is about 19 m higher than Alt. II-C-3.

- Disadvantage of Case II-C-3 is only total length of conduction main proposed. These are 1.2 km and 0.4 km longer than the Alt. II-C-1 and Alt. II-C-2, respectively.

However, differences of operation cost of pump are 5.7 million rupees for Alt. II-C-1 and 6.7 million rupees for Alt. II-C-2, respectively. On the other hand, increasing construction costs compared to two alternatives are about Rs. 21.1 million for Alt. II-C-1 and Rs. 7.1 million Rs. for Alt. II-C-2 respectively.

Alternative II-C-3 is recommendable as an optimum plan of case II-C accordingly.

d. Alternative II-B and II-D

Alternative II-B and II-D are possible ones. Result of comparison for Case-II are summarized in Table B.IV-4.

Layout and flow diagram of supply system for alternative-II are given in Figure B.IV-6 and B.IV-7. Summary of major conduction main are also shown in Table B.IV-4.

D. Alternative III

a. General description

Water supply systems arrangement of the alternative III is fundamentally from the intake point of the existing diversion work and or Khanpur reservoir to beneficiary area through long tunnel under Margala hill and Golra water treatment plant.

Two sub-alternatives for this category are considered as below:

- <u>Alternative III-A:</u> (Existing diversion work-Tarmakki and Golra Shaft-Golra)

  Intake point: Existing diversion work

  Facility: Conduit, syphon, pipeline, tunnel and vertical shaft, pumping station, water treatment plant and service reservoir

- <u>Alternative III-B:</u> (Intake tower on Khanpur Reservoir - Tarmakki and Golra Shaft-Golra)

  Intake point: Khanpur reservoir

  Facility: Intake tower, pipeline, tunnel and vertical shaft, pumping station, water treatment plant and service reservoir

The sub-alternative III-B will be further divided into two plans such as with two vertical shaft for Alt. III-B-1 and with one vertical shaft for Alt. III-B-2 respectively.

b. Alternative III-B

Summary of facilities concerned and water level for both two plans are shown in Figure B.IV-8 and Table B.IV-5.

Results of comprehensive comparison:

- Disadvantages and advantages of Case III-B-2 are;

a) Disadvantages

(1) Construction period of long tunnel would be one and half years longer than Alt. III-B-1.

(2) There are some unexpected factors such as drainage of spring water and its rehabilitation works, prevention and recovery of such accident.

b) Advantages

(1) Total length of conduction main is about 500 m shorter than Alt. III-B-1.

(2) There are no first vertical shaft with 97 m height.

As a result of the comparison, Alt. II-B-1 is slightly advantageous than Alt. III-B-2.

c. Alternative III-A

Summary of facilities and water level comparison for two plans are also shown in Figure B.IV-8 and Table B.IV-5.

Results of comprehensive comparison:

- Advantages and disadvantages of Case-III-B are;

a) Advantages

(1) Total length of conduction main and cost are slightly smaller than Alt. III-A.

(2) Water level of receiving well at Golra water treatment plant is about 5.1 m higher than Alt. III-A. Operation cost at Golra pumping station can be saved approximately 1.6 million rupees per annum.

b) Disadvantages

(1) Intake tower and pressure breaking facility shall be constructed in Khanpur reservoir.

(2) Remaining technical matters for the construction are almost same level.

Therefore, the recommendable alternative for long tunnel systems is Case-III-B, accordingly.

Layout and flow diagram of alternative III are shown in Figures B.IV-7 and B.IV-9, respectively.

4.2.2. Preliminary Design of Facilities

A. Raw Water Reservoir

Proposed raw water reservoirs are located at Sang Jani (for Alt. I-A, I-B, I-C), Khuram Paracha (Alt. II-A) and Julian (Alt. II-B) sites and their effective regulating capacities are as below:

Name of Reservoir	Capacity
Sang Jani	1.10 MCM (Two days)
Khuram Paracha	1.10 MCM (Two days)
Julian	0.55 MCM (One day)

A typical section of the reservoir is as shown in Figure B.IV-10.

Geology around the proposed Sang Jani Raw Water Reservoir consists mainly of silty clay with intercalated gravel beds. Silty clay is massive and compact, and its moisture content varies dry to wet. It is classified into CL-ML in terms of the Unified Soil Classification showing similar grain size distribution at any place with little amount of coarse grains except concretion. It is available for the impermeable material and its amount is sufficient for construction of the dam, whereas it is needed to protect the surface of dam because the clay is subjected to erosion. Any sealing treatment over the bottom of the reservoir is needed because the gravel layers may be permeabile. Geology and foundation at Khuram Paracha and Julian are smillar to Sang Jani.

B. Feeder Facility

Feeder facilities consist of head and cross regulators, feeder conduit, syphon, intake tower, diversion dam and improvement and rehabilitation works of Left Bank Canal. Layout plan of intake tower is illustrated in Figure B.IV-11.

C. Tunnel

a. General description

Short and long tunnel plans are proposed as follows:

Alternative	No. of Route	Total length m (mile)
II-A	1.0	2,600 (1.62)
II-B	2.0	6,200 (3.85)
II-C	3.0	5,510 (3.42)
II-D	1.0	5,000 (3.11)
III	2.0	11,700 (7.27)

Alternative II-B, II-C and II-D require an inclined shaft and Alternative III requires two vertical shafts. The diameters of tunnel of standard horse shoe type for Alternative II and of pressure tunnel for Alternative III are 2.10 m (7 feet), and that of free flow tunnel for Alt. III is determined at 2.4 m (8 feet) taking into account the workability as well as economic advantages. Typical cross-section and construction schedules are as per Figure B.IV-12 to B.IV-15.

b. Type of tunnel

Three tunnel types have been established for the study as given in Table B.IV-6, in which characteristic and standard measures of rock load and necessity and form of tunnel supports are also indicated for each tunnel type. As is explained in detail in the section of geology, rock classification is categorized into, including unconsolidated sediment layer, four types. Tunnel type suitable for each category of rock classifications is examined through field investigations as below;

Rock Type	Tunnel Type
Type I	Type A : 100%
Type II	Type A : 50%, Type B: 50%
Type III	Type B : 30%, Type C: 70%
Type IV	Type C : 100%

In general the fault zone is assumed to be several to ten several meters in thickness and, in many cases, organized by several adjoining layers of fracture. The thickness of the fractured fault zones are estimated at 20 meters in the study per location that is observed in the field or on the 1:20,000 scale aerophotos.

Among these fractured fault zones, those observed at Tarmakki and Shah Allah Ditta are worthy of special mention from geological point of view. Although it is possible to confirm its scale and surface structure at the site for Shah Allah Ditta, it is impossible for Tarmakki to investigate in detail because of well coated vegetations, and hence it is recommendable to be further examined by means of seismic survey and core boring. For the on-going feasibility study the thickness of the fractured faults zone was assumed as 50 meters for both Shah Allah Ditta and Tarmakki.

Engineering features of the fault zone vary in a wide range with slight to heavy earth pressure and, on the occasion of very high earth pressure during implementation, establishment of a different type of tunnel in consideration of increase of lining thickness, additional steel reinforcemnt and so on, would be needed.

c. Preparatory works for fractured fault zone

Existence of several fracture zones has been investigated for each alternative plan of tunnel. Among these fracture zones, those situated at Tarmakki vertical shaft, Bol inclined shaft and outlet of Shah Allah Ditta tunnel are important from geological structural point of view.

Alternative plans II-A, II-B, II-C and II-D pass two fracture zones at Bol and Shah Allah Ditta. Since earth covering is as shallow as 20 to 40 meters and the inclined shaft or outlet of tunnel is located near from those points, amount of spring water from the excavated surface of pit would relatively be small and method of sealing such water could be simple.

However for Alternative plan III, thickness of stratum aquifer as well as amount of spring water would be great due to the fact that the thickness of earth covering is as large as 100 to 200 meters at the passing point of these fracture zones. In the case tunnel excavation meets with such zones, spring water would increase in volume and continue for a longer period. These waters are required to be drained out through Tarmakki vertical shaft (97 meters in depth) or Golra shaft (59 m deep) and when the case of a large quantity of water extending over along period of time happens, tunnel excavation works would be discontinued causing not only prolongation of construction period but also considerable expenses for sealing works and for other miscellaneous treatment works.

These fracture zones should therefore be examined in detail by means of seismic survey and core boring to grasp their scales, structures and engineering characteristics, and to pass in safety these points, preparatory works for excavating such fractured fault zones should be preceded. Among various methods available, taking into account that earth covering is as deep as 100 to 200 meters, horizontal boring and chemical grouting method from pit-side is recommended.

There are three major faults as mentioned above. Among these, tunnel routes of Alternative II-A, Alternative II-B, C, D and Alternative III go across one, two and three faults respectively. Dimensions and conditions of these faults are not clear, but much care should be taken in excavating the faults because they are rather big from the aspect of structural geology and much earth pressure may occur.

In the clear days, there flows no water in the valley of Margala Hills except big ones and almost groundwater discharge occurs as springs around hills. This fact suggests the presence of routes in the mountain body, through which rain water is collected to springs. This means that if a tunnel route encounters and cuts this routes, issue of a considerable amount of groundwater may occur. Therefore, it is necessary to prepare some sealing method before excavation. Especially tunnel of Alternative-III have more possibility to encounter big issue of groundwater because it runs across under Nilan Kas river and under bigger mountain body.

Tunnel excavation may affect amount of spring water in Khurram Gujar on Alternative II-A, and in Shah Allah Ditta on Alternative II-B, C and D.

d. Designed tunnel type for each alternatives

Proposed types of tunnel cross section and length based on the design criteria and tunnel geology are summarized as below.

(Unit: m)

Alternative	Type A	Type B	Type C	Total
II-A	1,555	543	502	2,600
II-B	3,200	1,860	1,140	6,200
II-C	2,970	1,446	1,094	5,510
II-D	2,715	1,191	1,094	5,000
III	5,060	3,906	2,734	11,700

D. Water Treatment Plan

a. Facilities and capacity

Dimensions, specifications etc. of the major facilities of the treatment plant are proposed in accordance with the design criteria described in the preceding chapter. Figure B.IV-17 and B.IV-18 show the layout and the flow diagram of the proposed treatment plant.

1) Receiving well

a. No. of wells : 1 well

b. Dimensions : W 8.0 m x L 15.0 m x D 5.0 m = 600.0 cu.m.

c. Retention Times: T = 600 x 1/383.8 = 1.6 min

2) Mixing well

a. No. of basins : 4 wells

b. Dimensions : W 6.0 m x L 6.0 m x D 5.0 m
= 180.0 cu.m

c. Retention time : T = 180.0 x 4/383.8 = 1.9 min

3) Flocculation basin

a. No. of basins : 8 basins

b. Dimension : W 32.0 m x L 12.8 m x D 3.5 m
= 1,433.6 cu.m

c. Retention time : T = 1,433.6 x 8/383.8 = 29.9 min

d. Baffled channels: W 1.5 m x 2 channels, h = 34 cm
and loss of head: W 2.0 m x 2 channels, h = 14 cm
to be given (h) : W 2.9 m x 2 channels, h = 3 cm

4) Sedimentation basin

a. No. of basins : 8 basins

b. Dimensions : W 32.0 m x L 90.0 m x D 4.0 m
= 11,520 cu.m

c. Retention time : T = 11,520 x 8/23,025 = 4.0 hr

d. Overflow rate : R = 383.8 x 1/(8 x 32.0 x 90.0)
= 0.017 m/min

e. Average flow rate: V = 383.8 x 1/(8 x 32.0 x 4.0)
= 0.37 m/min

5) Rapid Sand filter

a. No. of filter beds: 40 beds-including spare beds

b. Filter area per one bed: W. 8.0 m x L 16.0 m
= 128 sq.m

c. Washing rate : Backwash 0.8 cu.m/min/sq.m
Surface wash 0.2 cu.m/min/sq.m

d. Loss of head : 2.0 m

6) Clear water reservoir

a. No. of reservoirs: 4 reservoirs

b. Dimensions : W 40.0 m x L 40.0 m x D 4.0 m = 6,400 cu.m

c. Rentention time : T = 6,400 x 4/23,025 = 1.1 hr

7) Elevated tank for backwashing

a. No. of tanks : 1 tank

b. Dimensions : Dia 21.0 m x D 3.0 m = 1,039 cu.m

8) Waste water basin

a. No. of basins : 2 basins

b. Dimensions : W 10.0 m x L 25.0 m x D 4.0 m = 1,000 cu.m

9) Administration building

a. Components : Administration Office, Laboratory, Chemical storage, Conference rooms, etc.

b. Area : 6,000 sq.m

10) Chemical feeding facility

Alum and chlorine feeding, feed systems which are shown in Figure B.IV-19 and B.IV-20.

b. Geology of proposed site

Sang Jani (Alternative I) and Golra (Alternative III)

According to the survey, the geology of the proposed treatment plant sites is alternating bed of silty clay and gravel or sand. Silty clay is massive and is well compacted. N valves of foundation indicate 20 to 30 which means that the foundation has enough bearing capacities for the water treatment plant (4 to 8 ton/sq.m). N valves observed at the sites suggest that, no special foundation treatment is taken. However, when heavy structures are mounted across the clay layers and gravel layers, differencial settlement may occurs. Detail survey and studies are required in detail design of structures.

Shah Allah Ditta (Alternative II)

Geology of the site is bedrock composed of sandstone and mudstone, however, some unconsolidated sediments develop at depression of mudstone. Boring and test pit survey was carried out in order to clarify the characteristics and depth of such unconsolidated sediments. It is learnt from the survey that the unconsolidated deposits consist of well compacted clayey sand, being associated with pebble of sandstone. Generally they seem to have enough bearing capacities, however, when heavy structures are laid across the unconsolidated deposits and bedrocks, there exists the possibility of differencial settlement in case of thick unconsolidated deposits.

E. Pumping Station

Layouts of the following three water source pumping stations are given in Figure B.IV-21 to B.IV-23.

- Alternative II-A: Khurram Paracha Station
- Alternative II-B: Julian Station
- Alternative II-C: Mohra Gota Station

In Alternative II-D, a Khanpur pumping station will be constructed in Khanpur reservoir, layout of which is illustrated in Figure B.IV-22. Figure B.IV-23 gives typical layouts of pumping stations to be installed in water treatment plants.

The specification of respective pump plants to be installed are summarized as below:

Alternative	Station Name	Specification
I-A	Sang Jani	700 mm x 1,830 kw x 6 unit
	Shah Allah Ditta	350 x 800 x 6
	Tomar	250 x 18.5 x 3
I-B	Sang Jani	700 x 1,380 x 6
	Tirnaul	400 x 220 x 6
	E-14	350 x 230 x 6
	Tomar	350 x 100 x 3
I-C	Sang Jani	400 x 690 x 6
	"	600 x 910 x 6
	Shah Allah Ditta	350 x 150 x 6
	Tomar	350 x 100 x 3
II-A	Khurram Paracha	800 x 1,900 x 6
	Shah Allah Ditta	350 x 160 x 6
	I-13	250 x 18.5 x 3
II-B	Julian	800 x 1,720 x 6
	Shah Allah Ditta	350 x 160 x 6
	I-13	250 x 18.5 x 3
II-C	Mohra Gota	800 x 1,450 x 6
	Shah Allah Ditta	350 x 160 x 6
	I-13	250 x 18.5 x 3
II-D	Khanpur	800 x 1,390 x 6
	Shah Allah Ditta	350 x 160 x 6
	I-13	250 x 18.5 x 6
III	Golra	350 x 430 x 6
	"	250 x 150 x 6
	J-11	250 x 15 x 3
	Westridge	350 x 220 x 3
	"	350 x 100 x 3

F. Pipe Works

The pipeline systems consist of discharge (rising) main, gravity main and main distribution line. The main distribution lines within the Project commanded area are excluded from the scope of this study, but included here for the purpose of comparison study.

The materials and grades of pipe were selected based on the allowable design pressure, earth pressure, and updown surging pressure in consideration of economic advantages. Suitable pipes selected for the system, as a consequence, are summarized as follows;

Discharge main:

Steel pipe: Diameter more than 1,350 mm

Ductile cast iron: Diameter less than 1,200 mm

Gravity main (Gravity flow):

PRCC: prestressed reinforce cement concrete pipe with steel core with diameter ranging from 1,650 mm to 400 mm

Distribution line:

Ductile cast iron: Allowable design pressure more than 12 kg per sq.cm.

PRCC (with steel core): Allowable design pressure less than 12 kg per sq.cm

Costs for miscellaneous appurtenant structures, such as stop value, air value and interconnection facilities, etc. are assumed at ten percent of the entire cost of pipe works.

Quantity of pipe works is summerized as follows:

(Unit: meter)

Alternative	Diameter (mm)	Discharge Main	Gravity Main	Distribution	Total
I-A	400-1,650	13,500	17,700	65,300	96,500
I-B	"	22,500	19,400	60,400	102,300
I-C	"	20,700	22,500	61,100	104,300
II-A	"	1,900	20,500	65,300	87,700
II-B	"	1,900	16,500	65,300	83,700
II-C	"	2,300	21,720	65,300	89,300
II-D	"	1,600	24,860	65,300	91,760
III	"	3,300	13,000	66,100	82,400

G. Service Reservoir

Two types of service reservoirs of flat slab type (RC) and prestressed concrete tank (PC) have been proposed. Selection of type depends on topographic conditions and economy of construction works. Proposed type and capacities of service reservoirs are given below;

Alternative	Name	Type	Capacity Effective (cu.m)	Capacity Design (cu.m)
I-A	Shah Allah Ditta	PC	26,100	18,800x2
	D-13	RC	14,100	15,300x1
	G-13	PC	84,400	27,100x4
I-B	E-14	PC	14,100	26,100x1
	Shah Allah Ditta	PC	26,100	18,800x2
	Tomar	RC	84,400	23,000x4
I-C	Shah Allah Ditta	PC	26,100	18,800x2
	D-13	RC	14,100	15,300x1
	Tomar	RC	84,400	23,000x4
II-A,B,C,D	Shah Allah Ditta	PC	26,100	18,800x2
	D-13	RC	14,100	15,300x1
	G-13	PC	84,400	27,100x4
III	Golra (1)	PC	26,100	16,500x2
	Golra (2)	RC	14,100	15,300x1
	H-11	RC	84,400	23,000x4

Typical layouts of RC tank and PC tank are illustrated in Figure B.IV-24 and B.IV-25, respectively.

Figure B.IV-1 LAYOUT, ALTERNATIVE I-A

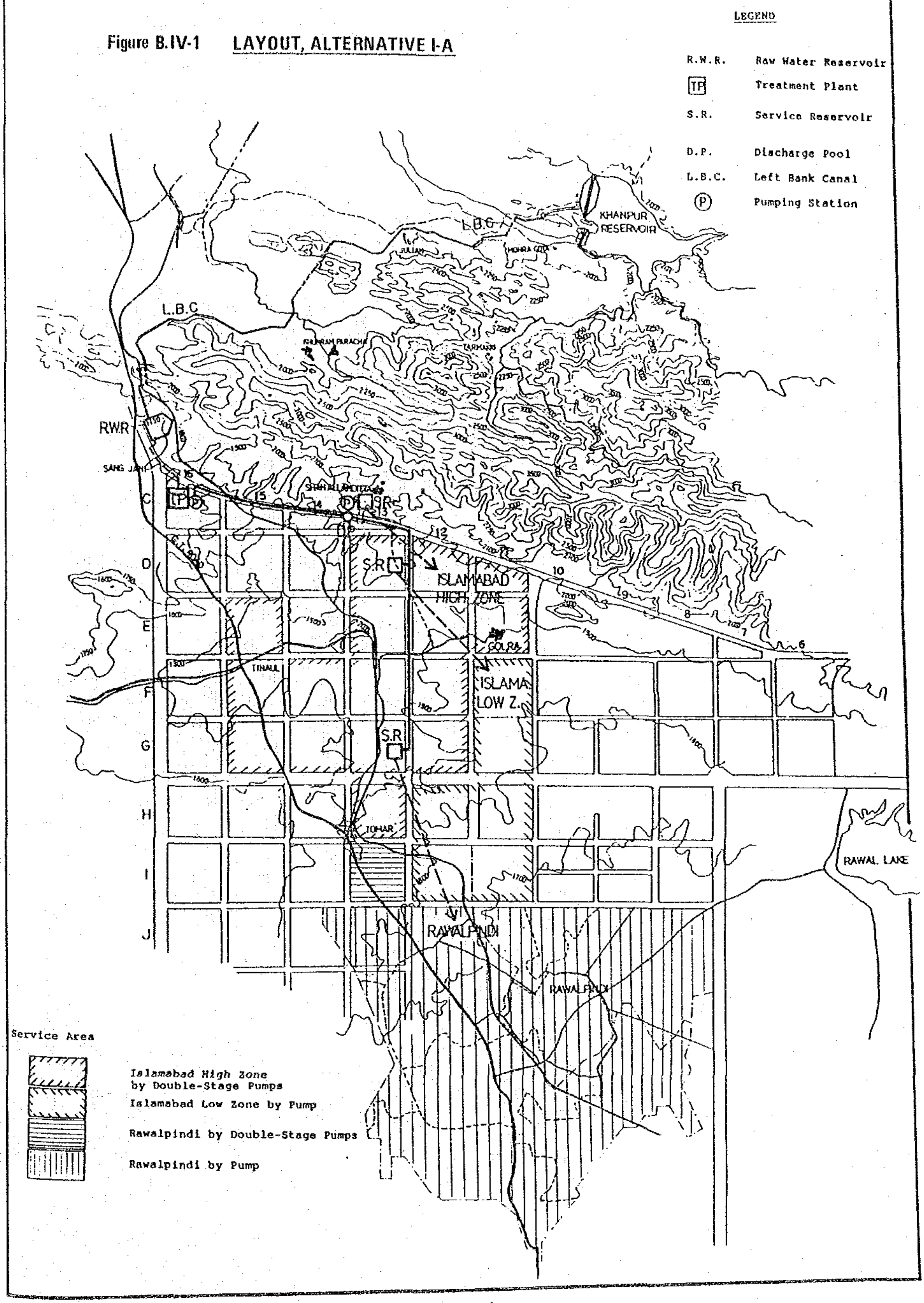


Figure B.IV-2 LAYOUT, ALTERNATIVE I-B

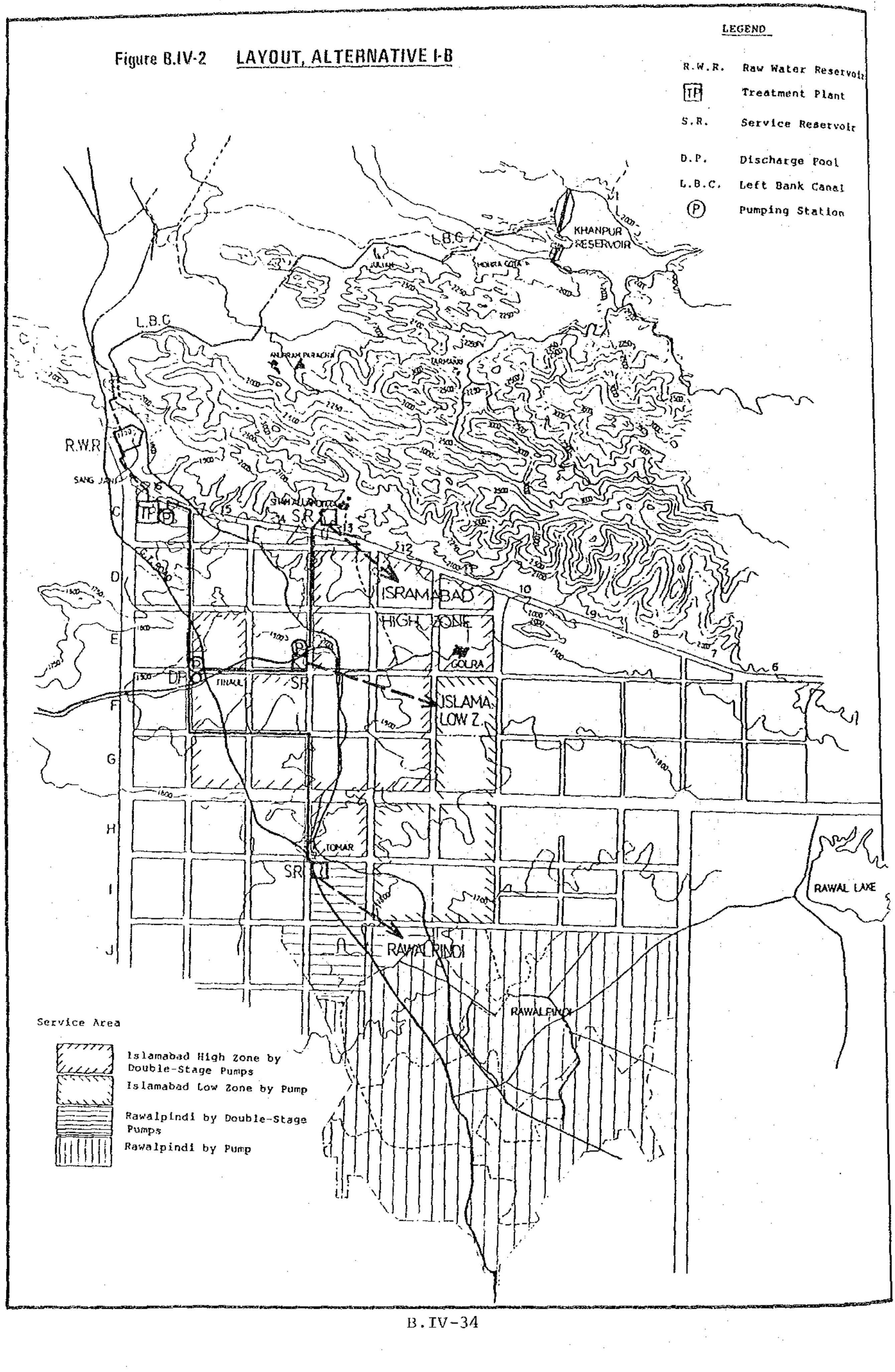


Figure B.IV-3 LAYOUT, ALTERNATIVE I-C

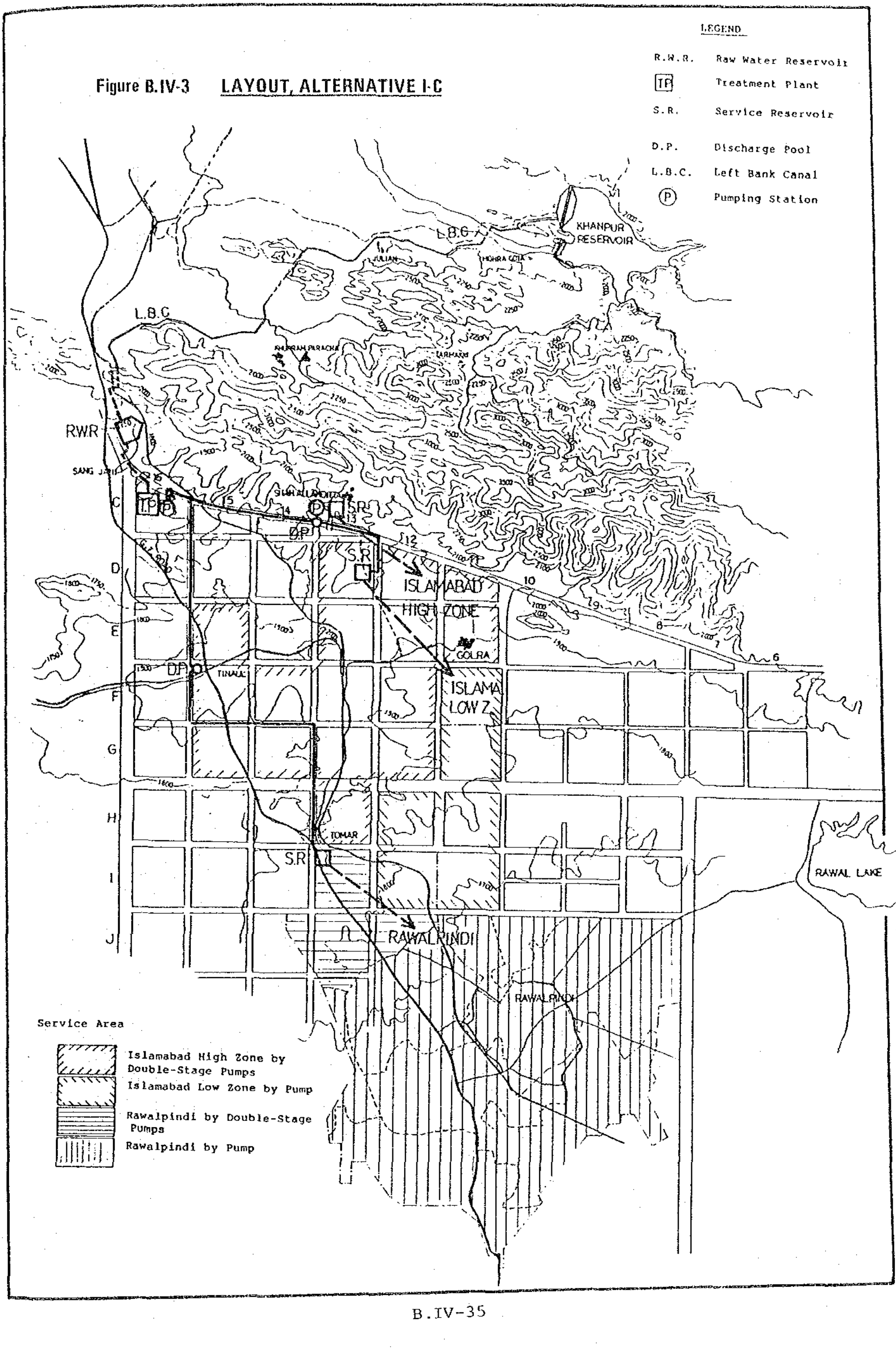
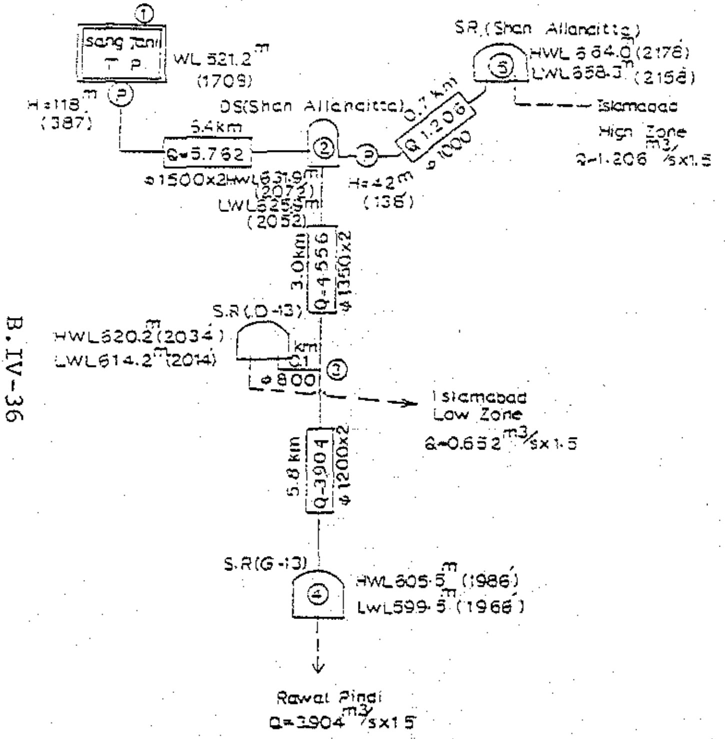
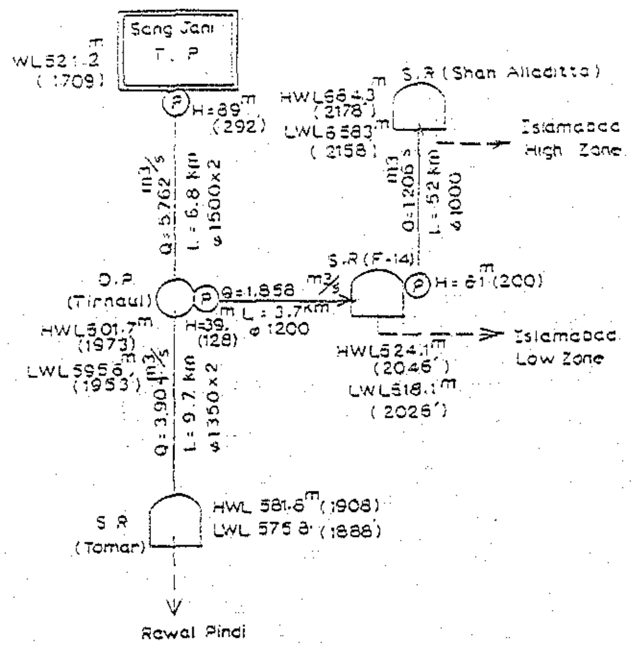


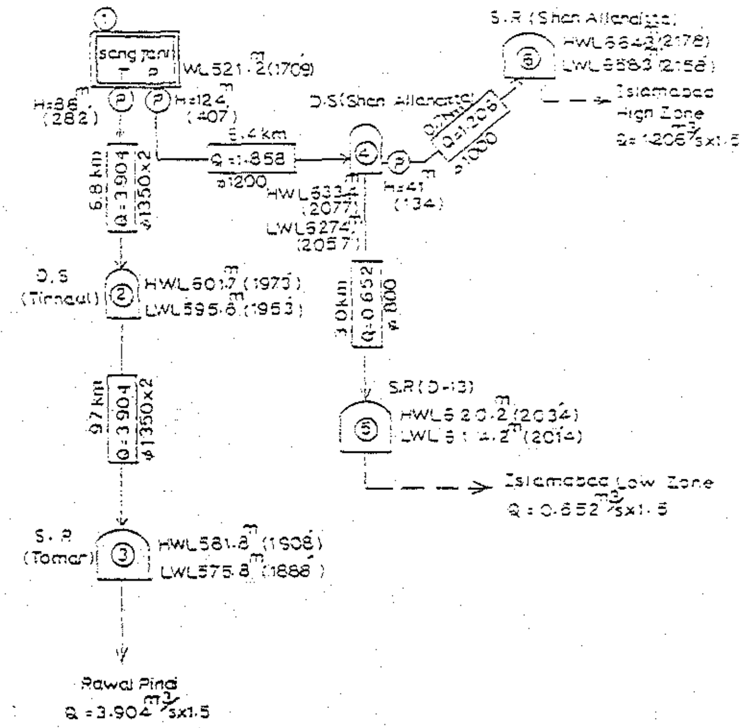
Figure B.IV-4 **FLOW DIAGRAM**



Alternative : I-A



Alternative : I-B



Alternative : I-C

FIGURE B.IV-5 POSSIBLE ALIGNMENT OF ALT. II-A AND ALT.II-C

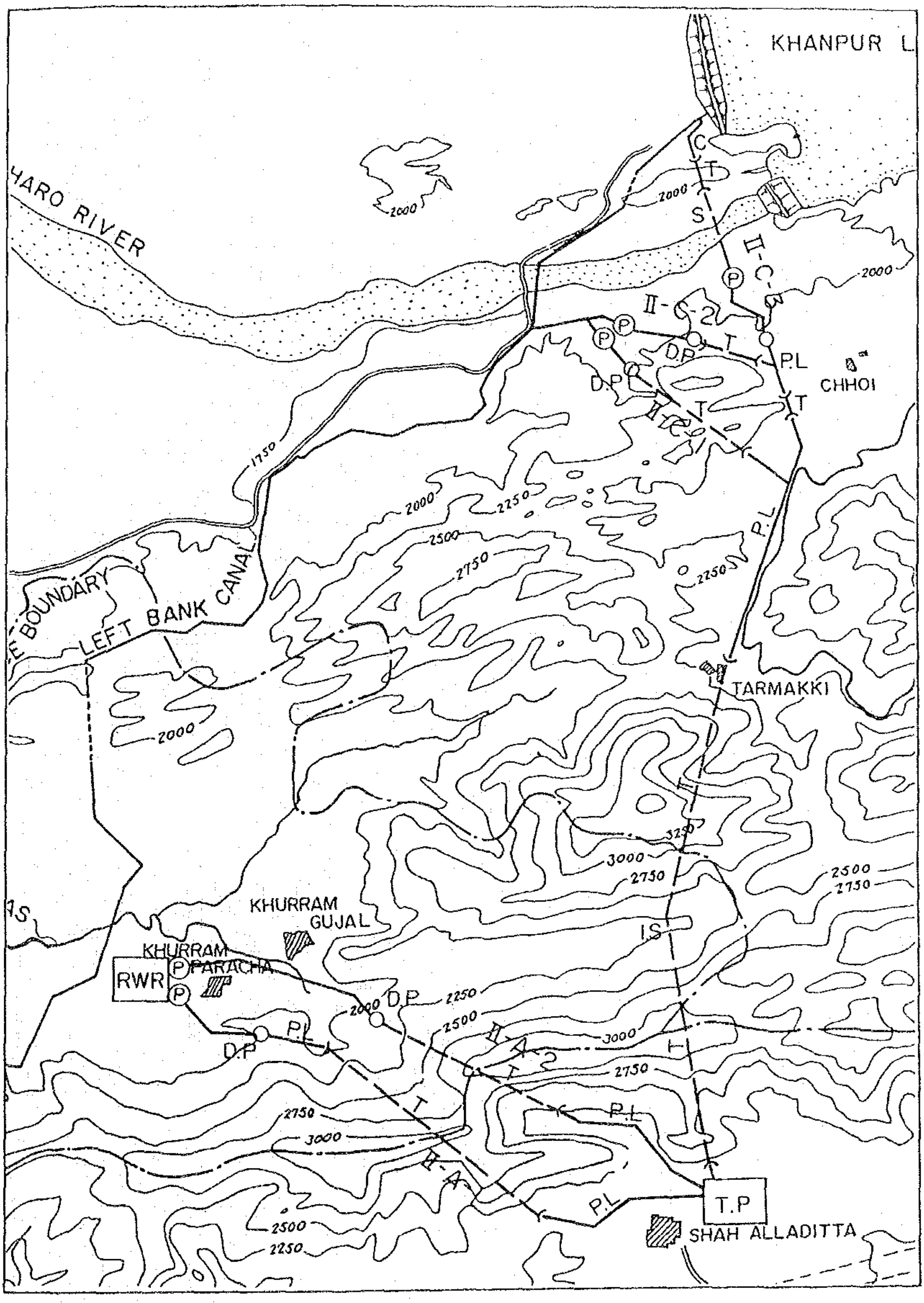


Figure B.IV-6 LAYOUT, ALTERNATIVE II

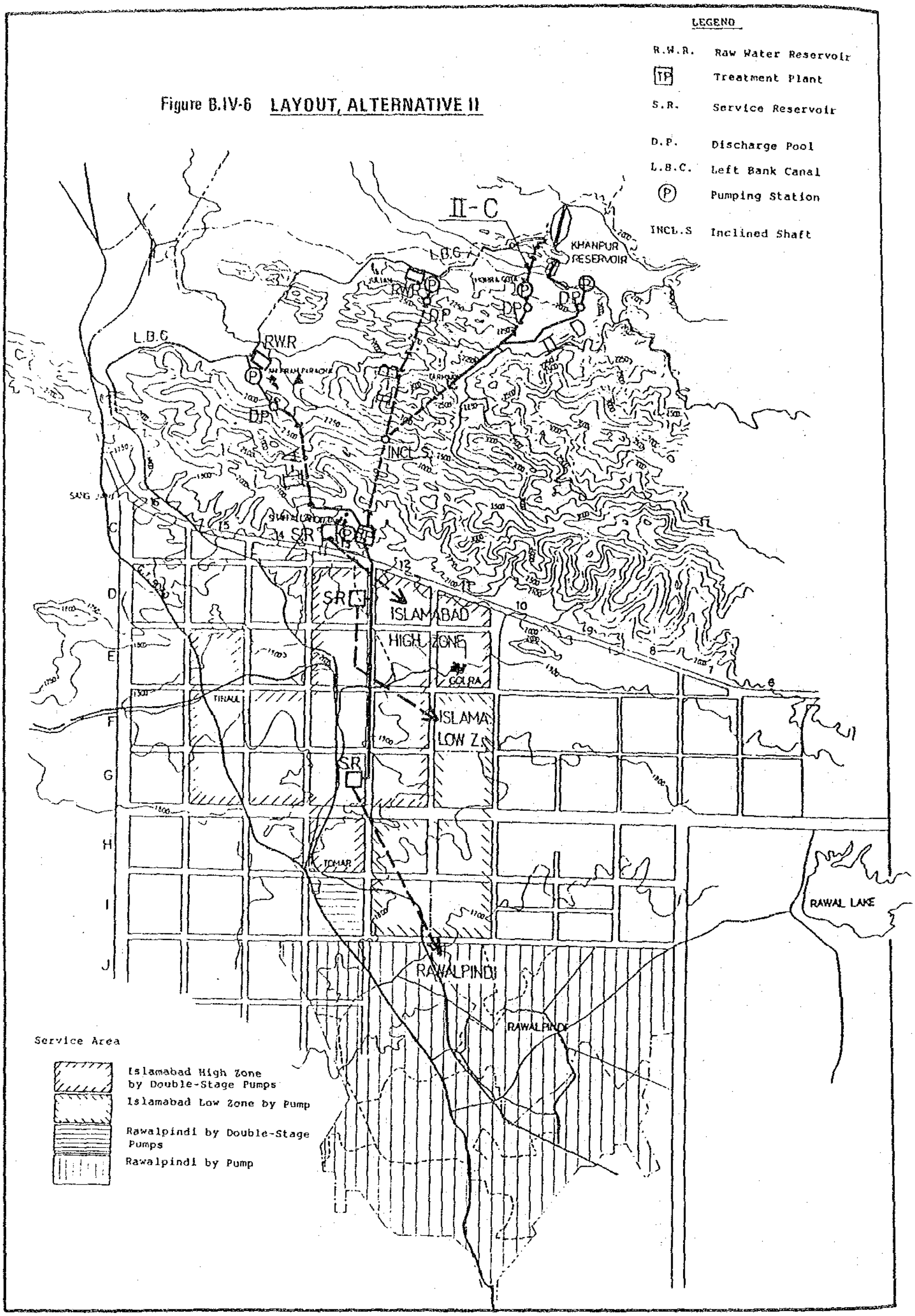
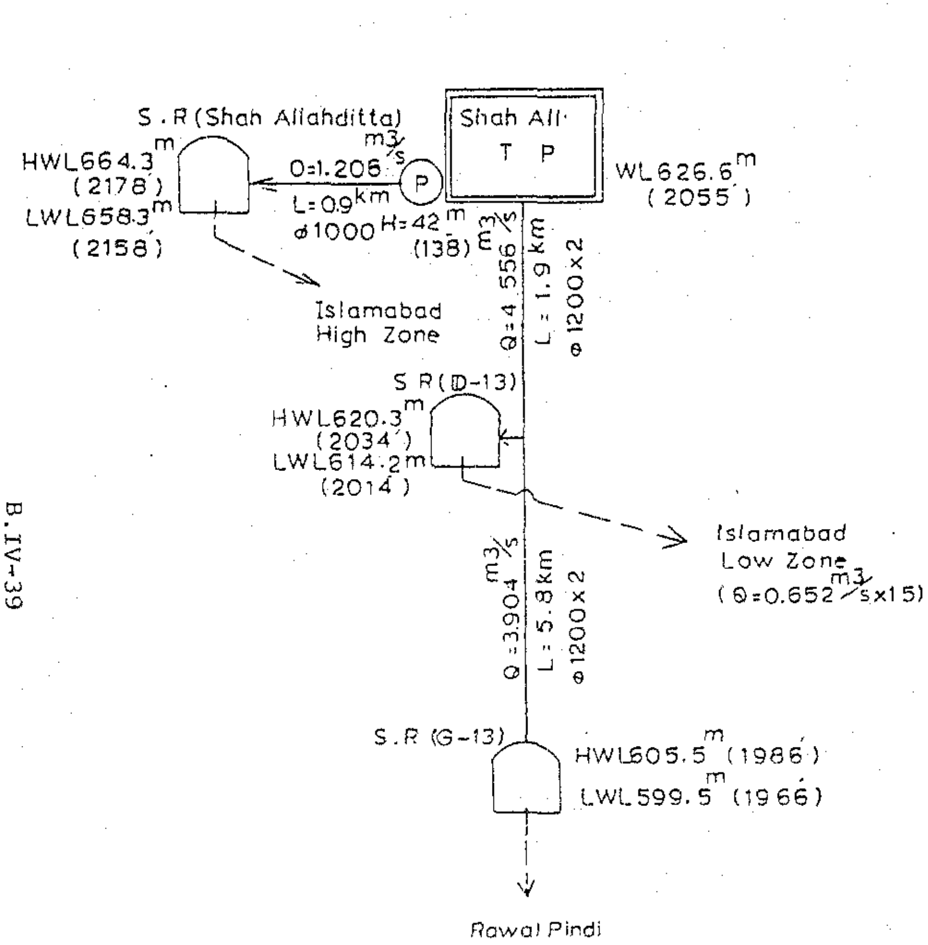
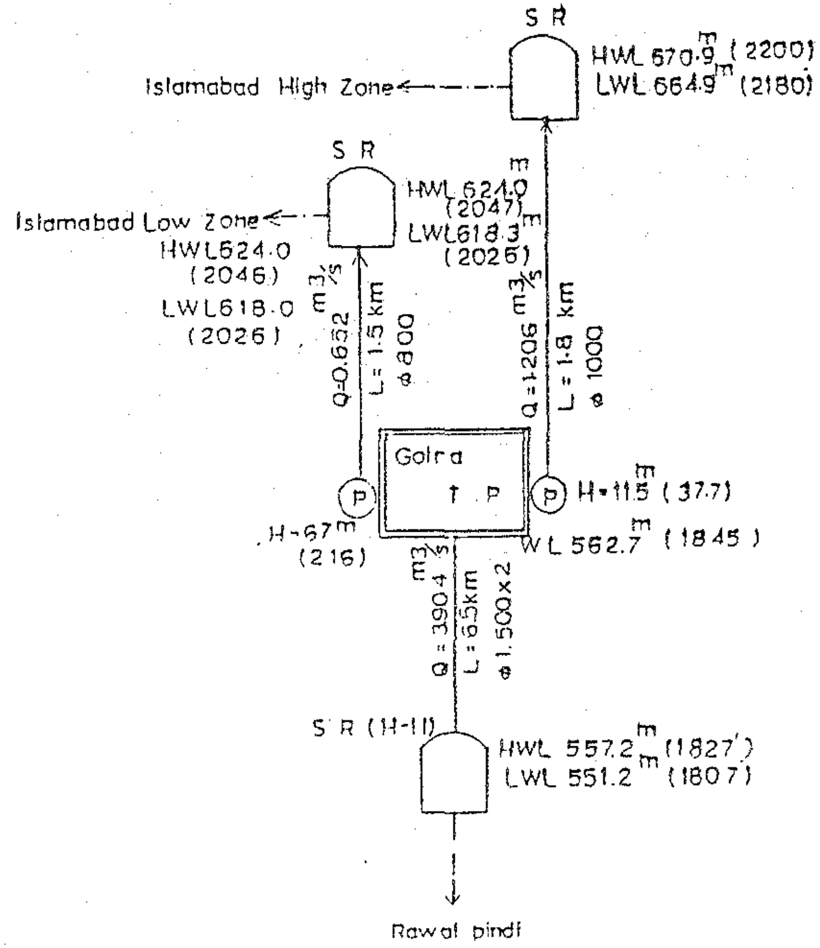


Figure B.IV-7 **FLOW DIAGRAM**



Alternative: II



Alternative: III

FIGURE B.IV-8 POSSIBLE ALIGNMENT OF ALT. III

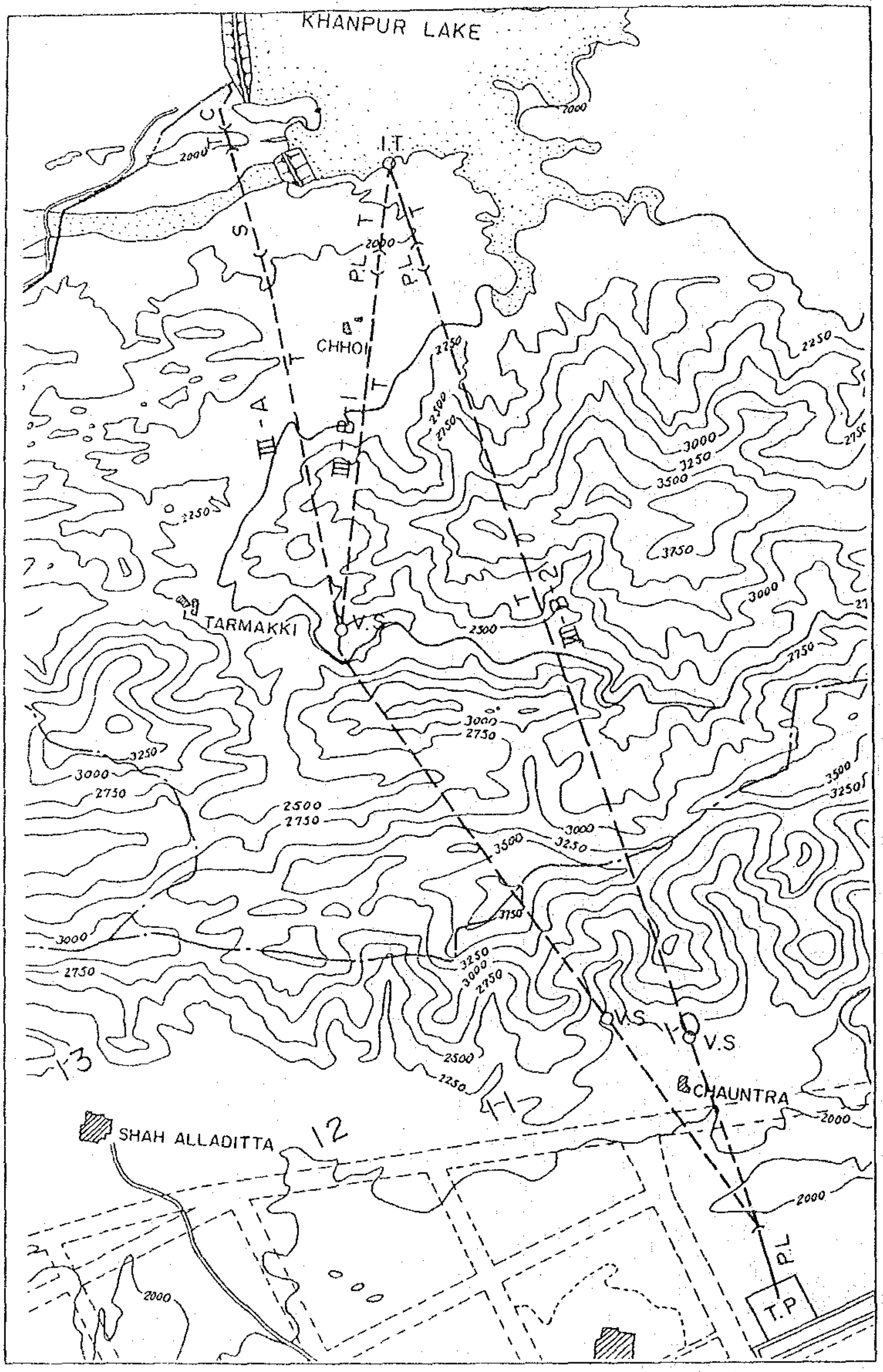


Figure B.IV-9 LAYOUT, ALTERNATIVE-III

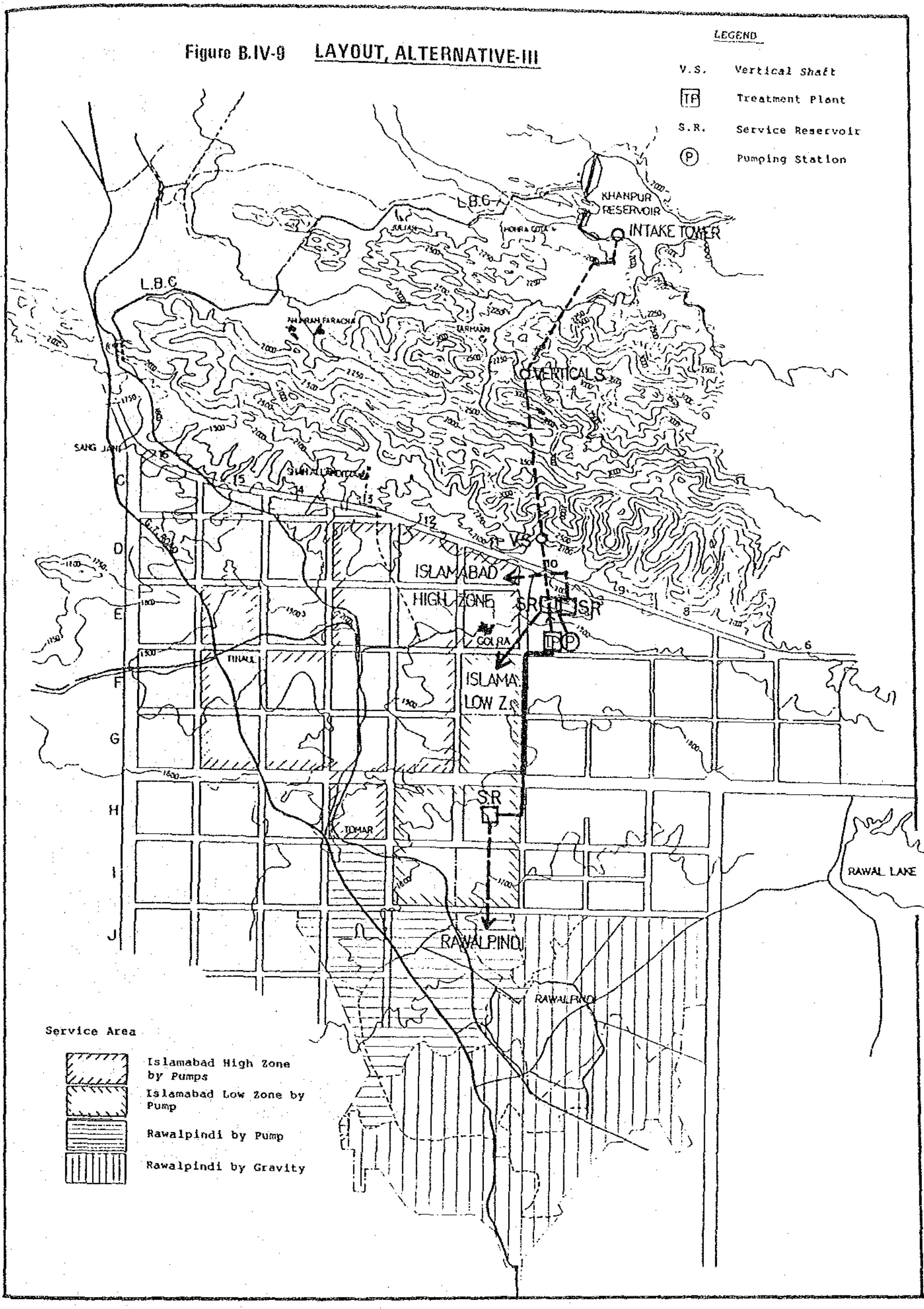
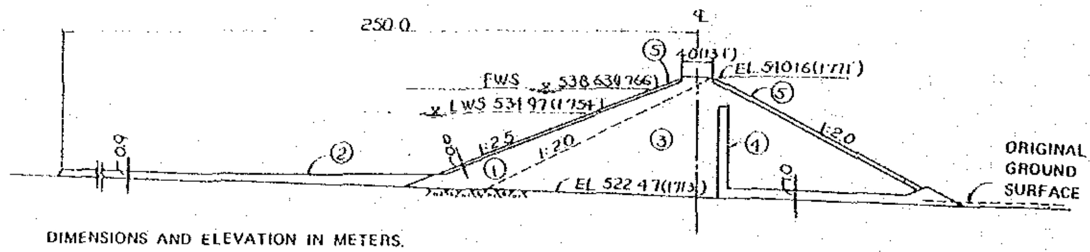


Figure B.IV-10 TYPICAL SECTION AND PLAN OF THE RAW WATER RESERVOIR (1)

LEGEND

1. IMPERVIOUS FILL
2. IMPERVIOUS BLANKET
3. RANDOM FILL
4. INTERCEPTER (DRAIN)
5. SLOPE PROTECTOR

HEIGHT OF DAM: 17.7 m (58')



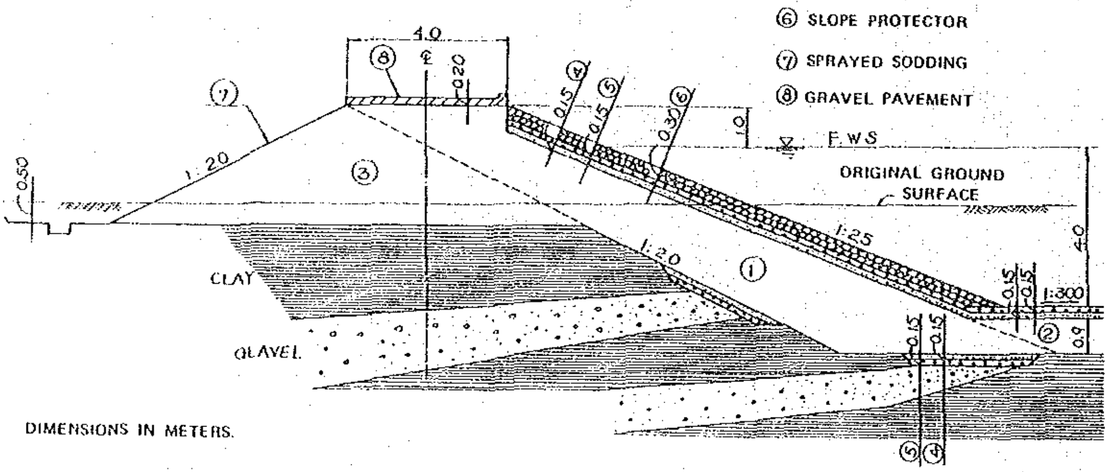
DIMENSIONS AND ELEVATION IN METERS.

ALTERNATIVE I (SANG JANI)

NOT TO SCALE

LEGEND

1. IMPERVIOUS FILL
2. IMPERVIOUS BLANKET
3. RANDOM FILL
4. FINE FILTER
5. COARSE FILTER
6. SLOPE PROTECTOR
7. SPRAYED SODDING
8. GRAVEL PAVEMENT



DIMENSIONS IN METERS.

ALTERNATIVE II (KHURRAM PARACHA AND JULIAN)

NOT TO SCALE

REMARKS	
DAM LENGTH	1,040 m
DAM HEIGHT	17.7 m(58')
DAM VOLUME	350,000 m^3
EARTH BLANKET VOLUME	100,000 m^3
TOTAL CAPACITY	1,888,000 m^3
EFFECTIVE CAPACITY	1,221,000 m^3

NOTE: CONTOUR IN FEET.

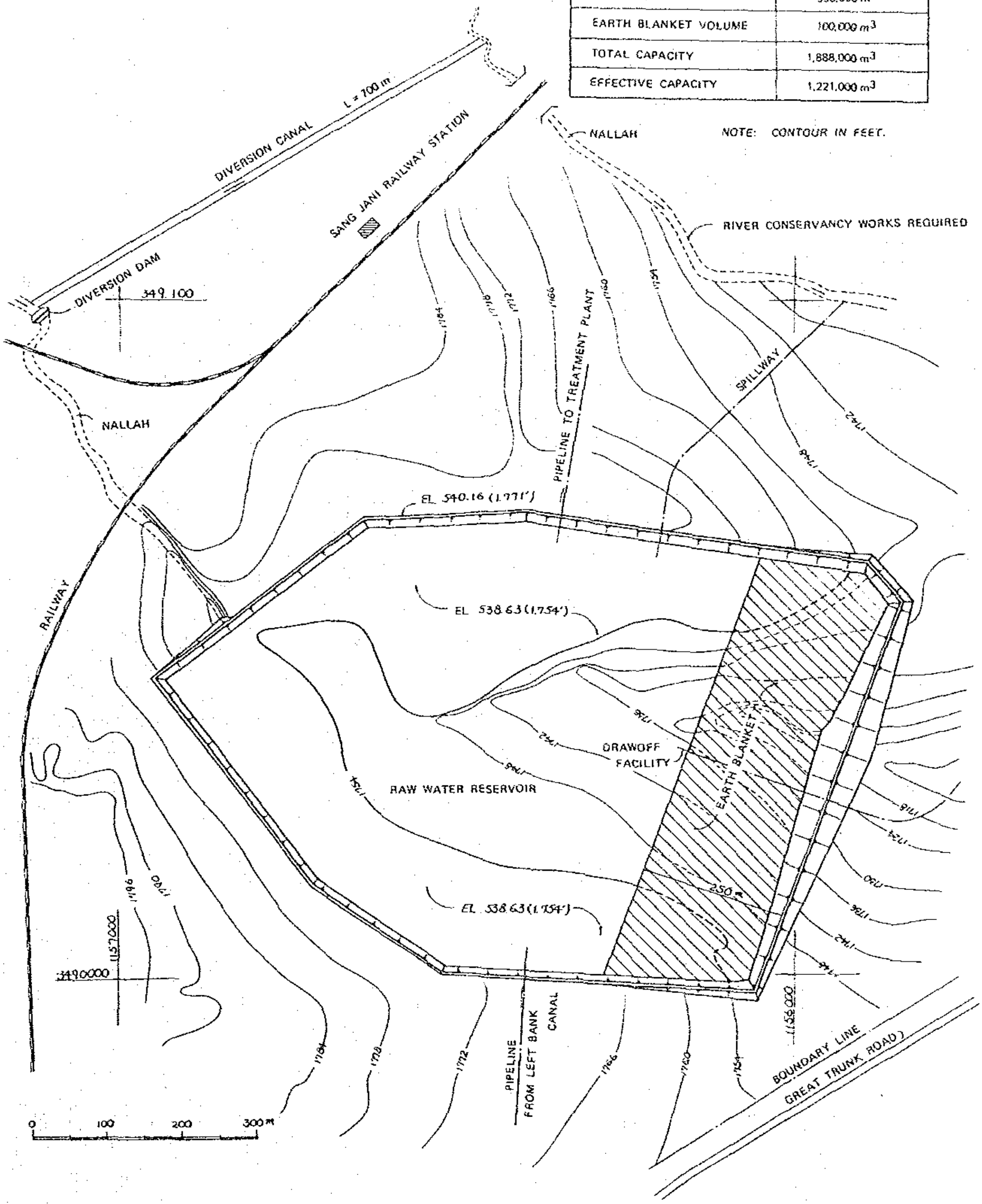
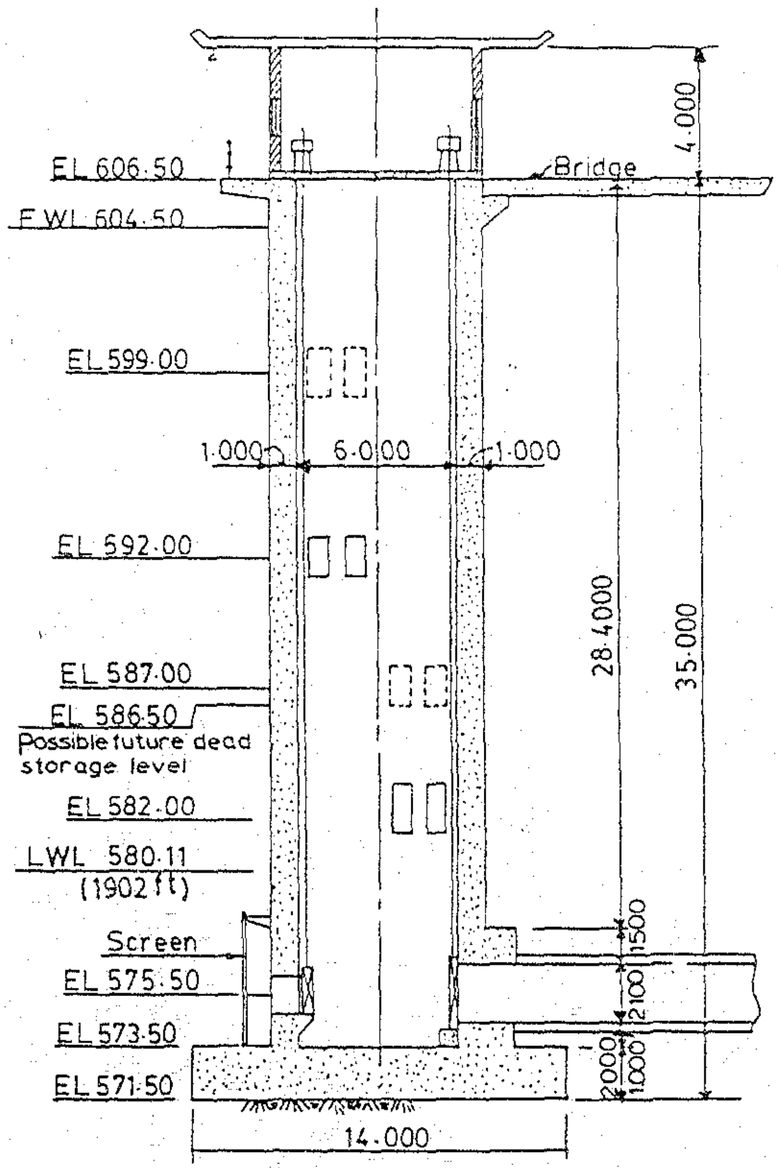
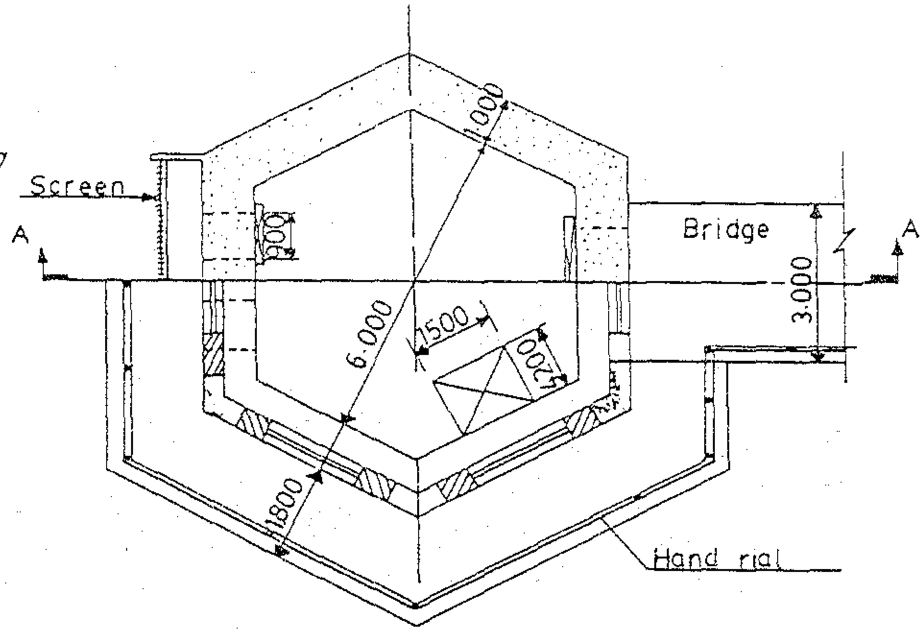


Figure B.IV-10 **TYPICAL SECTION AND PLAN OF THE RAW WATER RESERVOIR (2)**

Figure B.IV-11 INTAKE TOWER (ALTERNATIVE-III)

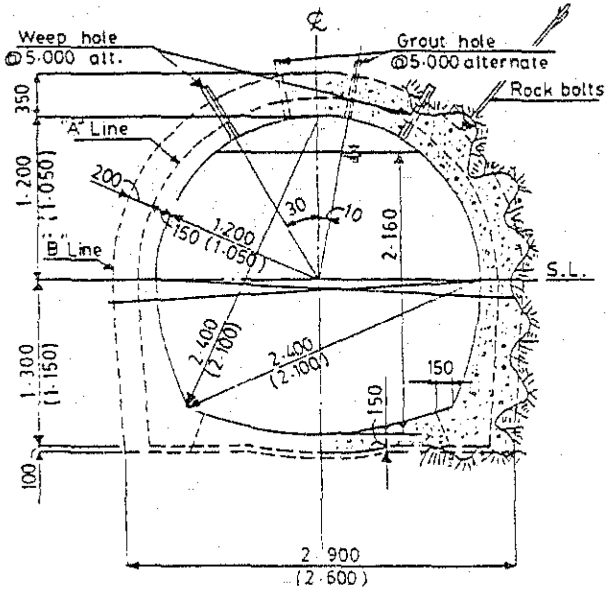


SECTION A - A scale 1:200

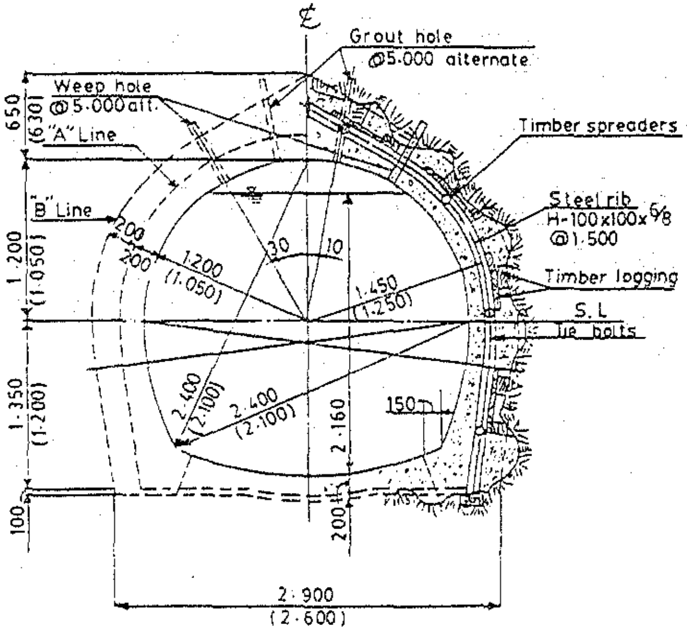


PLAN scale 1:100

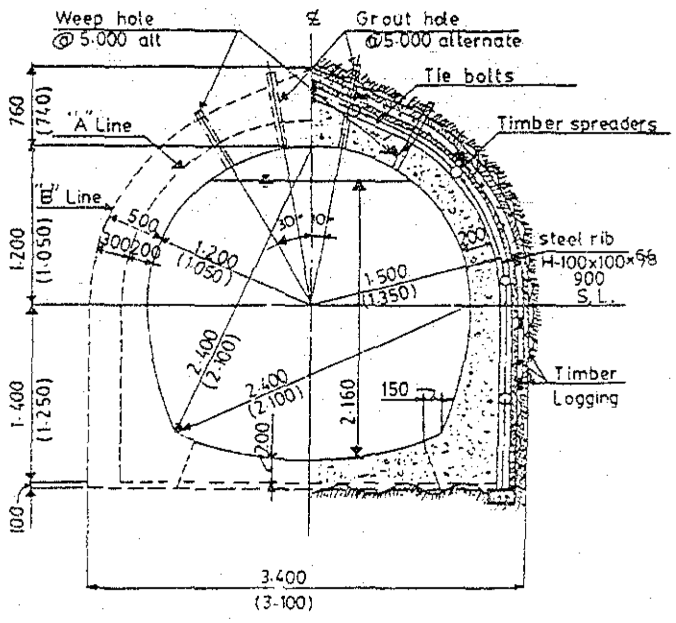
Figure B.IV-12 **TYPICAL CROSS SECTION OF TUNNEL**



TYPE - A



TYPE - B



TYPE - C

Note ( ) is size of 2.1 meter horse shoe tunnel
Weep hole are drilled free frowe tunnel only

Figure B.IV-13 TYPICAL CROSS SECTION OF VERTICAL AND INCLINED SHAFT

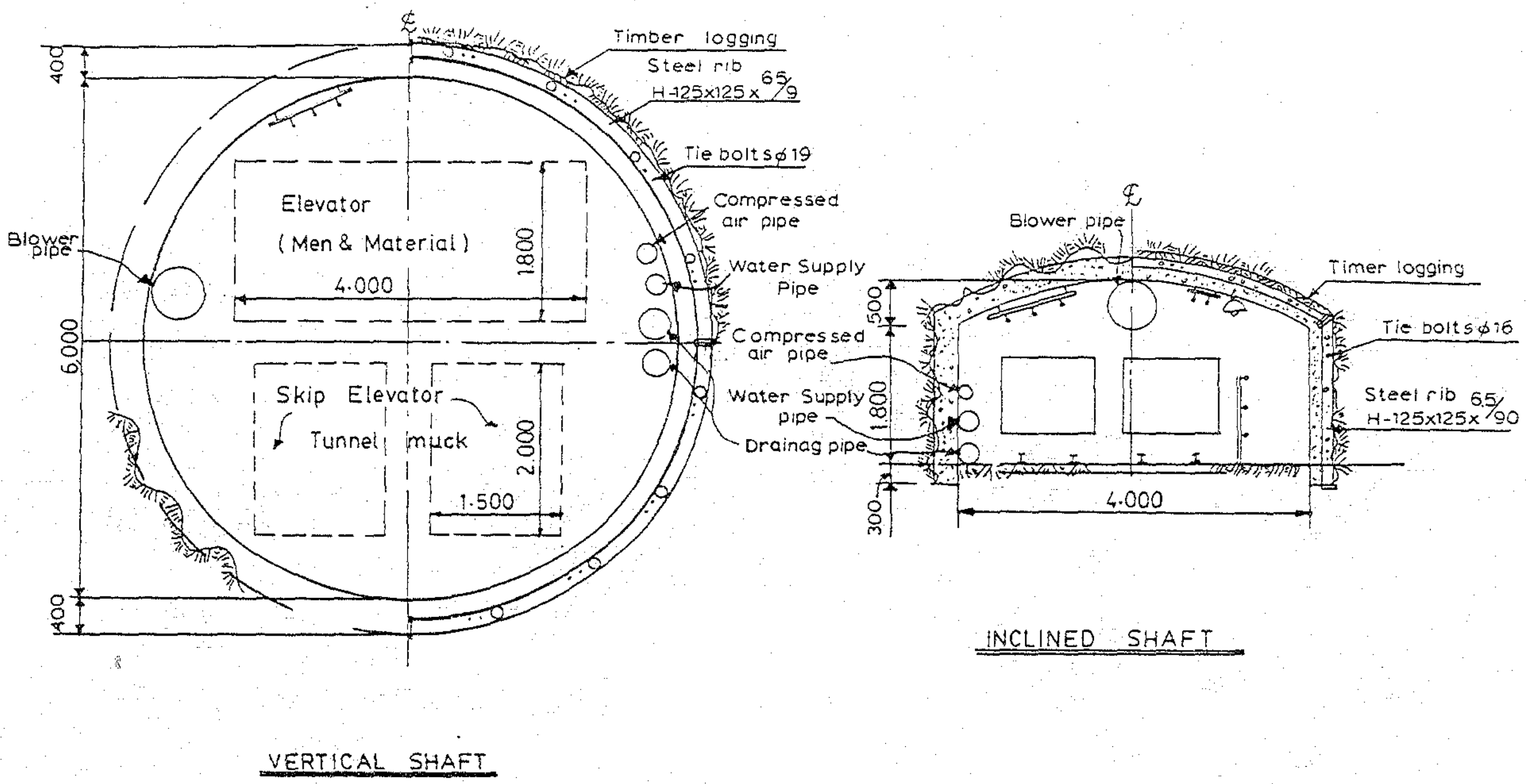


Figure B.IV-14 TUNNEL CONSTRUCTION SCHEDULE (ALTERNATIVE II-C, II-D)

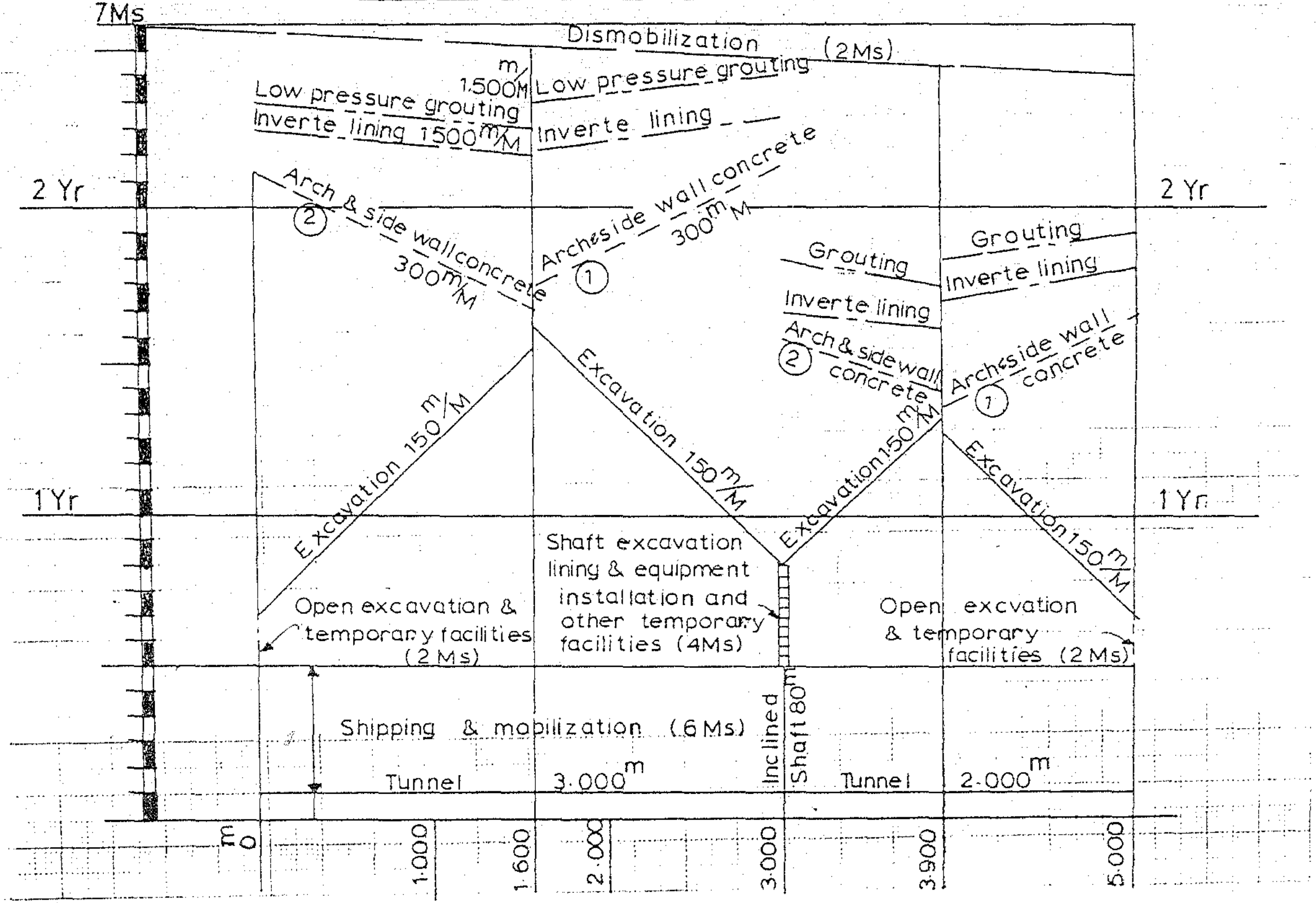
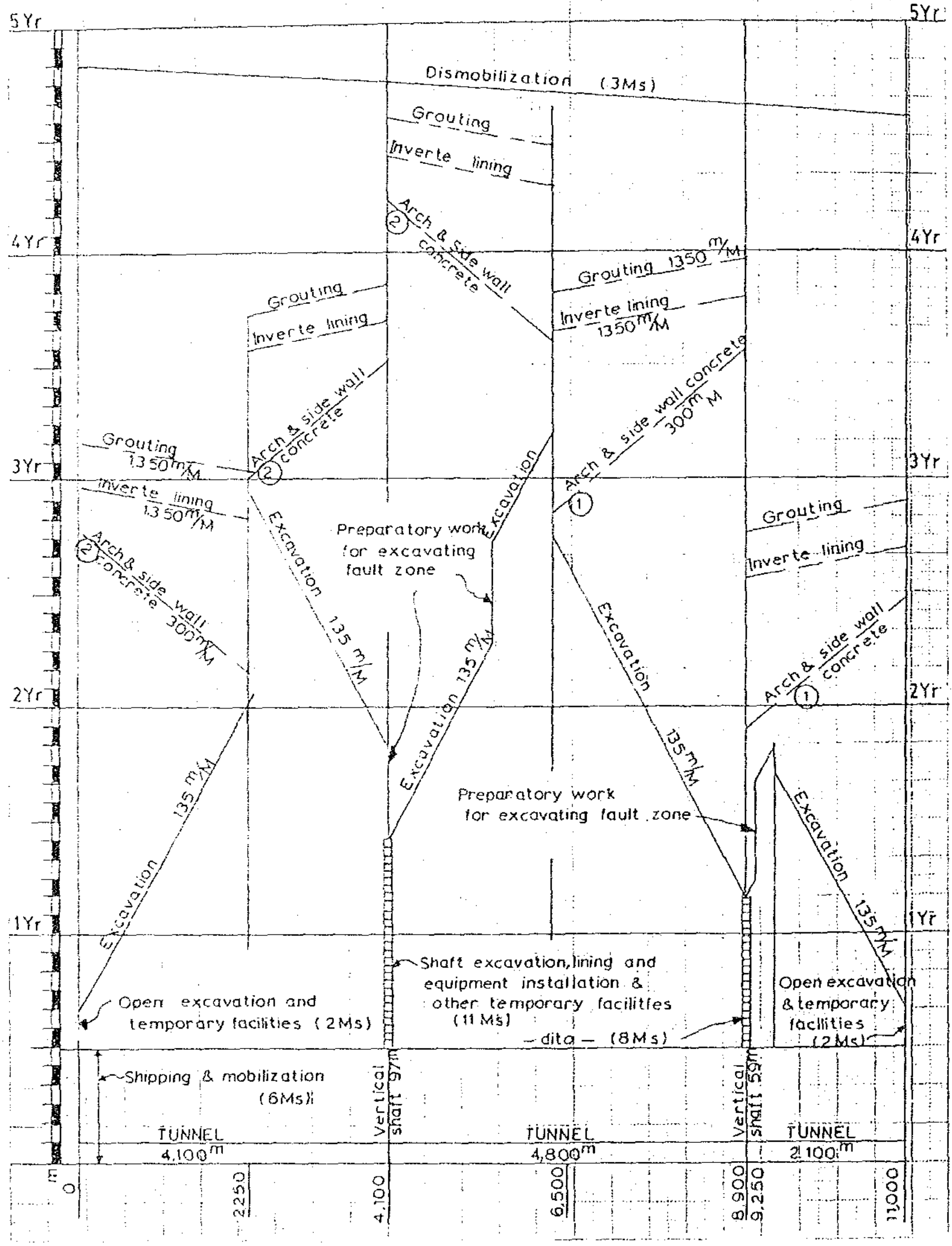


Figure B.IV-15 TUNNEL CONSTRUCTION SCHEDULE (ALTERNATIVE-III)



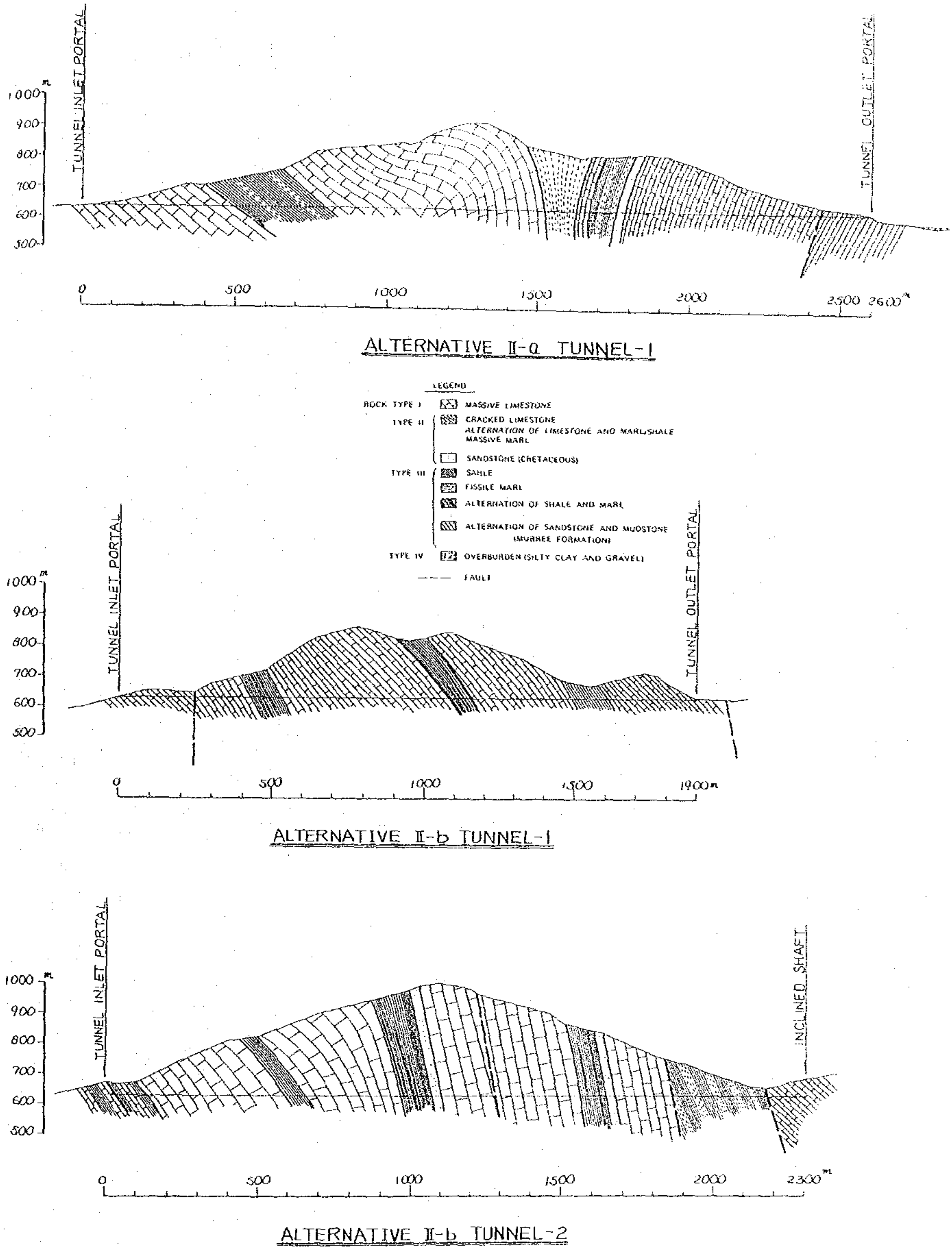
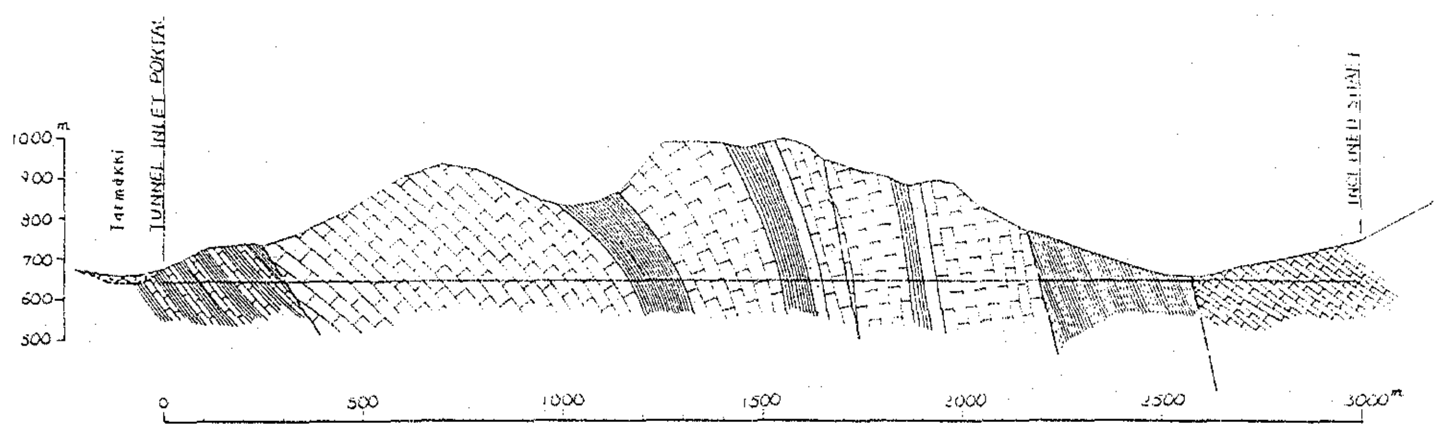


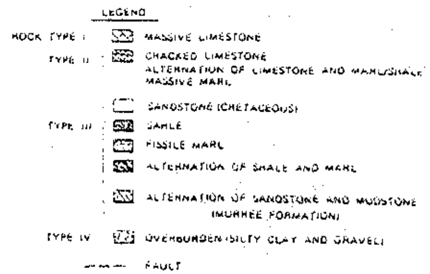
Figure B.IV-16 **GEOLOGIC SECTION ALONG TUNNELS (1)**

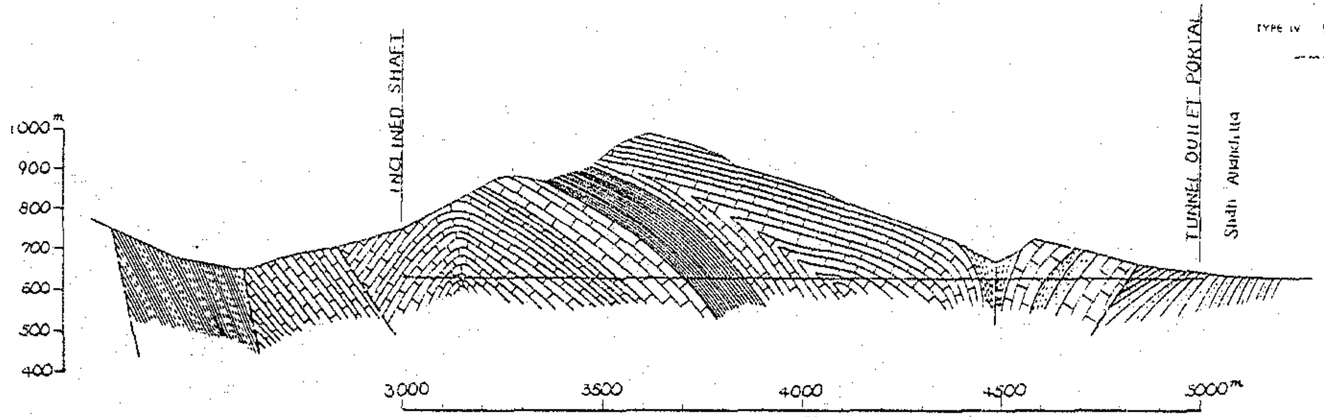
Figure B.IV-16 GEOLOGIC SECTION ALONG TUNNELS(2)



ALTERNATIVE II-C TUNNEL-3(UPPER PART)

ALTERNATIVE II-d TUNNEL-1(UPPER PART)



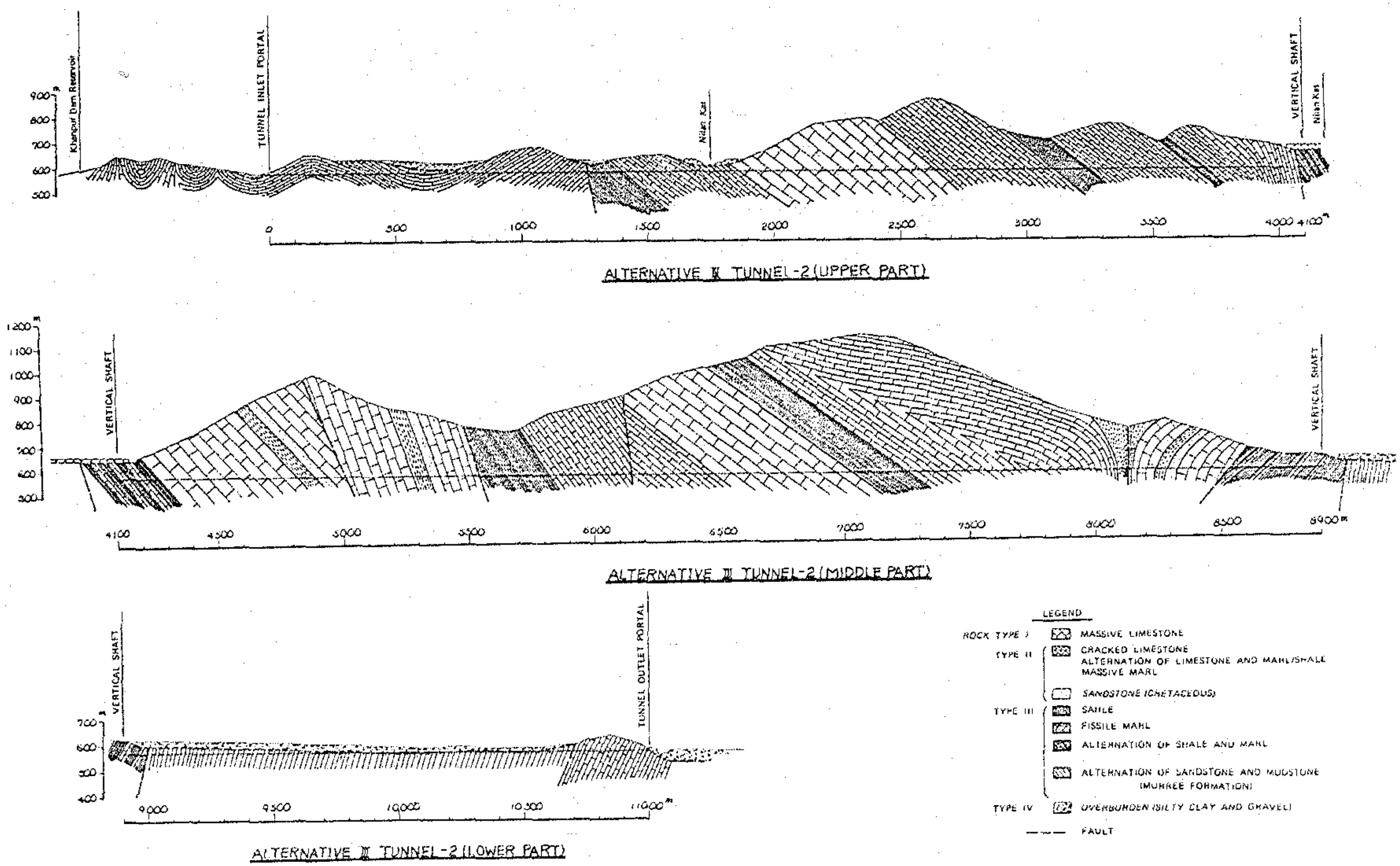


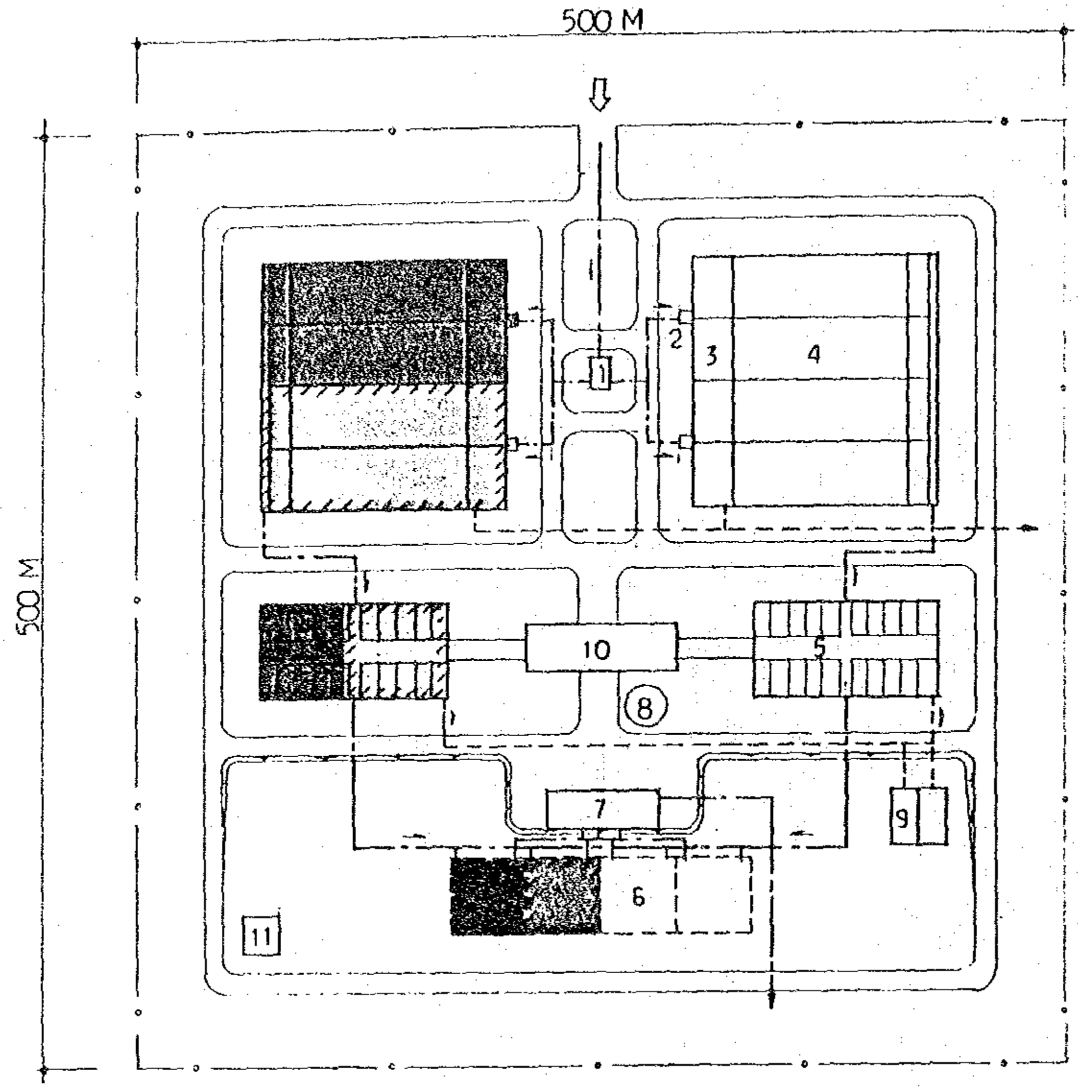
ALTERNATIVE II-b TUNNEL-2(LOWER PART)

ALTERNATIVE II-C TUNNEL-3(LOWER PART)

ALTERNATIVE II-d TUNNEL-1(LOWER PART)

Figure B.IV-16 **GEOLOGIC SECTIONS ALONG TUNNELS(3)**





1 RECEIVING WELL
2 MIXING WELL
3 FLOCCULATION BASIN
4 SEDIMENTATION BASIN
5 RAPID SAND FILTER
6 CLEAR WATER RESERVOIR
7 TRANSMISSION PUMP STATION
8 ELEVATED WASH TANK
9 WASTE WATER BASIN
10 ADMINISTRATION BUILDING
11 SUBSTATION

PHASE I :
PHASE II :
PHASE III :

Figure B.IV-17 **LAYOUT OF WATER TREATMENT PLANT**

SCALE 1 : 4000

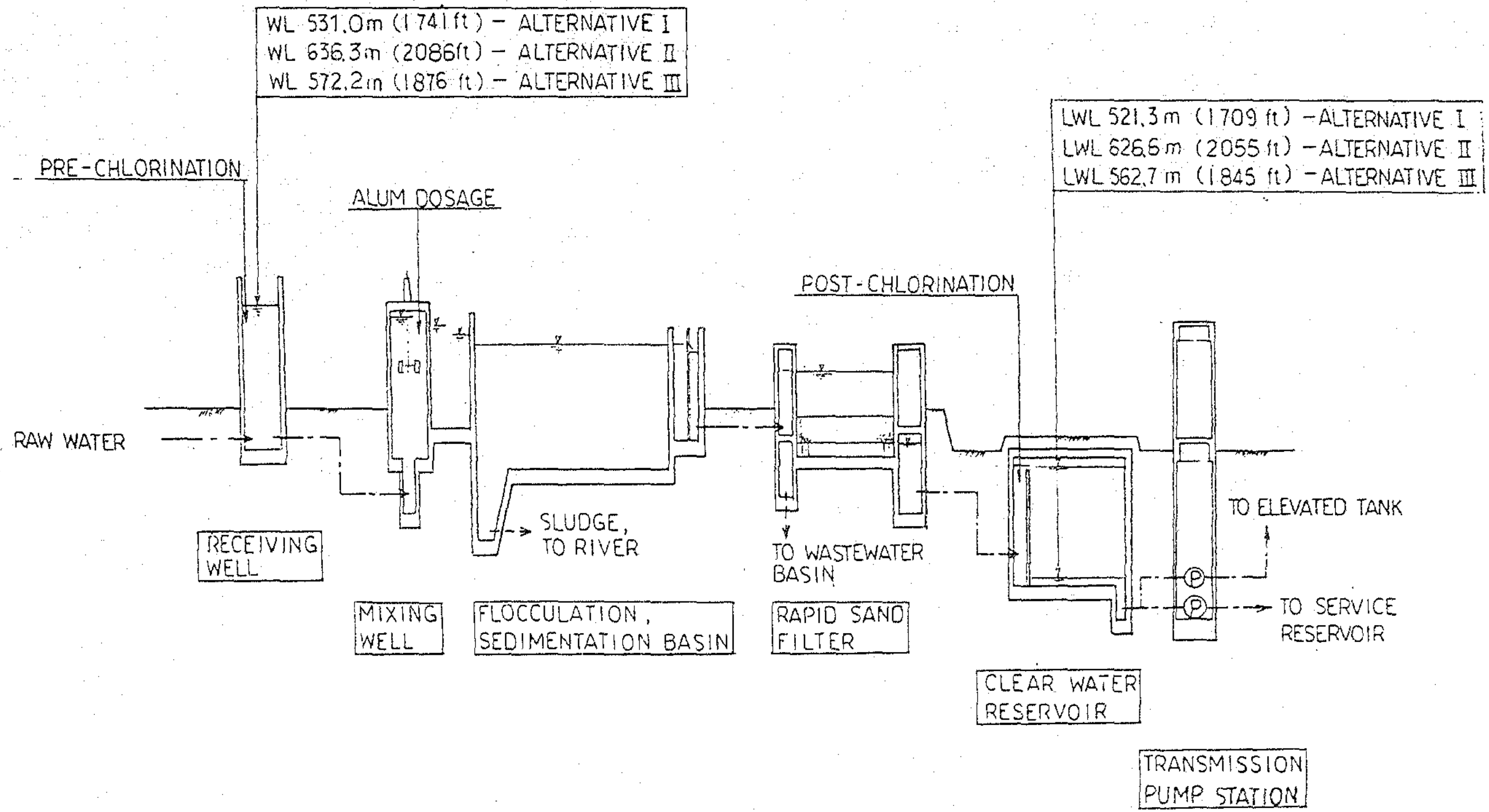


Figure B.IV-18 **FLOW DIAGRAM OF WATER TREATMENT PLANT**
NOT TO SCALE

Figure B.IV-19 ALUM FEED SYSTEM

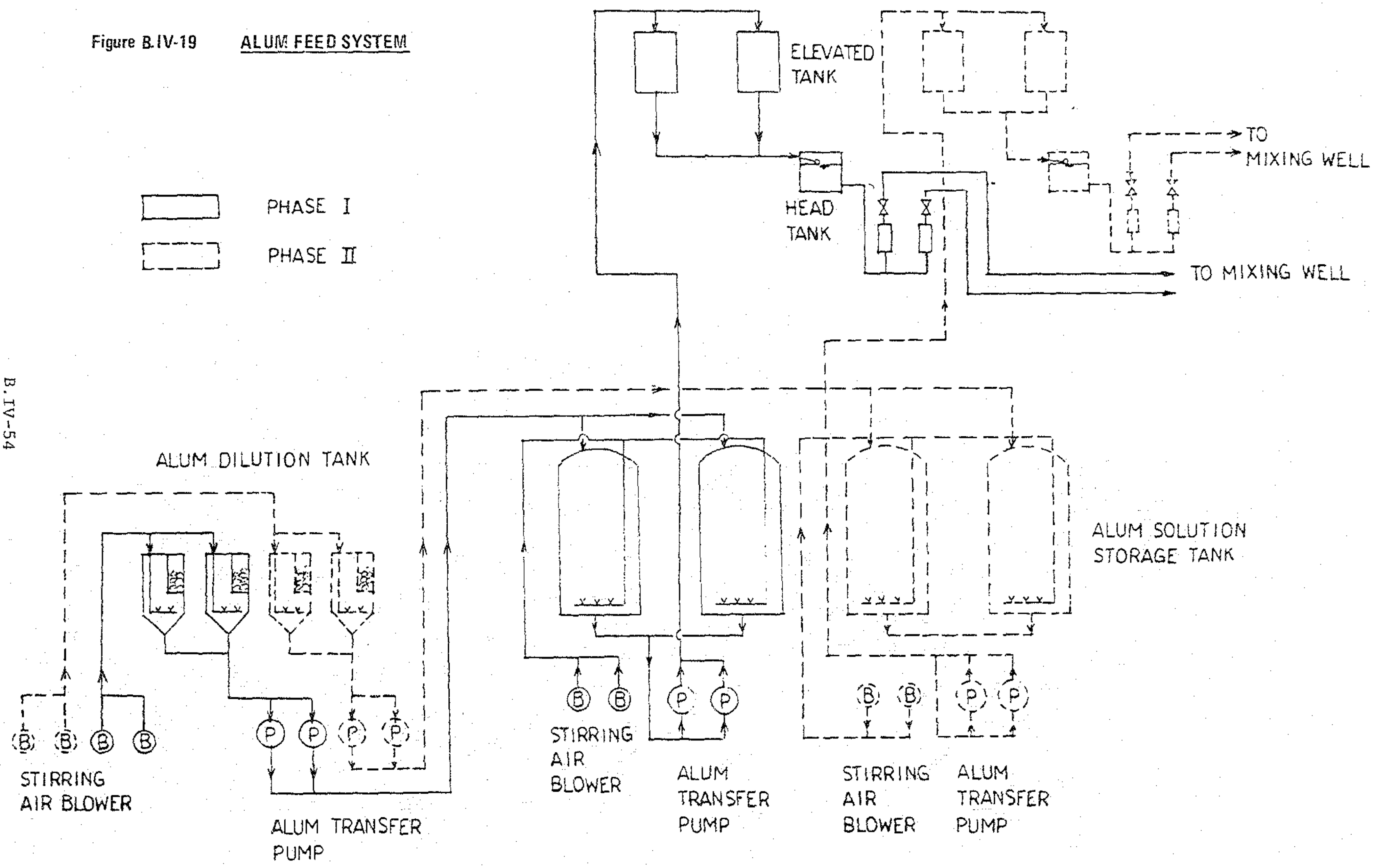


Figure B.IV-20 **CHLORINATION FEED SYSTEM**

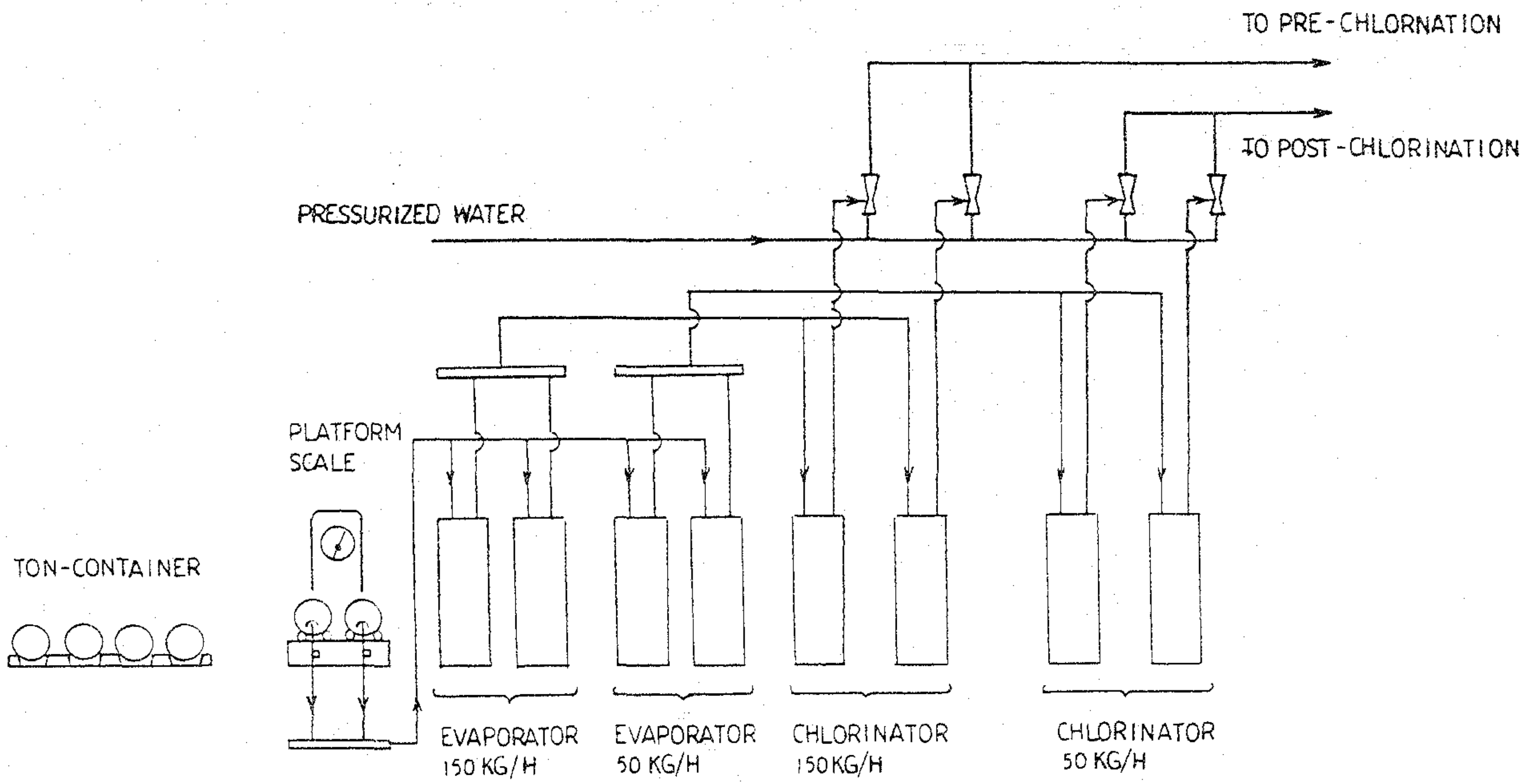


Figure B.IV-21 **PLAN OF MORA GOTA PUMPING STATION (ALT.II-C)**

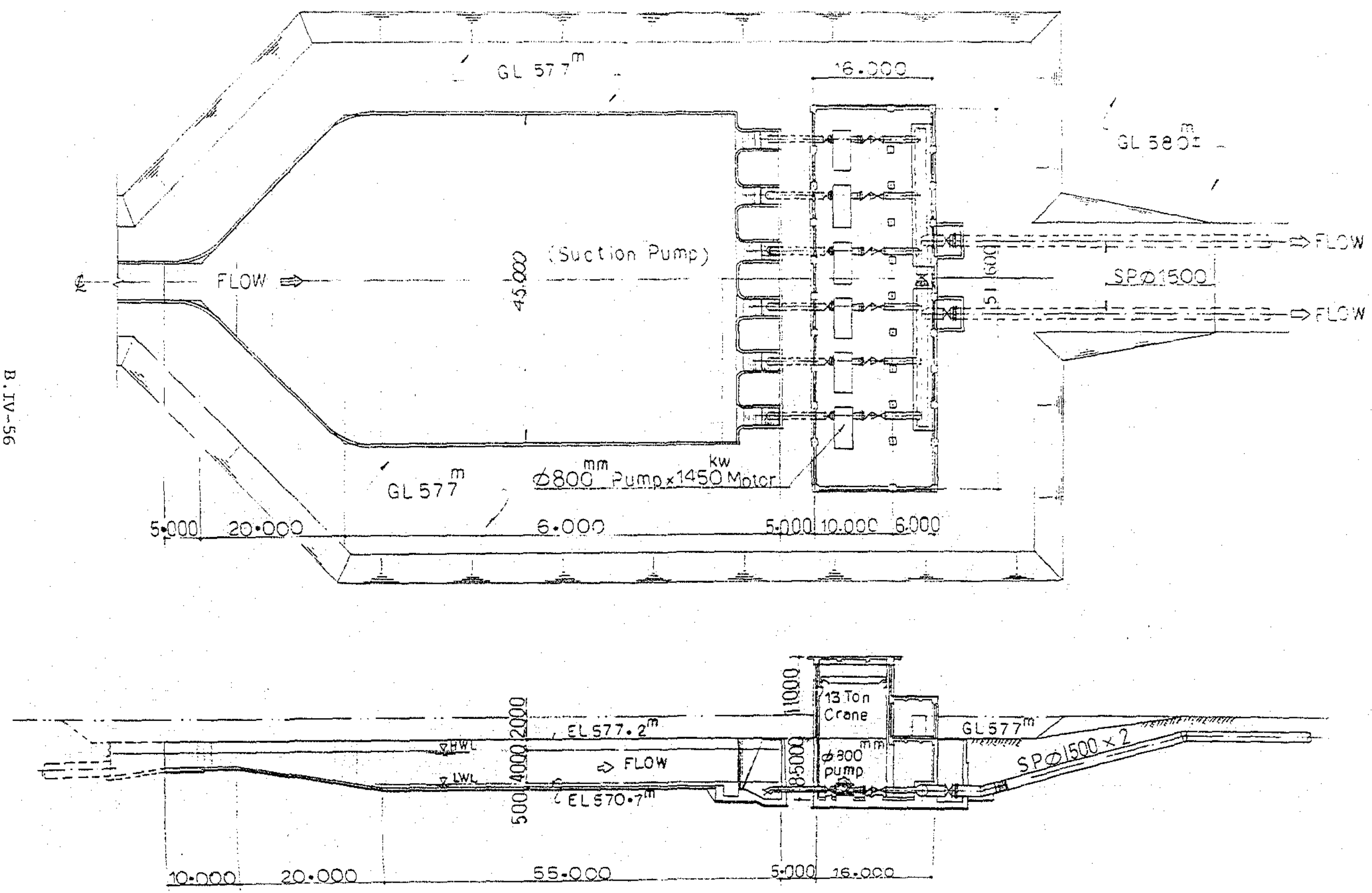


Figure B.IV-22 **PLAN OF KHANPUR PUMPING STATION**
(ALTERNATIVE II-0)

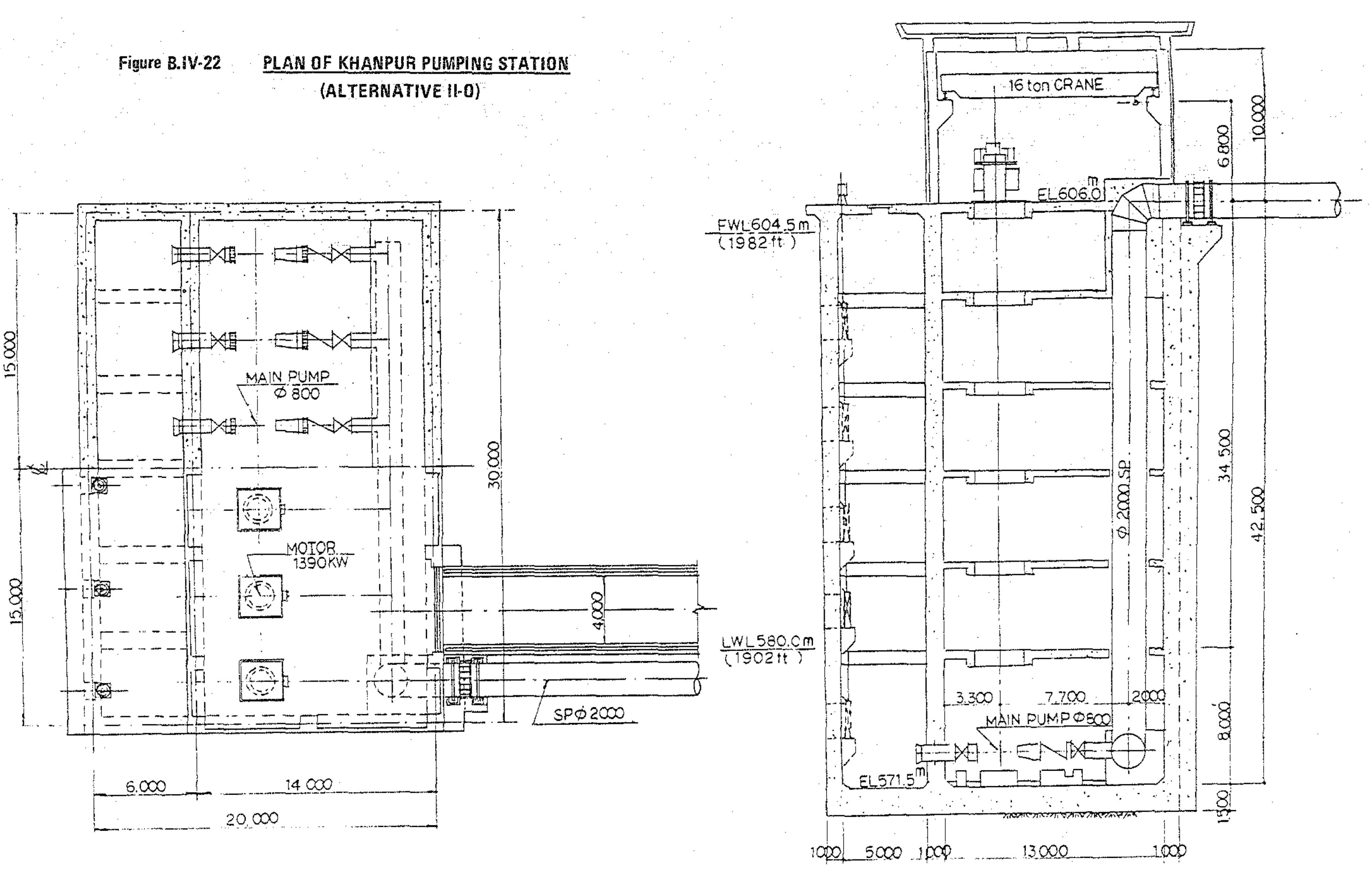


Figure B.IV-23 PLAN OF SANG JANI PUMPING STATION (ALTERNATIVE I-A)

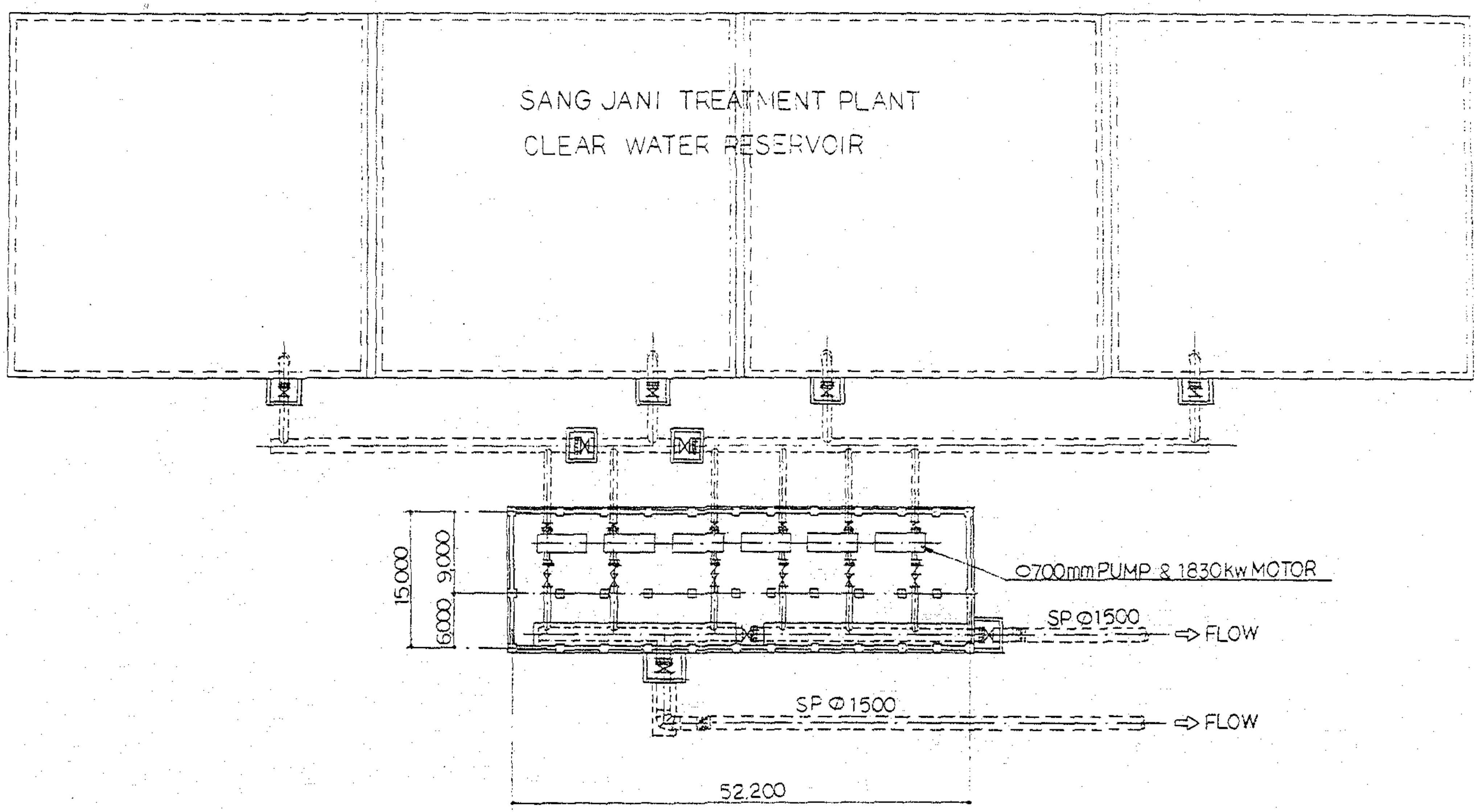
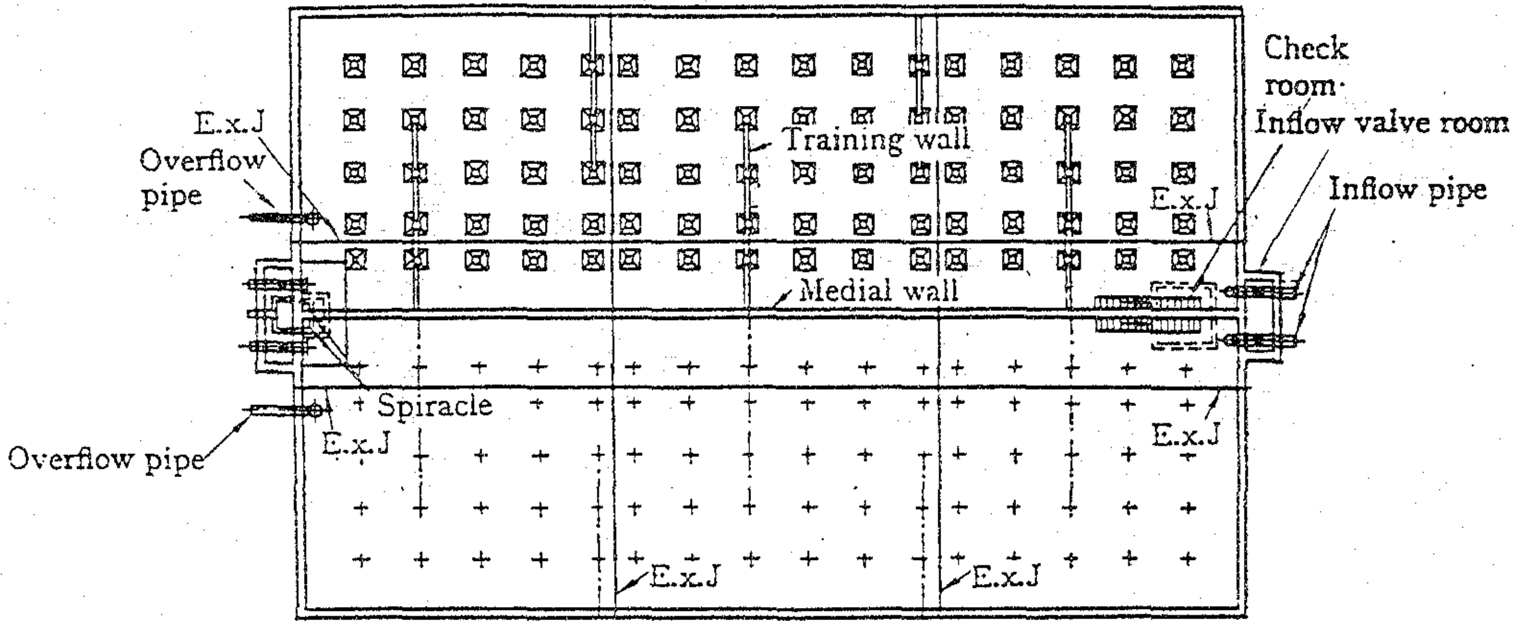
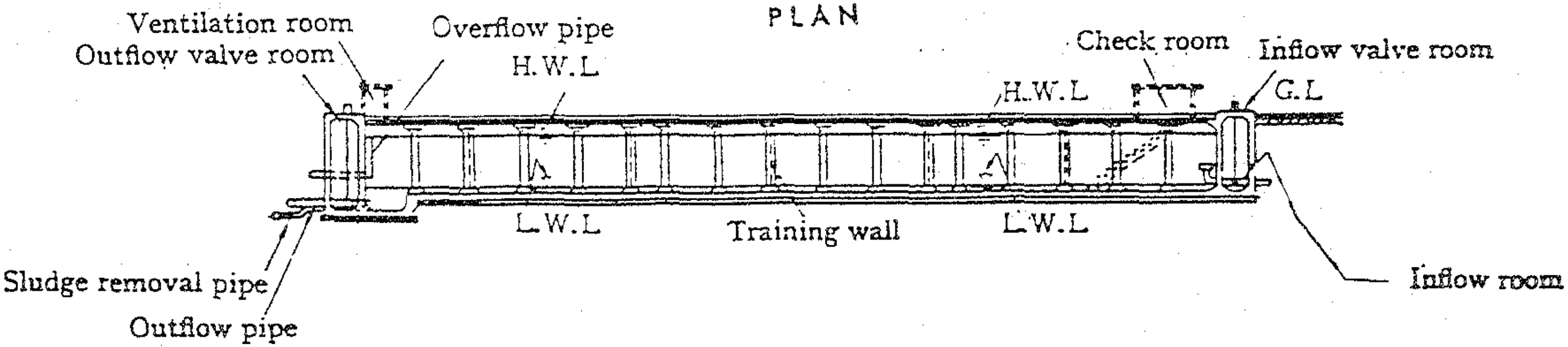


Figure B.IV-24 TYPICAL DRAWING OF SERVICE RESERVOIR (FLAT SLAB TYPE)

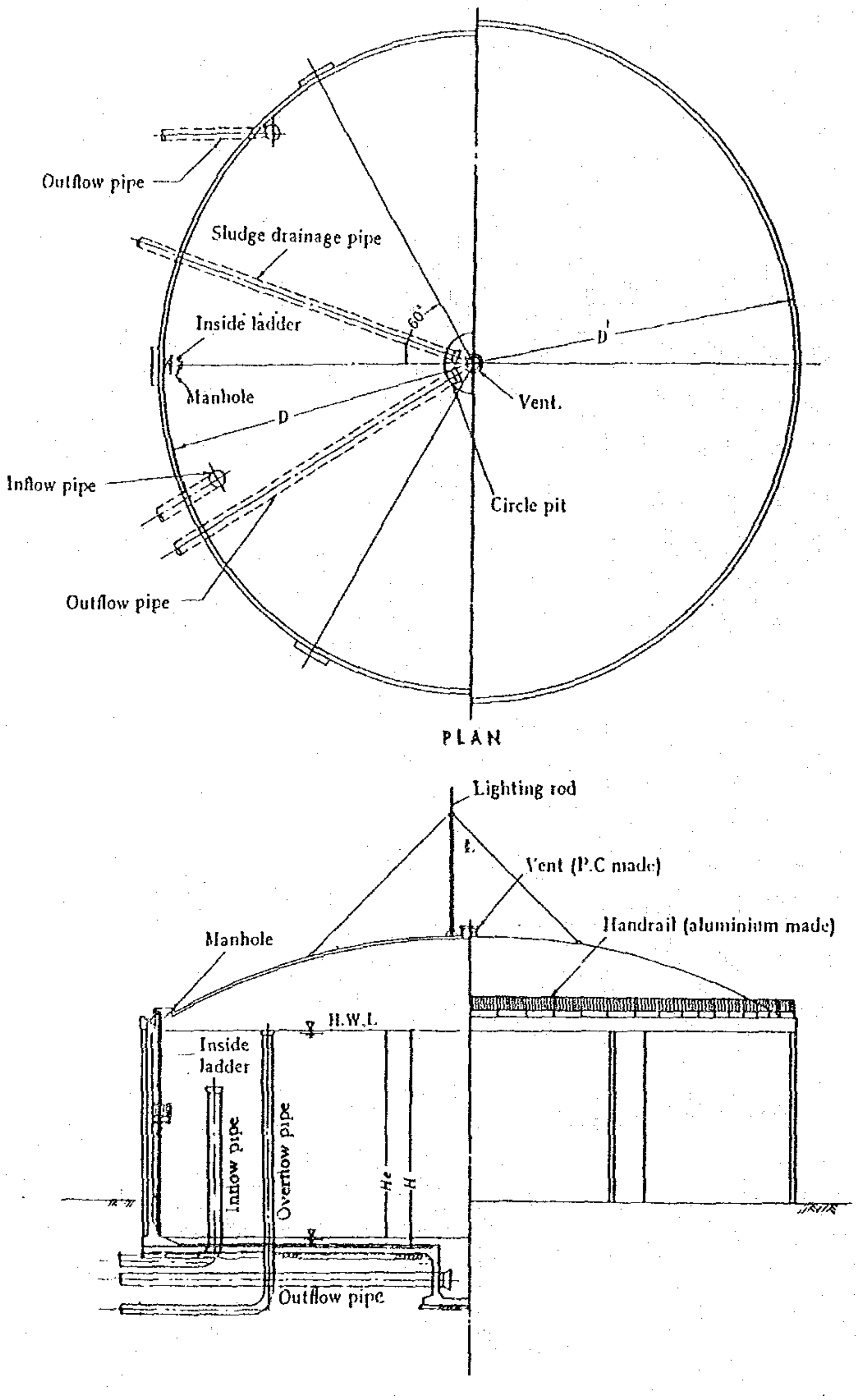


PLAN



CROSS SECTION

Figure B.IV-25 TYPICAL DRAWING OF SERVICE RESERVOIR
(PC-TANK)



CROSS SECTION

TABLE B.IV-1 IMPROVEMENT OF LEFT BANK CANAL

Facility and Specification		Alternative - I			Alternative -II				Alt.-III
		I - A	I - B	I - C	II - A	II - B	II - C	II-D	III-B
Gate facilities	(pl)	2	2	2	2	2	-	-	-
Conduit	(m)	550	550	550	290	-	-	-	-
Shoulder ditch	(m)	1,100	1,100	1,100	940	400	-	-	-
Berm ditch	(m)	5,120	5,120	5,120	4,860	2,880	-	-	-
Berm inlet	(pl)	52	52	52	52	52	-	-	-
Slop Protection	(sq.m)	20,100	20,100	20,100	20,100	12,600	-	-	-
Ditto (Mortar Spray)"		2,510	2,510	2,510	1,150	710	-	-	-
Drainage Canal	(m)	150	150	150	100	-	-	-	-
Others	(L.S)	6	6	6	3	2	-	-	-

TABLE B.IV-2 FACILITY AND WATER LEVEL (ALT.II-A)

Facility	Specification	II - A - 1 Length	II - A - 1 Water Level	II - A-2 Length	II - A-2 Water Level
		m	m	m	m
Intake point of L.B.C.			548.41		548.41
Head regulator		30		30	
			548.10		548.10
Feeder Conduit	2.50×2.50^{m}	820		820	
			547.60		547.60
Pumping Station		30		30	
			($HT=105^{m}$)		($HT=121^{m}$)
Delivery Pipeline	Dia.$1500^{mm} \times 2$	500		650	
			649.70		665.10
Discharge Pool	Effective depth (H=6.0m)	40		40	
			643.20		659.10
Pipeline (1)	Dia. $1650^{mm} \times 2$	800		1,000	
			642.00		657.10
Tunnel	Dia. 2100^{mm}	2,600		2,500	
			640.10		655.75
Pipeline (2)	Dia. $1650^{mm} \times 2$	1,000		1,400	
			638.60		653.65
Pipe Aquiduct	Dia. $1650^{mm} \times 2$	30		-	
			638.55		-
Pipeline (3)	Dia. $1650^{mm} \times 2$	700		-	
			637.45		653.65 (637.45)
Total		6,550		6,470	
(Reference)		Dia.800×500^{mm}		Dia.800×500^{mm}	
Pump Capacity (Q=6.40 cu.m/sec)	6 Units	Output 1800 KW		Output 2100 KW	

Table B.IV-3 FACILITY AND WATER LEVEL (ALT.II-C)

Facility	Specification	II - C - 1 Length	II - C - 1 W.L.	II - C - 2 Length	II - C - 2 W.L.	II - C - 3 Length	II - C - 3 W.L.
		m	m	m	m	m	m
Intake Point of L.B.C.			558.44		558.44	50	578.12
Head regulator		50	557.80	50	557.80	50	577.60
Feeder Conduit	2.30x2.30^{m}	400	557.50	500	557.40	400	577.30
Tunnel (1)	Dia. 2100mm	-	-	-	-	300	577.10
Syphon (Haro river)	Dia. 2000mm	-		-		700	
Pumping Station		30	- (HT=97^{m})	30	- (HT=99^{m})	30	575.20 (HT=80m)
Delivery Pipeline	Dia. 1500^{mm}x2	500	651.30	900	652.50	700	652.10
Discharge Pool	Effective Depth (H=6.0m)	40	645.30	40	646.50	40	646.10
Pipeline (1)	Dia. 1650mmx2	-	-	800	645.30	500	645.30
Tunnel (2)	Dia. 2100 mm	1,300	644.30	210	645.10	210	645.10
Pipeline (2)	Dia. 1650mmx2	1,940	641.40	2,510	641.40	2,510	641.40
Tunnel (3)	Dia. 2100mm	3,000	639.20	3,000	639.20	3,000	639.20
Shaft		-	-	-	-	-	-
Tunnel (4)	Dia. 2100mm	2,000	637.75	2,000	637.75	2,000	637.75
Pipeline (3)	Dia. 1650mmx2	100	637.60	100	637.60	100	637.60
		9,360		10,140		10,540	

(Reference)

		II - C - 1	II - C - 2	II - C - 3
1. Pump Capacity (Q=6.40 cu.m/sec)	6 Units	Dia. 800x500mm Output,1700 KW	Dia. 800x500mm Output 1700 KW	Dia. 800x500mm Output 1400 KW
2. Reducing Operation Cost by Tarmakki diversion dam	Reduction Volume 18,000,000cu.m Pump cap.per hrs 4,600 cu.m	3,920 hrsx1700 Kw ÷ 6,664,000 KWH 6,664x350 Rs = 2.33 M. Rs	Ditto 6,664,000 KWH Ditto 2.33 M. Rs.	3,920 hrsx1400 Kw ÷ 5,488,000KWH 5,488x350 Rs. =1.92 M. Rs.

Table B.IV-4 FACILITY AND WATER LEVEL (ALT.II)

Unit : Meter

Facility	Specification	II - A Length	II - A W.L.	II - B Length	II - B W.L.	II - C Length	II - C W.L.	II- D Length	II- D W.L.
Intake Point			548.41		557.23		578.12		580.12
Head regulator		30	548.10	30	556.80	50	577.60	-	-
Feeder Conduit	2.50×2.50^m	820	547.60	-	-	400	577.30	-	-
Tunnel	Dia. 2,100mm	-	-	-	-	300	577.10	-	-
Syphon	Dia. 2,000mm	-	-	-	-	700	575.20	-	-
Feeder Canal	3.30×2.00^m	-	-	200	556.60	-	-	-	-
Pumping Station		30	(HT≑105^m)	30	(HT≑95^m)	30	(HT≑80^m)	30	(HT≑77^m)
Delivery Pipeline	Dia. 1500mmx2	500	649.20	500	648.50	700	652.10	350	654.20
Discharge Pool		40	643.20	40	642.50	40	646.10	40	648.20
Pipeline	Dia. 1,650mmx2	800	642.00	-	642.50	500	645.30	4,580	641.40
Tunnel	Dia. 2,100mm	2,600	640.10	1,900	641.05	210	645.10	-	-
Pipeline	Dia. 1,650mmx2	1,000	638.60	-	-	2,510	641.40	-	-
Aqueductor Conduit	Dia. 1.600mmx2 1.90 x 1.90m	30	638.55	400	640.75	-	-	-	-
Tunnel	Dia. 2,100mm	-	-	2,300	639.10	3,000	639.20	3,000	639.20
Inclined Shaft		-	-	-	-	-	-	-	-
Tunnel	Dia. 2,100mm	-	-	2,000	637.65	2,000	637.75	2,000	637.75
Pipeline	Dia. 1.650mmx2	700	637.40	100	637.45	100	637.60	100	637.60
Total		6,550		7,500		10,540		10,100	
(Left Bank Canal Length)		(12,444)		(4,118)		(-)		(-)	
Grand Total		18,994		11,618		10,540		10,100	

TABLE B.IV-5 FACILITY AND WATER LEVEL (ALT.III)

Unit : Meter

Facility	Specification	III - A		III-B-1		III-B-2	
		Length	W.L.	Length	W.L.	Length	W.L.
Intake Point			578.12		580.11		580.11
Head regulator or intake tower		50		-		-	
			577.60		579.80		579.80
Conduit	2.30 x 2.30 m	400		-		-	
			577.30		-		-
Tunnel (1)	Dia. 2100mm	300		700		700	
			577.10		579.30		579.30
Syphon (Haro river)	Dia. 2000mm	700		-		-	
			575.20		-		-
Pressure break basin and conduit	2.40 x 240 m	-		550		550	
			-		578.00		578.00
Tunnel (2)	Dia. 2400mm	4,300		4,100		8,500	
			573.85		576.70		575.40
Vertical Shaft (1)	Dia. 6000mm	-		-		-	
			-		-		-
Tunnel (3)	Dia. 2400mm	4,800		4,800		-	
			572.35		575.20		-
Vertical Shaft (2)	Dia. 6000mm	-		-		-	
			-		-		-
Tunnel (4)	Dia. 2400mm	2,100		2,100		2,000	
			571.70		574.05		574.75
Pipeline	Dia. 1650mmx2	250		250		250	
			571.30		573.65		574.35
Total		12,900		12,500		12,000	

TABLE B.IV-6 CLASSIFICATION OF TUNNEL TYPE

Tunnel Type	Supports	Geological Condition	Rock Type 1/	Rock Load from Karl Terzaghi Hp(m)2/
A	No Supporting or Rock Bolt	Little cracked solid rock mass, moderately jointed	All of Type I Rock Part of Type II Rock	0 to 0.25 B
B	Steel Support H-100x100x6/8 @ 1.50m	Cracked and slightly weathered rock mass, or very consolidated soil	Part of Type II Rock Part of Type III Rock	0.25 (B+H) to 0.5 (B+H)
C	Steel Support H-100x100x6/8 @ 0.90-0.60m	Weathered rock, fault zone or consolidated soil	Most of Type III Rock All of fractured fault zone and unconsolidated sediment	0.6 (B+H) to 1.1 (B+H)

Note: 1/ Rock Type I : Massive limestone
II : Cracked limestone, alt. of limestone & marl, massive marl, sandstone
III : Shale, alt. of shale & marl, fissile marl, Murree Fm (sandstone & madstone) fault zone
IV : Unconsolidated Sediment, Overburden (silty clay, gravel)

2/ Height (H) and width (B) of tunnel excavation

3/ If type C with rock load greater than 0.8 (B+H), then interval of steel supporting should be shortened

Table B.IV-7 ROCK TYPES ENCOUNTERED ALONG TUNNEL ROUTES

Unit : Meter

Tunnel Route	Rock Type I	II	III	IV	Total
Alternative II-A					
Tunnel-1	1,220	700	680	0	2,600
Total	1,220	700	680	0	2,600
Alternative II-B					
Tunnel-I	0	1,600	300	0	1,900
-2 (Upper)	1,410	180	710	0	2,300
-2 (Lower)	390	1,150	460	0	2,000
Total	1,800	2,930	1,470	0	6,200
Alternative II-C					
Tunnel-1	0	300	0	0	300
-2	0	210	0	0	210
-3(Upper)	1,580	480	940	0	3,000
-3(Lower)	390	1,150	460	0	2,000
Total	1,970	2,140	1,400	0	5,510
Alternative II-D					
Tunnel-1 (Upper)	1,580	480	940	0	3,000
-1 (Lower)	390	1,150	460	0	2,000
Total	1,970	1,630	1,400	0	5.000
Alternative III					
Tunnel -1	0	700	0	0	700
-2 (Upper)	780	2,970	350	0	4,100
-2 (Middle)	1,900	1,630	1,270	0	4,800
-2 (Lower)	0	300	1,100	700	2,100
Total	2,680	5,600	2,720	700	11,700

Note : Rock Type
I- Massive limestone; almost no support needed.
II- Cracked limestone, alternation of limestone & marl, massive marl, Sandstone (cretaceous); supports partly needed.
III- Shale, alteration of shale & marl, fissile marl, alternation of sandstone & marl (Murree formation); supports needed
IV- Over burden (silty clay & gravel); supports heavily needed.

4.3. Preliminary Cost Estimate

4.3.1. Construction Costs

The preliminary cost estimates were made for major works such as tunnels and pipelines on the basis of work quantity calculated and unit prices provided by authorities concerned, and for other works such as buildings and service reservoirs relative costs obtained from similar works were applied. Prices of materials and equipment to be imported from foreign countries were estimated by the Team. Duties and taxes to be levied on such imports are included in the cost estimates in accordance with Pakistan Customs Tariff and Import Trade Guide.

The estimated costs include costs for construction works and procurement of materials and equipment. For the sake of comparative study of alternatives, the expenses needed for pre-construction works are not included in these cost estimates: they are costs for topographic and geological surveys, detailed design, consulting services, administration, supervision of construction works and so on. Physical contingencies (10%) were added to the construction costs.

Cost Summary of alternatives is given below; details are shown in Table B.IV-8 to 15.

For reference, total project costs of the representative three alternatives of I-C, II-C and III were calculated by adding costs for project offices, land acquisition, office equipment, engineering and administration, and contingencies. Physical contingencies (10%) were added to the estimated costs, and price increases were estimated by applying the annual rates of price escalation of 10% for local costs and 6% for foreign costs. The total project costs are summarized below (refer to Table C.IV-18 to 20).