MINISTRY OF HOUSING AND PHYSICAL PLANNING

DEPARTMENT OF WATER SUPPLY AND SEWERAGE

REGIONAL DIRECTORATE

COMMUNITY WATER SUPPLY AND SANITATION PROGRAMME

WESTERN REGION

DESIGN GUIDELINES

for

RURAL WATER SUPPLY SYSTEMS

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1. Introduction

Recent developments and long term observations on water supply systems made it necessary to revise the presently utilized Design Guidelines dated Shrawan 2044. The revision takes also care of the fact that in future the Community Water Supply and Sanitation Programme (CWSSP) will be active only in the Gandaki and Dhaulagiri zones of the Western Region. Lumbini Zone will be covered by the Rural Water Supply and Sanitation Programme (RWSS) a Finnida supported programme.
Two design flow charts are provided to guide the designer through the procedures of designing a rural water supply system namely:

- Annex I: Implementation of Water Supply Schemes Constructed with Community Participation
- Annex II: Preliminary Survey and Detail Design

Where necessary the charts refer to the chapter, where the issues are explained in detail.
As usual even the most detailed design guidelines will not be able to solve all the problems encountered when designing a water scheme. Under such circumstances the designer should not hesitate to discuss the issue with his superior and or the CWSS Programme Coordination (MHPP/DWSS RD Western Region).

2. Design Criteria

2.1 Population Projections

2.1.1 Present Population

The present population to be served by the scheme shall be estimated as follows:

Method A: (preferred method) Establishing the number of houses to be served by each individual tap. The population is derived from the assumption that in each house 5.5 persons are leaving.

Method B: The population figures are based on actual ward wise population figures adjusted accordingly if only parts of the ward are within the area of supply. This method should be used to counter check the population figures based on method A.

Population for rural areas and bazar areas should be established separately since different water consumption figures and peak flow rates are applicable.

2.1.2 Population Growth Rate

To forecast the future water demand it is necessary to make reasonable population projections. Due to the lack of reliable and statistically relevant data material it is necessary to introduce certain assumption in regard to the population growth rate. For Gandaki and Dhaulagiri zones the computed population growth rates

based on the 2028 and 2038 population census vary between -7.1 percent (Mustang) and +3.8 percent (Kaski). In comparison for the same time period the national growth rate was +2.66 percent and for Western Region only +2.32 percent.
Furthermore it should be appreciated that growth rate for rural areas and bazars can not be the same, therefore average growth rate for villages and bazars areas were established as follows:

Table 1: Annual Population Growth Rates (in percent)

Zone	Gandaki and Dhaulagiri [1]
Village area	2.3
Bazar area	3.0
School, Health post	3.2

Note: 1) excluding the Districts of Mustang and Manang.

1.3 Design Period and Useful Service Life

Taking into the considerations the investments made by all the parties concerned when constructing a water scheme, and the institutional capacity of the country as a large, are some of the factors which determine the wishful service life period of a water scheme. Presently and in the absence of any other supporting data it is assumed that a water scheme should serve the population for a period of at least 20 years.
Under certain circumstances it may be economically advantageous to consider phased implementation of a scheme e.g. by reducing the design period for certain parts of the scheme to 10 years like storage tanks or any other structure, but notwithstanding the overall implementation plan of the scheme will be based on a service life of 20 years.
It should be kept in mind that the economical savings of a phased implementation must be considerable to counter balance the logistic problems created e.g. re-mobilizing the communities, labour forces etc. or to get the project into the list of approved HMG projects at the time the additional structures are needed.
The service life of a scheme starts on the day of handing over the water supply to the users committee. Henceforth the time required to design and implement a scheme, which is for CWSS schemes about 2 to 3 years, must be added to the service life period to arrive at the design period. In general the design period is assumed to be 22 years if not stated otherwise.

1.4 Population Growth Factors

The population growth factors are the mathematical computation based on the population growth rate and design period as stated in the fore going chapters.

$$P_F = (1 + \text{Population Growth Rate}/100)^{\text{Design Period}}$$

$$P_F = (1 + P/100)^n \qquad \text{General Formula}$$

Hence the population growth factor for Village respectively Bazar areas are as follows:

Table 2: Population Growth Factors

Zone:	Gandaki and	Dhaulagiri
Design period (years):	12	22
Village area	1.31	1.65
Bazar area	1.43	1.92
School, Health post	1.46	2.00

2.1.5 Design Population

With all the data material available it is possible to forecast the population to be served by the scheme during various stages of its service life e.g.:

- Present Population at the time of designing the scheme[1)]
- Future Population after 10 years service life (needed only if phased implementation is considered)
- Ultimate Population after 20 years service life

1) This is a simplified assumption!. Normally the present population is computed for the time of commissioning of the scheme.

Example of "Design Population Computation" is shown on Annex III.

2.2 Livestock

Although it is appreciated that it may be advantageous to provide water for the need of the livestock, at the present time there is no economical justification to include water demands for livestock in CWSS schemes. In most rural areas indigenous cows and buffaloes are kept. Since the livestock milk production is very small it will not be greatly affected if they have to go some few hundred meters to a watering place.

2.3 Other Possible Users

On CWSS schemes other water users like institutions and or industries are not considered.
It should be borne in mind that the schemes are built with community participation and therefore individual / institutional or industrial needs above the per capita consumption can at the present time not be considered. Institutional requirements of schools, health post etc. can be included since they serve the whole population.

2.4 Water Consumption

2.4.1 Domestic Water Consumption

According to the CWSS Programme objectives the water supply shall reduce the prevalence of "water borne" and water washed diseases. The amount of water required is as follows:

- to prevent "water borne" infections of high quality water	10 lcd
- to prevent "water washed" infections of medium quality water	50 to 70 lcd
TOTAL	60 to 80 lcd

The CWSS Programme has adopted the following water consumption figures as recommended by WHO for rural schemes:

Table 3: Domestic Water Consumption Figures

Type of Supply Area	Daily Water Consumption
Rural Village	45 lcd
Bazar Area	60 lcd

A survey carried out in 1980 on some few CWSS schemes of the Western Region revealed that the actual consumption figures for a rural village is between 20 and 40 lcd and for a bazar area 50 to 70 lcd. For economical reasons it is not possible to increase the consumption figures and furthermore higher water consumption figures will reduce the number of feasible projects especially in areas where water is scare and henceforth water supplies are of great importance.

2.4.2 Institutional and Other Water Consumption

As mentioned in chapter 2.3, CWSS water supplies provide water for public institution like school and health post only. The respectively water consumption figures are given in Table 4.

Table 4: Institutional Water Consumption Figures

- School	6 ld	per student
- Health post	1000 ld	for out patient only and without improved sanitary condition e.g.WC
	3000 ld	for out patient only with improved sanitary condition

2.4.3 Water Loss Figures

It is an unavoidable fact but in any water supply system a certain amount of water is lost for various reasons (leakages, spillage, illegal use etc.). **Water loss** figures for rural water schemes vary

between 30 to 70 percent with a mean value of 50 percent depending on various factors like construction quality, operation and maintenance, occurring pressure etc.
For the sake of simplicity it is assumed that the above mentioned water consumption figure include an "unavoidable" water loss of approximately 20 to 30 percent. It is for this reason that the above mentioned water consumption figures can not be further reduced.

2.5 Water Demand Projections

Based on the population forecast and the water consumption figures it will be possible to forecast the water demand to be met by the scheme during various stages of its service live e.g.

- Present Water Demand	at the time of designing the scheme[1)]
- Future Water Demand	after 10 years service life (needed only if phased implementation is considered)
- Ultimate Water Demand	after 20 years service life

1) This is a simplified assumption! Normally the present water demand is computed for the time of commissioning of the scheme.

Water demands for standpost serving various users groups e.g. domestic (rural and school or bazar etc.) should be computed on the base of the various users groups population and consumption figures.
Example of "Water Demand Calculation" is shown on Annex III.

2.6 Proposed Source

2.6.1 Source Yield Assessment

At the time of carrying out the feasibility study the available source yield(s) and the water demand are for the first time assessed. The result of this assessment should be reflected in the chosen area of supply whose water demand should be met or surpassed by the safe yield of the source. At this stage after having computed the ultimate water demand it is necessary again to compare the available source yield versus the projected water demand. The available source yield, the number and time when such source yield measurements were taken must fulfill the following minimum criteria:

a) The source yield should be able to meet the stated demand of 45 resp 60 lcd plus institutional needs like schools health posts during the peak of the dry season.
b) For sources supplying less than 2000 persons (design population) the yield of the source must be measured at least twice at the peak of the dry season.

c) For sources supplying more than 2000 persons (design population) the yield of the source must be measured during at least two dry seasons to be able to compute the source yield recession curve.
d) If source yields are not measured at the peak of the dry season the flow measurements must be discounted as deemed necessary by the District Engineer, to arrive at the safe yield of the source.
However the CWSS Programme Coordination reserves the right to postpone the project implementation until reliable source yield measurements are available.
d) flow measurements taken during the monsoon season will not be considered at all.
e) Since most of the catchments are not able to trap all the water available the measured source yield must be reduced by a factor smaller then 0.9 to arrive at the safe yield of the source. The recommended reduction implies that at least 10 percent of the available water is lost through leakages on the catchment structures

2.6.2 Feasibility of Proposed Water Scheme

Based on the above mentioned condition it is possible to computed the available safe source yield and compare it with the forecasted water demand of the planned scheme. In case the source yield is insufficient to met the water demand then the project is regarded as not feasible. It is necessary either to re-dimension the area of supply, to develop additional source if possible, or to drop the project completely.

2.6.3 Type of Supply System

In the CWSS Programme two different types of supply systems are known, namely:

- Closed System and
- Open System

The basic difference between these two supply systems are:

- In an <u>open system</u>, as the name implies there is no device e.g. bib tap installed to stop the flow of water by means of closing the pipeline at the tapstand. Strictly speaking the name tapstand is wrong since they are not equipped with a tap, watering point may be more appropriate. In an open system the source yield must be sufficient to meet the peak flow demand without the need for a storage tank. Furthermore since there are no closing devices installed no Break Pressure Tanks are required to prevent the build up of excessive water pressure in the pipeline.
  To reduce the environmental problems caused by the continuous flow of water from the tapstands it has been observed that tapstands of open systems are equipped with a closing device. However such a system will be regarded as a "closed system" as defined below.

- In a closed system, as the name implies, devices are installed to stop the continuous flow of water by means of closing the bib tap which will result in a closed pipeline. In an closed system the source yield is not able to match the peak flow demand henceforth a certain amount of storage volume is required. Furthermore Break Pressure Tanks may be necessary if the max static pressure exceeds the pressure conditions as defined in Chapter 3.6.

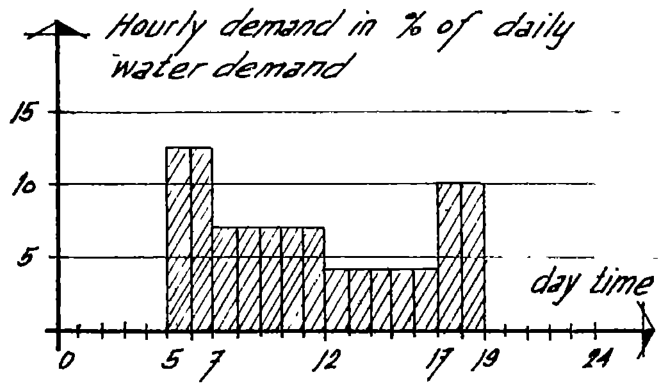
2.7. Pattern of Water Consumption

The pattern of water consumption influences directly the peak flow rates in the distribution system and the size of storage tank required.
The consumption pattern as shown in Table 5 and Figure 1 is based on some few field observations and on some assumptions. Nevertheless even the most detailed consumption pattern will not be able to provide much more accurate figures since consumption pattern vary from one supply system to another.

Table 5:
Pattern of Water Demand

Time Period	Consumption of daily water demand (%)
5^{00}	
	25
7^{00}	
	35
12^{00}	
	20
17^{00}	
	20
19^{00}	
	0
5^{00}	

Figure 1:
Pattern of Hourly Water Demand



2.7.1 Hourly Peak Demand Factor

Strictly speaking the above mentioned consumption pattern is valid only for.

- a max hourly peak demand factor of P_{HF} = 3,0 and
- an average hourly peak demand factor of P_{HF} = 1.7

The above made assumptions hold true for CWSS schemes serving a population between 500 and 2000 people. Therefore the computation of the required storage volumes is based on these hourly peak de-

mand factors. For bigger schemes the hourly peak demand factor should be decreased and for smaller schemes increased resulting in smaller respectively bigger storage tanks.

2.7.2 Instantaneous Peak Flow Factor

The peak flow factor is used to determine the pipe sizes of a distribution system. The peak flow factor shows the ratio between such a peak flow and the average flow rate based on 24 hours continuously flow.

Peak flow factor:
P_F = Peak flow rate/24 hours continuous flow rate

Basically the peak flow rate determines on how quick a water container can be filled. The higher the peak flow rate resp. the peak flow factor, the quicker a vessel will be filled. Thus a higher peak flow rate will provide for its users a greater convenience, but on the other hand increases the pipe diameter required. If the flow rate exceeds a certain amount it will cause a lot of water wastage due to spillage.
The following peak flow factors are used for CWSS schemes:

Table 6: Peak Flow Factors

	closed system	open system
Village (tapstand)	3.75	2.50
Bazar (tapstand)	3,00	2.50
School	Same as village or bazar systems. Peak water demand of village/bazar and school do not occur at the same time. In cases where the school is supplied by an individual pipeline of more than 500 m length a peak flow factor of 6 for closed system and 4 for an open system may be appropriate.	

Note: With the introduction of nominal tap flow rates the above mentioned peak factors are given for information and control purpose only.

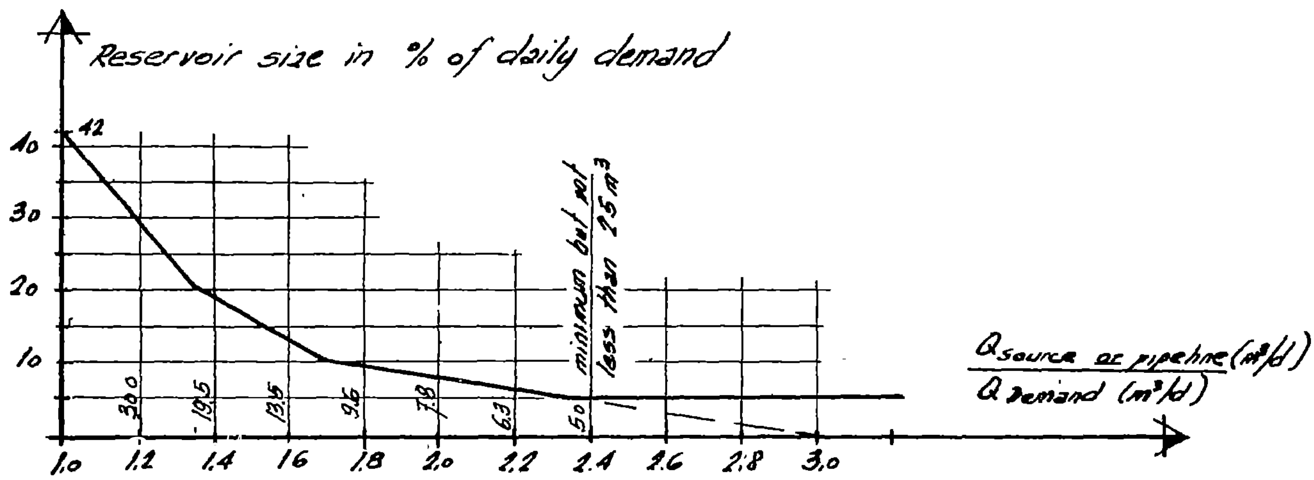
2.7.3 Required Storage Capacity

The function of a storage tank within a gravity supply system is to balance the continuous water production by the source and the greatly varying water consumption.
The size of the storage tank is determined by the following factors:

- average hourly peak water demand
- maximum hourly peak water demand
- the source yield or the pipeline capacity

For the given water consumption pattern the maximum storage capacity required is 42 percent of the daily water demand under the condition that the safe source yield resp. the gravity pipeline capacity is equal to the water demand. If the safe source yield resp. the gravity pipeline capacity is bigger, the storage tank volume can be reduced according to the graph shown in Figure 2. However the minimum storage volume to be provided shall be 5 percent of the daily water demand or 2.5 m³ which ever is bigger.

Figure 2: Storage Tank Capacity



3. Design Consideration

3.1 Friction Loss in Pipelines

At present the whole CWSS Programme utilizes the friction loss factor of k = 0.0 1 mm for PE pipes. From the point of view of a pipe manufacturer this may seem to be a reasonable friction loss factor, but from the practical point of view this friction loss factor is too low.
The following friction loss factors are proposed:

k = 0.1 mm for: PE pipes in distribution systems and gravity pipelines
k = 1.0 mm for: GI pipes and for PE gravity pipelines where there is a possibility of material deposits in the pipelines

Furthermore it has been observed that for velocities probably above 3 m/s the friction loss computed with e.g. Colebrook- White formula is too small, most likely due to the fact that we are dealing no more with pure water but an air/water mixture.
Head loss graphs for the above stated friction loss factors are attached to this Guidelines (ref. Annex IV).

3.2 Flow Velocities in Pipelines

3.2.1 Minimum Velocity

Special attention should be paid to the gravity pipe lines carrying water from river intakes to the storage tank. If no sedimentation is provided the minimum flow velocity shall be:

- in down hill stretches	0.8 m/s
- in up - hill stretches	1.0 m/s

If a sedimentation is provided the minimum flow velocity can be reduced to:

- in down hill stretches	0.4 m/s
- in up - hill stretches	0.5 m/s

3.2.2 Maximum Velocity

In an attempt to destroy surplus head small sized pipes are utilized resulting in very high velocity. On gravity systems, under normal operational conditions the water hammer phenomenon is hardly noticed, but on CWSS schemes water hammer development has been observed due to sudden closure of valve(s) combined with very high water flow velocities in the pipe lines.
The designer should be aware that in the case of a instant closer of a valve and the following water flow velocities in the pipes, a water hammer equal to the permissible working pressure ma develop:

- on PE pipes pressure class 6 kg/cm2	v = 2.3 m/s
- on PE pipes pressure class 10 kg/cm2	v = 2.8 m/s

Although these are pure theoretical figures it proves that a balance must be struck between destroying surplus head and the danger of providing an environment where water hammers can easily develop. Therefore the maximum velocity in pipe lines should be restricted to:

- desirable less than	2.5 m/s
- maximum	3.0 m/s.

3.3 Tap Flow Rates

In the past individual tap peak flow rates were computed and the pipe sizes based on these figures. To simplify the design work three nominal tap flow rates are introduced.

Table 7: Tap Flow Rates

Ultimate Water Demand per Tap (l/d)	Peak Flow Rate (l/s)	Remarks
3400 - 4600	0,15	Small cluster of houses
4600 - 5800	0,20	Village
5800 - 7000 (7600)	0,25	Bazar or Village with School / Health post.

(...) maximum

The sum of all tap flows should be roughly the same (not more than +/-20 percent) as the peakflow derived with the following formula.

Q_{PF} = (Ultimate Water Demand / 86400) x Peak Flow Factor
Note: For appropriate peak flow factors refer to Table 5

3.4 System Flow Rates

The nominal tap flow rates given in the foregoing chapter will result in the following peak flow factors depending on the number of people served:

Nominal Flow Rate	Design Population Served	Peak Flow Factor	Remark
0.15	75 - 100	3.8 - 2.7	Village
0.20	100 - 130	3.8 - 3.0	Village
0.25	100 - 120 (130)	3.8 - 3.1 (2.8)	Bazar

Although the given peak flow rate is for an individual tap it may be argued that not all the taps of a supply system are open at the same time therefore the peak flow rates for the pipes can be reduced. Experience shows that these peak-flow rates can be applied for a whole supply system serving up to 2000 people, since such a number of people are mostly served by two or more branch lines. Henceforth the system flow rates are directly based on the nominal tap flow rates.
Specially for big and or complicated supply systems it is advisable to prepare a schematic system lay out which shows the various branch lines, tap stands BPTs etc. The nominal tap flow rate is entered at each individual tap and the determining pipe flow rate is derived from the addition of all the tap flow rates served by the pipe portion under consideration.

3.5 System Lay Out

Definition:

- Gravity Pipelines: In the context of these guidelines a pipeline which is not taking part in the distribution of water and whose water is moved by the force of gravity from the inlet point to the outlet point is called gravity pipeline. In most cases grav-

ity pipelines are feeding storage tanks on a 24 hours basis (Intake - Storage Tank; Storage tank - Storage tank)
- Distribution pipelines: As the name implies these pipelines are used to distribute the water to the various consumers. The pipe sizes are determined by the peak flow rates.

3.5.1 Gravity Pipelines

Gravity pipelines are dimensioned for an continuous 24 hours average flow. For economical reasons it is important to place the storage tank(s), which is the termination point of the gravity pipeline as close as possible to the supply area.

3.5.2 Distribution Pipelines

Distribution pipe sizes are determined by the peak flow rate. It has been observed that the designer utilizes all the pipe sizes available in some cases just for some few meters. For the sake of simple O&M and keeping spare pipes the number of pipe sizes utilized should be reduced as far as economically justifiable.
At present the supply areas are covered with some branch lines from the top to the bottom. This requires a lot of BPT's each of them being a O&M headache. Following with the pipe alignment more the contour lines, although it may need more pipes, may reduce the number of BPTs.

3.6 Static Pressure

Definition:

The static pressure in a pipeline is equal to the difference in elevation between the point of consideration and the open end of that pipeline (where the water table is exposed to atmospheric pressure only) which is in most cases e.g. a Storage Tank or BPT. At the point of consideration the static pressure will occur only at the time when the flow in the pipeline is 0.

3.6.1 Gravity Pipelines

The maximum static pressure in gravity pipelines feeding storage tanks and BPT's on a 24 hours basis and which are located outside the distribution area is as follows:

- for PE pipes pressure class 10 kg/cm^2 not more than 100 m
- for GI pipes pressure class conforming to BS 1387 medium grade not more than 160 m

The maximum pressure shall not exceed 16 kg/cm^2 since most valves and fittings are dimensioned for a nominal pressure of 16 kg/cm^2. In addition it should be borne in mind that it needs a great skill to properly install a pipe line and to get the same water proof, if the static pressure is exceeding 16 kg/cm^2.

But at the inlet of the storage tank resp. BPT the maximum occurring static pressure shall not exceed:

- desirable 60 m
- in exceptional cases 100 m

Gravity pipelines within the distribution area must conform to the pressure criteria of the distribution pipelines (ref chapter 3.62)

3.6.2 Distribution Pipelines

In a distribution system there is a continuous change of flow condition caused by the opening and closing of numerous taps. These flow rates changes may create pressure waves, and if there is an abrupt change of flow e.g. by a sudden closure of a tap or valve a pressure wave of great magnitude will be created called waterhammer. In normal supply systems this pressure waves are not noticed at all, however in the case of a waterhammer occurrence a noise may be noticed and in extreme cases the pipeline may burst.
The development of waterhammer can be avoided by using appropriate design guidelines and installing slow closing bib taps and valves. However the continuous change of pressure in a pipe system cannot be avoided. The greater the pressure variations are the quicker aging of the pipe material and associated fittings will be observed. Bib taps are the most frequently operated fitting in a water supply system. They have to withstand the rough handling by the population concerned and they should remain water tight under all occurring pressure conditions. It is for these reasons that the maximum pressure in the distribution system shall be limited to:

- Maximum static pressure 60m even if pipe material with a permissible working pressure of 10 kg/cm2 is used.
- Exception to the above mentioned maximum will only be granted if a distribution pipe is crossing a not inhabited river valley, gully and where there is no chance that in future people will settle. Under such conditions the pressure may be increased to 80 m.

Self-closing taps with almost instantaneous closure mechanisms such as Jayson Taps should only be used where the static head at the tapstand is not more than 20 m and the design flow at the tap is less than 0.20 l/s.

3.7 Residual Pressure

Definition:

The residual pressure required at a tapstand, BPT or storage tank is determined by the following factors:

- Appurtenance head loss, which is caused by the design flow rate passing through a bib tap, float valve etc.

- Pipe installation head loss, which is created by the design flow rate passing through the pipe installation within the structure. For economical reason this part of the pipeline is often reduced in size and therefore the friction loss may be of considerable magnitude
- Safety head; to safeguard the system against survey inaccuracies

Considering the above mentioned factors the following residual pressure should be maintained:

Table 7: Residual Pressure

Structure		Residual pressure
- Tapstands:	ideal	5 to 10 m
	acceptable	10 to 15 m
- BPTs and Storage Tanks		10 to 20 m

Below some usual head loss figures for system appurtenances:

Note: The given head loss figures are equal to 50 % of the recommended residual pressure except for bib tap at nominal flow rates.

	Flow rate (l/s)	Head loss (m)
A) Float valve diam 1"	1.1	5
(12 mm orifice)	1.6	10
B) Float valve diam 2"	2.1	5
(piston type)	3.0	10
C) Bib tap diam 1/2"	0.29	2.5
	0.42	5
	0.51	7.5
Head loss for bib taps at nominal flow rates:		
D) Bib tap diam 1/2"	0.15	0.6
	0.20	1.1
	0.25	1.8

Note: Headloss figures are for guidance only they may vary considerable depending on the make and manufacturer.

3.8 Length of Water Supply System

Pipe length and water losses are somehow interrelated. It has been observed that very small source yields were proposed to be carried over kilometers of pipelines. To avoid waste of the meager resources it is proposed to limit the permissible pipe length in relation to the source yield as follow:

- source yield: permissible pipe length
  - < than 1 l/s 750 m per 0.1 l/s yield
  - > than 1 l/s 1000 m per 0.1 l/s yield

4. Water Supply System Components

4.1 Pipelines

Investmentwise the pipe material represents about 35 percent of the total cost of a water scheme (incl village contribution). However it has been observed that choosing pipe alignments does not receive the attention it deserves from the investment point of view.

Below the most important points when choosing a pipe alignment.

- The pipe track should avoid geological unstable areas, like landslides, gullies, streams etc. If this is not possible than such area should be crossed with suspended pipe bridges, if technically feasible.
- The pipe line gradient shall be uniform for an as long stretch as possible. To many changes in gradient will cause a lot of high points and not every high point created can be equipped with an air valve for economical reason.
  High points which are not very clearly distinctive will not always cause air block problems but they will reduce the flow capacity due to entrapped air.
  To minimize the number of pipe gradient changes the pipe shall be laid with a cover varying from the minimum of 80 cm to a maximum of 2 to 3 m.
- The minimum gradient of a pipeline shall be 2 percent which means that no pipeline should be laid horizontally.
- On distinct low resp high points wash out resp air release valves shall be installed.
- On river intakes "telescopic" pipe lines must be avoided e.g starting with a big diameter and ending with a small diameter pipe. Such pipelines tend to get blocked since bigger items may enter the pipe but are not able to leave it.
- Pipelines between intake (river, spring etc.) and Collection Chamber shall be at least of diam 40 mm, since water from a newly constructed catchment is always carrying soil and sand at the onset of the monsoon period.
- Pipe stretches parallel to the Hydraulic Grade Line (HGL) should be avoided since they create unstable condition whereby the entrapped air can not move in either direction. This point needs to be considered especially on gravity pipelines since on distribution pipeline the HGL is continuous changing in relation to the varying flow rates.
- It has been observed that following existing foot path is of advantage even if the pipe line length is increased. People using the path will observe any leakage and most likely the Village Maintenance Worker will be informed. In places where it is not possible to follow an existing path, it is advisable to create such a path, of course, in cooperation with the villagers concerned.

4.2 Structures

4.2.1 Intake and Collection Chamber

Each source intake irrespective whether river or spring, is unique and for these reasons there will never be a universally applicable intake design standard. Below some guidelines regarding intake structures (additional information on the design of intake structures are provided on the Annex VI & VII):

a) River Intake (for additional information refer to Annex VI):
Whenever practicable a river intake should be located:
- on a river whose main catchment area is covered with forest
- up stream of populated or farming areas
- up stream of a foot path, bridge, cattle watering and laundry washing places
- on a place where the immediate up stream area of the intake is not easily accessible to people and livestock
- where the ground is firm (e.g. rocky)
- outside of geologically unstable areas
- where it is possible to construct an intake with the available construction technique and skill to withstand monsoon floods
- at the outside of a river bend

b) Spring Intake (for additional information refer to Annex VII):
As compared to river intakes the locality of the spring intake is determined by the spring itself. Any spring intake which permits water contamination by surface water, people, livestock, etc. is regarded as a river intake.
If it is possible to choose between various springs then similar selection criteria may apply as for river intake.
1) the spring catchment area should be covered with forest
2) the following spring locations are preferable:
   - above populated or farming areas
   - above foot path, cattle watering and laundry washing places
   - where the immediate surrounding above the spring is not easily accessible to people and livestock
   - outside of geologically unstable areas
   - where the general terrain condition do not favour the development of swamps e.g. during the monsoon period

Collection Chambers are built for the following purposes:
- rough settling of coarse material contained in river or spring water
- to remove floating matters like leaves, branches etc. from river water
- safely dispose of surplus water caught and which is exceeding the pipe capacity
- to inspect the flow from the catchment, therefore if more than one source is utilized each source should have its own individual pipeline to the collection chamber for inspection purpose

If there are no emergency overflows installed at the spring catchment than the pipeline between catchment and collection chamber

must be able to safely discharge the maximum monsoon season discharge of the spring without causing any water impounding behind the spring catchment.
The overflow and drainage capacity of the collection chamber must be equal or greater than the maximum capacity of the pipeline between intake and collection chamber.

4.2.2 Sedimentation Tank

Water from streams or rivers may contain suspended matters like leaves, branches, soil and sand. These contamination may cause the following operational problems:

a) due to silt deposit
   - reduction of pipelines capacity until the extreme situation when a pipe becomes blocked
   - reduction of storage capacity
b) increase of wear and tear on system appurtenances e.g. valve stuffing boxes are worn out, valves can not be closed and air valves are leaking constantly.
c) in case of high water velocity it may cause scouring on the pipe surface

It should be well understood that plain sedimentation is not able to remove all contaminations. This fact is further exaggerated by the type of sedimentation unit utilized.
CWSS uses standard ferrocement storage tanks with small adaptations to suit a sedimentation tank (ref. Annex VIII). Since these tanks do not follow the design principal of plain sedimentation they do not have a very high efficiency factor.
Not all river intakes need to be equipped with a sedimentation tank. Depending on the catchment area (farming, forest) the river will have different level of contamination. If it is not possible to watch the streams behaviour during the monsoon season then at least the villagers should be consulted.
Furthermore in case of utilizing more than one source it may be possible to shut off the dirty source from the supply system during the rainy season since the flow rate of the clean source during this season may be sufficient to meet the water demand.
The size of sedimentation tank required depends mainly on the flow velocities in the pipe system especially the flow velocity in the gravity pipeline, and the type and size of particles carried by the river water. The following period of detention are recommended:

- 2 hours for rivers which carry relatively coarse material only and for pipe systems where the computed flow velocities in the main pipes is above 1 m/s.
- 4 to max. 6 hours for rivers which carry fine material and for distribution systems where the computed flow velocities in the main pipes is below 1 m/s.

4.2.3 Storage Tank

Storage tanks are the most costly structure build in a water sup-

ply system, and at the same time the most complicated ones. The sitting of the storage tank should be done very carefully keeping the following points in mind:

- it is cheaper to have a long gravity main which is dimensioned to carry the 24 hours average flow than to have long distribution pipe system whose sizes are based on the peak flow rate
- the storage tank should be located only 5 to max 10 m above the highest located tapstand of a supply area, single taps supplying just some few houses may be connected directly to the gravity pipeline via a small storage tank
- the storage tank site should be located on the lower edge of a terrace to avoid long pipelines carrying peak flows in flat terrain
- storage tanks must be accessible at any time and should not be located within rice fields (possibility of water contamination)
- it should also be considered, instead of building one big reservoir, to construct several small reservoirs cum BPTs.

4.2.4 Break Pressure Tanks (BPT)

At any kind of structure or device where the water is permitted to discharge freely into the atmosphere the hydrostatic pressure will be reduced to zero and therefore such a structure or device will act as a break in the build up of hydrostatic pressure. Henceforth any open vessel like storage tank, sedimentation tank, collection chamber, distribution chamber will act, besides its original purpose also as break pressure.
Break Pressure Tanks are build for the sole purpose of reducing the maximum occurring static pressure within the limit set (ref chapter 3.6). Tapstands and branch off - tees should be located just down stream of a BPT.

- BPT sites should be located on the lower edge of a terrace to avoid long pipelines carrying peakflows in flat terrain
- BPTs must be accessible at any time and whenever possible should not be located within rice fields (possibility of water pollution).

There are two types of BPT:

a) BPT equipped with float valve. This type is the most common one and is utilized in distribution systems (closed systems)
b) BPT without float valve. This type is also called interruption chamber and is mostly used on gravity pipelines

4.2.5 Distribution Chamber (DC)

To split a area of supply into easy manageable sub-systems of 10 to 15 taps it is also necessary to share the total amount of water available accordingly. The respective amount of water for each sub-system will be carried by separate feed pipelines to the various storage tanks.
The Distribution Chamber provides an environment in which from the hydraulic points of view the water can be distributed to a reason-

able accuracy. The distribution chamber used in the CWSS standard will work satisfactory up to a flow rate of 2 to (3) l/s.

4.2.6 Tapstands

Tapstands are the most frequently used component of the entire supply system. No other structure will face more abuse than these structure which in addition should fit in as closely as possible with local social and cultural needs. A tapstand if its location has been carefully chosen will be more than a just a physical structure, it will become a new and important meeting point for the villagers served. Properly located, designed and built tapstand will be a clean attractive and inviting place (Abstract from Handbook of Gravity Flow Water System by T.D. Jordan Jr.).
Special attention must be paid to a well protected tapstand surrounding and proper drainage. In places where it is not possible to dispose safely of the waste water soak pits of appropriate size must be constructed. Besides being the water collection point the tapstand area must also be suitable for cloth washing and bathing. The tapstand spacing is determined by the following two factors:

a) maximum desirable walking distance to fetch water
b) the number of people a tapstand is able to serve conveniently

The tapstand spacing guidelines used as far are appropriate in areas with high or medium population densities. However strict adherence to this guidelines in areas with a low population density leads to taps serving 2 or 3 houses only. To prevent, that in areas with low population densities the tapstand becomes a individual yard connection the following tapstand spacings are introduced (ref. Table 8).

Table 8: Tapstand Spacing

Tapstand spacing	desirable	low density area	
horizontal	150 m	250 m	350 m
vertical	50 m	42 m	35 m

Note: The various tapstand spacings are derived from the formula 4000m/300m x vertical dist. + horizontal dist. = approx. 816 which means 4000m horizontal distance is = to 300m vertical distance

The number of people to be served by a tapstand is determined by the nominal tap flow rate and the time period allotted to consume the daily amount of water, both parameters have been specified in the foregoing chapters. Based on these two factors a tapstand shall be able to serve the number of people as stated in Table 9:

Table 9: Population served per Tapstand

Population to be served (ultimate design population)	
maximum	120
minimum	90

Part of the population growth will be within the areas of supply of a tapstand, the other part of the additional population to be served will build new houses on new locations. Therefore it is impossible to avoid requests for additional taps even if the number of taps is based on the ultimate population to be served. It is therefore proposed to base the number of taps on the future population thereby accepting the need to construct 15 to 30 percent more taps at a later stage.
The number of tapstand within a supply system is decided by the Designer based on the above stated criteria and these criteria should be explained to the villagers. However tapstand locations must be chosen by the villagers themselves. To avoid the often observed heated discussions on tapstand location the following rule shall be applied:

- No tapstand shall be located within a house or court yard.
- If a certain tap location is liable to create internal friction within a community the designer must use his discretion to persuade the villagers to choose an alternative tap location.
- All the people served by the tap must have free access to the tap at any time.
- The often observed "abduction" of tapstands by certain social groups must be avoid by selecting a "neutral" tapstand locations.

4.3 System Appurtenances

4.3.1 Air Valves

Definition: In this context it may be considered that a high point is accentuated if it is situated 10 to 20 percent higher than the static pressure at preceding or succeeding low point.
Air release valves serve mainly three purposes namely:

- to release air from the pipeline during the filling process
- to release air from the pipeline during the normal operation of the water supply
- to prevent the development of vacuum in case a valve is closed upstream of the air valve

For pipe diameter up to 110 mm automatic "orifice" type air valves with a minimum orifice opening of diameter 2 mm will be suitable.
Air release valves are prone to malfunction and or water leakages. For this reason air release valves must be equipped with an isolating valve.
Air release valves connected to pipelines with high flow velocities are not very efficient since the draw off effect created by the air valve is too small.
Alternative air release should be considered like:

- manual operated air release valves
- connecting of tapstands on high points

However it should be born in mind that such alternative devices are not "automatically" releasing the air , but need to be operated manually on a routine basis.

4.3.2 Wash Out

Washout are used to clean the pipeline of sediments which have accumulated in due course. During filling of a pipe system, they may also act as an air release point.
When opening a washout valve a flow velocity must be created which is big enough to overcome the bond (shear stress) between pipe wall and sediment.
For this reason washout should have an minimum available head of 10 to 20 m without creating a vacuum at a high point For main pipe diameters up to 110 mm the washout shall have the same diameter as the main pipe.
It is important to place a washout at the first possible low point after an Intake.
In this context it may be considered that a low point is accentuate if the succeeding high point is situated on a 10 to 20 m higher level. Such points should be equipped with a washout

4.3.3 Sectional Valves

Sectional valves are installed for two different reasons which are as follows:

- for repair purpose; to enable the maintenance worker to shut off parts of a supply system
- for rationing of water; in case of a severe drought or other operational problems which necessitate water rationing

Sectional valves on long gravity pipelines should be located at a distance between (2) to 3 km. Care should be taken that the shut off part can be ventilated by means of air valves or tapstands to avoid the build up of vacuum.
Sectional valves on distribution pipes should be placed in such a way that they control 3 to 5 tapstands. Whenever possible sectional valves should be combined with an air release valve or washout but the sectional valve must be located upstream of these valves.
With the introduction of sectional valves in the distribution system tap flow control valves (stop cocks) shall no more be installed at the tapstands. If for whatever reason an individual branch line needs a sectional valve the valve shall be located away from the tapstand preferable at the branch-off location.

4.3.4 Valve Chamber

All system appurtenances shall be installed in a valve chamber with a minimum internal dimension of 75 x 75 cm. If more than one valve shall be installed in the same chamber the size shall be increased accordingly. The minimum dimension of the valve chamber is determined by the size of tools required to install or repair such a valve. Valve chambers for sectional valves or washouts shall have a chamber floor which permits condensed or leakage water to percolate into the ground. They do not necessarily require a drainage although such a drainage may prove to be of advantage when repairing a valve. Valve chambers for air valves require a

drainage with a free outlet. By all means it must be avoided that air valve chambers are filled with water since such water may enter the pipeline during times of low pressure.

5. Drawings

5.1 Lay Out

A general lay out plan of the planned water scheme which shows the supply system and other important landmarks shall be prepared. For smaller schemes supplying less than 2000 people (design population) a schematic sketch will be appropriate. For bigger schemes the layout plan of the pipelines must be based on survey data and shall be plotted in an appropriate scale. Other important features required for easy orientation may be sketched only.
A layout plan should contain besides the pipeline layout and landmarks, the following information:

- Intake Collection Chamber(s)
- Sedimentation Tank
- Storage Tank(s) and size(s)
- BPTs
- Distribution Chamber(s)
- Tapstand
- Pipe size, material and pressure class
- Air valves and Wash outs
- Sectional valves
- Name and safe yield of source(s)
- Locality names

For big schemes the drawing number of longitudinal profiles should be indicated on the respective locations. For small schemes the layout sketch and the pipeline flow diagram may be combined in one drawing.

5.2 Longitudinal Profile

The importance of the longitudinal profile is very often under estimated, and for these reason incomplete and untidy profiles are produced.
However a carefully surveyed pipe alignment and equally carefully plotted longitudinal profile will provide the designer with a lot of useful information. A longitudinal profile shall contain the following information inclusive the respective ground elevation.

- Sources Intake, Collection Chamber
- Sedimentation Tanks
- Storage Tanks
- BPT's
- Distribution Chambers
- Tapstands
- Wash outs
- Air valves
- Sectional valves
- Branch points
- River crossings
- Exceptional ground conditions like rocky or stony areas

- Each important location shall be indicated with a number or letter which should also be indicated on the lay out plan
- Hydraulic Grade line(s) and the horizon of the static level
- For each individual pipe line the following information shall be available on the drawing.
  a) horizontal distance
  b) tape length
  c) cumulative tape length, the chainage of the pipelines starts at the inlet and runs in the same direction as the flow
  d) ground elevation of the points whose distances are indicated above
  e) flow rates
  f) pipe diameter, material, length and pressure class
  g) residual heads on all the structures, appurtenances, high points and branch points

The scale to be used to plot the longitudinal ground profile shall be:

Horizontal / Vertical	
1: 2000 /	200
1: 5000./	500

Ground points shall be surveyed and plotted at least every 50 m or less if there are sudden changes in the terrain gradient like gullies, rice terraces etc.

6. Design Report

A comprehensive design report shall be prepared for each water supply project.
The following list of contents will generally be applicable for any size of project. However the contents of the report should always be modified to suit the requirements of the particular project.

1. Summary of approximately one page giving the most important information regarding the project
2. List of Contents
3. Introduction
3.1 Background
3.2 Previous Investigation and Reports
3.3 Scope of this Report
4. Summary of Design Data
4.1 Description of Area of Supply incl Location Map
4.2 Social Infrastructure
4.3 Existing Water Supply
4.4 Population Projections
4.5 Water Demand Projections
4.6 Hydrology
4.7 Selected Project Alternative
4.8 Studied but discarded Project Alternatives (incl unsuitable sources)
5. Intake
6. Treatment
7. Water Storage

8. Pipeline System
9. Cost Estimate
9.1 Construction cost
9.2 Operation and Maintenance cost
9.3 Revenue
10. Appendices
10.1 Survey Calculation
10.2 Hydraulic calculation
10.3 Structural calculation if any

7. Project File

For each water supply scheme 8 Project Files shall be prepared for the use and attention of the following Offices:

1) MHPP/DWSS Western Region RD's Office
2) MHPP/DWSS Western Region Store Section
3) CWSS/HELVETAS Programme Coordination
4) MHPP/DWSS DWSO District Engineer (Original File)
5) MHPP/DWSS DWSO Project Supervisor (Overseer)
6) MHPP/DWSS DWSO Construction i/c (WSST)
7) Water Supply Users Committee
8) District Treasury & Account Comptroller Office

The original file kept by the concerned District Engineer is the only file which contains all the data and information regarding a particular project. Thus it is of utmost importance to keep the original file on a safe place and to up date it on regular intervals.
The other files contain only the information required by the various Offices concerned to enable them to carry out their work or duties in connection with the implementation of the project. Annex V provides a list the of documents required for each file.

SUMMARY OF DESIGN CRITERIA and CONSIDERATION

Table 1: Annual Population Growth Rates (in percent)

Zone	Gandaki and Dhaulagiri [1]
Village area	2.3
Bazar area	3.0
School, Health post	3.2

Note: 1) excluding the Districts of Mustang and Manang

Table 2: Population Growth Factors

Zone:	Gandaki and Dhaulagiri	
Design period (years):	12	22
Village area	1.31	1.65
Bazar area	1.43	1.92
School, Health post	1.46	2.00

Table 3: Domestic Water Consumption Figures

Type of Supply Area	Daily Water Consumption
Rural Village	45 lcd
Bazar Area	60 lcd

Table 4: Institutional Water Consumption Figures

- School	6 ld	per student
- Health post	1000 ld	for out patient only and without improved sanitary condition e.g.WC
	3000 ld	for out patient only with improved sanitary condition

Water Demand Projections:

- Present Water Demand — at the time of designing the scheme[1]
- Future Water Demand — after 10 years service life (needed only if phased implementation is considered)
- Ultimate Water Demand — after 20 years service life

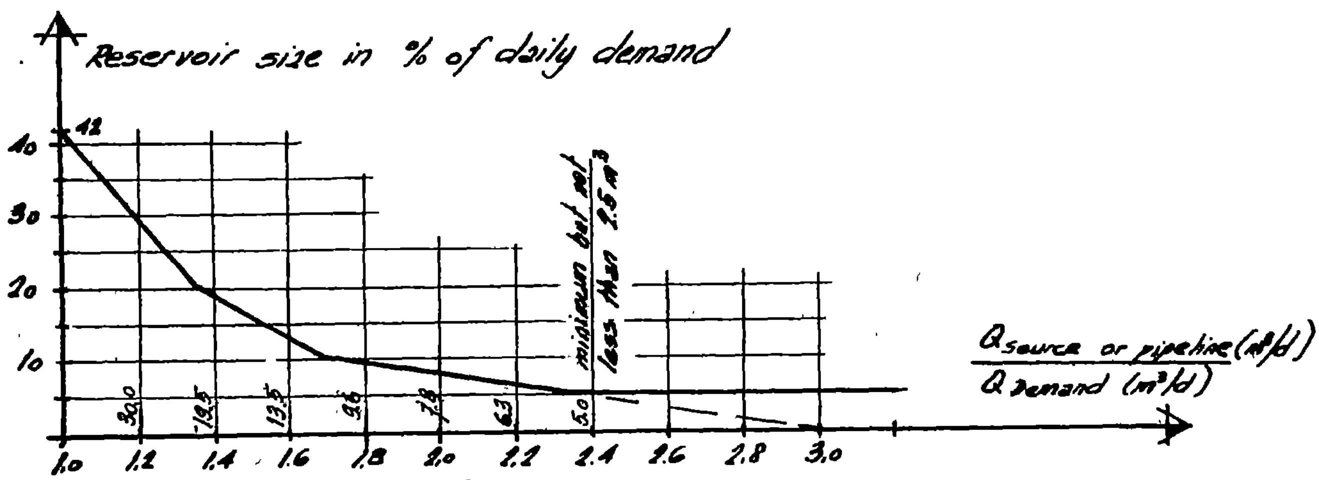
1) This is a simplified assumption! Normally the present water demand is computed for the time of commissioning of the scheme.

Safe source yield = 0.9 x measured source yield at the peak of the dry season

Table 6: Peak Flow Factors

	closed system	open system
Village (tapstand)	3.75	2.50
Bazar (tapstand)	3,00	2.50
School (usually the same as village or bazar tapstand, but in case of exceptional long individual pipeline	6.00	4.00

Figure 2: Storage Tank Capacity



Minimum Flow Velocity

On river intakes, if no sedimentation is provided the minimum flow velocity shall be:
- in down hill stretches 0.8 m/s
- in up - hill stretches 1.0 m/s

If a sedimentation is provided the minimum flow velocity can be reduced to:
- in down hill stretches 0.4 m/s
- in up - hill stretches 0.5 m/s

Maximum Flow Velocity
- desirable less than 2.5 m/s
- maximum 3.0 m/s.

Table 7: Tap Flow Rates

Ultimate Water Demand per Tap (l/d)	Peak Flow Rate (l/s)	Remarks
3400 - 4600	0,15	Small cluster of houses
4600 - 5800	0,20	Village
5800 - 7000 (7600)	0,25	Bazar or Village with School / Health post.

(...)=maximum

Static Pressure

Gravity Pipelines
- for PE pipes pressure class 10 kg/cm² not more than 100 m
- for GI pipes pressure class conforming to BS 1387 medium grade not more than 160 m

Distribution Pipelines
- Maximum static pressure 60m
- Exceptional cases 80m
- with self closing taps 20m (e.g. Jayson Taps)

Table 7: Residual Pressure

Structure	Residual pressure
- Tapstands ideal	5 to 10 m
acceptable	10 to 15 m
- BPTs and Storage Tanks	10 to 20 m

Length of Water Supply System

- source yield:	permissible pipe length
< than 1 l/s	750 m per 0.1 l/s yield
> than 1 l/s	1000 m per 0.1 l/s yield

Table 8: Tapstand Spacing

Tapstand spacing	desirable	low density area	
horizontal	150 m	250 m	350 m
vertical	50 m	42 m	35 m

Table 9: Population served per Tapstand

Population to be served (ultimate design population)	
maximum	120
minimum	90

Project\CWGL90_C

Implementation Flow - Chart for Water Supply Schemes Constructed with Community Participation

Stage	Activities by the committee:	Support Programmes by:
PROJECT IDEA		
District Dev. Board (if not considered → may be considered later; if considered ↓)	Formation of users committee	CU: Educational input regarding the rights and duties of the committee WIP: Makes sure women are put into the committee. Explanation of the various technical aspects of a water scheme.
FEASIBILITY STUDY (if not feasible → project is dropped; if feasible ↓)	Source selection, tap locations are decided at this stage	
SURVEY DESIGN ESTIMATE	Users committee shall approve design+estimate. It is important that at this stage a village assembly is conveyed	CU: Educational campaigns on health, nutrition and sanitation. Explaining to the population the components of a w/supply, the cost involved and the expected physical input by the village. WIP: Build up phase
District Dev. Board Construction Progr. (if not included → may be considered later; if included ↓)		
Procurement of Construction Material		
CONSTRUCTION OF WATER SUPPLY	Organization of community work, nomination of Village Maintenance Worker (VMW)	CU: Educational campaigns continuation. Special programmes concerning Operation+Maintenance WIP: Consolidating phase
HANDING-OVER OF COMPLETED SCHEME	Formation of O & M committee	WIP: Promotes the election of women into the committee. Follow-up phase Support of O & M committee
OPERATION AND MAINTENANCE OF THE SCHEME	Employment of VMW Organization of community contribution in kind and/or cash to pay the VMW and to keep the scheme in operational condition	

Note: CU: Communication Unit WIP. Women Involvement Programme

F E A S I B I L I T Y S T U D Y

Preliminary works:
1) Establishing of base population within proposed area of supply
2) Water demand projections

↓

3) Hydrogeological assessment of potential sources: ground-, surface-, or rain water within the area of supply

↓

4) Source selection

↓

5) Source yield and water quality assessment of the proposed sources

↓

6) Defining the type of service to be provided based on the water availability of the various potential sources

↓

7) Discussion with the population concerned regarding the type of service proposed and their expectations. Explaining to the villagers the pro and contra of the various type of services and their social/economical and technical implication.

↓

8) If required re-defining area of supply depending on the available source(s) and their possible area of supply

↓

9) Survey of each individual area of supply according to the standard format

↓

10) Rough ground survey with altimeter or abney level

↓

11) Lay-out sketch indicating:
- number and kind of sources to be developed
- location of structures like intake, collection chambers, storage tanks, sed. tanks, wells, rainwater catchment, water points etc.
- number of tapstands only locations to be decided by the population concerned

↓

12) Bill of Quantity

↓

13) Cost estimate and per-capita construction costs.
It is important to prepare for each independent sub-system the estimate as well as to carry out the health and social survey, since it is possible that the whole system is found not feasible but sub-system hereof may be feasible.

↓

14) 1st Assessment whether the proposed project is feasible or not

↓

If feasible continue with Detail Design

↓

D E T A I L D E S I G N

1) Hydrogeology survey if necessary

2) Final decision regarding tapstand, well, water point locations. But even storage tank or intakes locations should be discussed with the villagers

3) Final yield assessment of the proposed sources

4) Final adjustment of area of supply based on information regarding source potential and the perception of the future beneficiaries of the system concerned

5) Establish actual number of people leaving with in the area of supply
   - establish for each tapstand number of houses served
   - number of people per household = 5.5
   - Compute present (or base) population for each tapstand - — ref. 2.1.1

6) Continuous source yield monitoring

7) Population projections present, future, and ultimate population served by each tapstand — ref. 2.1.4, 2.1.5

8) Water demand projections based on population forecast for each individual tapstand — ref. 2.4, 2.5

9) Assessment of safe source yield: measured yield x 0.9 = safe yield — ref. 2.6.1

10) 2nd assessment whether the proposed project is feasible or not by comparing water demand versus available source yield — ref. 2.6.2

11) Decision regarding type of system: (open or closed) — ref. 2.6.3

12) Decision regarding storage tank capacity and whether one big, or several small tanks — ref. 2.7.3

13) Detail ground survey with Abney level or Theodolite (altimeter survey not acceptable). With terrain points at last every 50 m or less if there are important features like rivers, rice terraces, sudden change of ground inclination. For big and complicated schemes a proper lay-out survey must be done. For the intake site a proper ground plan must be surveyed to enable the designer to prepare a detail plan for the intake structure.

14) Survey calculations

15) Plotting of longitudinal profile and lay-out plan (draft) with all the necessary information required to design a water scheme — ref. 5.

↓

16) Deciding on important location of structures
like: - storage tank(s)
- break pressure tank(s)
- distribution chamber(s)
those locations are determined by the maximum permissible static pressure within a supply system — ref. 3.6

↓

17) Computing of: - tap flow rates
- pipe flow rates — ref. 3.3

↓

18) Deciding on gravity pipeline capacities versus storage capacity requirements (cost analysis) — ref. 4.2.3

↓

19) Preparation of schematic lay-out with pipe flow rates indicated — ref. 3.4

↓

20) Hydraulic calculation based on the calculated flow rates taking into consideration:
- residual pressure requirements
- maximum/minimum flow velocities — ref. 3.2, 3.6, 3.7, 3.7

↓

21) Determining the number and location of:
- air valve (A/V)
- washouts or scour valves (W/A)
- pipe bridges (suspended)
- river crossings
- any other structure deemed to be necessary — ref. 4.

↓

22) Designing not standardized structures like:
- river intakes
- spring catchment
- river crossings etc.

↓

23) Review of the design by DE and CWSS PCM
if necessary requests for additional information
e.g on the chosen alternative; source selection

↓

24) Necessary amendments on:
- population and water demand forecast
- supply system and structures

↓

25) Final drawings
- longitudinal profile
- lay-out etc.
- not standardized structures — ref. 5.

↓

26) Bill of Quantities
- for standardized structures
- for not standardized structures
the quantities have to be estimated — ref. Standardization

↓

27) Cost estimate inclusive rate analysis

↓

28) Review of Bill of Quantities and cost estimate by DE and CWSS PCM

↓

29) Preparation of Design Report

↓

30) Preparation of original project files for approval procedures — ref. 7.

↓

31) Technical approval for donor

↓

32) Approval of project by District Engineer and or Regional Director

↓

33) Approval for the Beneficiaries

↓

34) Preparation of the various project files — ref. 7.

W A T E R D E M A N D P R O J E C T I O N

Name of Project: ....

Name of Community: .... Ward No.: ....

District: ....

Number of people per household 5.5
Present year: 1990
Future year: 2002 (present year +12)
Ultimate year: 2024 (present year +22)

Tap No.	Locality	No.of Houses	Population Present	Population Future	Population Ultimate	Schools and Institutions Present	Schools and Institutions Future	Schools and Institutions Ultimate	Consumption Figures (l/d) Popul.	Consumption Figures (l/d) School	WATER DEMAND Future l/d	WATER DEMAND Ultimate l/d	Tap flow nominal l/s	Remarks
1		13	72	94	118				45		4,215	5,309	0.20	
2		9	50	65	82				45		2,918	3,675	0.15	
3		12	66	94	127				60		5,663	7,603	0.25	Bazar
4		5	28	36	45	100	146.0	200.0	45	6	2,497	3,242	0.15	
5		11	61	87	116				60		5,191	6,970	0.25	Bazar
6		7	39	50	64	1	1.5	2.0	45	1000	3,730	4,859	0.20	
7														
8														
9														
10							Peak flow: 31658/86400*3.75 = 1.37							
11														
Total for control purpose		57	314	426	551	100	146.0	200.0			24,213	31,658	1.20	
						School only								

PROJECT\CWGL90_Z

ANNEX 3a

MHPP/DWSS COMMUNITY WATER SUPPLY AND SANITATION PROGRAMME WESTERN REGION Page: ......

W A T E R D E M A N D P R O J E C T I O N

Name of Project: ....

Name of Community: ....: Ward No.. ....

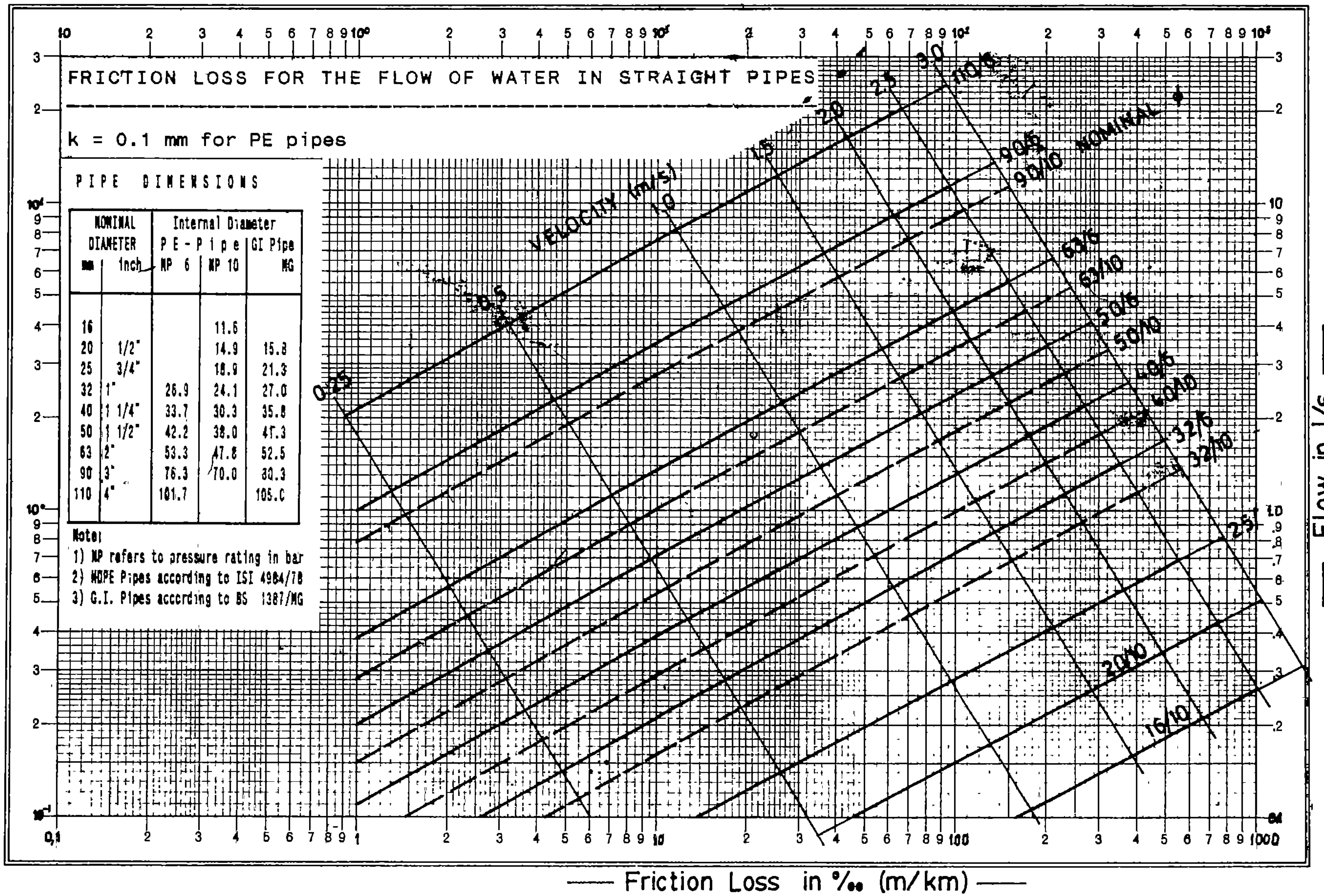
District: ....

Number of people per household: .....
Present year: ......
Future year: ...... (present year +12)
Ultimate year:...... (present year +22)

Tap No.	Locality	No. of Houses	Population Present	Population Future	Population Ultimate	Schools and Institutions Present	Schools and Institutions Future	Schools and Institutions Ultimate	Consumption Figures (l/d) Popul.	Consumption Figures (l/d) School	WATER DEMAND Future l/d	WATER DEMAND Ultimate l/d	Tap flow nominal l/s	Remarks
Total for control purpose														

PROJECT\CWGL90_Z

ANNEX 3b



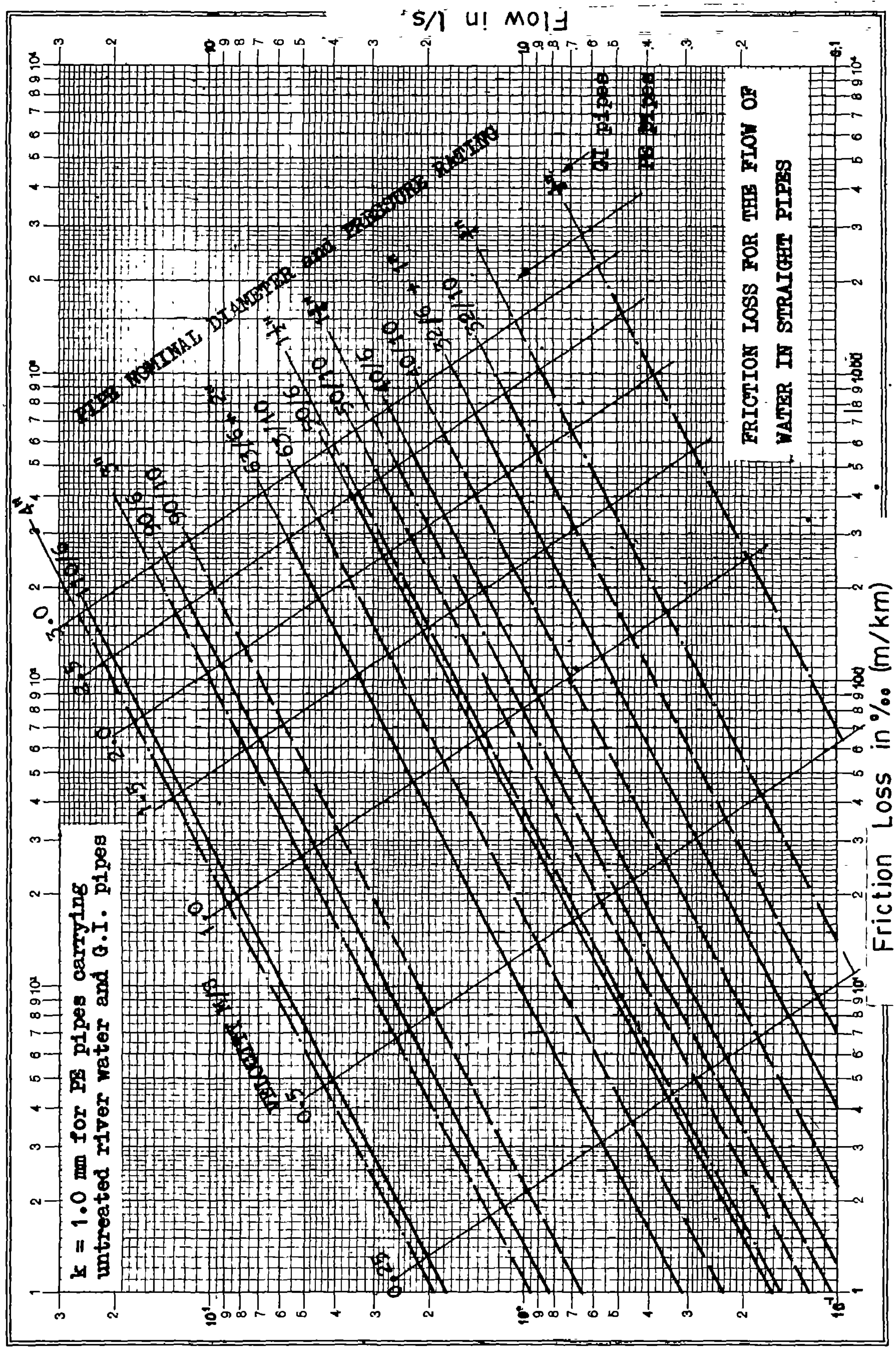
PIPE DIMENSIONS

NOMINAL DIAMETER mm	NOMINAL DIAMETER inch	Internal Diameter PE-Pipe NP 6	Internal Diameter PE-Pipe NP 10	Internal Diameter GI Pipe MG
16			11.6	
20	1/2"		14.9	15.8
25	3/4"		18.9	21.3
32	1"	26.9	24.1	27.0
40	1 1/4"	33.7	30.3	35.8
50	1 1/2"	42.2	38.0	41.3
63	2"	53.3	47.8	52.5
90	3"	76.3	70.0	80.3
110	4"	101.7		105.0

Note:
1) NP refers to pressure rating in bar
2) HDPE Pipes according to ISI 4984/78
3) G.I. Pipes according to BS 1387/MG

Annex IV a

Annex IV_b


FRICTION LOSS FOR THE FLOW OF
WATER IN STRAIGHT PIPES
k = 1.0 mm for PE pipes carrying
untreated river water and G.I. pipes
PIPE NOMINAL DIAMETER and PRESSURE RATING
VELOCITY M/S
GI pipes
PE pipes
Flow in l/s
Friction Loss in ‰ (m/km)

ANNEX V: DOCUMENT CONTENTS OF THE VARIOUS PROJECT FILES

RECEIVER:	RD-Office		CWSSP	D W S O F F I C E			Water	Audit	
	RD	Store	HELV.	DE	Super visor	Constr i/c	Users Comm.	Office	Total
TYPE OF DOCUMENT									
a) Prel. Survey Report			x	x					2
b) Design Report	x		x	x	x	x	x		6
c) Design Report Appendicis									
1) Field Survey Data and Survey Calculation				x					1
2) Hydraulic Calculation				x					1
3) Structural Analysis				x					1
d) Drawings complete set	x		x	x	x	x	x		6
e) Summary of Design+Estimate	x	x	x	x	x	x	x	x	8
f) Recapitulation of									
1) Material and Labour			x	x	x	x	x		5
2) Bill of Quantity			x	x	x	x			4
3) Pipes and Fittings			x	x	x	x			4
g) Weightlist				x					1
h) Material Transport calc.				x			x		2
i) Rate Analysis				x				x	2
j) District Rates				x				x	2
k) UNICEF Rates				x				x	2
l) Material Issue Record		x	x	x	x				4
m) Correspondence	x		x	x					3
Total number of document/file	4	2	9	17	7	6	5	4	54

R I V E R I N T A K E

River intake structures have to conform or to take care of various requirements like:
- geological condition
- continuous water abstraction irrespective of river water level
- safely discharging of flood flows without endangering the structure

General Notes:

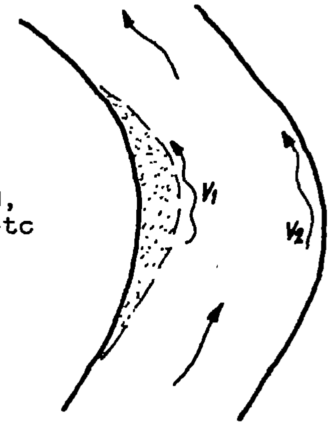
A) River intakes shall be located within stable reach of a river. Neither shall the river be in silting nor in a erosive state at the intake site.
If a river intake shall be placed within an unstable river stretch only temporary intake structures shall be considered.

B) Rivers with great slope gradients combined with high run off figures are unsuitable for any kind of intake structures (flash floods with a lot of debris)

C) The dam should be perpendicular to the river axis.

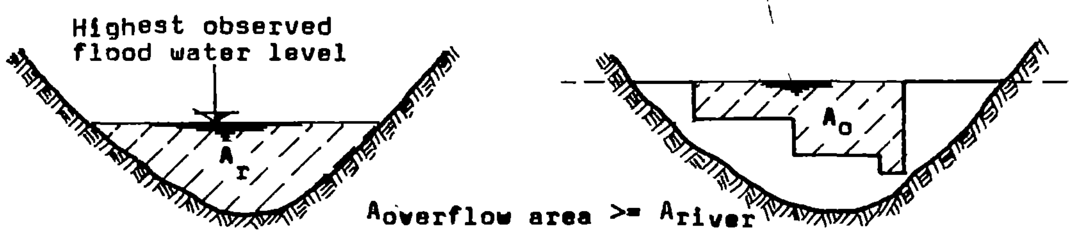
D) The intake chamber should always be at the outside of a river bend. As illustrated below, on the outer edge of a river bend the water is flowing with the greatest velocity which reduces the risk of sediment deposits within the intake area.

v_1 = low velocity deposits of sand, debris, leaves etc



v_2 = high velocity no or little sediment deposits

E) On river intakes to be built in unconfined river beds the overflow area of the dam (A_o) must be large enough to safely discharge flood water levels without being overtopped. The overflow area can be computed if sufficient and suitable river measurements are available. The required overflow area will be derived from a projected or measured 100 years flood (return period). In absence of any reliable flow measurements the overflow area shall be at least equal to the river cross section during an observed flood (villagers will be able to give such water level indications if no debris deposit on the river banks indicate such a flood water level).



F) The intake dam shall not be dimensioned to retain water or for sedimentation purpose (the storage volume will be silted up in short time and the sedimentation will not be efficient due to turbulence just in times when most need). Therefore the dam height which depends on the amount of water to be abstracted, the minimum as well as maximum river flow, shall be kept as little as possible

G) Any intake structure must be analysed for its safety regarding:
- over turning
- sliding
- uplift
- foundation pressure etc.

H) Water leakages under and around the dam shall be minimized with appropriate measures like:
1) Leakage under the dam: If the dam cannot be founded on an impermeable strata it is necessary to construct a cut-off trench. The cut-off trench shall be:
   - either bounded into an impermeable layer or
   - if it is not possible to reach such a layer the trench shall be of appropriate depth which will reduce the seepage to an acceptable level.
2) Leakage around the dam shall be prevented with wing-walls which are projecting into the surrounding terrain. Special attention should be paid to dams which are ending against boulders, since often the water finds a way around such boulders.

Note: Water leakages under and around the dam may endanger the stability of the whole structure !

I) A collection chamber shall be built for all intake structures without an intake and/or valve chamber. The distance between the collection chamber and the river intake shall be as short as possible e.g. not exceeding 50 m. If a sedimentation tank shall be built no collection chamber is required.

K) A floating intake may be a viable alternative in relatively large rivers with variable water levels.

The following river intake sketches are classified according to the above mentioned requirements. However it is important to realize that each river is unique and for this reason no universally applicable design rules are available.

Type of river bed	size of river	Type of Dam	Ref. to RIVER INTAKE Type Drg: Min. river overflow rate (1/s) Q_o <= (5-10)	Q_o >= (5-10)
Confined	small/medium	complete	A_1	B_1
Confined	medium/large	partial	A_2	B_2 Note 1)
Unconfined	small/medium	complete	A_3	B_3
Unconfined	medium/large	infiltration	C	
Unconfined	medium/large	partial	Note 2)	

Note: 1) This case has not been illustrated but in principal it is a combination of river intakes type A_1 and B_2.
2) Partial dam in unconfined river bed is in most cases not possible, since the water course may change due to the flow obstruction caused by the partial dam.
3) Although the river intake sketches have been drawn to scale only few measurements are indicated since the drawings should be used for illustration purpose only.

Definition:

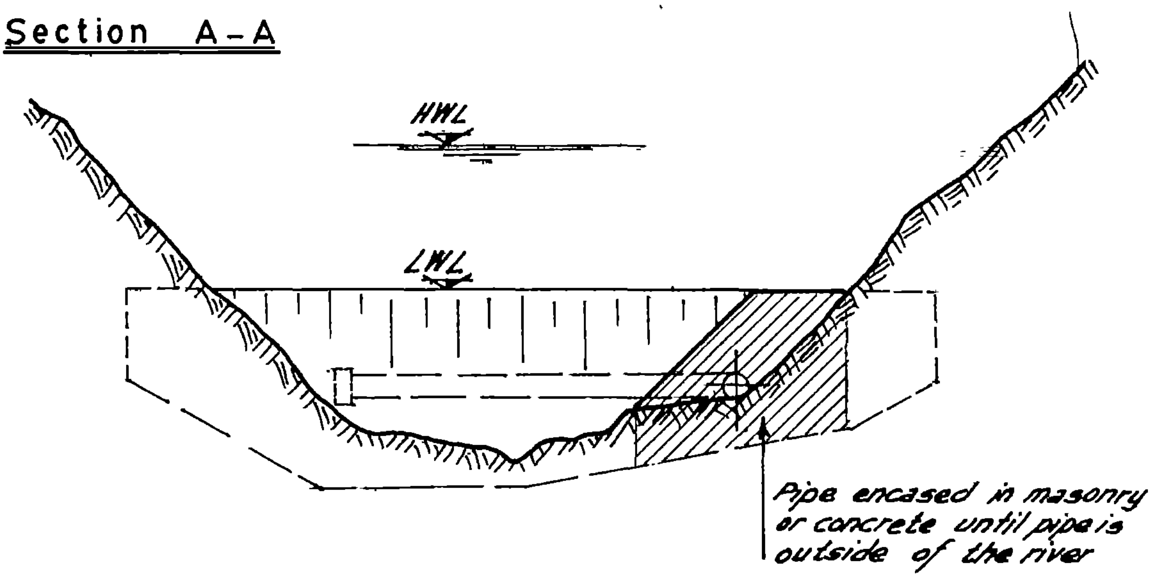
A) Confined river beds are formed with either:
- huge boulders
- rock outcrops
- complete flow channel in rock

B) Unconfined river beds are formed in comparable easy erodible material.

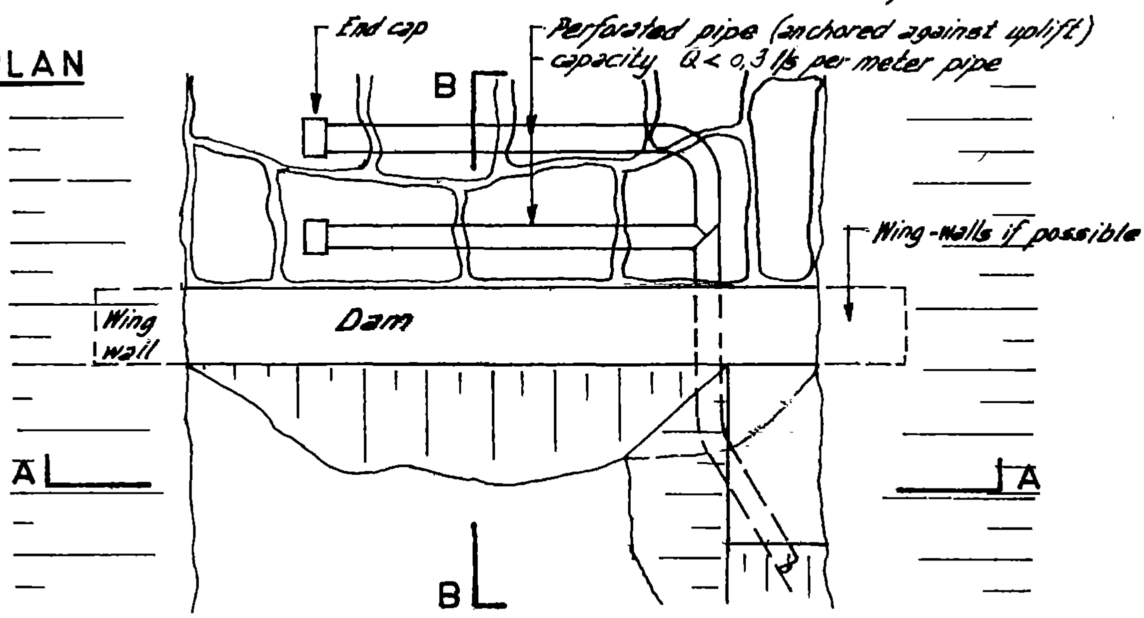
C) Size of river:
- small <5 m
- medium 5 to 10 to 20 m
- large >20 m

RIVER INTAKE Type A_1

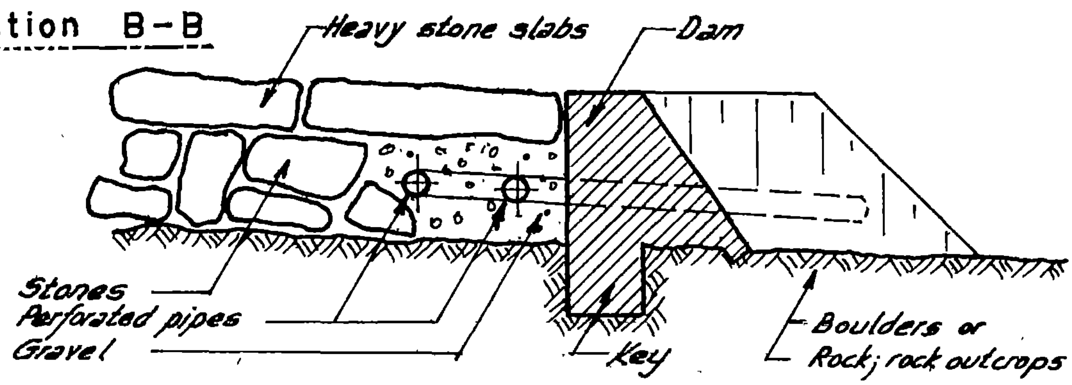
Section A – A



PLAN

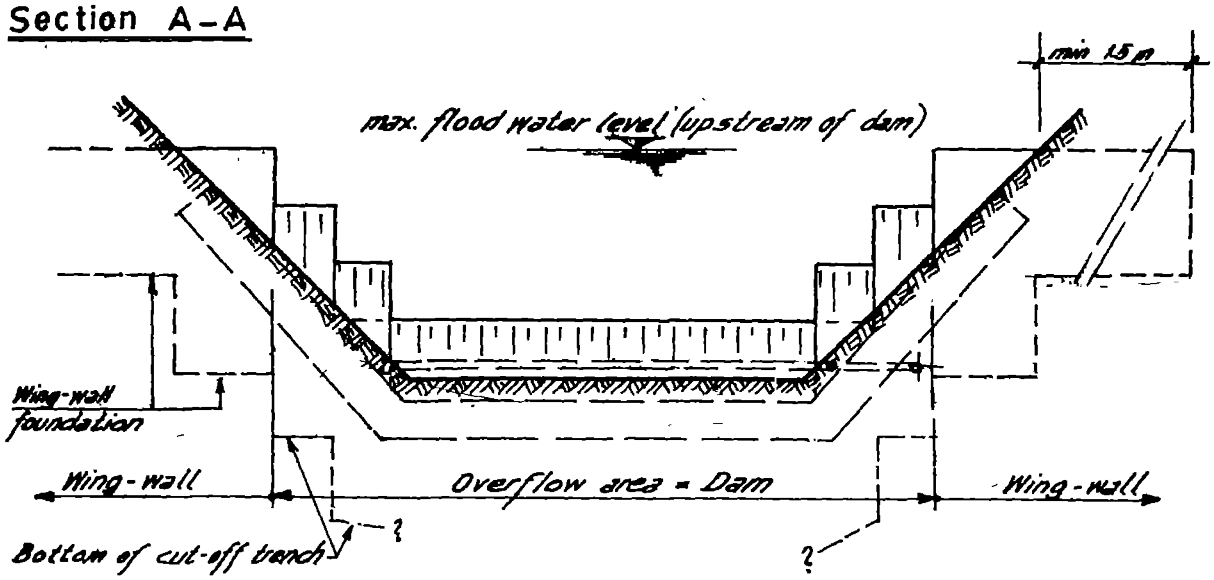


Section B – B

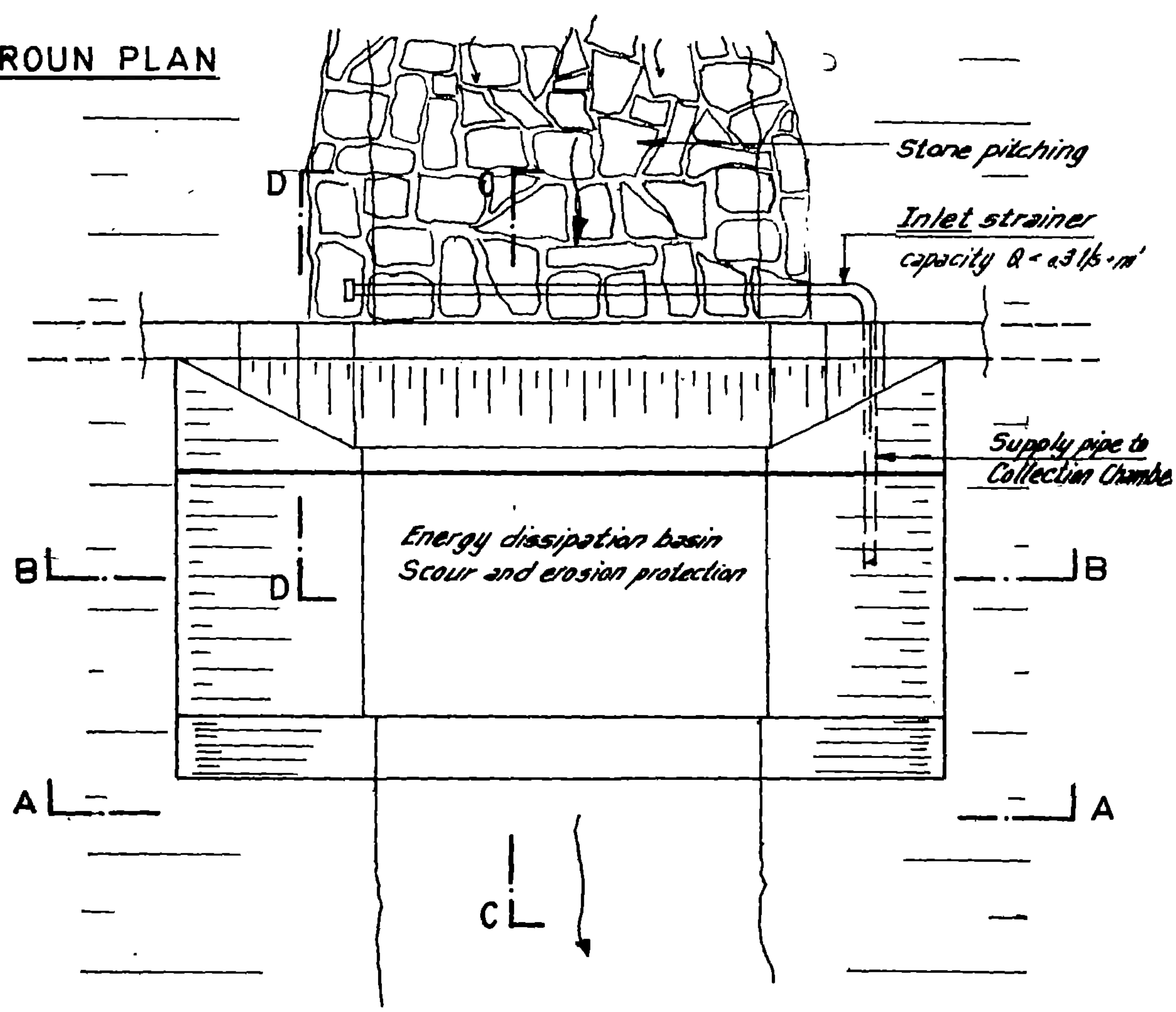


RIVER INTAKE Type A 3a)

Section A-A

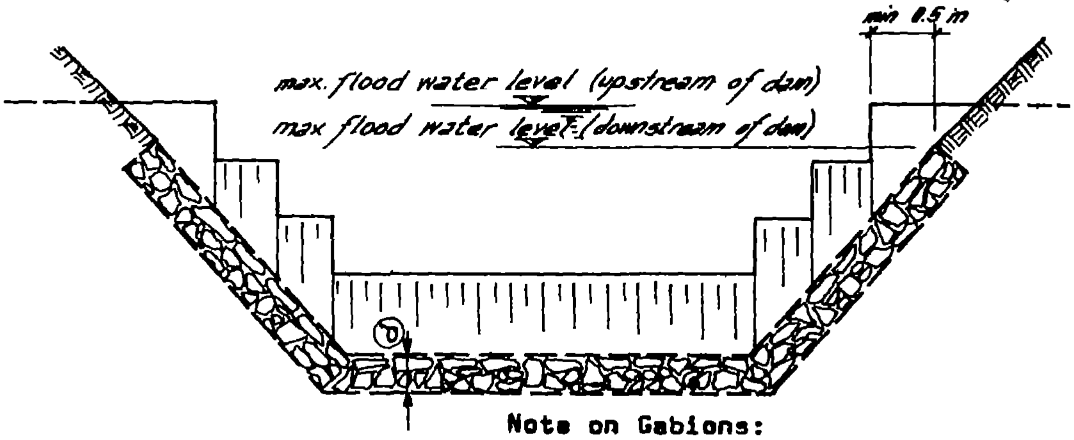


GROUN PLAN

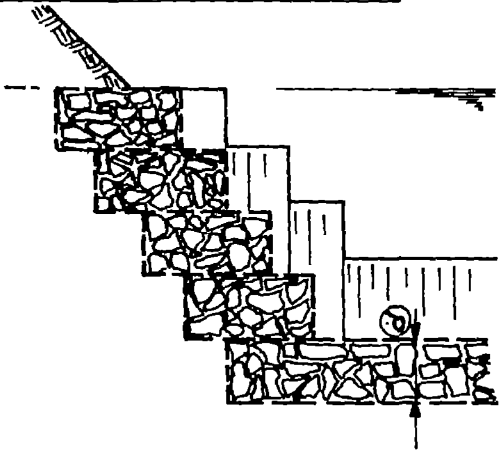


RIVER INTAKE Type A 3b)

Section B-B



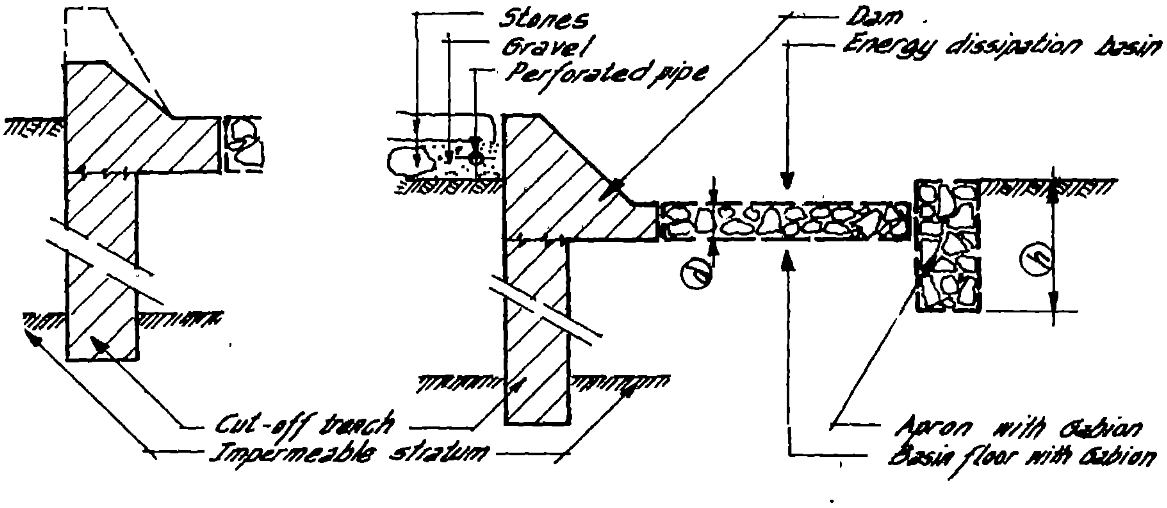
Alternative Section B-B



Note on Gabions:

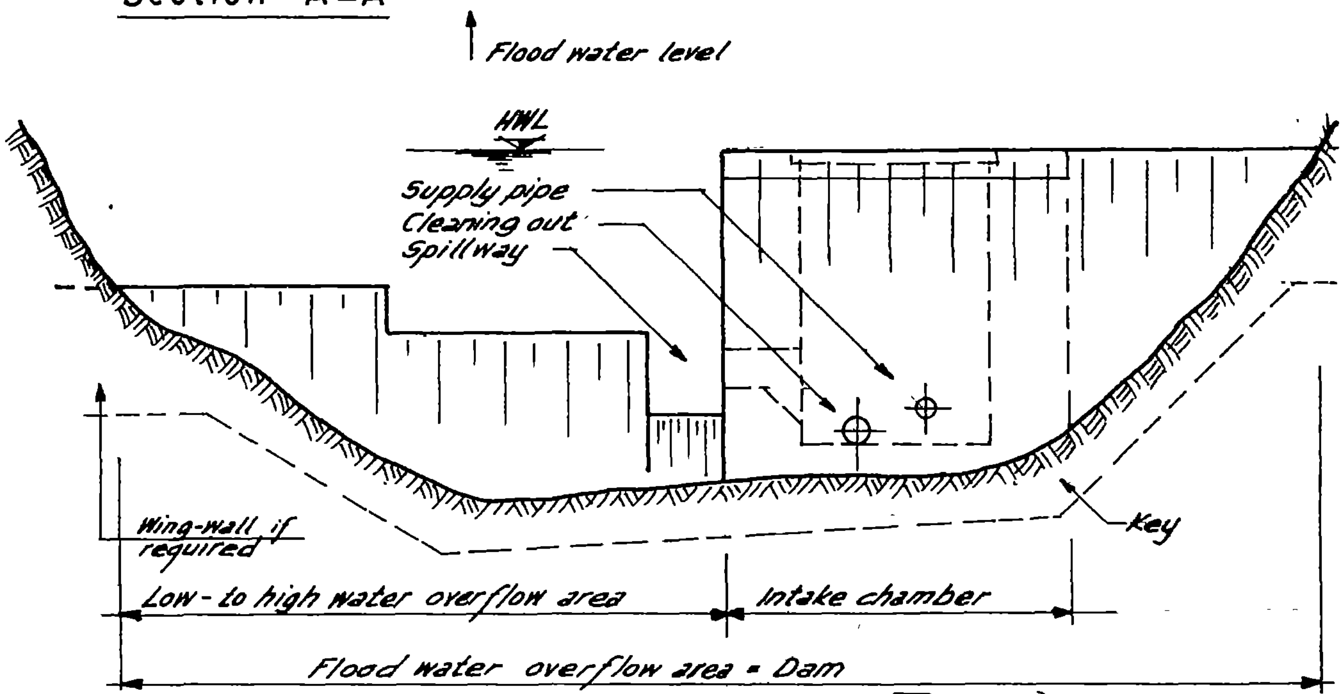
1) Gabion mattresses for basin floor construction. Thickness ⓓ = min. 30 cm for low velocity rivers (v <3 m/s). Thickness ⓓ may increase up to 1oo cm for high velocity rivers !
2) Apron height ⓗ shall be at least 2 x d but not less than 1oo cm
3) Round edged stones shall not be used for gabion filling. If no other stones are available than the stones must be broken to have at least one sharp edged face.
4) Minimum size of stones to be used shall be at least twice the biggest gabion mesh dimension.
5) Gabion mattrasses shall be anchor to the dam structure with steel bars or steel cables.
6) Chicken wire is unsuitable for gabion baskets.

Section D-D Section C-C

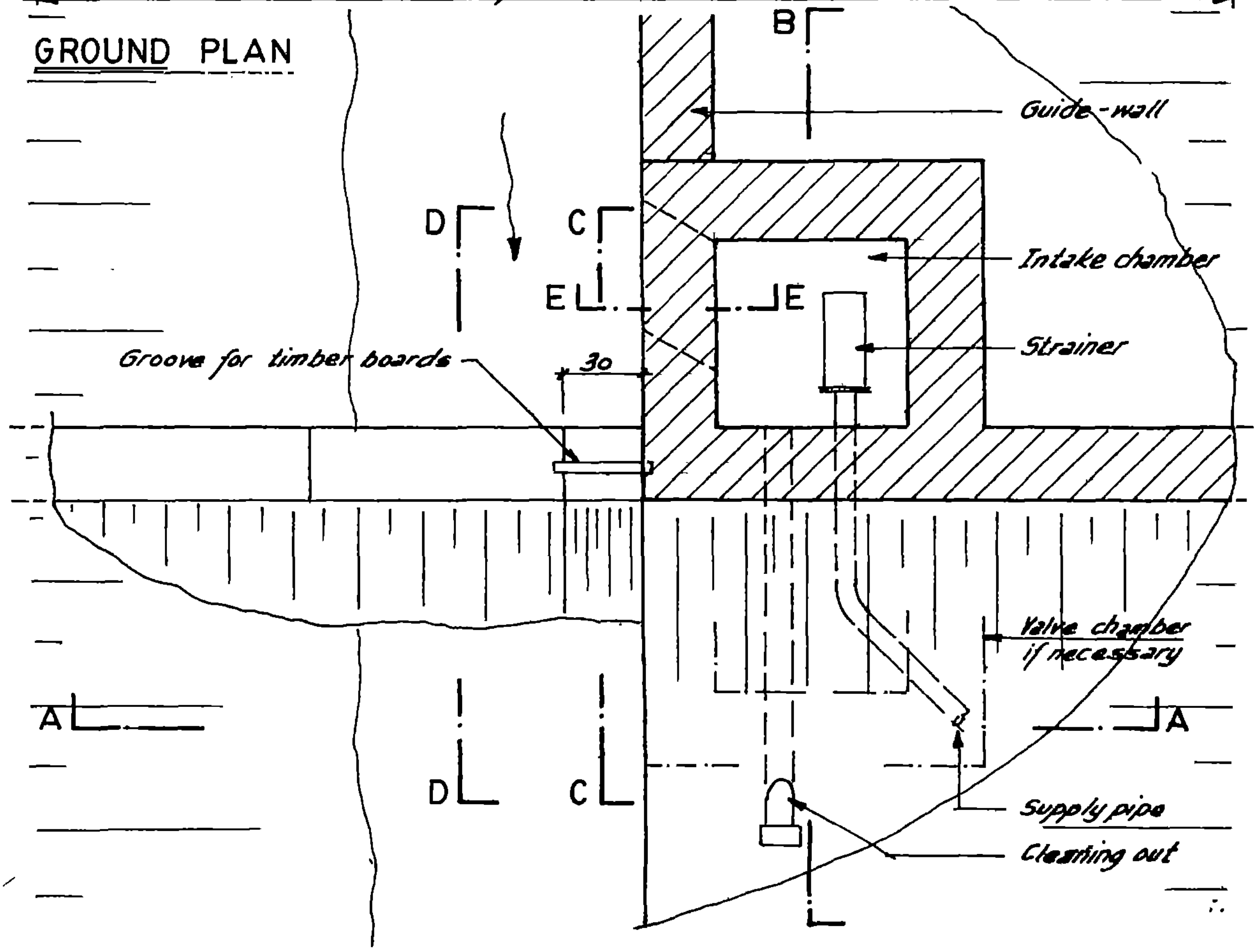


RIVER INTAKE Type B1a)

Section A–A

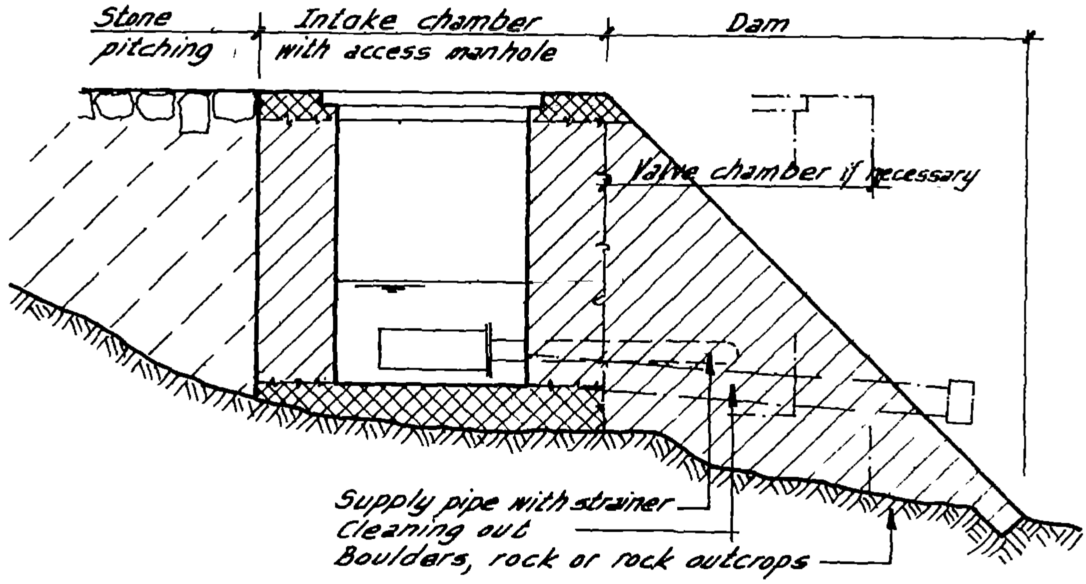


GROUND PLAN



RIVER INTAKE Type $B_{1b)}$

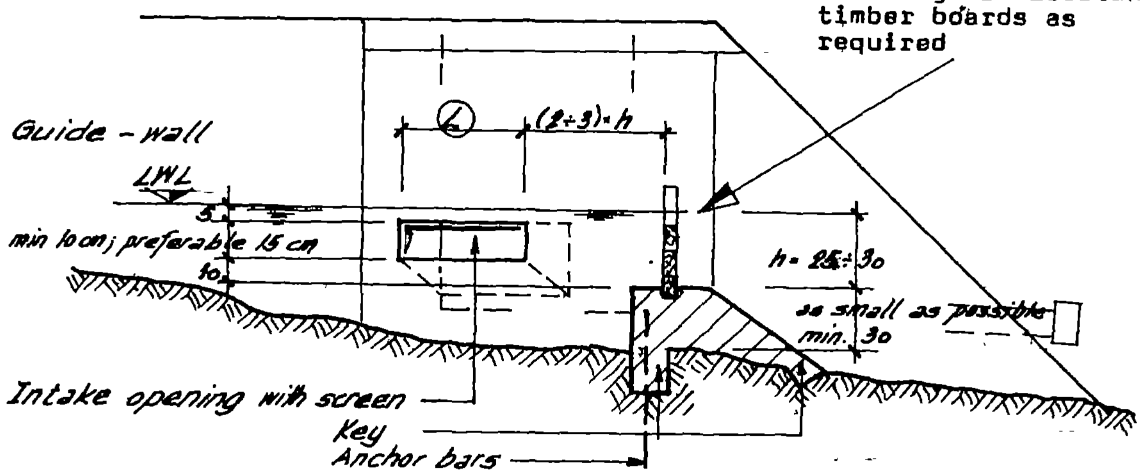
Section B-B



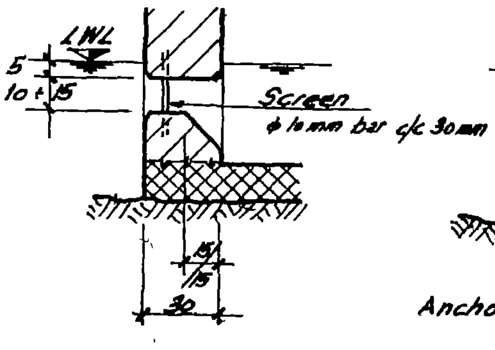
Section C-C

1) Measurement Ⓛ
Minimum 60 cm or more to keep the intake velocity below 0.1 m/s

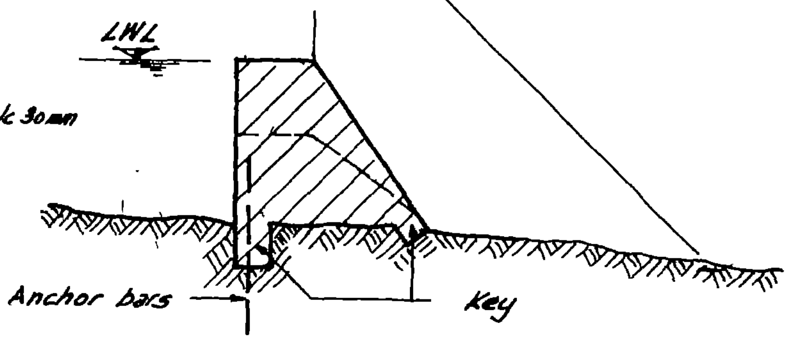
2) The low water level (LWL) is maintained by removing or inserting timber boards as required



Section E-E

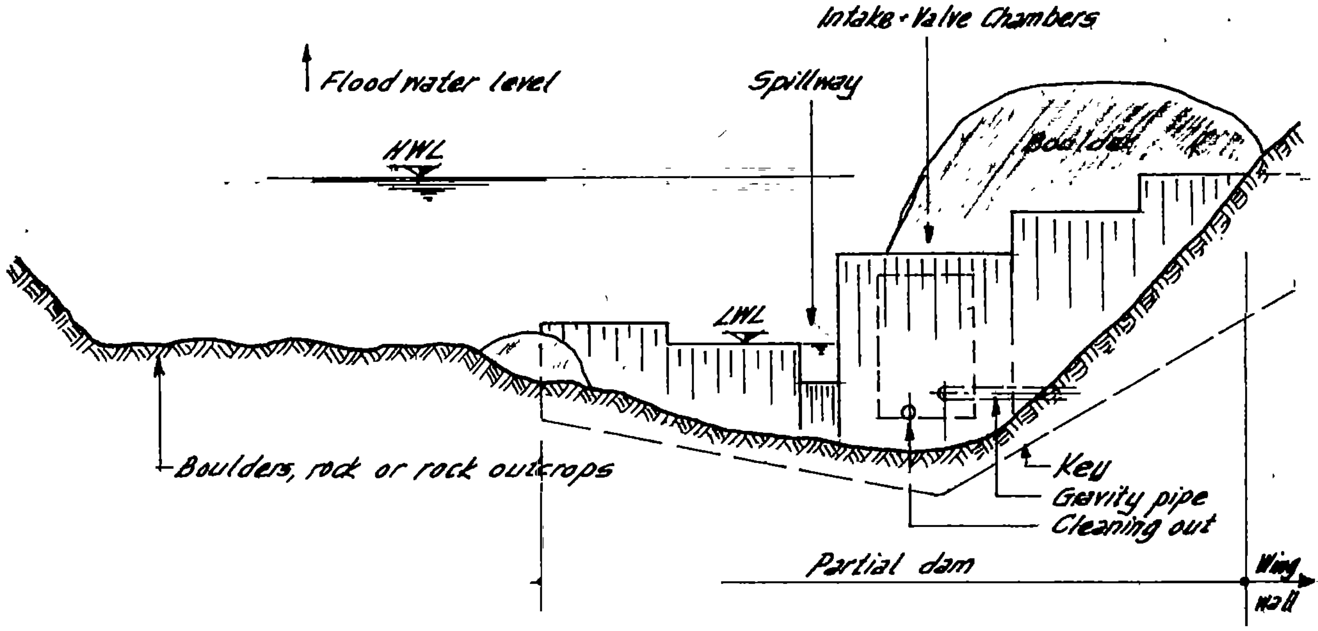


Section D-D



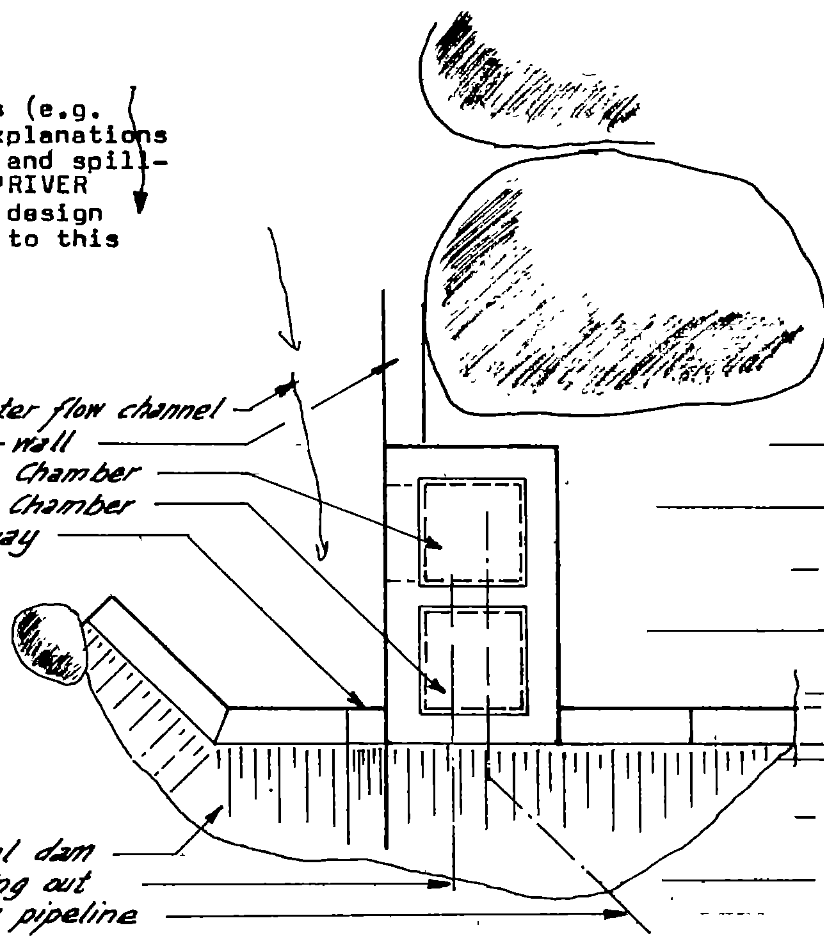
RIVER INTAKE Type B_2

Sectional Elevation

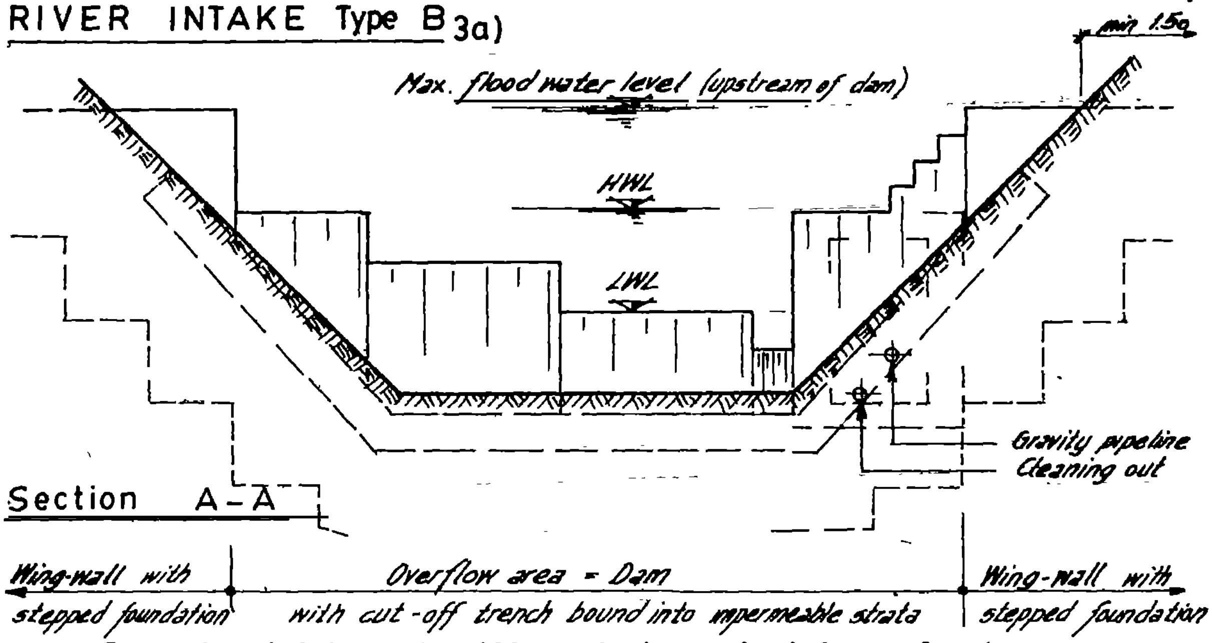


PLAN VIEW

Note: For more details (e.g. cross sections) and explanations (e.g. on water intake and spillway design) refer to "RIVER INTAKE Type B_1" whose design principles apply also to this river intake



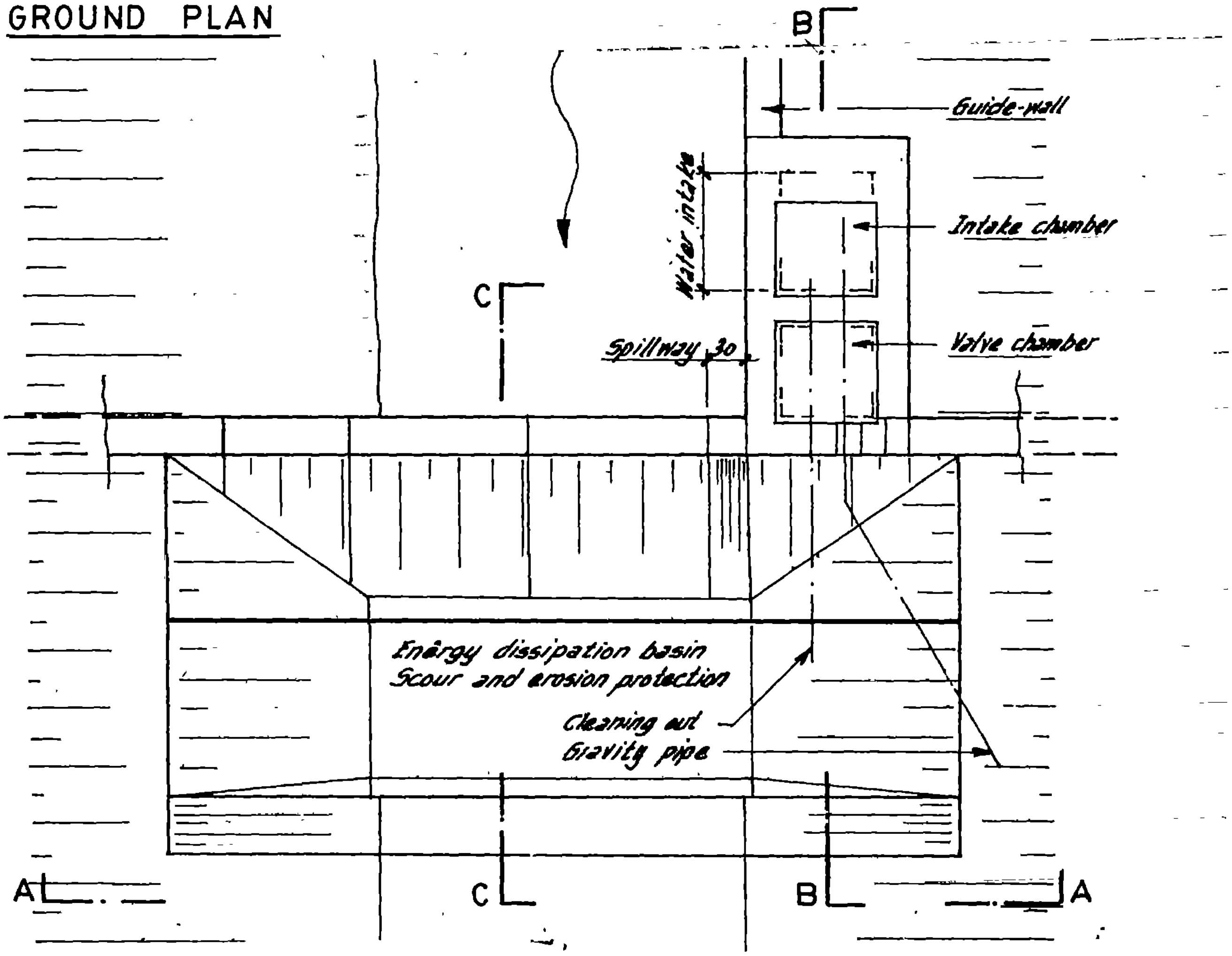
RIVER INTAKE Type B 3a)



Section A-A

Note: For water intake and spillway design principles refer to "RIVER INTAKE Type B_1"

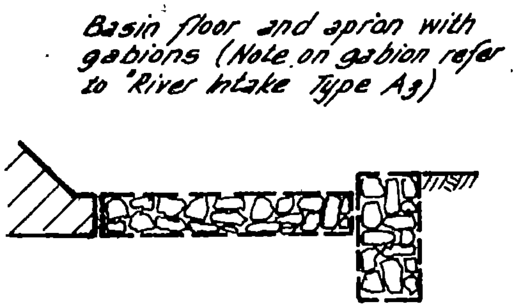
GROUND PLAN

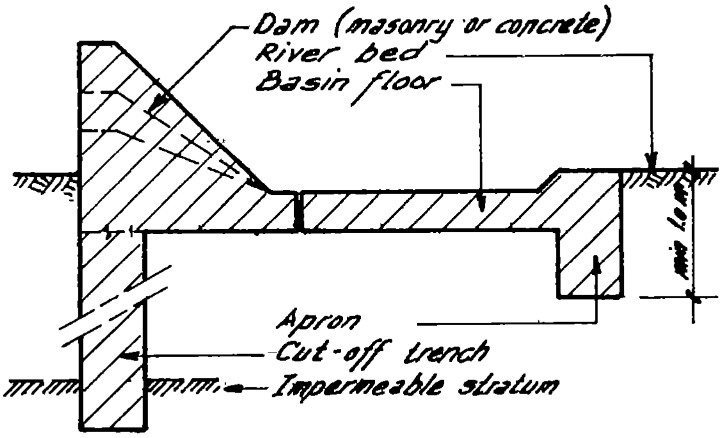


RIVER INTAKE Type $B_{3b)}$

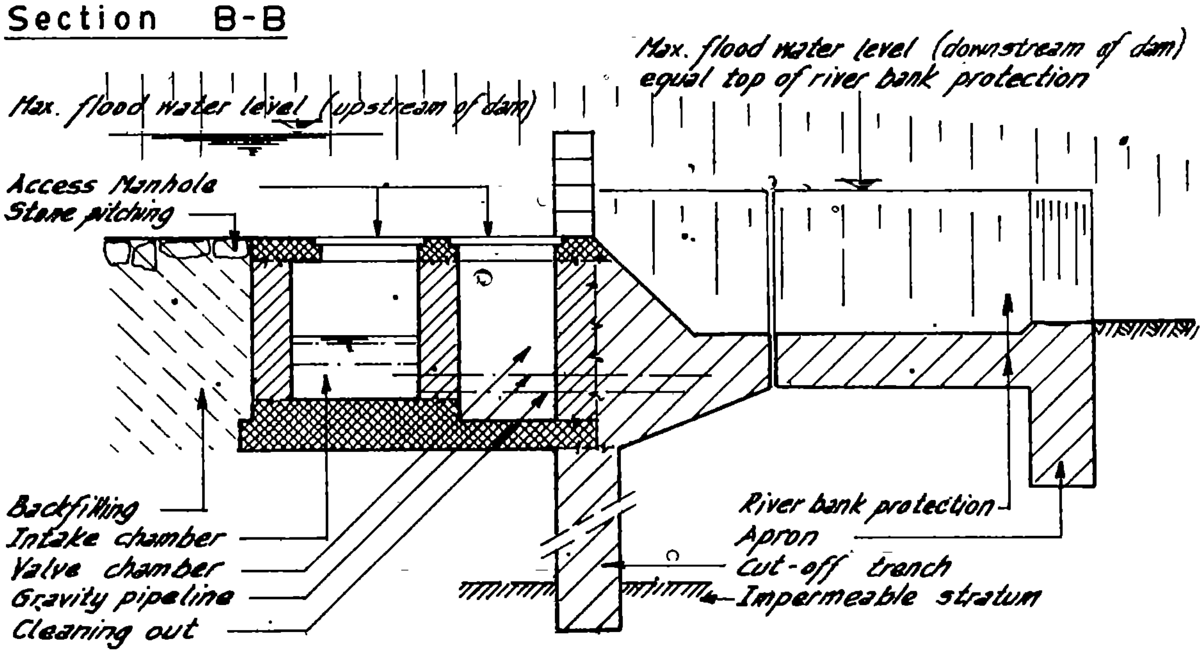
Section C-C

Alternative Section C-C





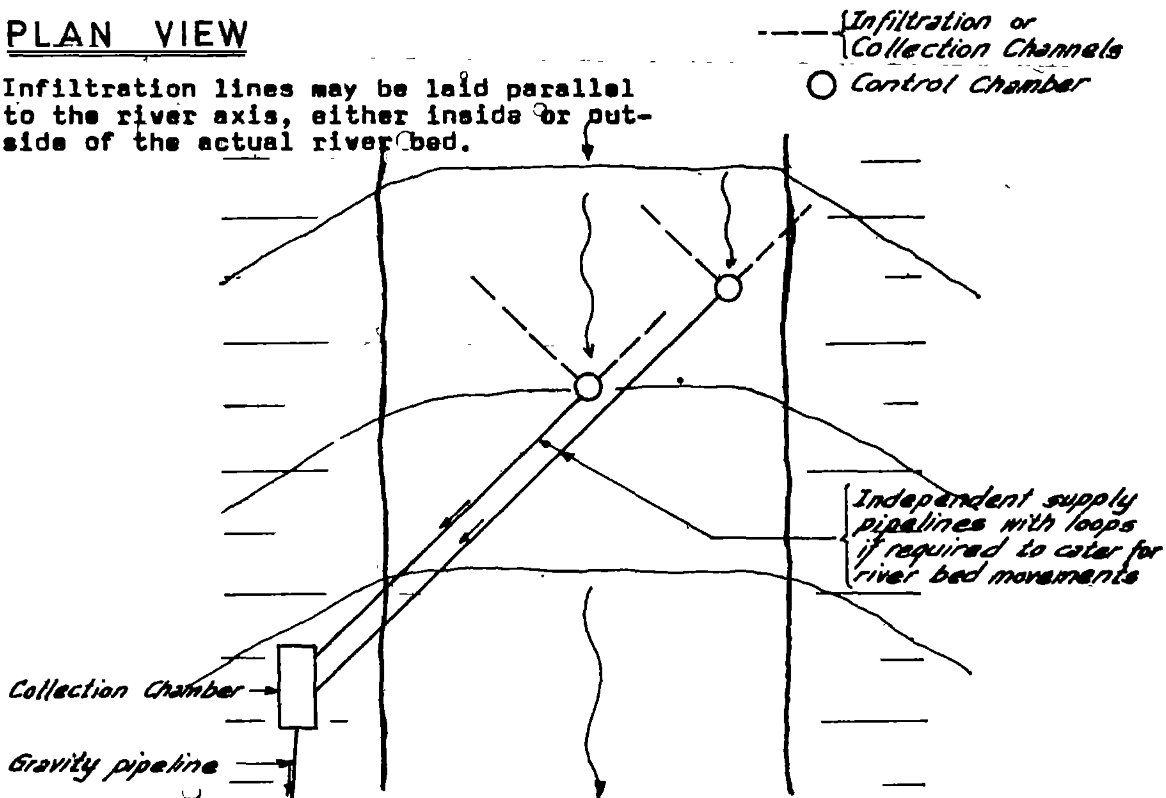
Section B-B



RIVER INTAKE Type $C_{a)}$

PLAN VIEW

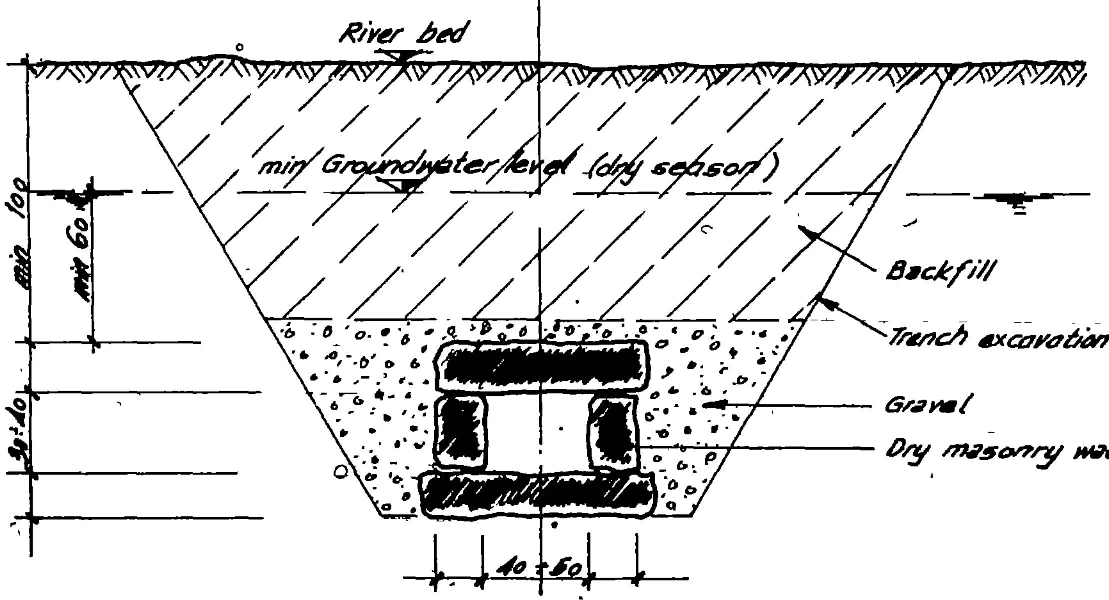
Infiltration lines may be laid parallel to the river axis, either inside or outside of the actual river bed.



Difference in elevation between Control- and Collection Chamber (add to the below mentioned figures the supply pipe friction loss):
- in stable river beds: greater than 0.5 m; desirable 2.0 m
- in unstable river beds: greater than 5.0 m

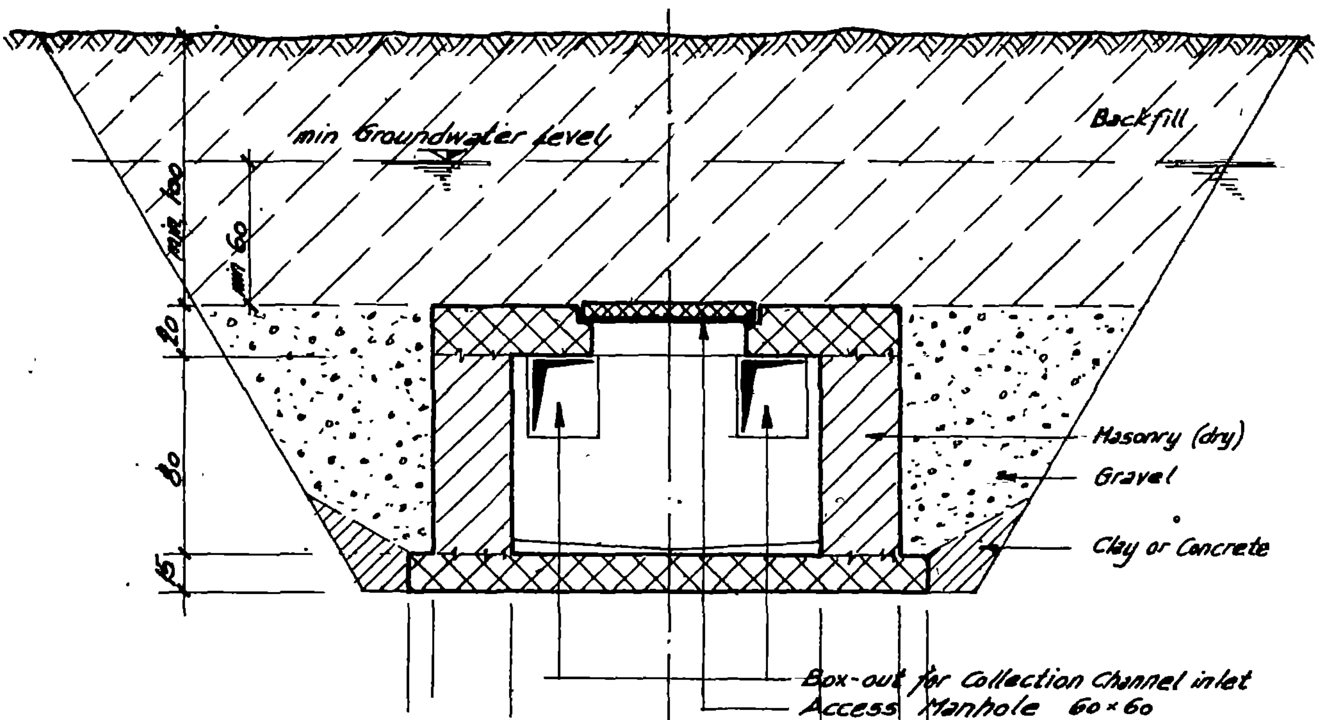
The following length of infiltration channel is required to produce 1 m3 water per day:
- good aquifer > 0.2 m
- poor aquifer > 2.0 m

Cross Section Collection Channel

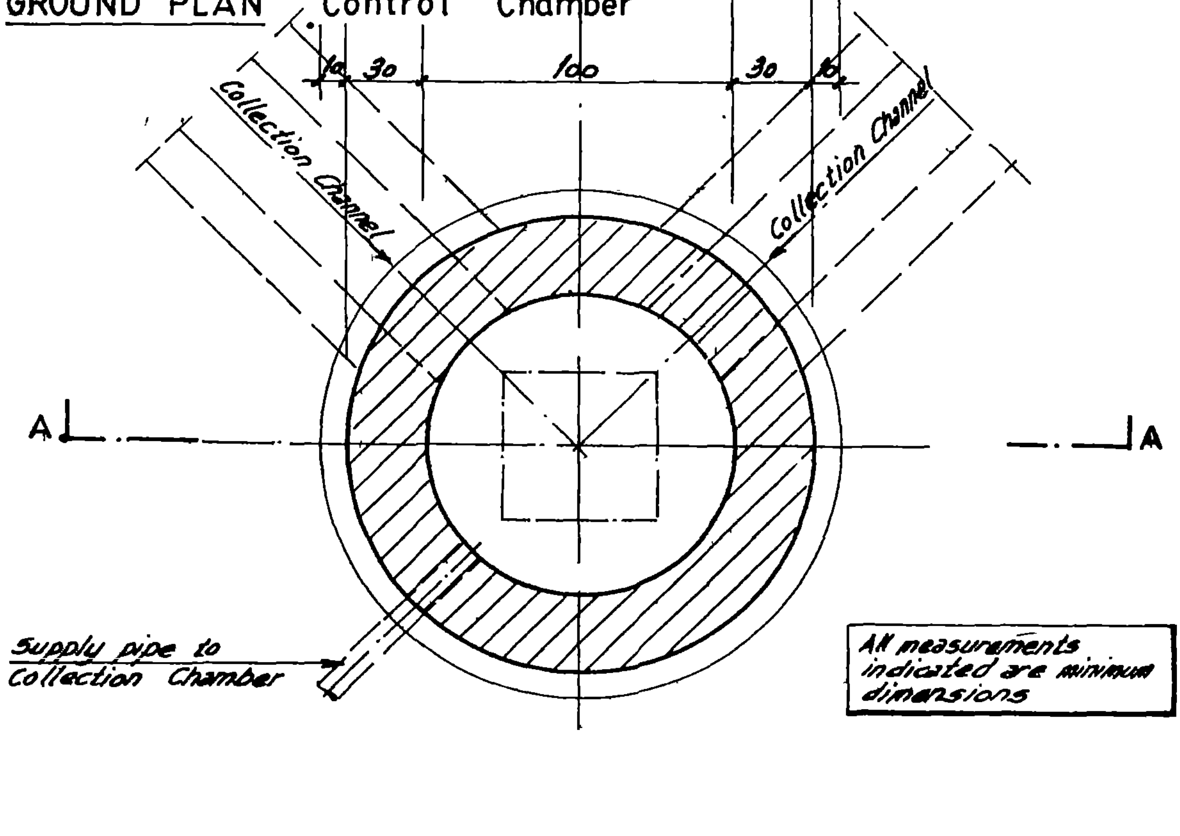


RIVER INTAKE Type C b)

Section A-A



GROUND PLAN Control Chamber



S P R I N G   C A T C H M E N T

A) Spring Protection Zone

The catchment area consists of the area which is situated just above the catchment and whose surface water may drain into it. This area must be properly protected. It is proposed to fence this area either with barbed wire, stone walls or living thorn hedges such as roses. The size of the protected area depends on the thickness and type of covering material above the actual catchment. Spring catchments constructed according to this guide lines require a protected catchment area of approximate 50 m in radius. Shallow spring catchments or spring catchments covered with unsuitable material require a much bigger protected area.
Within this area no farming, no domestic animal grazing, no fish ponds, rubbish pits or permanent/temporary structures are permitted. Existing surface water drains must be channeled in such a way that they are not able to pollute the spring water.
For easy surveillance and maintenance it is advised to plant grass within the perimeter of the catchment structures. The rest of the area should be afforested with bushes.
The extended catchment protection area deals mostly with the watershed area of the source. In most cases this area is agriculturally used and therefore protection measures are extremely difficult to implement. However the use of fertilizer within the first 200 to 300 m above the catchment should be minimized. In case the extended catchment area is still covered with forest or bush the community should be encouraged to declare the whole area as a protected zone. This means that the area should not be turned into agricultural land but the timber or fire wood produced by the area should be utilized.

B) General Construction Rules

It is of great importance to construct the spring catchment carefully because it is the heart of the water supply. Failure to do so, may cause a total breakdown of the entire water supply system.
A spring catchment should be built in a simple and practical way. The type of catchment to be utilized depends on the topographical situation, type of source and the prevailing geological condition.
No attempt should be made to change the springs natural flow regime. If there is any flow obstruction either during construction or later the water may be tempted to find an other route.
The depth of the catchment construction depends on geological and hygienic considerations. Under normal circumstances the catchment should be covered at least by 3 m soil. If this is not possible it is necessary to make special protective arrangements. Whenever possible the catchment should be built right onto the impermeable strata. Blasting near springs or for catchment excavation is not permitted.

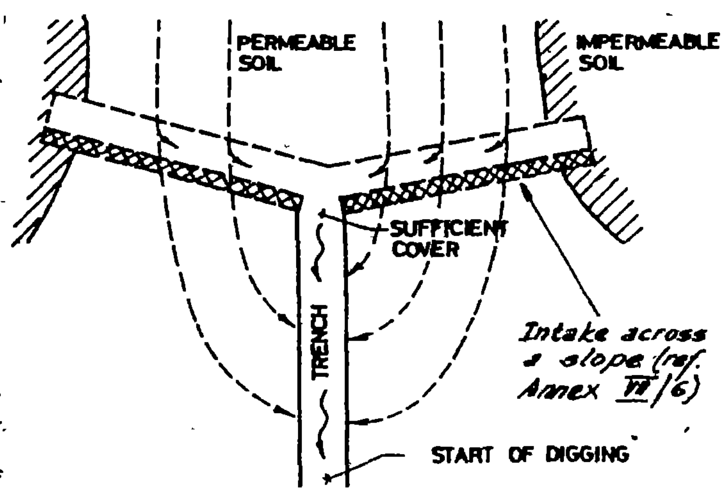
On bigger spring catchments it is a good practice to install two supply pipes between the catchment and the control or collection chamber. This additional pipe should be located slightly higher than the first one. During rainy season may be both of them will be operational. But if the higher located pipe is operational only it indicates that a problem within the catchment has occurred which needs urgent attention before the spring water finds a way around the catchment.
The most common faults observed on spring catchments are illustrated on this Annex VII page 11.

Much experience is required to design and construct a spring catchment. Due to the nature of work it is impossible to prepare standard designs for spring catchments. The following notes and sketches are provided for information and illustration purpose only.
For each spring catchment a drawing or sketch should be prepared. This document is not only required for work estimation but outlines the designers ideas on what kind of structure is envisaged. However it is of great importance that the Technician is guided during the construction of a spring catchment by an experienced Overseer or Engineer.

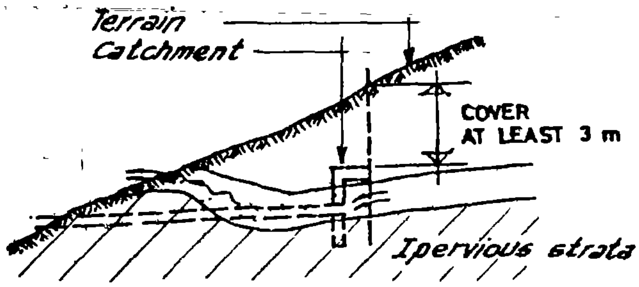
C) Spring Catchment Excavation

Usually excavation starts at the point where the water emerges from the ground. While the digging follows the flow of water it is important to keep always a drain open to ensure a free outflow of the spring water. The following examples are given as a guideline and illustrate just a few situations encountered during the construction of a spring catchment:

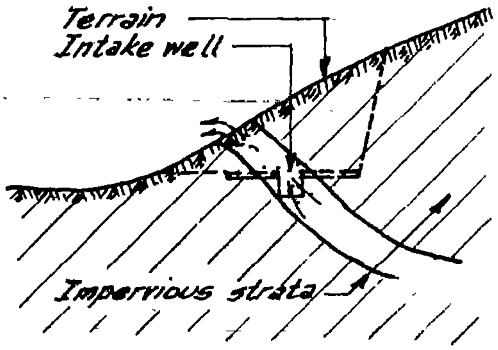
Example 1:
The amount of water coming out at the head of the trench decreases with further excavation. Therefore, water is entering on one or both sides along the trench. In this case the trench must be split up in a V or T shaped trench as indicated by the flow of water. The junction is located at the place where a sufficient covering of the water bearing strata has been achieved. Digging on the branch trenches continuous until there is the impression that even with further digging the flow of water will not increase any more.



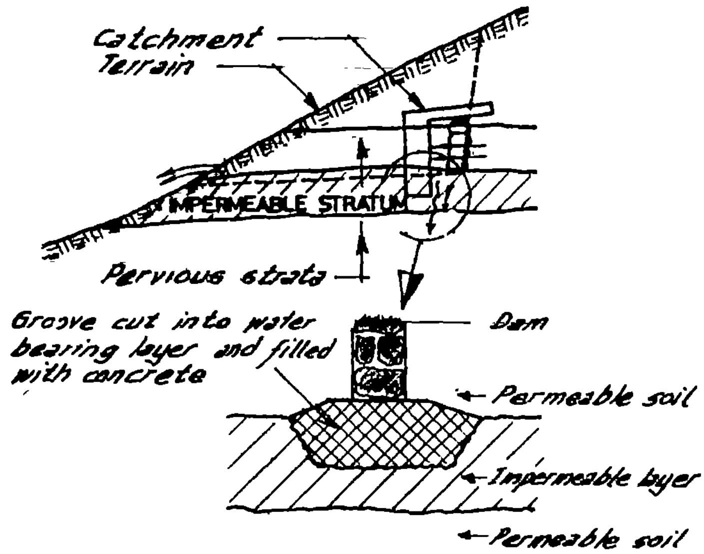
Example 2:
Spring water is coming up from the ground. A drain has to be excavated until the water bearing strata is met out of which the water originates. In case the cover is insufficient the digging will continue along the flow of water until an acceptable covering of the water bearing strata has been achieved.



Example 3:
In case the drain cannot be excavated to such a depth as required to find the water bearing strata the catchment construction must be done in a similar way as for artesian well (e.g. dug well construction with prefabricated well casings).



Example 4:
Utmost care must be applied when constructing spring catchments on relatively thin water bearing strata. If during construction the excavation penetrates this impermeable layer the spring water may disappear through such a damaged layer. If it is suspected or known that the water bearing strata is extremely small it is necessary to cast the foundation of the dam directly into the groove provided in the water bearing layer. Afterwards the dam may be built in the usual manner.



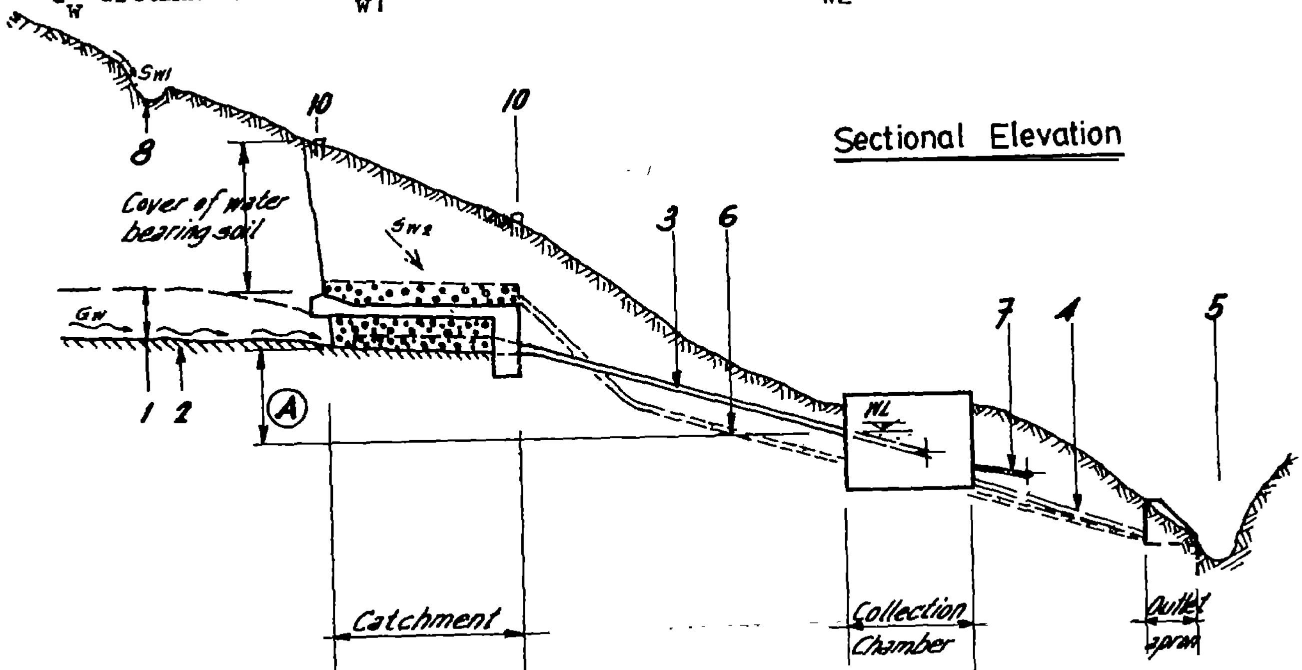
Example 5:
The distance between the catchment and any tree should be large enough to ensure that no roots are able to enter the catchment. This point needs special attention in case of shallow springs catchments, since neither plastic nor concrete as watertight cover are root proof, or with other words the roots will find their way through such construction materials.

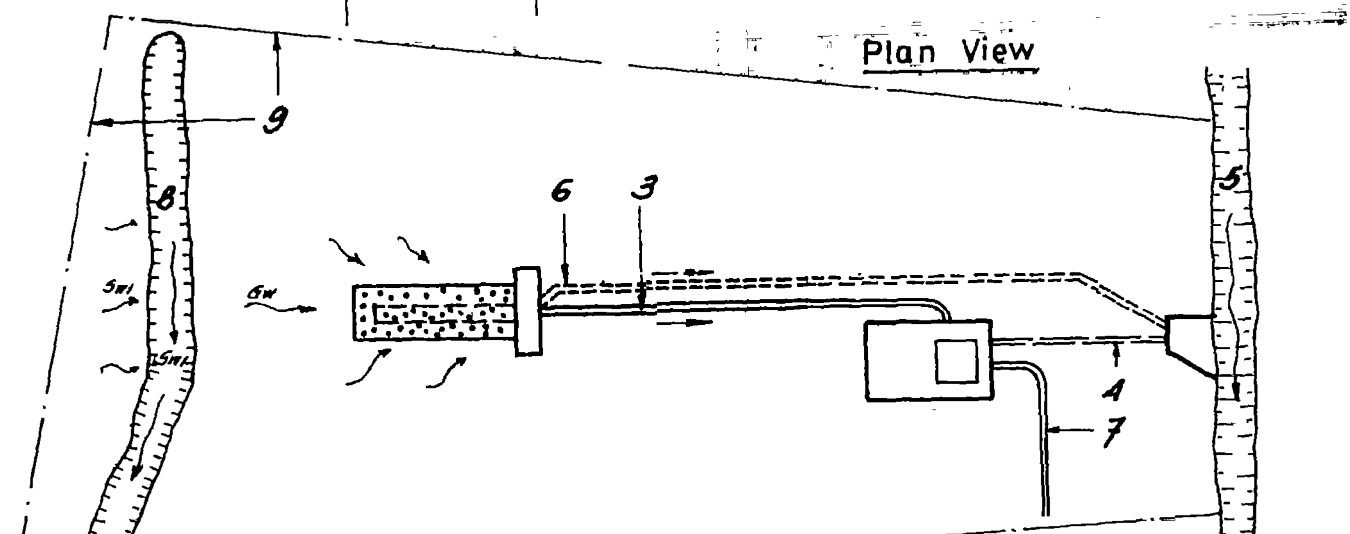
SPRING CATCHMENT GENERAL ARRANGEMENT

LEGEND

1 Water bearing soil
2 Impervious strata
3 Supply pipe
4 Overflow/drain pipe
5 River
6 Seepage water drain pipe
7 Gravity pipe line
8 Surface run off water interception drain
9 Delimitation of spring protection area
10 Benchmarks for catchment location

G_W Groundwater S_{W1} Surface run off water S_{W2} Seepage water



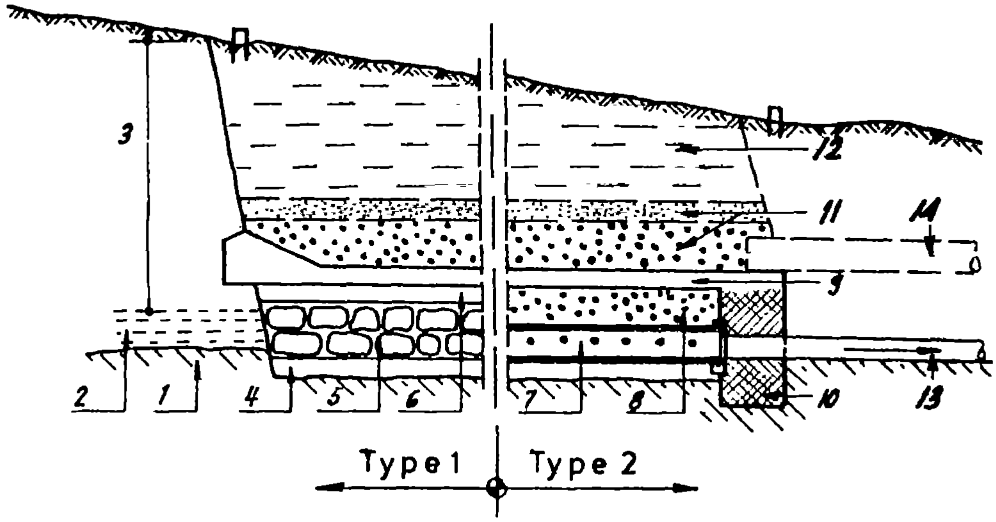


Ⓐ Difference in elevation between SPRING HORIZON and WL COLLECTION CHAMBER:
- in flat areas greater than 0.5 m
- desirable 2.0 m
- catchment in geological unstable zone 5.0 m

Note: Friction head loss in supply pipe must be added to the above indicated figures

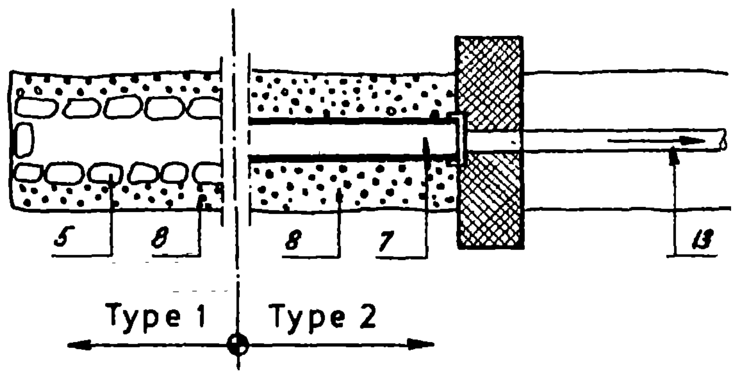
Intake-details

Sectional Elevation

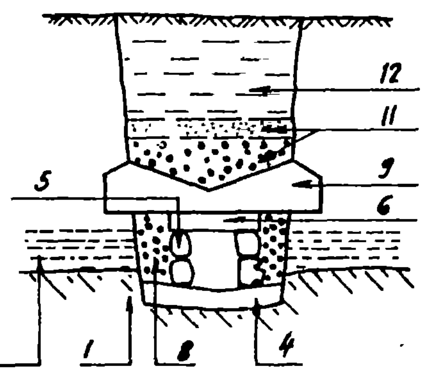


1 Impervious strata
2 Water bearing soil
3 Cover of water bearing soil
4 Bed plate
5 Dry wall
6 Cover slab
7 Perforated pipe
8 Gravel
9 Watertight cover
10 Dam
11 Permeable material
12 Impermeable backfilling
13 Supply pipe
14 Seepage water drain pipe

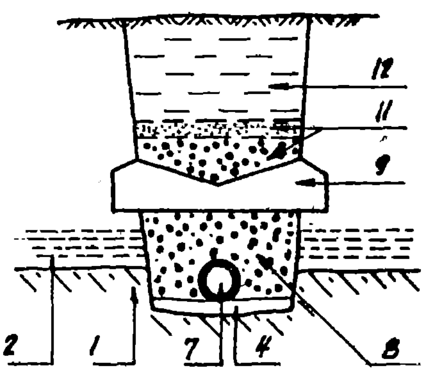
Plan



Cross section type 1

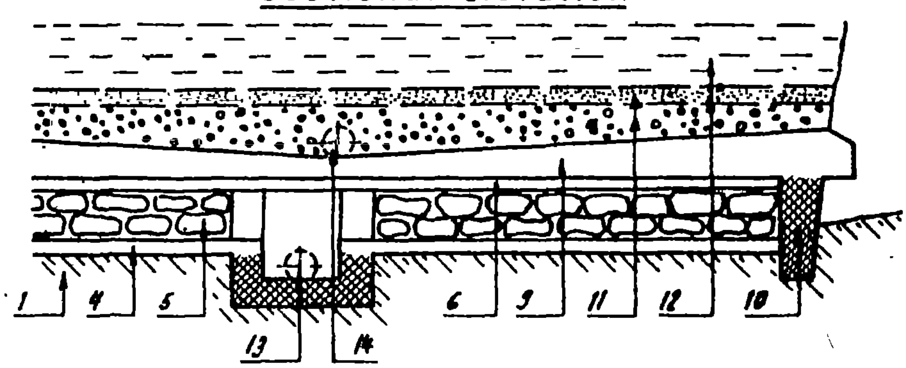


Cross section type 2



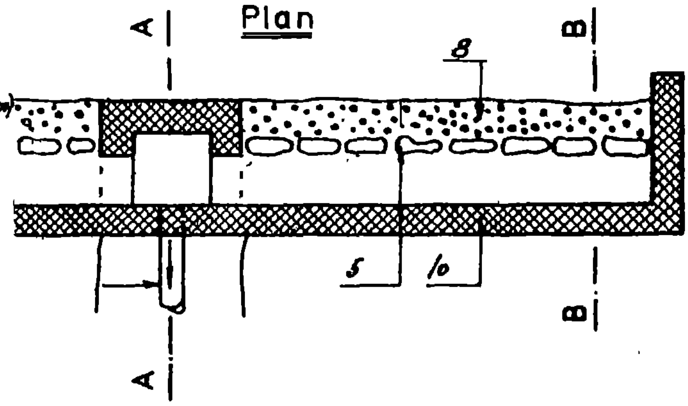
Intake across a slope – details

Sectional elevation

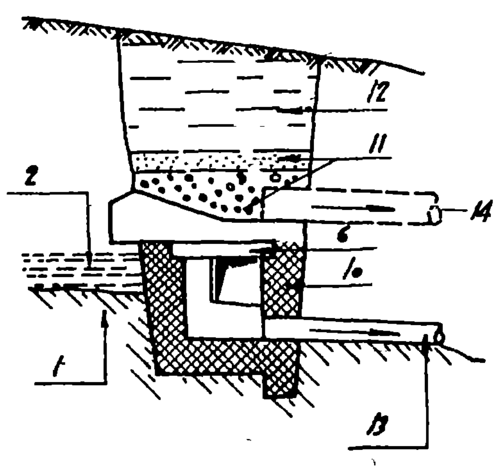


1 Impervious strata
2 Water bearing soil
3 Cover over water bearing soil
4 Bed plate
5 Dry masonry wall
6 Cover slab
7 Perforated pipe (not shown)
8 Gravel
9 Watertight cover
10 Dam
11 Permeable material
12 Impermeable back-filling material
13 Supply pipe
14 Seepage water drain pipe

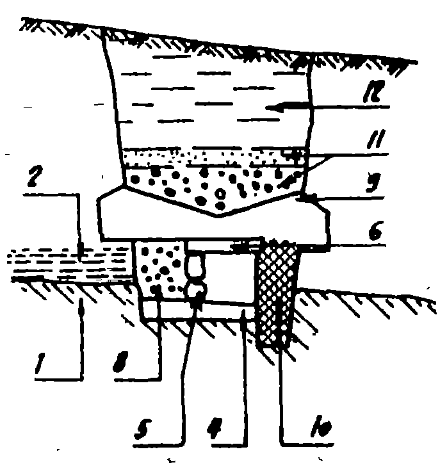
Plan



Section A-A

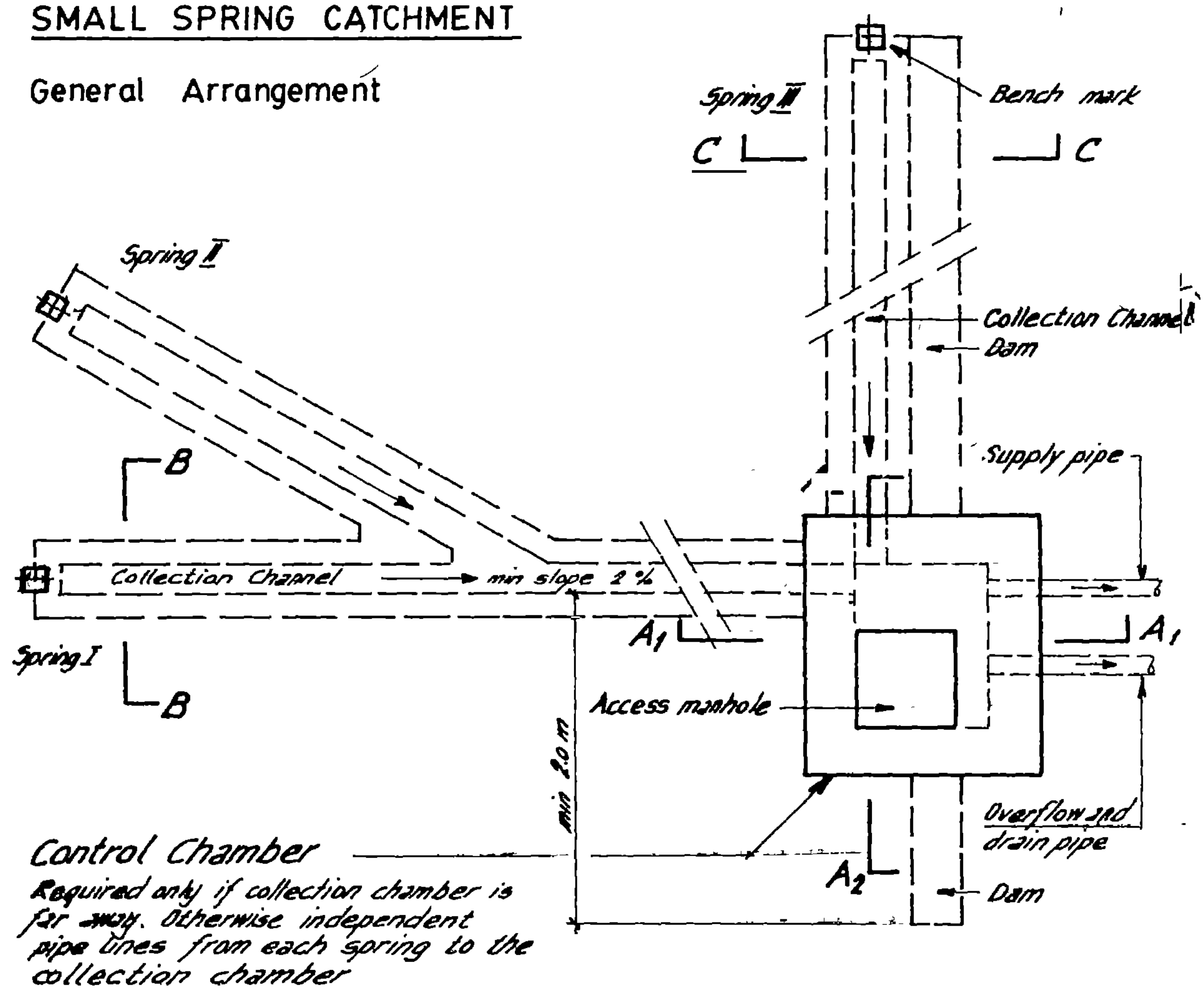


Section B-B

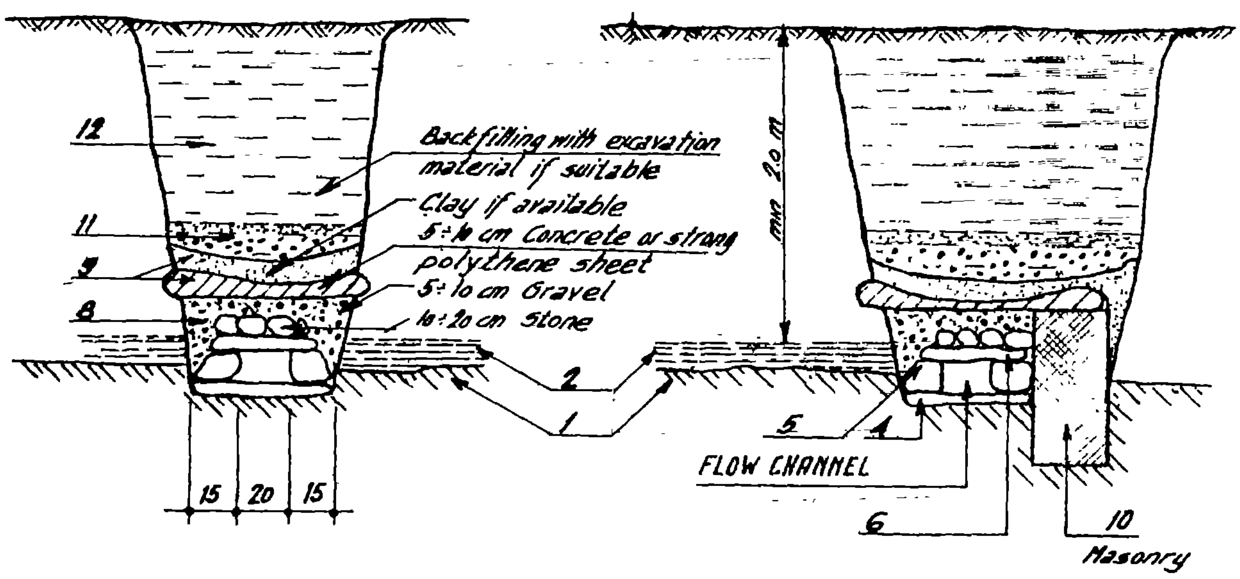


SMALL SPRING CATCHMENT

General Arrangement

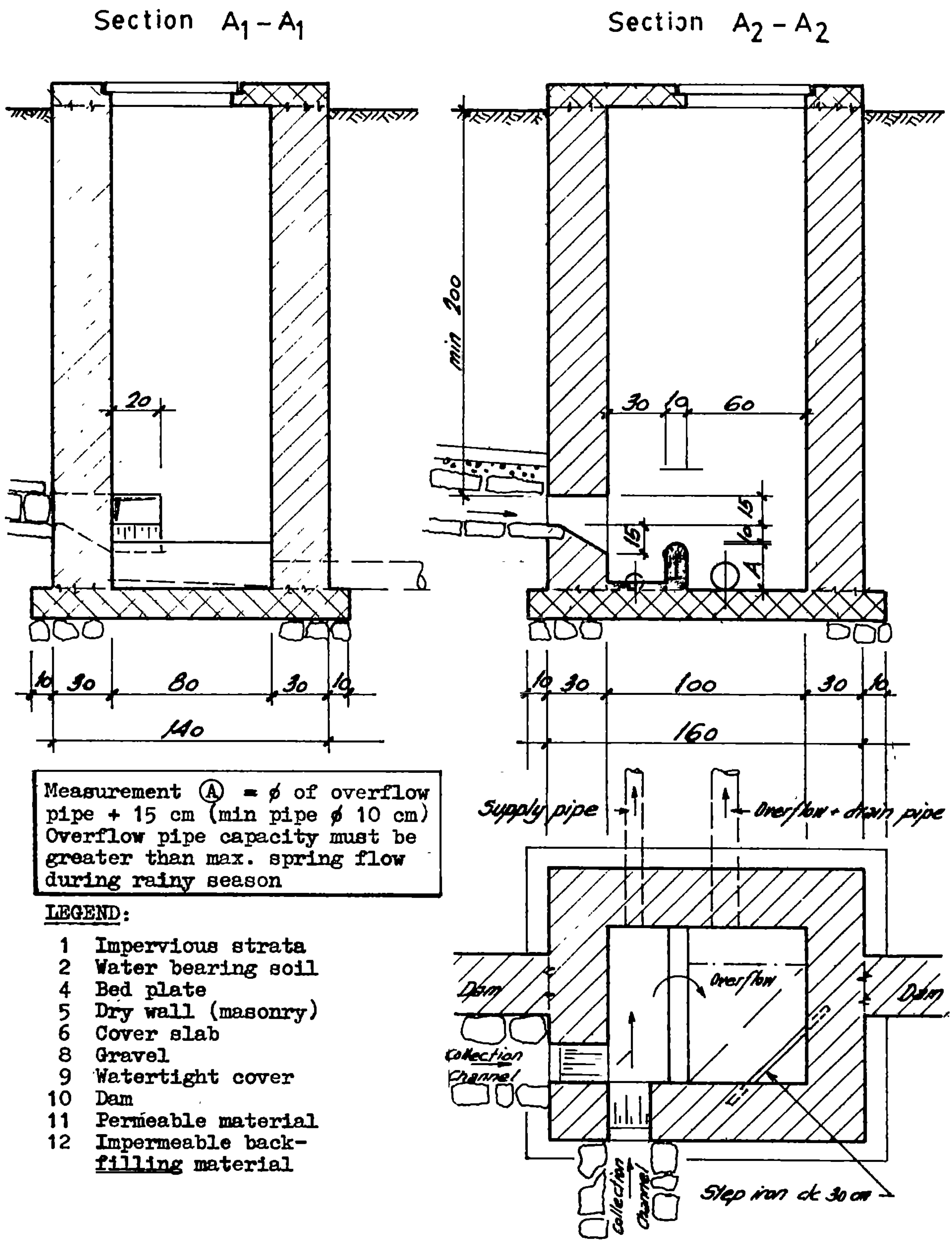


Section B-B   Section C-C



SMALL SPRING CATCHMENT

Control Chamber Details



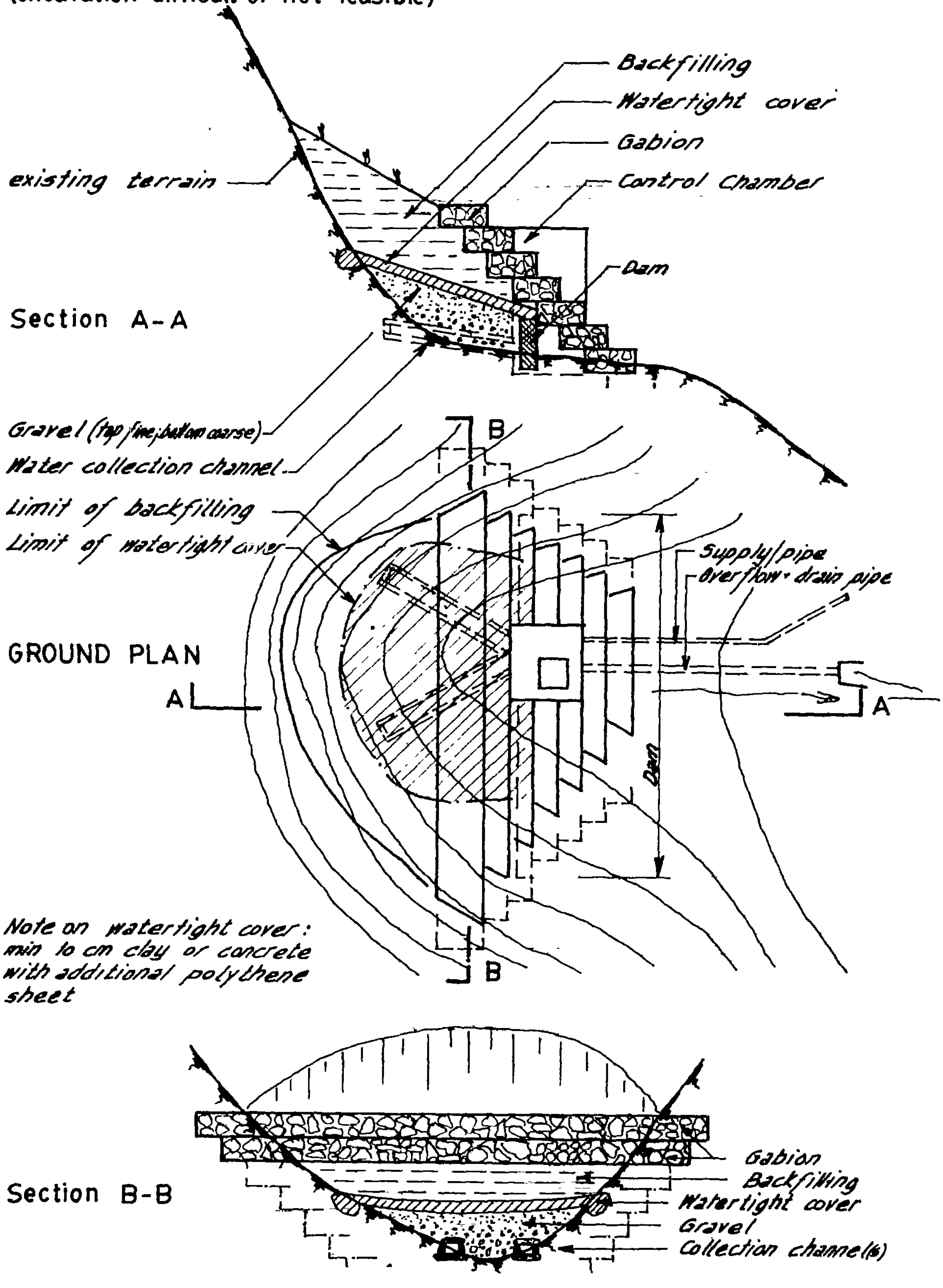
Measurement Ⓐ = ø of overflow pipe + 15 cm (min pipe ø 10 cm) Overflow pipe capacity must be greater than max. spring flow during rainy season

LEGEND:

1 Impervious strata
2 Water bearing soil
4 Bed plate
5 Dry wall (masonry)
6 Cover slab
8 Gravel
9 Watertight cover
10 Dam
11 Permeable material
12 Impermeable back-filling material

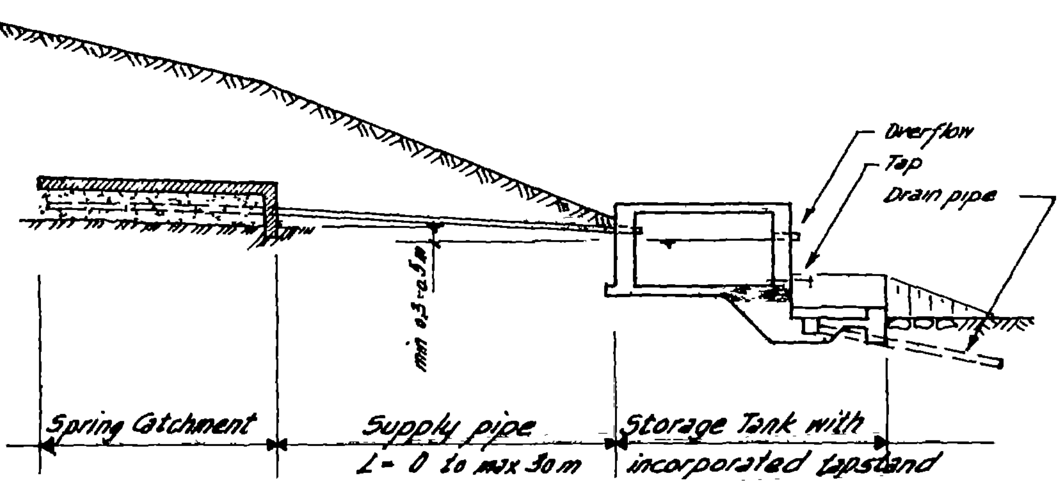
SPRING CATCHMENT in Rocky Areas or Steep Hill Sides

(excavation difficult or not feasible)



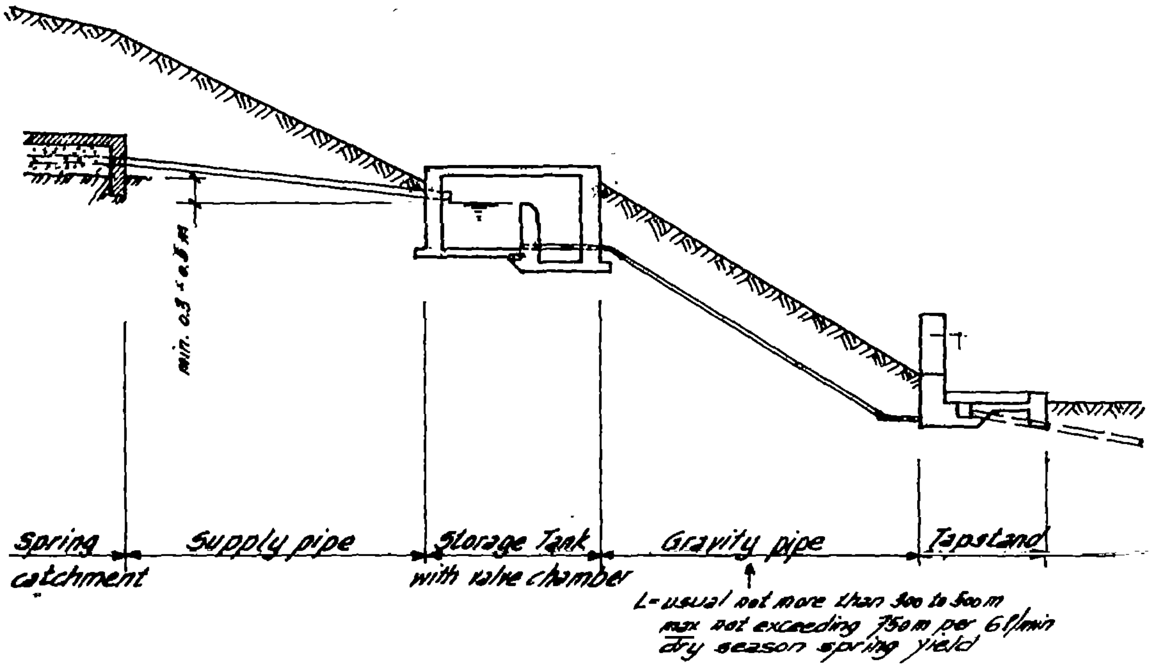
SOURCE IMPROVEMENTS

Small Water Point

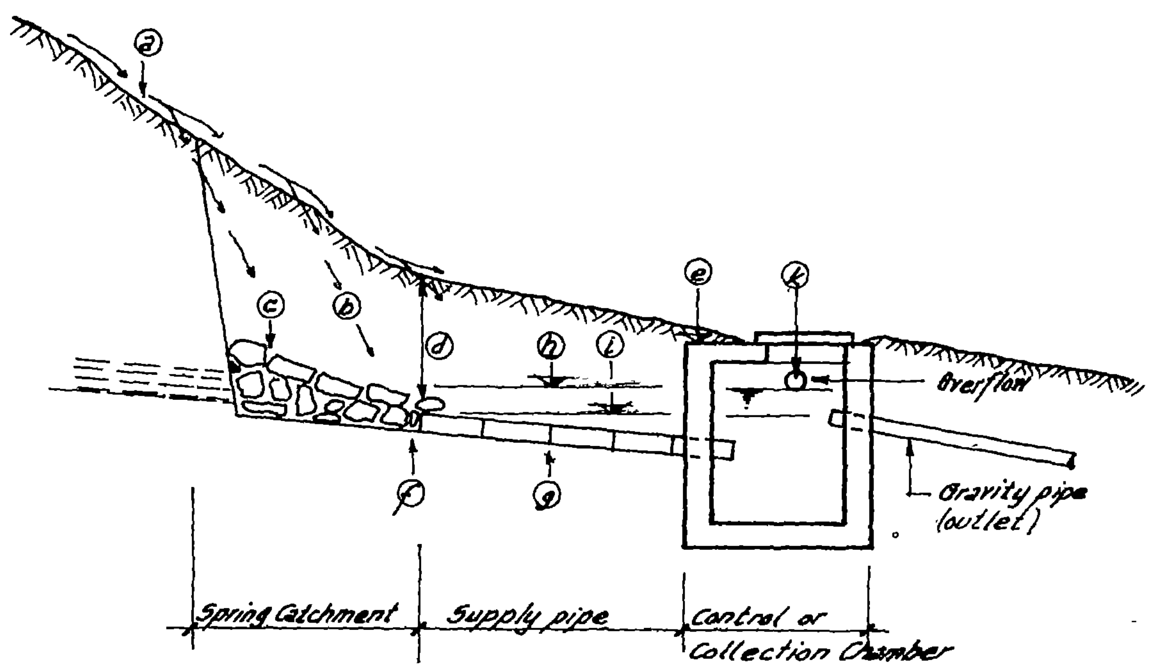


TYPE OF WATER POINT	SMALL	LARGE
- Minimum spring yield	>1 l/m	>2 l/m
- Water availability	15 ÷ 30 lcd	25 ÷ 45 lcd
- Daily water demand calculation: future population times	30 lcd	45 lcd
- Storage Tank capacity:	Ref chapter 2.7.3: except minimum volume required 5% of daily demand but not less than 400 l	

Large Water Point



MOST COMMON FAULTS ON SPRING CATCHMENTS



L E G E N D :

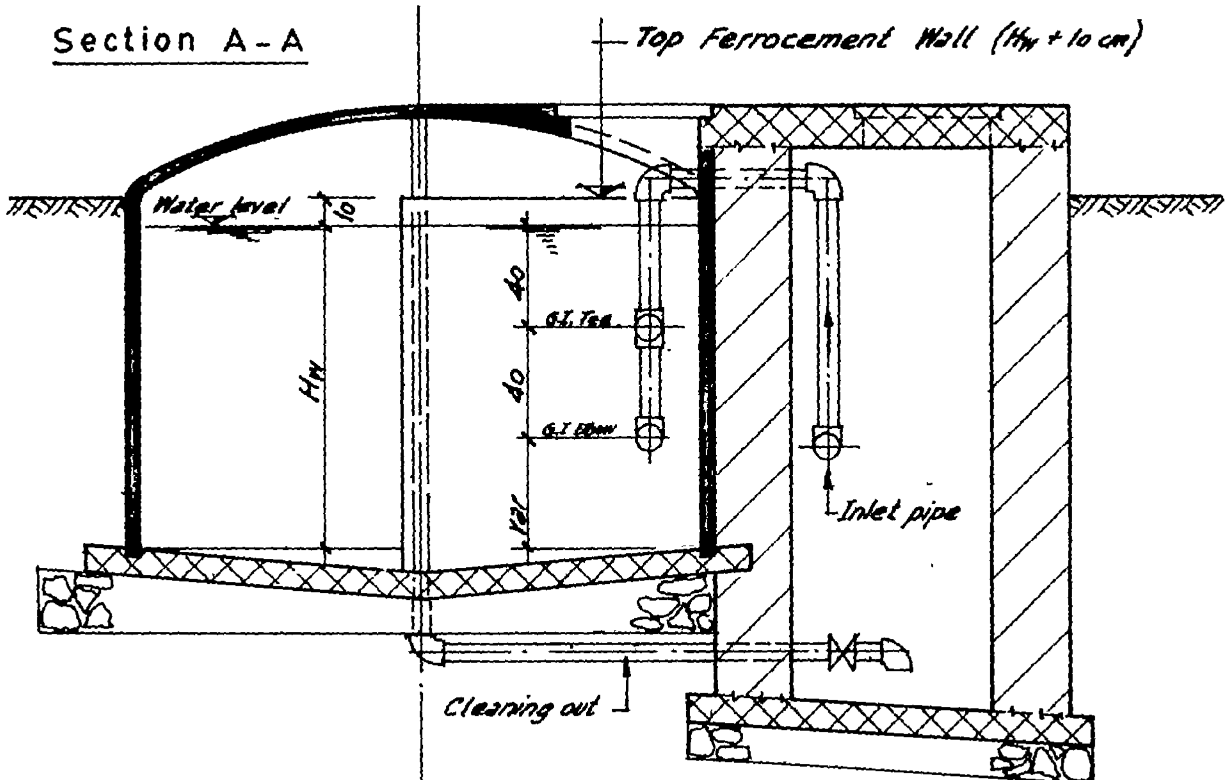
ⓐ no surface run off water interception drain ⓑ unsuitable backfilling material (permeable) ⓒ no watertight cover ⓓ cover of water bearing soil inadequate ⓔ top of control/collection chamber below ground level ⓕ no dam or dam with too short wing-walls and/or insufficient depth of dam foundation	surface water is able to pollute spring water
ⓕ (continued), ⓖ leakages from pipe joints, ⓗ, ⓘ	loss of spring water
ⓗ position (elevation) of overflow to high 1) ⓘ position (elevation) of outlet to high 1) ⓚ insufficient overflow capacity	obstruction to spring flow and impounding of spring 2)

ⓖ leakages from pipe joints

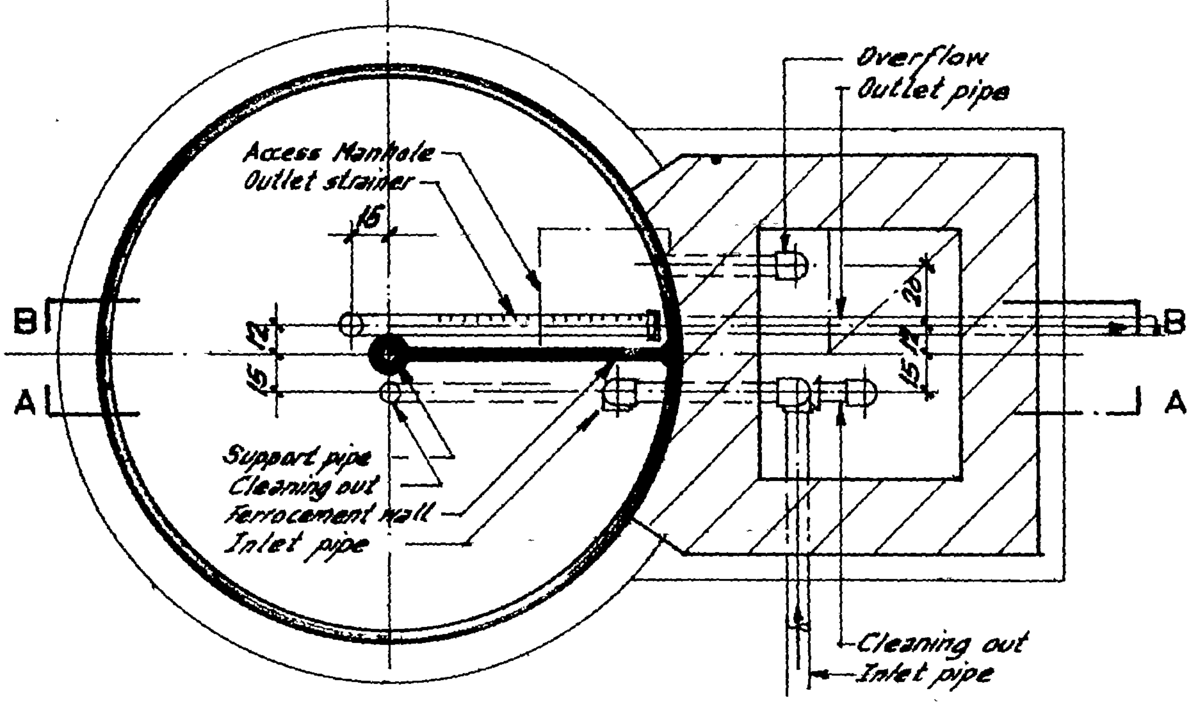
Note: 1) in relation to spring horizon
2) spring may find an other outlet to overcome the obstruction

SEDIMENTATION TANK

Section A - A

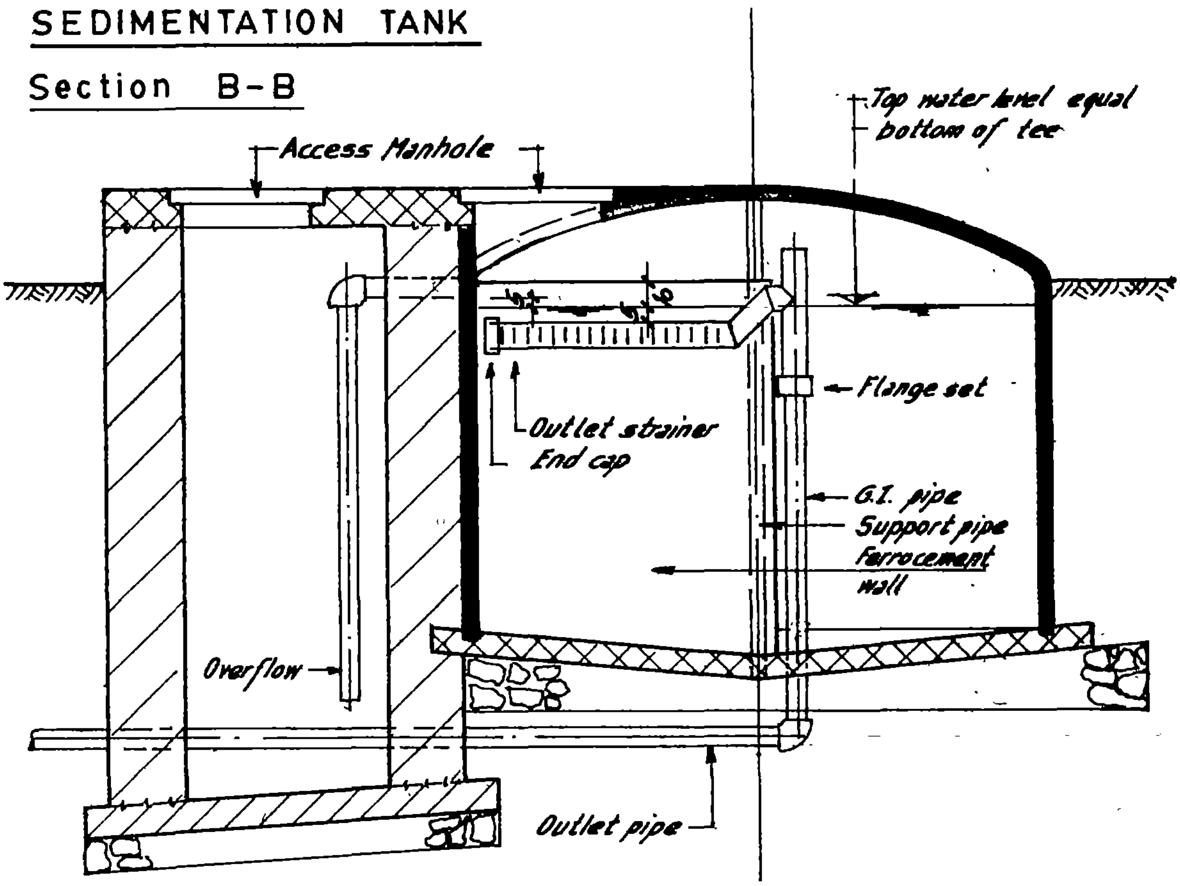


GROUND PLAN



SEDIMENTATION TANK

Section B-B



REMARKS TO THE SEDIMENTATION TANK

1) The sedimentation tank structure is based on the CWSS standard storage tank design.
2) The ferrocement partition wall shall be fixed to the support pipe by bending some reinforcement bars and chicken wire around the pipe and than applying a layer of mortar (minimum 3 cm).
   On the opposit site a small groove in the ferrocement tank wall is required as a key for the partition wall.
3) Pipe installation for the sedimentation tank as compared to the standard storage tank (confirm whether pipe sizes are appropriate).

Pipe	Location	Pipe and Fitting requirements
a) Overflow b) Cleaning out	different	same
c) Inlet d) Outlet	different different	within operation chamber and under the tank floor the same, but within sed. tank additional pipes and fittings are required (ref. drawing),
e) Drainage	as usual	as uaual

4) The outlet strainer is made of PE pipe, with saw blade cuts c/c 2 cm.
   The strainer shall be fixed to the partition wall.
   The open ended PE outlet tee will control the water level, therefore it is not necessary to install the strainer exactly horizontal.

[illegible]