WATER SUPPLY
FOR
RAILWAY ENGINEERS

MAY 2005

INDIAN RAILWAYS INSTITUTE OF CIVIL ENGINEERING
PUNE 411001

PREFACE TO REPRINT

The original book on 'Water Supply for Railway Engineers' was brought out in 1998. The book covered various topics related to water supply and served as a handy reference book for officials entrusted with construction and maintenance of water supply systems. In view of its great demand, the book became out of stock. Therefore, it has been decided to reprint the book for the benefit of field engineers.

The reprint has been brought out with better quality paper and printing. Shri Sudhanshu Sharma, Professor Bridges and Shri A. K. Rai, Professor Works have carried out the updating and proof-checking of the book.

MAY, 2005
PUNE

SHIV KUMAR
DIRECTOR / IRICEN

FOREWORD

An efficient water supply scheme leads to customer and staff satisfaction. These schemes require heavy investments. Hence, there is an imperative need for proper planning and execution of such works. I am glad IRICEN is bringing out this booklet dealing with water supply. I hope this publication will be useful to Railwaymen who are responsible for construction and maintenance of water supply installations.

V. K. AGNIHOTRI
MEMBER ENGINEERING
RAILWAY BOARD
NEW DELHI

PREFACE

The subject of water supply is of considerable importance to the field officials who are engaged in this aspects of work of the civil engineering department. This booklet aims to bring together relevant details for the field engineers which are available in scattered form in various text book, handbooks, codes and manuals.

It is hoped that engineers in the field connected with water supply will find this volume a useful source of knowledge and guidance.

This book has been prepared by Shri Rajesh Agarwal, Professor of this Institute. If there are any suggestions for improving the book or if any error/discrepancy is noticed in its contents, kindly write to the undersigned.

November, 1998
Pune

Vinod Kumar
Director
IRICEN

ACKNOWLEDGEMENTS

While covering the subject of water supply during various courses at IRICEN, the absence of a document covering all aspects of water supply was acutely felt. Information on water supply is available in various technical literatures but it is scattered.

This IRICEN publication is result of the desire to fill the gap and produce a documentation which would be useful for all practicing civil engineers on Indian Railways. All possible care has been taken to collect the information from authentic sources. However, readers are requested to refer original references for more details.

This booklet has been prepared with the able assistance of Shri A.V.Limaye, ex faculty of IRICEN. Through discussion with him, many grey areas in the subject and requirement of field engineers could be assessed and dealt with suitably. The services provided by him are praiseworthy.

It would not be out of place to acknowledge the support and assistance rendered by IRICEN faculty and staff in the above efforts. Word processing of the manuscript and numerous editing thereof has been done by Mrs. Vidya Jamma. The preparation of the drawings have been done by Shri Anil Padmane. The printing of this booklet has been done departmentally with the help of Shri P.D.Khajindar and Shri Pradeep Tawade.

Above all, the author is grateful to Shri Vinod Kumar, Director IRICEN for his encouragement and guidance in improving the document.

November, 1998
Pune

Rajesh Agarwal
Professor (Works)
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WATER SUPPLY

1.0 Introduction : The subject of water supply involves assessment of requirement of water, development of its sources, purification of raw water, storage and conveyance. The common problems like borewell verticality, yield, determination of suitable diameter of pipes, network analysis and the details required for maintenance of water purification plants have been dealt. Each stage involves design aspects. Hence, wherever necessary illustrative examples have been included to supplement the text.

2.0 Requirement of water : It is the responsibility of Civil Engineering Department to arrange supply adequate quantity of water of acceptable quality to Railway premises. To discharge this responsibility, it is essential to assess the requirements of water supply. This exercise is done for various locations and then total requirement of water is worked out. The yardsticks for water requirement are laid down in Indian Railways Works Manual (IRWM). The latest yardsticks as per the Manual are as under:

Table 1

Particulars	Demand (litres per day)
a) Residential area. Household consumption for officers & staff	200 per capita (includes 45 Litres required for flushing) per head
b) Office and work shop - Offices - Workshops	 45 per capita 30 per capita
c) Station and Platform - Apron washing - Platform washing - Passengers on Railway Station* - Washing of Carriages on washing lines - Cleaning of Carriages on Platform - Carriage watering**	 10 per m^2 5 per m^2 25 per Passenger 3600 per carriage B.G. 2600 per carriage M.G. 500 per carriage As per actual
d) - Miscellaneous - Garden per hectare of Lawn area - Hospital - Fire service	 22500 approx 450 per Bed Occasional sumps and hydrants to be adequate for emergencies

* Number of Passengers entraining at the station plus half of the passengers detraining.

** Full tank capacity for originating station and 75% for other stations. One B.G. coach requires app. 900 litres. There are 4 tanks, each of 225 litres in BEML coach & 275 x 4 litres in ICF coach.

The requirement of water worked out on the basis of Table-1 has to be further enchanced based on future developmental growth of the station.

The quantity of water based on above assessment has to be increased to cater to the peak requirement on some days on account of festivals, holidays etc. Generally, a factor of 1.8 is adopted to convert the average daily demand to peak daily demand. It is essential to take into account the requirement of water due to "fire demand" also. This "fire demand" can be assessed according to local requirements and in the absence of any specific guidelines, the method stipulated as per IS:9668-1980 can be followed, as one of the method.

Example-1 Take the case of Divisional Headquarter for calculation of water requirement and storage capacity. The data is only for illustration purpose.

Number of Quarters	= 1500
Number of members per family	= 5
Total number of occupants	= 1500 x 5 = 7500
Average per capita consumption	= 200 lpd
Total average demand	= 7500 x 200 = 1500 klpd
Average daily demand on maximum day	= 1.8 x 1500 = 2700 klpd

Note : (lpd = litres per day, klpd= Kilolitre per day)

Fire demand as per IS 9668 - 1980 = 1800 l/min for every 50000 population or part thereof. The fire reserve specified should be maintained for at least 4 hours. Local authority should frame suitable by-laws based on merits of each case.

Hence total water requirement for fire = 1800 x 60 x 4 = 432 klpd.

1/3 of this fire demand will be provided in the service storage. The balance requirement may be distributed in several static tanks at strategic points.

Hence, volume to be provided in service reservoir = 1/3 x 432 = 144 klpd

= 1.44 lakh litres.

Operational requirements :

1 DRM office + subordinate offices
Assuming total staff strength of 500 persons
Requirement of water at 45 liters per capita = 500 x 45 = 22500 lpd
= 22.5 klpd

2 Divisional Hospital
Assuming 25 beds
Requirement of water at 450 liters per bed = 25 x 450 = 12250 lpd
= 11.25 klpd

3 Garden of DRM office, Hospital & Children park
Assume total area 2 hectare
Requirement of water per hectare is 22500 liter/day
Total requirement of water = 2 x 22500 = 45000 lpd
= 45 klpd

4 Passenger's requirement
Assume total number of passengers = 2500 per day
Requirement of water per passenger = 25 liters per day
Toatal requirement of water for passengers = 2500 x 25 = 62500 lpd
= 62.5 klpd

5 Apron washing
Assume 2 aprons of 400 metre length, 3 m. wide
Total area of aprons = 2 x 400 x 3 = 2400 m^2
Requirement of water for apron washing is 10 litres per m^2/day
Total requirement of water for aprons = 2400 x 10 = 24000 lpd
= 24 klpd

6 Rake washing
Assume 2 rakes of 18 coaches each are maintained at divisional headquarters
Requirement of water for one coach = 3600 litres/day
Total requirement of water for coach washing = 2 x 18 x 3600
= 129600 lpd
say 130 klpd

7 Carriage filling
Lump sum quantity = 300 klpd.
Otherwise take actual no. of trains to be watered. (Each B.G. coach has 4 x 225 litres.)

8 Misc. requirement like gardening for quarters, institute, schools, Small workshops, losses etc. = 10 klpd.

Total requirement of water = 3305.25 klpd say 3300 klpd
Average hourly requirement of water = 3300/24 = 137.5 klpd.

The average hourly demand as worked out above has to be modified to bring out variation in hourly demand of water at different period of the day. Normally for residential colonies, the requirement of water will be higher during morning and evening hours, whereas the requirement on stations area will be higher during train timings.

This diurnal variation should be the basis of calculating the storage capacity of water tanks and the pumping arrangements. As a matter of practice, capacities of various components of a water supply scheme are designed as shown in Table 2.

Table 2

STRUCTURES	CAPACITY
a) The intake structure for fetching water from a source like river, pond, lake etc.	Maximum daily demand
b) The pipeline mains from intake to treatment plant	Maximum daily demand
c) Treatment plant	Maximum daily + some reserve or twice of average daily demand
d) Pumps for delivering water from treatment plant to storage tanks	Twice of a average daily demand
e) Storage capacity with efficient standby pumps	Higher of i) One forth of maximum daily demand ii) One third of average daily demand
f) Distribution system	Peak hourly demand

3.0 Storage of water : Water meant for distribution is required to be stored in storage tanks located at suitable locations. The storage may be in the form of ground level reservoirs or high level storage tanks (overhead storage). These reservoirs are generally located near the place of consumption of water and on a high ground to reduce the staging cost. The capacity of these storage tanks is determined based on a "Mass Curve" plotted on the basis of daily pumping and consumption pattern. The distribution pattern of demand of water varying throughout the day is illustrated in Table 3. The distribution pattern is in terms of IS Handbook SP:35:1987.

Table 3
FOR 8 HOURS OF PUMPING

Time							
From (Hour)	To (Hour)	Hourly demand	Cumu-lative demand	Avg rate of Pumping per hour (24xa)/8	Cumu-lative pumping	Cumulative deficit or surplus Surplus:+ Ve Deficit: -Ve	Starage in Reservoir at the end of Period
0	4	0.2a	0.8a	3a	12a	+ 11.2a	16.6a
4	5	0.4a	1.2a	3a	15a	+13.8a	19.2a
5	6	0.8a	2.0a	3a	18a	+ 16a	21.4a
6	10	2.25a	11.0a	-	18a	+ 7a	12.4a
10	12	a	13.0a	-	18a	+ 5a	10.4a
12	13	0.6a	13.6a	-	18a	+ 4.4a	9.8a
13	14	2.25a	15.85a	-	18a	+ 2.15a	7.55a
14	17	0.7a	17.95a	-	18a	+ 0.05a	5.45a
17	18	2.25a	20.20a	-	18a	-2.2a	3.2a
18	20	0.9a	22.0a	-	18a	-4a	1.4a
20	22	0.7a	23.4a	-	18a	-5.4a	0
22	23	0.4a	23.8a	3a	21a	-2.8a	2.6a
23	24	0.2a	24.0a	3a	24a	0	5.4a

Capacity of storage reservoir = maximum deficit + maximum surplus = 5.4a + 16a = 21.4a Where a = Average hourly demand on the maximum day.

The Data of Table 3 is plotted in the form of Mass-Curve in Fig-1

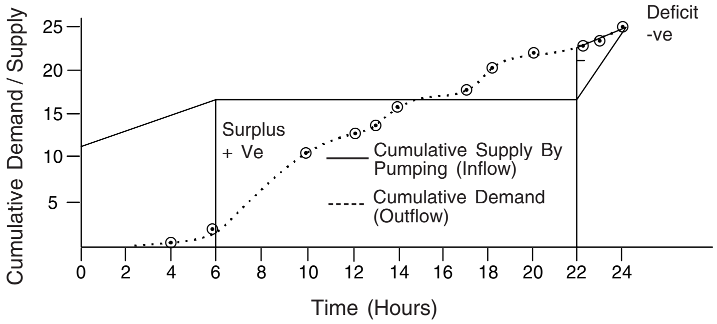


Fig. 1 : Mass Curve for Storage Capacity Calculations

3.1 Storage requirements in buildings : Storage in buildings is required to be provided against interruption of supply caused due to repairs. This also reduces maximum rate of demand on the mains. It also helps in the periods of intermittent supply and maintenance of storage for fire fighting.

The quantity of storage depends on hours of supply, rate & regularity of supply, consequences of exhausting storage, types of sanitary fixtures and fire fighting requirements.

Storage requirements for some of the common types of buildings are given below for general guidance.

1. Dwelling houses = 70 litres per resident
2. Commercial buildings without canteen =35 litres per person
3. Commercial buildings with canteen = 45 litres per person

In buildings where flushing cisterns are installed the water storage requirements depends on number of water closets seats and urinals. The following yardsticks can be used for calculating storage for flushing purpose.

1. Tenements having common conveniences, 900 litres per w.c. seats.
2. Residential premises other than 1 above., 270 litres for one w.c. seat and 180 litres for additional w.c. seat.
3. Workshops - 900 liters per w.c. seats and 180 litres per urinal.

The above data is based on BIS publication SP35:1987 - Handbook on Water supply and Drainage.

It is some times desirable to have a minimum storage of half a day's supply and maximum of one day's supply for overhead tank. The ground level tank, where provided, should have a minimum capacity of 50% of the overhead storage tank.

If the storage capacity of overhead tank is more than 5000 litres it is advantageous to arrange it in a series of tanks so interconnected that each tank can be isolated for cleaning and inspection without interfering with the supply of water. In large storage tank the outlet shall be at the end, opposite the inlets to avoid stagnation of the water. The outlet pipe shall be fixed 50 to 75mm above the bottom of the tank and fitted with the strainer.

4.0 Sources of Water : The demand of water is to be met from a suitable source. This source should satisfy the requirement in terms of quantity as well as quality. The usual sources of supply of water are rivers, canals, lakes and ponds, under ground water (wells) etc. Railway may have its own arrangement for collecting water or may depend on local government resources for supply of full or partial requirement of water.

4.1 Some important features regarding the borewells (tube wells) which is one of the important source at many stations are described below. The details of other sources of water are readily available in various text books.

The important features of bore-wells which normally are a cause of dispute between civil engineering and electrical engineering deparment are as given below:

a) Verticality of bore well
b) Development and yield testing of bore wells.

The verticality of bore-wells is to be checked according to the procedure laid down in IS:2800 (Part II), 1979. The details are briefly described below:

The equipment required for the above purpose is tripod stand with guide pulley, a string of adequate length and strength. A special type plumb is required which will permit passage of water through it. It is normally fabricated using a piece of diameter 6mm less than the inside diameter of the casing pipe.

The arrangement of tripod with string and plumb over the well is shown in Fig. 2.

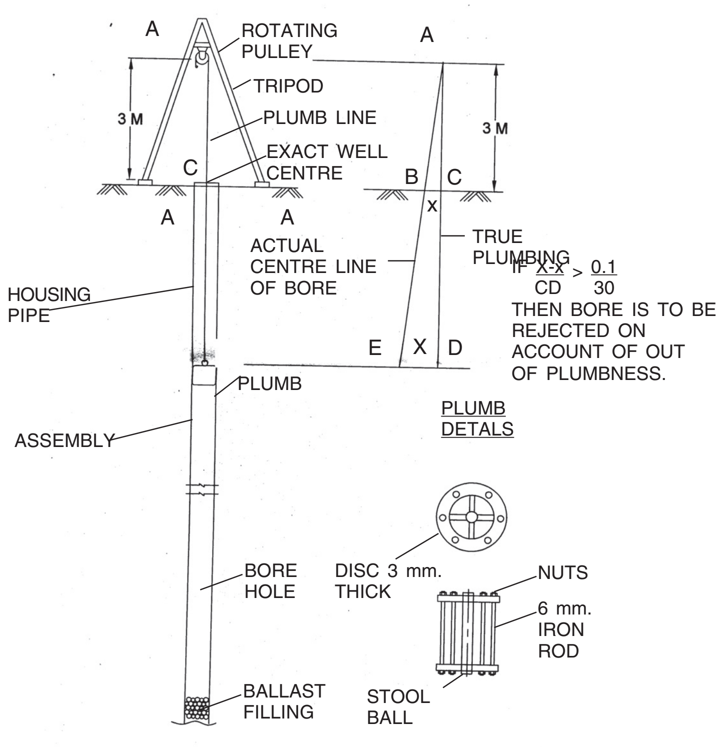


FIG. 2: METHOD OF TESTING PLUMBNESS OF A BORE WELL

The plumb is lowered into the well. The location of plumbline with respect to centre of bore well at ground level is to be recorded for every successive 3m. depth. This way the location of centre of bore hole at various depth is plotted as shown in Fig 3. In this figure the number opposite to each mark indicates the depth below ground level.

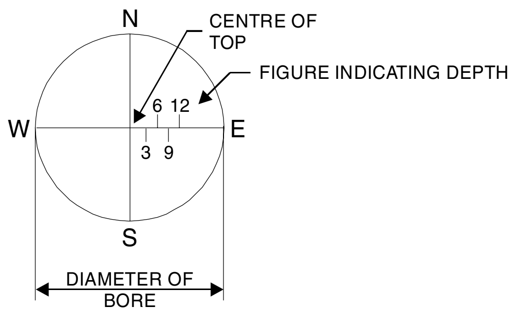


FIG. 3: LOCATION OF CENTRE OF BORE AT VARIOUS DEPTH AS INDICATED AT THE TOP OF BORE WELL

The next step is to plot circles of diameter equal to internal diameter of casing pipe. These circles will be plotted after deriving the actual deviation at respective depths duly calculated based on field data. Say for example in Fig.3, at 9m depth the deviation observed at top of bore well is 5mm, it will mean that actual centre at 9m depth will be (5/3) x (3+9) = 20mm. The direction of this deviation of 20mm will be same as that of observed 5mm deviation at top with reference to centre of bore at ground level. Such points are now located and using each point as a centre, circles of diameter equal to internal diameter of casing pipe are plotted. This way we will get a plot similar to that shown in Fig. 4 below.

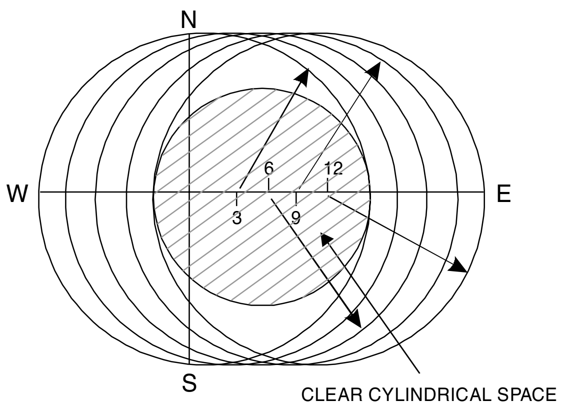


FIG. 4: BORE HOLE POSITIONS AT DIFFERENT DEPTHS OBTAINED DURING PLUMB TEST

The diameter of the circle which fits into the clear inner space of above Fig. 4 is the diameter of the cylindrical space available for insertion of submersible pump and water pipes for carrying water out of well. Such cylindrical space should not be less than the clear cylindrical space available in a hypothetical tube well of the same size but having a deviation of 10 cm. per 30 m in one direction and in one plane only.

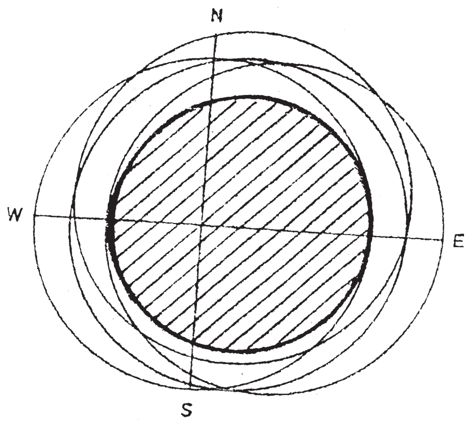


FIG. 5(a) : RESULT OF PLUMB TEST OF
A BORE WELL (ACCEPTABLE TYPE)

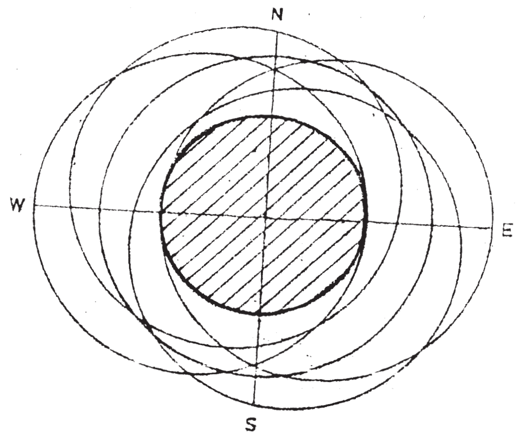


FIG. 5(b) : RESULTS OF PLUMB TEST OF A BORE WELL
(REJECTABLE TYPE)

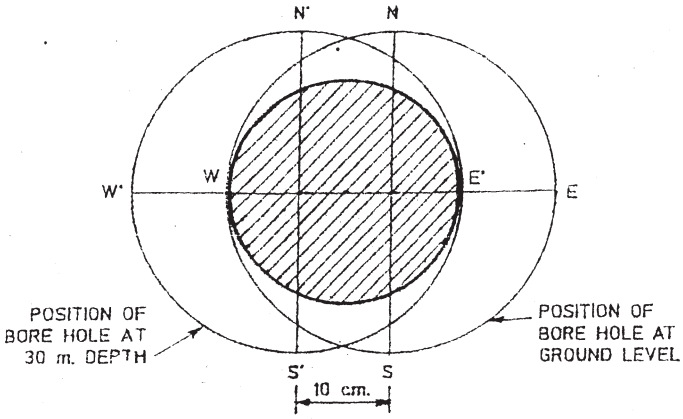


FIG. 5(c) : SKETCH INDICATING PERMISSIBLE INCLINATION FOR BORE WELLS

As an example, the results of plumb test of two borewells are presented in Fig.5 (a) and Fig. 5 (b). These are compared with permissible limits as indicated in Fig.5 (c).

In the above figures, borewell pertaining to Fig. 5(b) shall be rejected, and that of Fig. 5(a) shall be acceptable.

The Criteria as per IS:2800 (Part II) does not check the out of straightness of the bore well. Fig. 6 shows the difference between a straight but inclined and curved bore.

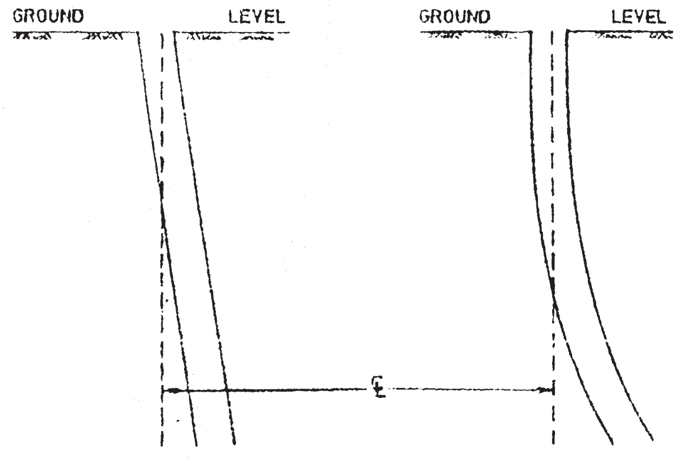


FIG. 6(a) : STRAIGHT BUT INCLINED BORE

FIG. 6(b) : CURVED BORE

In Fig. 6 (a) straight pump can be inserted where as in case of Fig. 6 (b) a long pump can not be installed due to curvature. Hence we have to check the curvature of bore well. To assess the straightness of the boring, the normal practice is to insert a blank pipe of 3m length, whose outer diameter is 12mm less than the internal diameter of well casing. If this blank pipe is able to go to the required depth, then such bore well will be acceptable.

4.2 Water level assessment in bore well : It is essential to know the water level before and after pumping in order to assess the yield of the bore well. The water level can be ascertained as per the methods described in IS:2800 (Part II)-1979. A brief description of these methods is given below :

a) **Electrical Method :** Fig. 7 shows the apparatus used for electrical method. As soon as the two electrodes come in contact with water, an indication will be seen in the voltmeter. The depth of free surface of water can be assessed from the length of the electrical cable inserted in the tube well.

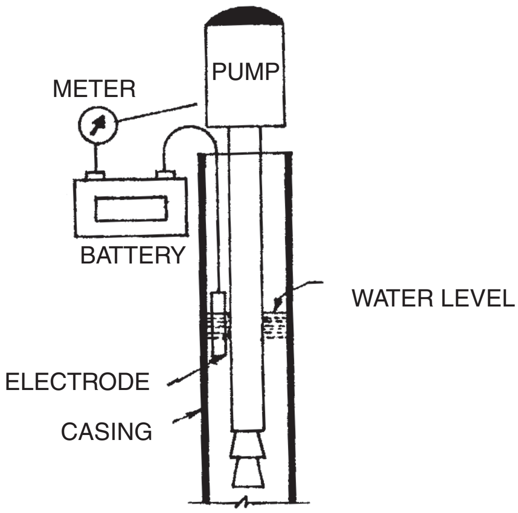


FIG. 7: ELECTRICAL METHOD FOR DETERMINING WATER LEVEL IN BORE HOLE

b) **Air Pressure Line Method :** Fig. 8 shows the apparatus used for air pressure line method. The pressure gauge will indicate the "A" depth. By deducting "A" depth from "B" (length of air line tube) the depth "C" which is the depth of free surface of water can be known.

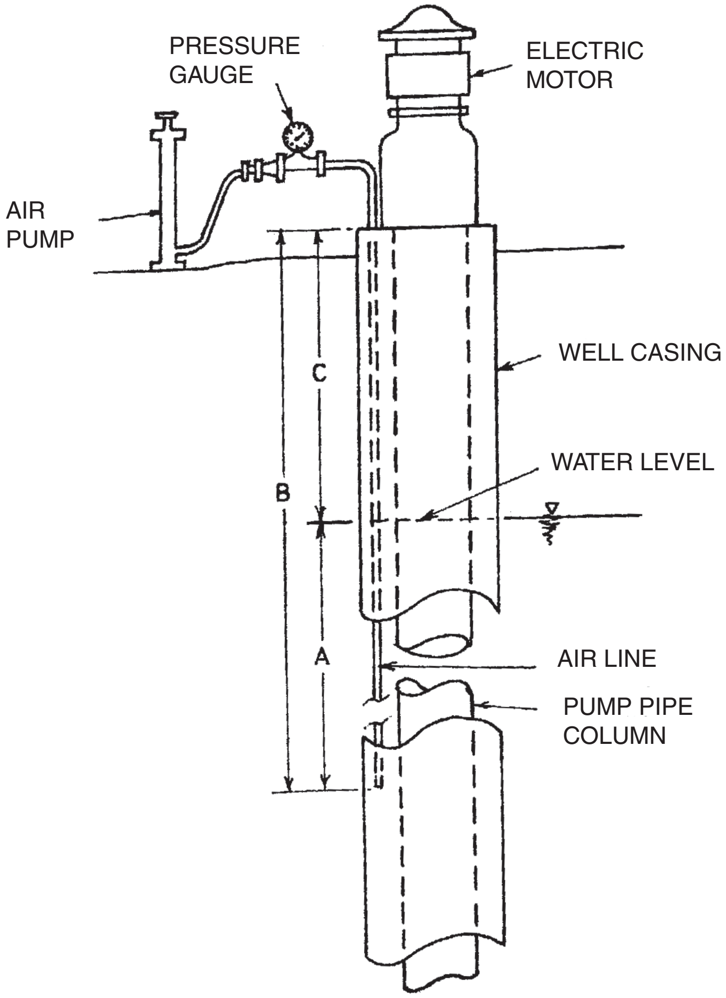


FIG. 8: AIR LINE METHOD FOR DETERMINATION OF WATER LEVEL

4.3 Yield testing of bore wells : A test pump is inserted temporarily in the bore well and water is pumped out by keeping the desired level of draw-down. In case use of a notch (e.g. V-notch, rectangular notch) is made for assessing the rate of flow, it is essential to record the corresponding draw-down of the tube well. Normally draw-down should be about 2 meters. If it exceeds, there are chances that fine sand particles may be drawn into the bore well. Pumping rate, in general, shall not exceed 60 percent of the yield determined by test. Geological advice may be taken in specific cases.

4.4 Development of bore wells : During drilling operation, fine mud particles are generated. These particles will clog the pores of acquifer (water bearing strata) thereby reducing the flow rate of water into the well. One of the most common method used for the development of well is compressed air method. The details of this method as well as other methods are described in IS:11189-1985 (Methods of Tubewell Development).

5.0 Conveyance of water : The laying and maintenance of pipe lines is one of the important job of civil engineers. It is essential to select the pipes of suitable material and diameter. Different material like, mild steel, copper, cast iron, cement concrete, HDPE, PVC, Asbestos cement are used for manufacturing pipes. Out of all these the most commonly used material are cast iron (C.I.) and mild steel (G.I.).

5.1 C.I. Pipes : These pipes are useful for pressure mains and laterals where large quantites of water are to be carried. Due to their strength and corrosion resistance, CI can be used in soils and for waters of slightly aggressive character. Due to their weight, they pose transportation problems.

5.1.1 IS: 1536-1989 Specifications for centrifugally cast (spun) iron pressure pipe for water, gas and sewage

Method of manufacture : Centrifugally Cast (Spun pipes)
Diameter 80 - 1000 mm
Length 3.66 - 6.0m
t (thickness) : 10/12 (7+.02 DN) in mm for LA Class
DN : Nominal diameter of the pipe in mm.

Table - 4
Classiffication of pipes

Classification	LA	A	B	C	D	E
Thickness	t	t+10%of t	t+20%of t	t+30%of t	t+40%of t	t+50%of t
Test pressure after installation for S&S joint	1.2 MPa	1.8 MPa	2.4 MPa			
Flanged joint	Slightly less					
Hydrostatic Test pressure at 3.5 Mpa (35 kgf/cm^2) at factory						

5.1.2 IS:1537-1976. Specification for vertically cast-iron pipe for water, gas and sewage

Method of manufacture : Vertically Cast
Dia : 80 mm - 1500 mm
Length : 3.66 m to 5.5m (S & S)
Test pressure for spigot and socket joints

Table - 5

Diameter	Class A	Class B
For S & S joints upto 600mm	2.0 Mpa (20 kgf/cm^2)	2.5Mpa (25kgf/cm^2)
600-1000mm	1.5 Mpa (15 kgf/cm^2)	2.0Mpa (20kgf/cm^2)
1000-1500mm	1.0 Mpa (10 kgf/cm^2)	1.5Mpa (15kgf/cm^2)

The weight of the spun pipes is about 3/4 of the weight of the vertically cast pipes of the same class.

5.2 Mild steel pipes (galvanised) : Normaly these pipes are available up to100mm diameter. There are three classes of pipes as per IS:1239(part I)-1990. These classes are 'Light' (L), 'Medium' (M), and 'Heavy' (H). However, code provides for up to 150mm diameter pipes in class M & H. For easy identification, the pipes shall be distinguished by colour bands which have to be applied at the ends before the tubes leave the factory.This colour scheme is as follows:

Light tubes: Yellow band
Medium tubes: Blue bands
Heavy tunes: Red bands

For building water supplies, normally Railway specifications recommend the use of medium class pipes(blue band). For all G.I. pipes, the IS code No. IS:1239 (Part I) along with class code 'L' , 'M' and 'H' are engraved on the pipe which should be very clearly visible. Further check should be exercised by checking the weight per unit length and the shell thickness as specified in IS:1239 (Part I) Table 1 to 3. Fittings for these pipes are governed by IS:1239 (Part 2) 1992.

The features of commonly used sizes of medium class of G.I. pipes are given in Table 6.

Table 6
Features of medium class G.I. Pipes

Internal Diameter (mm)	Shell Thickness (mm)	Mass of Tube (Kg/m) with Plain end
15	2.6	1.21
20	2.6	1.56
25	3.2	2.41
40	3.2	3.56
50	3.6	5.03
80	4.0	8.36
100	4.5	12.2

5.3 PVC pipes : Un-plasticised PVC pipes are manufactured according to IS:4985-1988. These pipes are classified into 4 classes based on the pressure viz. Class I for 2.5 kg/cm^2, Class 2 for 4 kg/cm^2, Class 3 for 6 kg/cm^2 and Class 4 for 10 kg/cm^2. These pipes are jointed together using couplers as spigot and socket made specially to form a close fit. Solvent cement is used to make the joint leak-proof.

Pipes with threaded ends able to withstand pressures as high as 40-60 kg/cm^2 are also available in market. These pipes are being manufactured as per American Standards according to manufacture's claims. The threads of these pipes are similar to those of GI pipes.

5.4 Selection of diameter of pipes carrying flow under pressure :

The selection of diameter of pipes carrying flow under pressure can be made using any one of the methods as described below:

5.5 Hazen & William's Formula :

$$V = 0.849\, C\, R^{0.63} \times S^{0.54}$$

Where V = Velocity in meters per second
R = Hydraulic Radius in metres
= Area of Flow / Wetted perimeter
S = Hydraulic gradient (meter per meter)
C = Hazen & William's Coefficient

For Circular pipes, $R = \frac{\pi D^2}{4} \times \frac{1}{\pi D} = \frac{D}{4}$

Where D = Internal Diameter, in mm,

Substitute for value of R by D/4x1000 (to convert D in m), we get

$$V = 4.567 \times 10^{-3} \times C \times D^{0.63} \times S^{0.54}$$

To develop the formula for (m^3/s)

$Q = (4.567 \times 10^{-3} \times C \times D^{0.63} \times S^{0.54}) \times$ Cross- section Area of pipe in (m^2)

$= (4.567 \times 10^{-3} \times C \times D^{0.63} \times S^{0.54}) \times \pi D^2 / 4 \times 1000 \times 1000$

$= 3.587 \times 10^{-9} \times C \times D^{2.63} \times S^{0.54}$

Where Q = Discharge in m^3/s

D = Diameter in mm

S = Hydraulic gradient (m/s)

Table - 7
Recommended Value of Hazen and William's Coefficient 'C'

Conduit Material	Recommended Value of C	
	For New pipe	For Design purpose
Cast Iron	130	100
Galvanized Iron > 50 mm	120	100
Galvanized Iron 50 mm and below (used for house connections)	120	55
Steel, riveted joint	110	95
Welded joints, lined with cement or Bituminous enamel	140	110
Steel, welded joints	140	100
Concrete	150	120
Asbestos Cement	150	120

An example of using Hazen and William's equation is given below to illustrate it's use.

Example : One overhead tank at an elevation of 10m has to supply water to a public toilet's storage tank at an height of 5m. The two tanks are located at a distance of 100 metres. The desired rate of flow is 20 lit/min. Calculate the diameter of GI pipe to be provided.

Scl : The mean Value of 'C' from above table can be taken as 100

$Q = 20$ lit/min $= 3.33 \times 10^{-4}\ m^3/s$.

Hence using Hazen & William's equation.

$3.33x10^{-4} = 3.587 \times 10^{-9} \times 100 \times d^{2.63} \times (10-5)^{0.54}/100$

Solving the above equation

D = 24.86 mm. Say 25mm pipe is to be provided.

4.6 Darcy - Weisbach Equation : This equation is based on the head loss incurred due to friction on the inner walls of the pipe. The head loss is equal to :

$$H_L = fLV^2/2gd$$

Where H_L = Head loss,
f = friction factor,
L = Length of pipe,
g = acceleration due to gravity,
d = internal diameter of pipe.

The Friction factor 'f' depends on Reynold's Number, and relative roughness (e/d) of the pipe, where e is the roughness projection & 'd' is the internal diameter of pipe.

Reynold's Number $R_y = \frac{Vd\rho}{\mu}$ (dimensionless Number)

Where V = Velocity of flow
d = Internal diameter of pipe
ρ = density of water = approx. 1000 kg/m^3 at 20^0 C
μ = dynamic viscosity of water
= 0.001 kg/meter second at 20^0 C.

Having calculated R_y, 'f' can be calculated by using the formula :

$$f = \frac{64}{R_y} \qquad \text{if } R_y = 2100 \text{ to } 4000$$

$$\text{or } f^{-0.5} = 1.14 + 2.0 \log\left(\frac{d}{e}\right)$$

where d = internal diameter of pipes in mm

e = roughness projection, in mm approximately equal to

= 0.0127mm for GI pipes

As an alternative, Moody's diagram can also be used for finding out the value of friction factor 'f'.

The procedure for using Moody's diagram is to find out the value of Reynold's number and e/d. Select the appropriate curve according to e/d and located a point on it corresponding to Reynold's number. Now, the value of friction factor 'f' can be read on the left side 'y' axis.

As an illustration an example is given below. In this example, the head-loss is calculated for pipes of different diameter ranging from 15mm to 40mm. It is seen that frictional head loss decreases with increase in diameter of pipe. This loss is also called "Major loss" as compared to ‘‘Minor loss’’. The later occur on account of fitting like elbow, tee, valves etc.

Example : The Dynamic viscosity of water (μ) at 20^0 C is 0.001 kg/m.s. It is to be pumped at the rate of 20 lpm through a G.I. pipe line. The length of pipe line is 100m. Mass density of water is 1000 kg/m^3. Calculate the head loss through 15, 20, 25, 32, and 40 mm dia pipes. Assume e = 0.0127 mm for G.I. pipes.

Table - 8

Solution:

Dia (mm)	X area (mm^2)	V m/s	$R_y = \frac{Vd\rho}{\mu}$	e/d	f from Moody's Diagram	$H_L = fLV^2/2gd$
15	176	1.89	28350	.00084	0.025	30.34
20	314	1.06	21200	.00063	0.0270	7.73
25	490	0.68	17000	.00050	0.0275	2.59
32	804	0.41	13248	.00039	0.0295	0.805
40	1256	0.26	10600	.00031	0.031	0.277

5.7 Losses due to Fittings and Transition in pipes : These losses are called "Minor losses" as compared to major loss due to friction in pipe line. In long pipelines minor losses may be negligible. Losses in fittings depend on the velocity head.

$$h_L = K\frac{V^2}{2g}$$

Where h_L = minor loss, V = Velocty, g = acceleration due to gravity
K = Resistance coefficient depending upon the type of fittings

The Value of K for elbows varies from 0.15 to 0.60 whereas it varies considerable for valves. Reference may be made to Table 7 of SP:35 (S&T) - 1987 - Handbook on Water Supply and Drainage isued by BIS. The number of bends in pipe lines should be as minimum as possible to avoid loss of head. Exit connections from water tanks should be "bell mouthed" with radius greater than 0.14d where 'd' is diameter of outlet pipe, to avoid excess loss due to sudden contraction. For outlets without bell mouth arrangement, the head loss is increased almost ten times.

5.8 Series and Parallel Pipes : There are instances when pipe lines consist of pipes of varying diameter in continuity. This situation may arise due to non availability of pipes of same diameter of the required length. At the time of replacement of such pipe on account of compulsive conditions, a question arises as to what should be the diameter of new pipe for equivalent flow conditions.

The diameter of new pipe line can be calculated by the use of following equation.

$$\frac{L}{D^5} = \left\{\frac{L_1}{D_1^5} + \frac{L_2}{D_2^5} + \frac{L_3}{D_3^5} + \dots\dots\dots\right\}$$

Where L = Total Length of equivalent pipe
D = diameter of equivalent pipe
L1, L2, L3 etc = Length of individual pipes in series
D_1, D_2, D_3 etc. = Diameters of individual pipes in series. This equation is based on the assumption that the friction factor 'f' as described in Darcy- Weisbach equation in Para 4.6 above, is same for all the pipes under consideration, including the equivalent pipe.

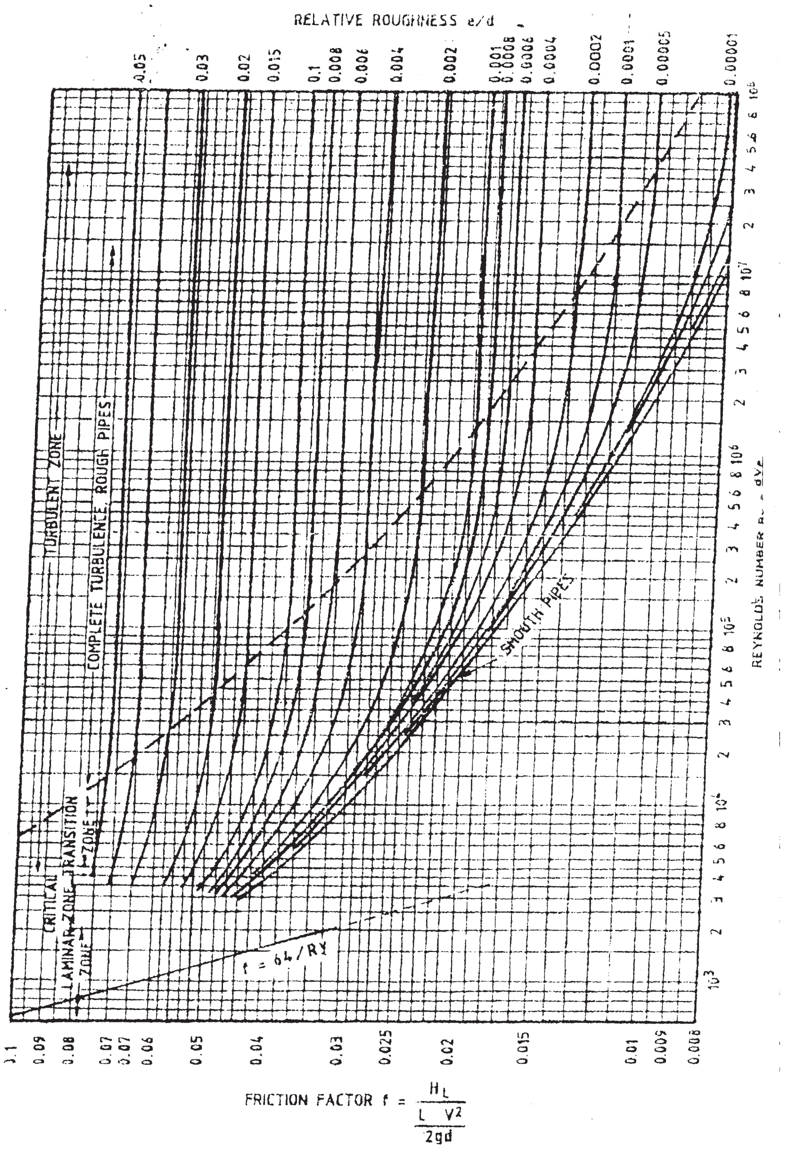


FIG. 9: MOODY'S DIAGRAM

5.9 Branching of Main Line : The flow carried by main line is distributed through branch lines. The number of connections which can be provided from one main is a matter of concern. This situation can be analysed in a simple manner by making following assumptions.

(i) The flow carried by all branches will be equal.

(ii) The material, diameter and length of all branches will be same.

The above assumptions imply that the head loss through all the branches will be equal. The solution to above problem can also be used for calculating the number of service connections of a particular diameter and with above assumptions from a available main line.

The following equation can be used for such solution.

$$D = dn_b^{2/5}$$

Where D =diameter of mainline,

d = diameter of proposed service branches,

n_b = number of branches.

To illustrate above solution, an example is given below :

Example: Calculate the number of 15mm service connections which can be provided from a main pipe line of 50mm.

Solution : Here D = 50mm, d = 15mm

hence using $D = dn_b^{2/5}$, we calculate n_b = 20

Hence 20 such connections can be provided.

6.0 Joints in water pipe lines :- Two common types of joints are used in water pipe lines accordingly to the type and pressure of the pipe lines.

6.1 Threaded joints : For G.I. pipes of small diameter threaded joints are used. The specifications of threads for pipes are governed by IS:554:1985. Depending upon the size of pipe, the number of threads in 25.4mm length, pitch, depth of thread, Gauge length etc. are tabulated. The total threaded length will depend on the dimensions of fittings given in IS:1239 (Part2):1992. Few common sizes of pipes used in Railways are 15mm, 20mm, 25mm, 40mm, 50mm and 100mm. Table 9 - gives the length of threaded portion along with size of threads.

Table 9
Threading details for pipes

Nominal size of pipe (internal) diameter (mm) (1)	Minimum Length of threaded end of pipe (mm) (2)	Number of threads in 25.4 mm length (3)	Minimum number of threads in threaded length given in col (2) (4)	Depth of thread (mm) (5)	Gauge length (mm) (6)
15	19 mm	14	11	1.162	8.2
20	20 mm	14	11	1.162	9.5
25	23 mm	11	10	1.479	10.4
40	26 mm	11	12	1.479	12.7
50	30 mm	11	13	1.479	15.9
100	44 mm	11	19	1.479	25.4

*Gauge length can be taken as length of taper portion of threaded end. This taper is 1 in 16.

6.2 Spigot and socket joint : These joints are used for C.I. pipes. These joints are preferred over flanged joints due to their flexible nature. To make this joint, spigot end of the next pipe is inserted in socket of first pipe. The socket end of pipe is kept against the flow of water. In case of slopes, the socket end shall face upstream.

A twisted spun yarn is filled into the gap and it is adjusted by the yarning tool. It is caulked well. A rope is then placed at the outer end of this socket and is made tight fit by applying wet clay, leaving two holes for the escape of entrapped air inside. The rope is then taken out and molten lead is poured into the annular space by means of a funnel. The clay is then removed and lead is caulked with caulking tool. Lead wool may be used in wet conditions. Quantity of lead to be consumed per joint is specified in Zonal Railway's specifications, according to the diameter of pipe.

7.0 Testing of pipe lines : Leakages in pipelines have to be tested for withstanding pressures likely to be developed under working conditions in accordance with IS:3114:1965.

7.1 Pressure-test : In case of gravity pipes, maximum working pressure shall be two-thirds of the test pressure to which pipes are subjected to in the manufacturer's plant before the lot is released for despatch. The details of test pressure to be adopted at factory for C.I. pipes are given in para 4.1

For G.I. pipes of all the classes, test pressure for testing at manufacturing plant is 50 kg/cm^2. Hence for these pipes the maximum working pressure in field will be 33.33 kg/cm^2.

The test pressure to be imposed in field on the laid pipe line should be $1^1/_2$ times the maximum sustained operating pressure. However, it should not be less than 2/3 of test pressure to which pipe is subjected to in manufacturing plant. This test pressure should be applied and maintained for at least four hours. If the visual inpection satisfies that there is no leakage, the test can be passed.

7.2 Leakage Test : The leakage is defined as the quantity of water to be supplied into the pipe line which is necessary to maintain the specified leakage test pressure after the pipe has been filled with water.

Pipe line installation can be accepted only if the leakage is less than (in cm^3/hr) as determined by the formula given below. (BIS publication SP35:1987 Handbook on Water Supply and Drainage)

$$q_\ell = \frac{ND\sqrt{P}}{3.3}$$

Where q_ℓ = allowable leakage in cm^3/h (=ml/hr)

N = Number of joints in the length of the pipeline

D = Diameter in mm and

P = The average test pressure during the leakage test in kg/cm^2.

8.0 Pipeline networks : The design of distribution systems involves making a preliminary design of the network, analysing the same and modifying it to suit the analysed results. It is an iterative process. Few important aspects which should be kept in mind while designing a network are presented below. It is followed by the design equations and to illustrate the whole process an illustrative example is also given.

8.1 For any network few conditions are to be fulfilled.

* Flow into a junction = flow out of a junction
* The loss of head clockwise direction = the loss of head in anti clockwise direction
* Darcy-Weisbach equation is to be satisfied.

$$H_L = rQ^n = \frac{fL}{2g(\pi/4)^2}\frac{Q^n}{d^5} \quad \text{hence} \quad r = \frac{fL}{2g(\pi/4)^2 d^5}$$

Generally the value of n is taken as 1.72 to 2.0

It is seen from above that the value for 'r' is dependent only on the characteristics of the pipes alone. The characterstics are

Where r = pipe parameter as defined above
f = friction factor for the pipe material
L = length of each branch of the pipe line.
g = acceleration due to gravity
d = diameter of the pipe (internal)

Using the above equations, the method of solving the network analysis by Hardy-Cross method can be used. This method is also called the method of successive approximations. Given a network the first step is to assume a most suitable distribution of flow that satisfies continuity at each junction.

With the assumed valued of Q for each pipe of the network, the value of head loss is computed. In any one circuit net head loss is made zero. The convention of +ve head loss in clockwise direction and -ve head loss in anti-clockwise direction is followed.

If Q_0 is the assumed discharge and ΔQ is the correction in assumed discharge and Q is the correct discharge, then.

$$Q = Q_0 + \Delta Q$$

And the head loss for the pipe is

$$H_L = rQ^n = r(Q_0 + \Delta Q)^n$$

Thus for complete circuit

$$\sum H_L = \Sigma rQ^n = \Sigma r(Q_0 + \Delta Q)^n$$

By expanding the term in the brackets by binomial theorem

$$\Sigma rQ^n = \Sigma r\left[Q_0^n + nQ_0^{n-1}\Delta Q + ...\right]$$

If ΔQ is small compared with Q_0, all terms of the series after the second one may be dropped. Thus.

$$\Sigma rQ^n = \Sigma rQ_0^n + \Sigma rnQ_0^{n-1}\Delta Q$$

For the correct distribution the circuit is balanced and hence

$$\Sigma rQ^n = 0$$

Therefore $$\Sigma rQ^n + \Delta Q \Sigma rnQ_0^{n-1} = 0$$

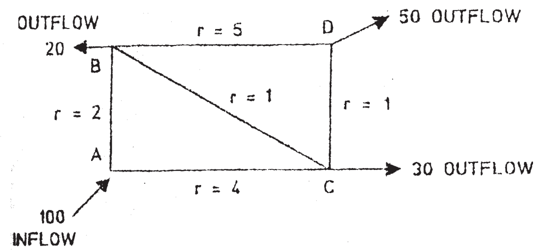
In the above expression ΔQ has been taken out of the summation as it is same for all the pipes in the circuit

Solving for ΔQ

$$\Delta Q = -\frac{\Sigma r Q_0^n}{\Sigma r n Q_0^{n-1}}$$

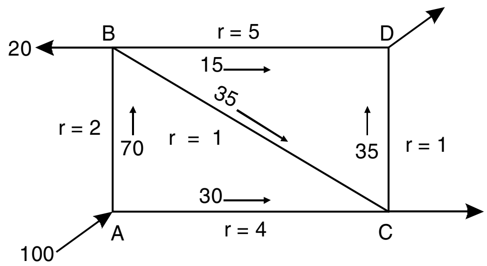
To illustrate the above principles an example is solved below.

Example: For a pipe network shown in the figure, determine the flow in each pipe. the value of n may be assumed as 2.0.



The logic which has been followed for the intial approximation is:
If r is more, it means that d is less and the rate of flow will be less.

1st approximation : for the quantity of flow for various branches in the nework.



For this distribution the correction ΔQ for the loop ABC are computed as follows :

Loop ABC

Pipe	rQ^n	rnQ_0^{n-1}
AB	$2 \times 70^2 = 9800$	2x2x70=280
BC	$1 \times 35^2 = 1225$	1x2x35=70
AC	$-4 \times 30^2 = -3600$	-4x2x30=240
Total	7425	590

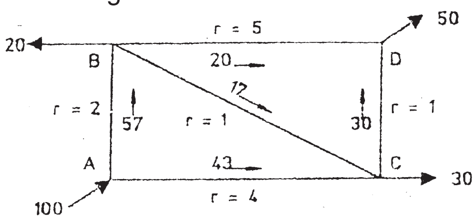
$$\Delta Q = \frac{-7425}{590} = -12.58 \quad \text{say -13}$$

Loop BDC

Pipe	rQ_0^n	rnQ_0^{n-1}
BD DC CB	$5 \times 15^2 = 1125$ $-1 \times 35^2 = -1225$ $-1 \times 35^2 = -1225$	5x2x15 =150 1x2x35 = 70 1x2x35 = 70
Total	-1325	290

$$\Delta Q = \frac{-(-1325)}{290} = 4.56 \quad \text{Say 5}$$

Applying above corrections the modified discharges for the various pipes are shown in diagram below :



2nd Approximation

Loop ABC

Pipe	rQ^n	rnQ_0^{n-1}
AB BC CA	$2 \times 57^2 = 6498$ $1 \times 17^2 = 289$ $-4 \times 43^2 = -7396$	2x2x57 = 228 1x2x17 = 34 4x2x43 = 344
Total	-609	606

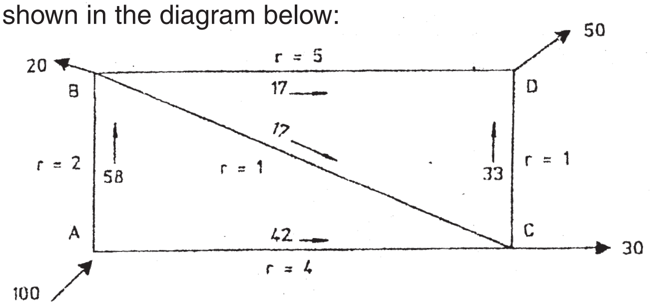
$\Delta Q = -(-609)/606 = 1.015$ Say 1

Loop BCD

Pipe	rQ^n	rnQ^{n-1}
BD	5 x 20^2 = 2000	2x5x29 = 200
DC	-1 x 30^2 = -900	2x1x30 = 60
CB	-1 x 17^2 = -289	2x1x17 = 34
Total	811	294

$$\Delta Q = \frac{-811}{294} = -3$$

Applying above corrections, the modified discharges for the various pipes are shown in the diagram below:



3rd Approximation:

Loop ABC

Pipe	rQ^n	rnQ_0^{n-1}
AB	2 x 58^2 = 6728	2x2x58 = 232
BC	1 x 21^2 = 441	1x2x21 = 42
CA	-4 x 42^2 = -7056	4x2x42 = 336
Total	113	610

ΔQ = 133/610 = 0.185 which is negligible

Loop BCD

Pipe	rQ^n	rnQ^{n-1}
BD	5 x 17^2 = 1445	2x5x17 = 170
DC	-1 x 33^2 = -1089	2x1x33 = 66
CB	-1 x 21^2 = -441	2x1x17 = 42
Total	-85	278

ΔQ = -(-85)/278 = 0.305 which is negligible

Hence, the values as adjusted for third iteration can be taken as final.

9.0 Valves : Valves are designed to serve as regulators of the flow. There are special purpose for effecting scour, air esape, non-return of flow etc. Brief description & uitilities of some important commonly used valves are given below.

9.1 Line Valves : These valves are fixed at a suitable location in the pipe line to regulate the flow. These valves are designed for uni-directional flow but under pressure reversal, flow can take place in wrong direction also.

Depending on the type of obstruction these are classified as Gate valve, Globe valve, Plug valve etc. In gate valve, a plate like obstruction moves up/down to regulate the opening which in turn controls the rate of flow. Contrary to this Globe valve consists of an spherical obstruction which in closed position sits over a horizontal opening and by varying the gap between the bottom of sphere and top of opening, the flow is regulated.

Gate valves are preferred in large diameter pipes. Most of the Gate valves will operate properly only when installed in a vertical position. Globe valves are having poor hydraulic characteristics as the loss of head through them is more compared to Gate valves. The primary application of globe valves is in domestic connections. These valves can be installed both in horizontal and vertical directions. It is to be remembered that pipe line can not be fully drained when Globe valve is in position.

Plug valves consists of a socket in the shape of a frustum of a cone. This socket has openings in the direction of flow. An obstruction in the shape of closely fitting tapered plug moves about a vertical axis inside the socket. This plug also has slot cut through it along horizontal axis. By rotation of plug, the matching of hole in socket & plug is controlled which, in turn, regulates the flow. Such valve offers negligible resistance to flow & by only half rotation full regulation is achieved & hence it is less time consuming to operate this valve.

9.2 Check Valves, Reflux Valve, or Non Return Valves : These valves open only in the direction of flow and automatically prevent reversal of flow in a pipe line. These are particularly useful in pumping mains when positioned near pumping stations to prevent back flow when pumps shut down. These are also provided on the rising mains at the foot of long upward inclines. These are placed at intervals (say 300m) in long pumping main to prevent back pressure on the engine. Check valves installed at the end of a suction line are called as "Foot Valves".

9.3 Air Valves or Air Relief Valves : These are fixed on trunk and secondary mains at the highest points, on undulating water mains, on long streches of nearly level mains, and at summits of all changes of gradient. Their function is to allow the accumulated air to escape when pipe is filled and to permit the air to enter when the pipe is emptied. Air and vacuum relief valves are essential on large diameter steel pipes which would collapse if subjected to a vacuum on the inside.

Air valves consist of a floating ball in a chamber which allows the air to escape and closes as soon as the air is expelled under pressure of water. Single air valves are usually provided on mains of 75mm diameter and double air valves as as mains of 100mm diameter and above. The usual sizes of air valves as compared to main diameter are as under:

Table - 10
Suitability of Air Valves

Pipe size	Size of Air valves
upto 100 mm	40 mm
100 to 200 mm	50 mm
250 to 300 mm	80 mm
400 to 500 mm	100 mm
600 mm	150 mm

Air valves are usually bolted on to a standard flanged tee. The common practice in respect of placing of air valves on long water mains while ascending or descending is at 400m to 800m intervals. The air valve is fixed in a suitable masonry pit for protection and a weep hole is provided for the escape of water which may pass air valve with escaping air.

9.4 Scour Valves or Blow off valves : These are small Gate valves in pressure conduits, which are provided at the bottom of all depressions and dead ends to drain out the waste water or sediment collected. The exact location of scour valve is frequently influenced by appurtenances to dispose off the water. Scour valves are usually 80 to 150mm diameter leading from the main from a flanged tee branch to a ditch with sluice valve control. A combined stop cock and scour valve is frequently used for building. Frequency of operation depends upon the quality of water carried, especially as slit load.

9.5 Pressure Relief or Safety Valves : These are fixed at the down stream ends of long length of mains or where water hammer is likely to occur, to relieve excessive pressure. These are automatic valves that close when pressure becomes excessive on downstream side. They are heavily weighted spring controlled valves which open under pressure exceeding those for which they are set.

They are best connected to the mains with flanged joints so that they can be easily removed, repaired and re-inserted without disturbing the rest of the pipe line.

10.0 Pumps : Pumping of water is required to overcome gravitational force, frictional force and to enhance the rate of flow at the discharge end such as in case of fire fighting pumps. The two popular classifications of pumps are displacement pumps and centrifugal pumps.

10.1 Displacement Pumps : These pumps work in reciprocating mode. These pumps are of two types, i.e. reciprocating and diaphram pumps. The reciprocating pumps are good on high end head (high pressure) duties.

10.2 Centrigugal Pumps : These pumps contain rotational impeller which imparts high velocity to water so that it can overcome gravitational and frictional forces. These pumps are further divided into two main types.

(i) Priming type

(ii) Submersible type

Priming type of pumps are installed above the water surface of the source. There is a suction pipe between the source and pump. Priming is required to be done to force the air out of the suction pump. The distance between the impeller to water surface is called suction lift. Normally, suction lift is limited to 7 meters. The depth from which water may be raised by an ordinary suction pump is limited to the ability of the pressure of the atmosphere to support a column of water in a vacuum. When the barometer stand at 762mm of mercury, the height of water column will be 762x13.6 (specific gravity of mercury) = 10.36m. In this allowance must be made for (a) variations in atmospheric pressure,(b) lowering of water level in the well or pump where from it has to be lifted and (c) efficiency of the pump. A total allowance of at least 2.44m must be made for these. Thus 7.92 m is the maximum height at which a pump can be expected to work satisfactorily at all times. However, the suction lift of an average pump is limited to a maximum of 4.5m especially with centrifugal pumps.

The horse power of a pump can be calculated as shown below :

$$W = \frac{L_g p\ g(H_T+h)}{60}$$

$$= \frac{L_g p\ x9.81(H_T+h)}{60}$$

$$W = 0.1635L_g(H_T+h)\text{watts}$$

$$= \frac{L_g(H_T+h)}{4562} \text{ (in horse power)}$$

Where L_g = discharge in litres per minute

p = density of water (1000 kg/m^3)

g = acceleration due to gravity (9.81 m/s^2)

H_T = Total head including discharge head and suction head (in metres)

h = Head loss in metres

W = Horse power of pump

The above formula does not account for the working efficiency of the pumps. While deciding the provision of pumps based on any above equation the efficiency of the type of pumps in question may be taken into account.

The total head is from the surface of water in the well after draw-down, upto the point of delivery of water. It should be noted that in tube wells, the draw-down should be normally restricted to 2.0 m to avoid fine sand being drawn along with the flow of water into the well. Further while deciding the capacity of pump, it should be borne in mind that the pumping rate shall, in general, not exceed the 60 percent of the yield of well determined by the test.

The hours of pumping have bearing on the storage capacity of tanks. This storage capacity will increase with the decrease in pumping hours.

10.3 Hand Pumps : Hand pumps are used for water supply at isolated locations such as small colonies, level crossings, small stations where the requirement of water is less and it is not economically feasible to provide water supply, using electrical pumps or through long lead pipe lines from neighbouring areas. At some places the hand pumps may be provided as standby in case of failure of electrical supply or major breakdowns of the existing water supply arrangements.

10.31 Shallow Well Hand Pumps : These are manufactured as per IS: 8035 -1976, "Specification for shallow well Hand pumps" and are suitable for lifting water from the shallow wells of depth not exceeding 8.0 meters. The pumps are designated according to the nominal internal diameter of the bore of the pump cylinder. These standard sizes are 65mm, 75mm and 90mm. The discharge of water, collected out of twenty strokes shall be 6.8, 11.3 and 17.0 litres respectively for above sizes. These quantities of discharge are for static suction lift of 1.5m and to be measured after the pump body is completely filled with water.

The body of this type of hand pump is made of cast iron and piston rod is made of mild steel.

10.32 Deep Well Hand Pumps : These hand pumps are to be manufactured in accordance with IS: 9301 -1990 "Deepwell Hand Pumps-

Specifications "(Third revision)". This type of deepwell hand pump is also popularly known as "India Mark II hand pump". These hand pumps are usefull for lifting water from wells of depth 20m upto 50m as below this depth, the efforts put in may not give adequate results in terms of discharge of water.

The nominal bore size of G.I. Pipe conforming to IS: 1239 part(I) medium quality size for this pump is 150mm. The riser pipe conforming to IS 1239 (Part I) is 32mm. This riser pipe will be screwed and socketed in 3m length.

While testing, the discharge of water out of 40 continuous strokes in one minute shall not be less than 15.0 litres. For the purpose of discharge test, the pump is fixed in a barrel of 200 litres and is fully primed.

11.0 Purification of Raw water : The water to be supplied for human consumption should be free from impurities. The standards of water quality have been laid down by Bureau of Indian Standards in IS:10500. The raw water from the source may need treatment to conform to these standards.

The step wise procedure for water purification is shown in following flow diagram : (Fig. 10)

Raw Water → Screening → Sedimentation with coagulation → Filtration
↓
Disinfection
↓
Storage and Distribution

FIG. 10 : FLOW DIAGRAM

11.1 Screening : Screens are used at surface water intakes to prevent the entrance of leaves, twigs etc. The opening of screens is generally 6mm.

11.2 Sedimentation : The removal of suspended particals may be effected by sedimentation in a basin. As the water enters this basin, the flow velocity gets reduced. Due to gravitational pull, the suspended particles move towards bottom of this basin. The floor of the basin is given a slope towards the far end. The basin is designed in such a manner that a particle entering at the inlet settles vertically and gets trapped inside before reaching the outlet zone.

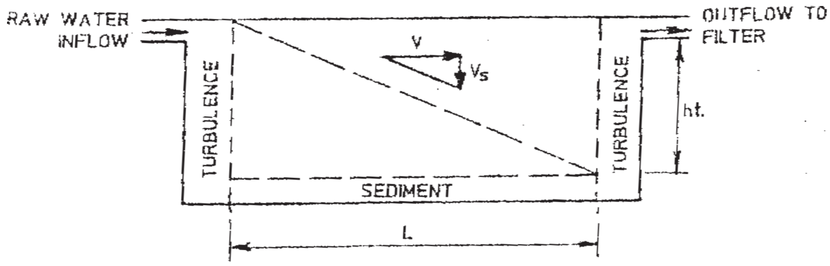


FIG. 11: SEDIMENTATION TANK

If Q is the rate of flow, then, the horizontal velocity is Q/A where x-sectional area (A) is width (w) x height (h_t) of sedimentation chamber.

$$\text{Hence } V = Q/A = Q/Wh_t$$

If the settling velocity is V_s, then for a particle to settle to bottom of basin, it will take time = h_t/V_s seconds. During the same time, the particle travels a longitudinal distance of L_t.

$$\text{Hence} \quad \frac{h_t}{V_s}=\frac{L_t}{V} \qquad \text{Therefore,} \quad V_s=\frac{Vh_t}{L_t}=\frac{Q}{WL_t}$$

which is also known as surface overflow rate. The value of V_s can be obtained by using Strokes Law :

$$V_s=\frac{g}{h_t}(\rho_s-\rho)d_\rho^2 \text{ for Reynold's Number} < 0.5$$

Where g = acceleration due to gravity

ρ_s = mass density of particles.

ρ = mass density of water

h_t = Depth of water above sediment

L_t = Length of tank excluding turbulence zone

d_ρ = diameter of particle

μ = dynamic viscosity of water.

Removal of very fine clay particles (colloidal particles) etc. may take very long time to settle because their settling velocity is too less. When some chemical like alum is added to water and thoroughly mixed, smaller particles combine together and form bigger woolly masses which settle fast. The chemical technique of destabilization of electrical charge around small particles is known as coagulation. The slow mixing technique which promotes the agglomeration of these stabilized particles is called flocculation. The chemical used commonly for coagulation is Alum $Al_2(So_4)_3$.

The optimum PH range of water for using Alum is 4.0 to 7.0. The dosage of Alum is to be determined experimentally using flocculator. It may vary from 0.03 to 0.13gm/litre.

11.3 Filtration : The water coming after sedimentation is passed through filter media. There are two common types of filtration beds, namely slow sand filter and rapid sand filter. For very small works pressure filters may also be used.

11.3.1 Slow Sand Filters : These are conventional beds of fine and less carefully graded sand. Effective Size(D_{10})* of 0.2 to 0.3mm and Uniformity Coefficient ** of 2 to 3 are commonly used. Thickness of sand bed may vary from 1.0 m to 0.75m. The minimum satisfactory depth is 0.5m. The maximum loss of head is 60cm. When this head is reached the filter is taken up for cleaning. A normal period of operation between cleaning may be about 6 weeks with the turbidity of raw water not exceeding about 30 JTU (Jackson Turbidity Unit). For cleaning slow sand filter, top layer of sand bed is scrapped. The depth of removal at a time may vary from 20 to 30mm. The layer of sand is supported by a layer of gravel about 0.3m thick which is graded from an effective size of about 5mm at the top to 50mm at the bottom. The filtration rate of slow sand filters is approximately 0.1 to 0.15 $m^3/m^2/hr$. The purification achieved is 99%.

11.3.2 Rapid Sand Filter : Rapid sand filter also consists of layers of sand. Compared to slow sand filter, the sand used in this filter is coarser, hence the flow rate of water is higher. Normally the filtration rate of water through Rapid Sand Filter is 4.8 to 6 $m^3/m^2/hr$. The sand used in rapid sand filter should be free from dirt, be hard and resistant to abrasion and preferably be quartz or quarzite. It should not lose more than 5% by weight after 24 hours immersion in 40 percent hydrochloric acid. Sand with an effective size* of 0.45 to 0.70 mm and a Uniformity Coefficient** from 1.3 to 1.7 is used in Rapid sand filter. The depth of sand layer is 0.60-0.75m. This sand is supported on a gravel layer of depth 0.45m.

* The effective size is the sieve size in millimeters which permits 10 percent of the sand by weight to pass through it.

** Uniformity Coefficient is D_{60}/D_{10}

The effective size of gravel is 2 to 5mm at top and 50mm at bottom. Minimum of 2 units should be provided at any water works so that at least one unit will be available for filteration when other unit is under repair or cleaning operation.

The cleaning of rapid sand filter is effected by reversal of flow. First air is passed at the rate of 600-900 lpm/m^2 at the pressure of 0.35 kg/cm^2 for 5 minutes in the reverse direction. After this water is passed which removes the dirt by bubbling action. The dirty back wash water flows out by a system of drains. The quantity of back wash water is about 2% of the total filtered quantity. The turbidity of effiuent water from Rapid sand filter should not exceed 1 JTU. The loss of head in a clean filter should not exceed 0.15m. The purification achieved trough Rapid sand filter is 90%.

11.4 Disinfection of water : This is the process of destroying organic matter and bacteria from the water before it is consumed. Among the most common agents of disinfection are chlorine, ozone and ultraviolet irradiation. Later two are mostly used in package units marketted by various companies. For water supply plants, chlorination is the generally adopted process, due to its cost effectiveness.

11.5 Principles of chlorination : The water to be chlorinated should be free from turbidity. The chlorine demand of water is to be estimated. This chlorine demand of water is the difference between the amount of chlorine added to water and the amount of residual chlorine remaining at the end of a specific period at contact (usually 60 minutes) at a given temperature and pH value of the water. It is the amount of chlorine that is needed to destroy bacteria and to oxidise all the organic matter and ammoniacal substances present in the water. The point at which the chlorine demand of the water sample is met is called the "break-point". If further chlorine is added beyond the break point, free chlorine in the form of HOCL (Hypochlorous Acid) and OCI (Hypochlorite ion) begin to appear in the water.

Both hypochlorous acid and hypochlorite ion are disinfectants, but hypochlorous acid is more effective. The total molecular chlorine in hypochlorous acid and hypochlorite ion in water is called free available chlorine. The presence at free residual chlorine for a contact period of at least one hour is essential to kill bacteria and viruses. This means that water should be consumed only after a lapse of minimum one hour after the chlorination treatment.

The minimum recommended concentration of free chlorine is 0.5 mg/litre, at the treatment plant. The free residual chlorine provides a margin of safety against subsequent microbial contamination such as may occur during storage and distribution. The sum of the chlorine demand of the specific water sample plus the free residual chlorine of 0.5 mg/l constitutes the correct dose of chlorine to be applied.

11.5.2. Method of chlorination : Chlorine is applied to water as chlorine gas or chloramine for large scale chlorination. Chlorine gas is cheap, quick, efficient and easy to apply. Chlorinating equipment is necessary for injecting chlorine into water.

Chloramines are loose compounds of chlorine and ammonia. They give a more persistent type of residual chlorine. Their reaction is slower. They are suitable for long lead of treated water and for long hours of storage.

Disinfection of water on small scale can be done by using bleaching powder, ($CaOCl_2$) which is white amorphous powder with a pungent smell of chlorine. It is an unstable compound. On exposure to air, light and moisture, it rapidly loses its chlorine content. hence it should be stored in dark, cool, dry place in a closed container that is resistant to corrosion. The chlorine content of Bleaching powder stocks should be frequently checked. Roughly 2.5 grams of good quality bleaching powder would be required to disinfect 1000 litres of water. This will give an approximate dose of 0.7mg of applied chlorine per litre of water. When freshly made, bleaching powder contains about 33 percent of "available chlorine".

List of abbreviations used

C - Hazen & William's Coefficient

C.I. - Cast Iron

d_p - Diameter of particle

D_d - Internal diameter of pipe

D_{10}, D_{30}, D_{60} - Size nomenclature of sand where Dx is size below which x% of contents will lie

e - Roughness projection

f - Friction factor

g - Acceleration due to gravity

G.I. - Galvanised iron

h_L - Minor loss

h_t - Depth of water above sediment

H_L - Head loss

H_r - Total head including discharge head & suction head (in meter)

K - Resistance Coefficient

L - Length of pipe

L_q - Discharge in litres per minute

L_t - Length of tank excluding turbulance zone

n_b - Number of branches

N - Number of joints

P - Average test pressure

q_3 - Allowable leakage (in cm^3/hr.)

Q - Rate of discharge

r - Pipe parameter

R - Hydraulic radius

R_y - Reynold's Number

S - Hydraulic gradient

V - Velocity of flow

V_s - Settling velocity

w - Width of water tank

W - Horse power of pump

ρ - Density of water

μ - Dynamic viscosity of water