LIFT IRRIGATION METHODS AND PRACTICES

ACTION FOR FOOD PRODUCTION

AFPRO

25/1A, INSTITUTIONAL AREA, PANKHA ROAD
D BLOCK, JANAKPURI, NEW DELHI - 110 058

LIFT IRRIGATION
METHODS AND PRACTICES

BY
Dr. M. K. MAITRA
Head, WRD Dept.

ACTION FOR FOOD PRODUCTION

25/1A, INSTITUTIONAL AREA, PANKHA ROAD
D BLOCK, JANAKPURI, NEW DELHI - 110 058

Published by : Action For Food Production, 25/1A, Institutional Area
Pankha Road, D-Block, Janak Puri, New Delhi- 110058.
Printed by : Sunstar Enterprises, New Delhi. Tel. : 5555708.

Foreward

Water Resource Development is an important area of activity in AFPRO. Over the years AFPRO has extended its services in developing minor irrigation projects for the benefit of poor and marginal farmers. Though the technical knowledge pertaining to Lift Irrigation is available to a large number of qualified engineers, the services of these engineers are not readily available to grass roots Non-Governmental voluntaray agencies (NGVOs) involved in the rural development in general and rural poor in particular. As a result these voluntary agencies are forced to use whatever little expertise is available with them in order to complete the projects rendering at time the schemes either ineffective or uneconomical .

AFPRO believes that production and dissemination of technical information written in an easy language to understand can go a long way in the transfer of technology to a large number of groups. It is infact to fulfil this mission, AFPRO produces technical literature from time to time for the use of grass roots voluntary agencies.

I am sure this manual compiled by Dr. M.K. Maitra, Head WRD Department, would be immensely helpful to the technical staff members of NGVOs working in the field of agricultural and rural development. The manual would also be helpful as a reference book to development and financial institutions in making necessary decisions.

We are very happy to bring this document out as an AFPRO publication which is to be made widely available to grass-roots NGVOs.

Raymond Myles
Executive Director
AFPRO, New Delhi.

CONTENTS

LIST OF FIGURES

CHAPTER 1

LIFT IRRIGATION

1.1 DEFINITION

Irrigation is the science of applying water to soil by artificial means for sustenance of plant growth. While Lift Irrigation would simply mean lifting of water for irrigation, the term Lift Irrigation Scheme (LIS) in the present context would mean a scheme in which an appreciable quantity of water would be raised by means of a pump (lifting device) from a dependable source of water to an appropriate height for irrigation of an area located at a level higher than that of the water source.

In conventional Lift Irrigation practice, water is released at an elevated location so that irrigation of the desired area is performed under gravity flow through open channels or buried pipelines. In sprinkler and drip irrigation systems however, water is applied by means of a pump directly to the crops.

Both the surface and groundwater reservoirs are used as the dependable source of water for irrigation. While the common surface water sources are rivers, streams, canals and tanks, groundwater resource through open dug wells,tube wells and dug-cum borewells is also used as a dependable source under favourable hydrogeological conditions.

LIS can be classified by a number of criteria based on its source of water, source of power, size of the command area, type of energy consumed, type of installation and management etc. In practice, however, several of these criteria are grouped together to describe a scheme more comprehensively. Following terms enumerate some such examples:

DST :	(Dieselised Shallow Tubewell)
DDW :	(Dieselised Dug Well)
DDT :	(Dieselised Dep Tubewell)
MDP :	(Mobile Diesel Pump)
EST :	(Electrified Shallow Tubewell)
EDT :	(Electrified Deep Tubewell)
ERL :	(Electrified River Lift)
DRL :	(Dieselised River Lift)
RLI :	(River Lift Irrigation)
CLI :	(Canal Lift Irrigation)

1.2 FEATURES OF LIS

- LIS allows irrigation of a specified area as per availability of water.

- Irrigation can be provided in any given area as per the choice of the user.

- The nearness of the source to the command area helps in reducing conveyance losses thus increasing the "Duty" of water.

- Irrigation of an elevated area is made possible which can not be served otherwise by any normal flow irrigation .

- Controlled application of water helps in preventing problems related to waterlogging and drainage.

- Decisions relating to investment, gestation period, extent of command area, water sharing, cropping pattern, operation and maintenance of the scheme etc.can be made commensurate to the capabilities of the users of the scheme.

- LIS as compared to canal irrigation enables greater use of local knowledge, skill and other resources thus developing self help attitude.

- Scheme based on dependable groundwater as source is effective even during the periods of drought.

- While the controlled extraction of groundwater enhances the recharge intake into the groundwater reservoir, over extraction leads to falling water table which in turn increases pumping cost.

- LIS depends entirely upon the timely supply of energy necessary for operating the pumps.

- Since water is obtained at a considerable cost, the return from the crops needs to be judiciously matched with the cost of water.

- Proper management of a LIS is of utmost importance, lack of which would lead to financial losses and eventual failure of the scheme.

1.3 PLANNING

Planning involves making rational choices amongst feasible courses of investments and other development possibilities on the basis of socio-economic considerations. In agriculture water is the most critical input. Lack of timely and adequate water supply not only retards crop growth but also renders use of other inputs ineffective.

Planning for LIS therefore necessarily demands matching of the available water with crop production in the overall development perspective (Macro-economics). Reliable data on the irrigation potential developed so far in India through LIS alone is not available separately. The trends in development of irrigation potential in our country is however clear.

In 1950 at the beginning of the first 5-year plan the total area irrigated was 26 million hectare (mha) which by the end of 1979 had risen to 52.2 mha. Out of this area, 27.3 mha are covered under Minor Irrigation Schemes. A large percentage of minor irrigation is achieved by pumping of water from wells, tranks, canals and rivers.

A study of the power consumption pattern can also indicate the trend in the growth rate of LIS. It has been estimated that electric power consumption during 1973-74 was 4242 million kilowatt hour (mkw-hr) which is expected to increase to 14204 mkw-hr by the year 1988-89. The diesel consumption during the corresponding period was expected to rise from 559 to 1116 million litres. In order to meet the ever increasing demands of irrigation, progressively higher investment are likely to be incurred in the subsequent years. Large amqunt of investments are called for not only in the installation of lifting and distribution systems but also meeting the operating costs mostly in terms of energy consumption.

A study conducted by Agricultural Refinace and Development Corporation (1980) in some selected states revealed startling figures of faulty selection and installation of small pumping sets by individual farmers. Lack of adequate knowledge on the use of pumping machinaries will not only affect the end user adversely but will also lead to the erosion of the scarce national resource.

1.4 FEASIBILITY STUDY

While studying the feasibility of a proposal for LIS, priority is naturally attached to those proposals where the source of water is perennial. In case the source of water is seasonal, study should be made to ascertain if the source can support irrigation for one or two full agricultural seasons.

Kharif crops normally meet their major water requirements from rainfall. Supportive irigation from a LIS during Kharif season is required only when the rainfall is not adequate or improperly distributed. It is therefore customary to plan a scheme specifically for the Rabi season which automatically ensures that the scheme will cover a larger area for Kharif crops. The coverage for summer crops, is however likely to be the least.

Feasibility of community LIS depends upon the study of a series of interralated parameters. In practice however many of the required data may not be available readily. Since, for a small and isolated scheme much time may not be available for the collection of detail field data, information available from the adjoining area and feed back received from the local inhabitants may be used.

When interpreted properly by experienced personnel ... information are likely to provide a working data base necessary for the feasibility study . It is however, essential that during a feasibility study the folllowing parameters are looked into with greater interest.

1.4.1 Dependability Of Source

It is obvious that the source of water, whether surface water or ground water, should be dependable. The first step is to note whether the source is perennial or seasonal. The minimum available quantity of water during the lean period for perennial sources should be ascertain. Similarly, for seasonal sources, both the period upto which water is avaliable as well as the quantity during that period should be established. Attention should also be directed to ascertain, if the water has any adverse quality.

Special care is called for situations where industrial effluents are discharged into the upstream area of a surface water source. The extent of saline water contamination of the sources in the coastal region, if any, should also be noted with care. Source of water prone to quality deterioration needs regular monitoring even after the commissioning of a scheme.

1.4.2 Location of Command Area

Relative elevation of the command area and distance of the delivery point from the source are the two important factors in deciding the cost of a scheme. Initial capital cost for a scheme with too high a lift even though may be considered acceptable but the high operating cost due to the use of higher horse power may completely upset the economical viability. Similarly, use of too long a pipeline will not only increase the initial cost but also will result in a higher operating cost due to increased frictional head loss.

From technical feasibility point of view, however, there is no definite cut-off values for maximum permissible lift or maximum permissible distance. Viability of such schemes are to be decided either on the basis of a unit cost norm or socio-economical considerations as the case may be.

1.4.3 Shape of Command Area

Topography and shape of the command area plays an important role in determining design and distribution layout. In most design, water is raised to the highest location of the command area by a "raising main". Water is then distributed to the other parts of the command area by gravity flow. Existing land slope therefore, not only influences the method of irrigation and design of distribution system but also the extent of irrigable command area and cost of irrigation per unit area. The topography of the command area however does not assume so much importance in the case of application through sprinkler and drip system.

1.4.4 Soil Condition

A study of soil not only helps in the choice of crops but also in deciding the extent of farm area. Soils likely to cause high water loss in transmission can bring about a vast difference between planned and actual command area. A preliminary inspection of soil is important for knowing its general characteristics. Attempts should be made to categorize the thickness, texture and water holding capacity of the soil.

Information about soil fertility can be gathered by queries made to experinced local farmers. Details of soil quality, however should be collected in the course of time by chemical analysis of the properly collected samples.

1.4.5 Agro-Climatological Condition

Collection of reliable agro-climatological data is of utmost importance. Detail Information about rainfall and temperature variation should be collected preferably on monthly basis for at least 10 previous years. Areas where these data are not available from local sources data published by Indian metereological department for the nearest observatory could be used.

Analysis of rainfall data helps in establishing monsoon reliability, extent of precipitation, run-off etc. Analysis of hydrometeorological data is also used in estimating factors like effective rainfall and potential evapo-transpiration etc. needed for computing water requirement of crops.

1.4.6 Cropping Pattern

Introduction of irrigation should necessarily bring about certain changes in the existing agricultural practices. It is therefore necessary to study if the farmers can adopt easily to such changes like modification in cropping pattern, increase in input application, increase in production etc. Care should normally be taken due to socio-cultural considerations not to disturb the established Kharif crop.

New Rabi and summer crops may be introduced as per availability of water. Suitable cropping system should be introduced in areas where some Rabi and summer crops are already being grown from the existing but poorer source of irrigation. Study of existing and proposed cropping pattern also aids in the benefit cost analysis of the scheme.

1.4.7 Commitment of the Source

Care should be taken to study if the proposed scheme, from source point of view, will affect any existing scheme in the area. Number and type of schemes operating both in the up and down stream sides of a river, canal, stream etc. should be noted so that the existing pattern of water sharing is not disturbed inadvertantly.

Design of the scheme should therefore be based on actual available quantity of water. For well lift schemes, safe distance between two wells should be such that pumping from one does not lower the water level adversely in the other i.e. a proper well spacing criteria is followed.

1.4.8 Water Table and Drainage

It is important to carry out preliminary investigation about the hydrogeological condition of the command area by conducting inventory of wells. If the groundwater available is adequate to meet the water requirement of the crops, there would be no need to transport water from elsewhere. Groundwater potential of a command area under a LIS is likely to get augmented due to the recharge from the return irrigation. Supplementory irrigation from wells in the command area of a LIS not only helps in covering a larger area but also helps in preventing water logging which is a major cause for increasing soil salinity.

Uncontrolled application of irrigation water to crops demands that adequate facilities for drainage be also provided. Water table, in the absence of drainage facility may be raised very close to the ground level causing water logging. Such upward movement of water followed by evaporation from the ground surface tend to concentrate dissolved salts in the soil. Lack of leeching due to poor drainage subsequently renders the soil saline or alkaline.

1.4.9 Source of Energy

Conventional prime movers are electric motor and diesel engine. If electricity is available at the LIS site, it is not only imperative to note if the type of supply can withstand the proposed "load" but also to note if the supply is regular, more so during the peak agricultural season.

If electricity is not available at site, the possibilities of extension of the same to the site should be probed through discussions with the appropriate authorities. Should the choice be made in favour of operating a scheme based on diesel engine, supply of diesel, maintenance facility and overall economic viability of the scheme should be critically analysed. If non-conventional prime mover is chosen, it should be based on established performance data obtained after adequate experimentations.

1.4.10 Land Holding

It is important to note the existing land holding pattern in the command area. Collection of cedestral map (revenue map) indicating the survey numbers is an important prerequisite. This map togetherwith the list of land holders help in planning the distribution layout. It also helps in working out equitable distribution of water and making other management decisions. Knowledge of land ownership is also necessary in case institutional finance is to be induced into the area.

1.4.11 Availability of Infrastructure

Availability of water not withstanding, the size of a scheme should be planned commensurate to the available infrastructure in the area. Availability of standard construction material, pumping machinaries, maintainance facilities, dependable source of energy and important agricultural inputs must be looked into. Other related aspects like consumption, storing, processing transport and marketing of the produce should be kept in view in order to assess the probable socio-economical impact of such schemes.

1.4.12 Social Demands

There may be proposals which do not satisfy the technical norms in its strictest sense but may deserve priority when the benefit is aimed at the poorer section of the society. Less stringent standards of economic viability could be applied to some of these schemes considering the social desirability. Provision of subsidy under the national policy, if available could otherwise justify implementation of such schemes.

Conversely, even a very technically feasible scheme may be considered undesirable where the prevailing socio-economic condition is not condusive for a balanced and sustainable growth.

1.5 SURVEY AND DATA COLLECTION

Once a scheme is found feasible from the initial study, the next step would be to collect detail data to the extent possible. While some data could be collected from similar other areas or even at a later date but certain other set of more important data must be collected at the time of initiation of the scheme itself. These are socio-economic and Topographical data both of which are obtained by conducting actual field surveys.

1.5.1 Socio-Economic Survey

Socio-economic data is needed basically to note the existing resource base. Data should be collected with the involvement of local beneficiaries either in a given format or through other participatory methods. This "bench mark" survey helps in ascertaining at a later date the incremental benefit accrued from the scheme. Moreover the involvement of the beneficiaries from the very initial state enhances the opportunities of successful implementation, maintenance and operation of the scheme. Knowledge of socio-economic situation is also essential for the introduction of measures for equitable distribution of water.

For example, water may be distributed family wise or to irigate only a standard size plot for each family. Even "water-coupons" may be issued to the landless settlers within a project area as a measure to benefit the entire community. The basic socio-economic data to be collected are:

- Land holding
- Size of the family
- Trades and skills
- Agricultural practices
- Cattle and other livestock
- Customs and habits
- Infrastructural facilities

1.5.2 Topographical Survey

Contour map or topographical map gives the levels (relateive elevations) at any location within the area surveyed. The purpose of topographical survey in a LIS are :-

- To measure elevation difference between the water body and the delivery point

- To decide upon the orientation and laying of the raising main

- To locate and design water impounding structures like bunds/weirs etc. on a seasonal stream/river

- To prepare an effective distribution layout

- To plan and design the locations of different hydraulic structures for the scheme

- To help implementation of the scheme as per the design

Topographical survey for the preparation of contour map of the project area could be conducted by standard methods using either a dumply level taking levels of the ground surface by a "staff" at equal size grid corners in a systematic manner or by using a Theodolite taking measurements of both horizontal and vertical angles of a fixed point of the "staff" placed at some convenient locations chosen randomly.

Any permanent land mark could be taken as the temporary "bench mark" and a value could be assigned (say 100 m) as reference for elevation. The relative elevation of the surveyed area with respect to the bench mark is noted to prepare the contour map with respect to this assigned bench-mark.

Once the elevation of the measuring points are recorded on paper, the contour map is prepared by joining lines through the points of equal values of elevation. When the levelling data are recorded at each corners of standard square grids, contouring can be done easily by transferring the data to a graph-sheet. While larger contour interval is appropriate for steeply sloping land surface, relatively smaller contour interval will be necessary for flat terrain.

Contour maps should preferably be prepared in 1 cm = 40 m scale so that the same could be superimposed over the cedestral (revenue) map. While conducting the topographical survey, the following levelling data is to be collected which are important design prerequisites.

- The level of water surface
- The level of drawdown if any
- The bed level of the water body
- The level of adjacent ground surface
- The level of delivery (highest) point in the command area

Once the points of water withdrawal and that of delivery are decided, the line (longitudinal section) drawn between these two points along a vertical scale gives the orientation and layout of the raising main. Topographical survey is also necessary for the development of a stream to a dependable water source particularly where water impounding structure is to be constructed.

Cross sections of the stream bed should be taken at a number of sites both in the upsteam and down stream sides so as to find the best possible site for construction of the check weir. Similarly, longitudinal section of the stream bed should be taken in the downstream side upto a considerable length in order to make decisions about the height of the weir and also the extent of the submerged area.

1.6 DESIGNING LIS

Design of a LIS, irrespective of its size, involves selection of best possible alternatives and preparation of specific details concerning the following major aspects of the scheme.

- Development of water source
- Assessment of water requirement for crop
- Decision on pumping equipments and distribution layout
- Preparation of cost estimates

A good design helps not only in containing the unit costs of implementation but also helps in ensuring smooth and trouble free operation over a longer period. It is therefore imperative that a design engineer be adequately familiar with various aspects of a LIS like development of water source, water requirement of crops, selection of pumps and pipes, installation, operation and maintenance as well as cost and benefit of such schemes. Layout plan of a standard LIS is presented in Fig. 1.1 and the longitudinal section of a raising main is presented in Fig. 1.2.

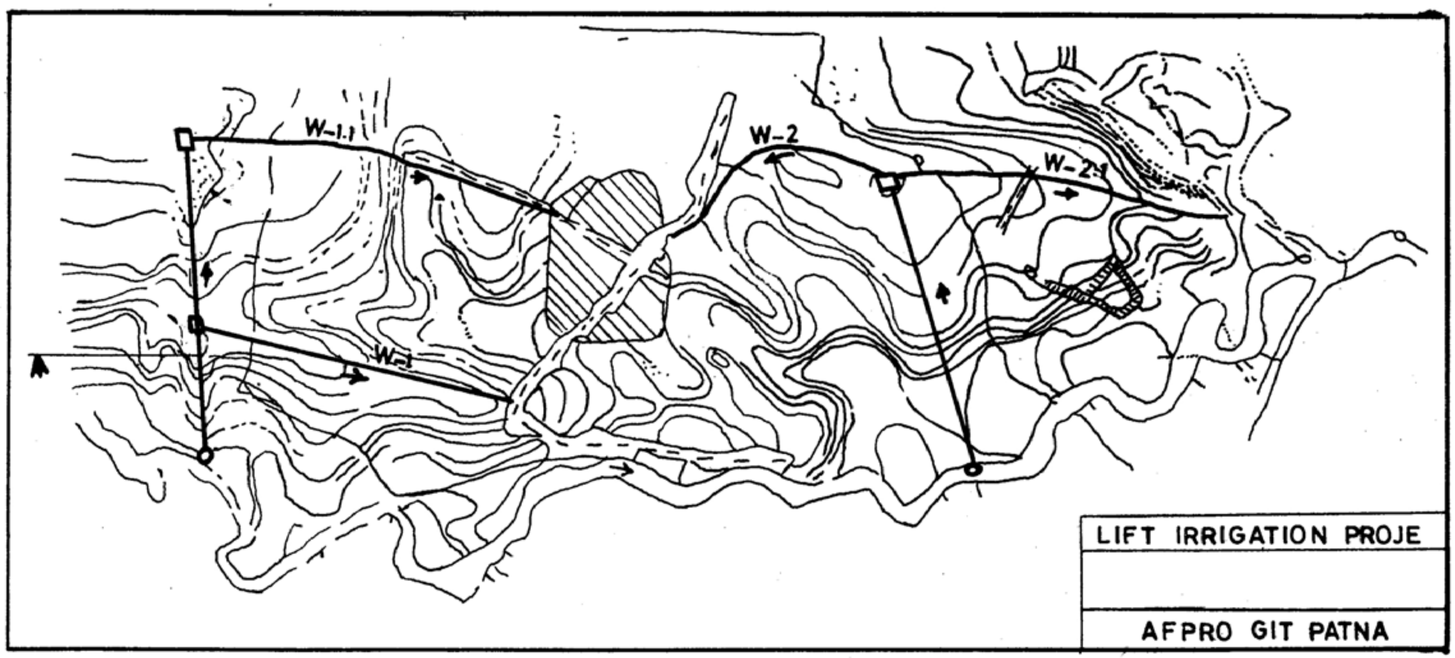

Fig. 1.1 Layout plan of a standard LIS

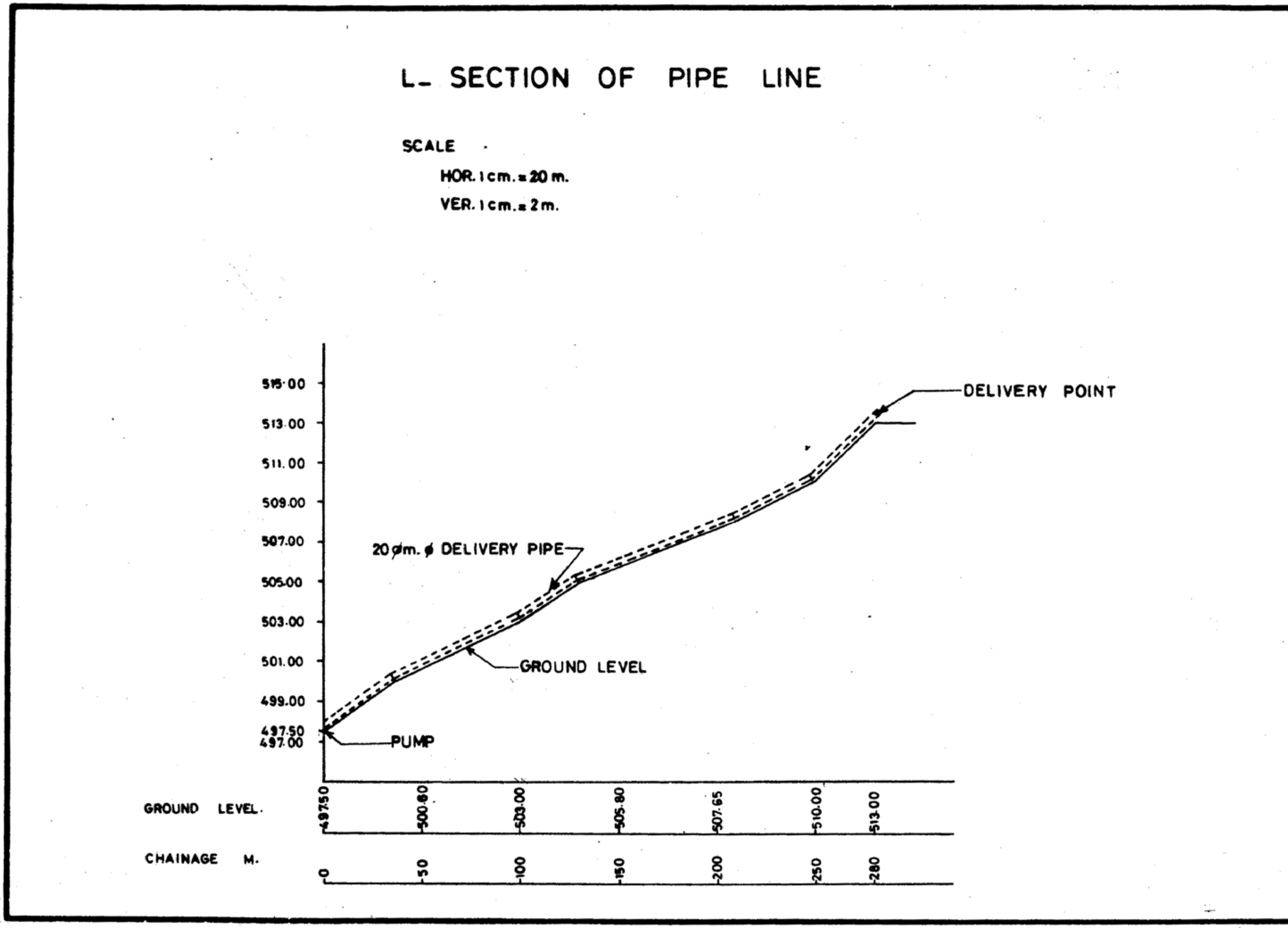

Fig. 1.2 Longitudinal section of a raising main

CHAPTER 2

DEVELOPMENT OF WATER SOURCE

2.1 GROUNDWATER

All groundwater occurs as an integral part of the "hydrologic cycle" wherein a part of the rainfall percolates downwards to recharge the groundwater reservoir. Water occupying all the pores within a geological substratum to its level of saturation is groundwater. This saturated zone is differentiated from the overlying unsaturated zone (aeration zone) by "water table" which is subject to vertical fluctuation with season.

A porous and permeable formation which can hold and transmit sufficient water under normal field condition is an "aquifer". An aquifer which is bound at its bottom surface by an impervious layer but open at its top surface is an unconfined or water table aquifer. Water in unconfined aquifer occurs under atmospheric pressure, the top surface of which is water table (Phreatic surface). The aquifer which is bound at both its top and bottom surface by impervious layers is known as confined aquifer. Water occurs in confined aquifer under pressure which is higher than the atmospsheric pressure and hence rises to a level higher than the top surface of the aquifer when penetrated by a bore-hole.

2.2 DUG WELL

Dug wells are simple and most extensively used water development structures. These are constructed normally to tap shallow water table aquifers with poor to moderate yield potential. A dugwell should preferably tap the entire thickness of the saturated zone. Dugwells are constructed by excavation and require a lining upto the depth to which the formation is loose and collapsible. In alluvial formation dugwells are lined along its entire depth.

Dugwells in hard rock formation are partly lined and receives water as percolation through the weathered and fractured rock aquifer. Static water level in a dugwell is water table which rises and falls in response to the seasonal variation in rain-fall. A dugwell is dried up when the water table vanishes completely or receeds below the bottom of the well. High water level fluctuation is observed in a well which receives water from an aquifer having low permeability and vice versa. Centrifugal pumps are used most conveniently to extract water from dugwells.

2.3 TUBEWELL

Tubewells are normally constructed to tap deep confined aquifers having moderate to high yield potential. These are constructed by adopting a suitable drilling techniques as is applicable to the formation in question.

In alluvial formation drilling is done commonly by simple cutting tools using rotational motion and the drill cuttings are flushed out by mud circulation using a mud pump. Casing pipe is inserted all along the length of the bore hole having slots (screen) placed against the aquifer to allow entry of sand free water into the tubewell.

Large quantity of water can be pumped out from tube wells by deploying vertical turbine or submersible pumps. Common centrifugal pumps are also used if the water level in the tube well is shallow and the drawdown is limited. The housing pipe i.e.the upper part of the well casing should have relatively larger diameter to house the bowls assembly (impellers) of the pump.

In hard rock area, tubewells are drilled by cutting and hammering (down the hole hammer) or simply by cutting (cable tool, calyx etc.) holes into the ground. The drill cuttings are flushed out by using either compressed air or by physical scooping. The well casing is placed only upto the loose near surface formation (overburden).

Water level in a tubewell taping a deep confined aquifer is known as piezometric surface which due to the prevailing pressure head, rises quite closer to the ground surface. Piezometric surface normally do not fluctuate much in response to seasonal variation in rainfall.

2.4 DUG CUM BOREWELL

An area where both shallow unconfined and deep confined aquifers are present, groundwater is tapped most profitably by constructing dug cum borewell in which vertical boreholes are drilled from the bottom of the dugwell.

While the dugwell taps water from the shallow aquifer, the bore taps deeper confined aquifer, the piezometric surface of which rises sufficiently high to reach into the dugwell for accumulation. Water received from both the aquifers as stored in the dugwell can then be pumped out using ordinary centrifugal pump.

2.5 ESTIMATION OF YIELD

Yield of a well is determined by actual test pumping. When water is pumped out from a well, groundwater from the surrounding aquifer starts flowing into the well (Fig. 2.1). The rate of flow is proportional to the permiability of the aquifer and hydraulic gradient of the flow. If the rate of pumping is higher than the rate of flow of water into the well a decline in water level is observed which is the drawdown. Steady state is said to have reached when the drawdown for a given discharge rate remains unchanged with time.Theoretical drawdowon in a well under constant discharge rate is computed by one of the following formulae.

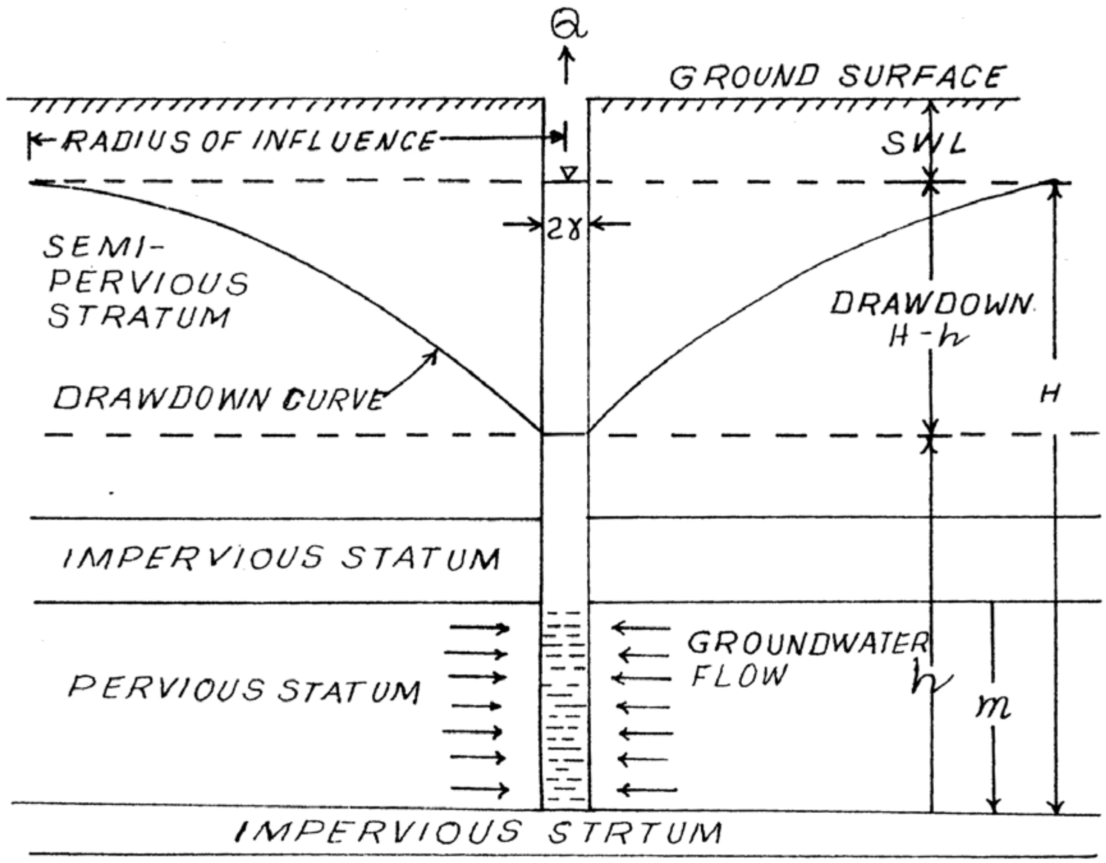

Fig. 2.1 Groundwater flow in a tubewell

For fully penetrating, unconfined aquifer with steady state of flow

$$s = \frac{2.3\ Q}{2\ \pi\ T} \log \frac{R}{r} \qquad \ldots\ 2.1$$

For fully penetrating confined aquifer with unsteady state of flow

$$s = \frac{2.3\ Q}{4\ \pi\ T} \log \frac{2.25\ Tt}{r^2 S} \qquad \ldots\ 2.2$$

where,

Q = Constant rate discharge, cu.m/day
T = Transmissivity of the aquifer, sq.m/day
t = time of pumping, day
R = Radius of influence, m
S = Storage co-efficient
r = Radius of well, m

Transmissivity (T) is the product of permeability (K) and thickness (b) of the aquifer and Storage co-efficient (S) is the capacity to store or relase water from an aquifer and is unit less.

One important question that needs to be resolved while dealing with the issue of well yield in the context of a LIS is whether a well should be designed as per the water requirement of the crop or the command area is to be restricted as per the availability of water from a well. Since yield of a well depends on the nature of the aquifer, it is advisable that the cropped area be decided based on the safe yield of a well.

2.6 YIELD OF DUGWELL

Since dugwells do contain an appreciable quantity of water in storage, the full flow in the aquifer in response to the pumping starts rather late. Because of this delay in establishing full flow in the aquifer, the conventional yield testing formulae are not applicable to dugwells of large diameter. Yield of a dugwell is best understood from its specific capacity value and rate of recuperation values.

2.6.1 Permissible Drawdown

Specific capacity of a well is the discharge per unit drawdown and is given by Q/s. Specific capacity values for different discharge rates are obtained by step drawdown test. Under this test, the well is first pumped for a particular time at a lower discharge rate than the well can sustain and the drawdown is noted. The discharge is then increased in steps of equal time and the corresponding drawdowns at the end of each step are noted for as many steps as possible.

Specific capacity for each step is obtained by dividing the discharge rate with the corresponding drawdown. The specific capacity value in each step differs (decreases) when the flow is unsteady and the effect of time of pumping is pronounced on the drawdown.

In order to determine the yield of a dugwell, it is therefore customary to first decide the permissible drawdown of the well. The corresponding discharge is then found out for that given permissible drawdown by test pumping of the well. Since centrifugal pumps are used in most dugwells, the permissible drawdown should not normally exceed 3-4.5m keeping in view the limiting suction lift of such pumps.

It is evident from Eqn. 2.1 that drawdown in a well varies directly with discharge Q and the factor log R/r and inversely with transmissivity T. Radius of influence (R) is determined by studying drawdown in an observation well located at a reasonable distance away from the test well.

Though the value of R in a water table aquifer vary widely, the value of the factor log R/r do not vary appreciably. Normally for dugwells the log R/r value ranges from 1 to 3. Taking the average value of the log R/r as 3.33 the following approximate relationship was suggested by logan.

$$T = 1.22\ Q/s \qquad \ldots\ 2.3$$

It is however important that permissible drawdown should be only that much which recuperates adequately before the pumping is started again.

2.6.2 Recuperation Test

Water level in a well begins to rise the moment pumping is stopped. The rate of rise in water level is the rate of recuperation. Higher the drawdown in a well faster will be the initial rate of recuperation. Complete recovery of a well takes much longer time than say its initial 50% of recuperation. Recuperataion is measured by residual drawdown (s') which is the distance between the static water level and the recovered water level at any given time. In order to estimate recuperation period, the modified slichter's formula is used.

$$t = \frac{2.3\ As}{Q} \log \frac{s}{s'} \qquad \ldots\ 2.4$$

where,

t = Time of recuperation, min
A = Area of the well, sq.m

Q = Discharge rate, cu.m
s = Permissible drawdown, m
s' = Residual drawdown, m

It is generally assumed that near complete recuperation has taken place when the ratio of permissible drawdown (s) and residual drawdown (s') is about 10. Thus,

$$t = \frac{2.3\ \pi\ r^2 s}{Q} \log 10 = 7.23 \frac{r^2 s}{Q} \qquad \ldots\ 2.5$$

2.7 YIELD OF TUBEWELL

In step drawdown test, a well is first pumped at constant discharge rate for a given period of time. The discharge is then increased and kept constant in the next step for the same time interval. Use of equal time interval eliminates the effect of time on drawdown and cosequently on the measured specific capacity. Drawdown in the well at the end of each step is measured. A minimum of three such steps are nessesary to form three simultaneous equations to solve the thrtee unknown flow coefficients. Drawdown for any discharge rate beyond the discharges for which the well has been actually tested is obtained by extrapolation.

Use of data from step drawdown test is illustrated in Fig. 2.2. It is obvious from the figure that the Well-I is located in an area which has lower yield potential (transmissivity) than the well - III. By extrapolating these curves, the discharges from these three tubewells for a drawdown of 6 m works out to be 86 cu.m/hr , 136 cu.m/hr and 186 cu.m/hr respectively.

Theoretical drawdown at the end of any given time of pumping (t) in a taubewell fully penetrating a confined aquifer is computed by using Eqn. 2.2 when the value of T and S of the aquifer is known. An additional amount of drawdown originated due to friction of flow met in the well (well loss) is however required to be added with the theoretical drawdown in order to obtain the observed (actual) drawdown. Extent of well loss is obtained from the analysis of Step drawdown test data . One of the simple method of analysis is the algebraic method, as shown below.

s = Theoretical drawdown + Well loss

$$s = BQ + CQ^n \qquad \ldots\ 2.6$$

Where,
s = Actual drawdown, m
B = Aquifer loss coefficient
C = Well loss coefficient
Q = Discharge rate, cu.m
n = Well loss exponent taken normally equal to 2

Value of C is first obtained from the following equation

$$C = \frac{\Delta s_2/\Delta Q_2 - \Delta s_1/\Delta Q_1}{\Delta Q_1 + \Delta Q_2} \quad \ldots 2.7$$

where,

$$\Delta Q_1 = Q_2 - Q_1; \ \Delta Q_2 = Q_3 - Q_2$$
$$\Delta s_1 = s_2 - s_1; \ \Delta s_2 = s_3 - s_1$$

Value of B is then obtained by replacing C value in any one of the simultaneous equations.

2.8 EFFECT OF DIAMETER ON YIELD

It is observed from Eqn. 2.1 that discharge of a well under steady state condition is inversely proportional to the factor log R/r. Thus by doubling the radius (r) of a well, the discharge (Q) of the same is increased only marginally (logerithmically) and not double as may be commonly believed. Increase in diameter however helps in increasing the storage capacity of a well.

Example 2.1

Compute the theoretical drawdown from a 200 mm dia. tubewell in a fully penetrating confined aquifer having average transmissivity (T) and storage co-efficient (S) as 6000 Sq.m/d and 0.0001 respectively when the well is pumped for 12 hrs. at a discharge rate of 250 Cu. m/hr.

Find the increment in discharge for this particular drawdown, if the diameter of the same well increased from 200 m to 800 mm.

Solution

$Q = 250 m^3/hr$

$T = 6000 m^2/day = 250 \ m^2/hr$

$r = 100 \ mm = 0.1 \ m$

$S = 0.0001$

$t = 12 \ hr$

Using eqn 2.2 we have

$$s = \frac{2.3 \times 250}{4 \times 250} \log \frac{2.25 \times 250 \times 12}{0.01 \times 0.0001}$$

$$= 0.183 \times 9.83$$
$$= 1.8$$

The theoretical drawdown is 1.8 m

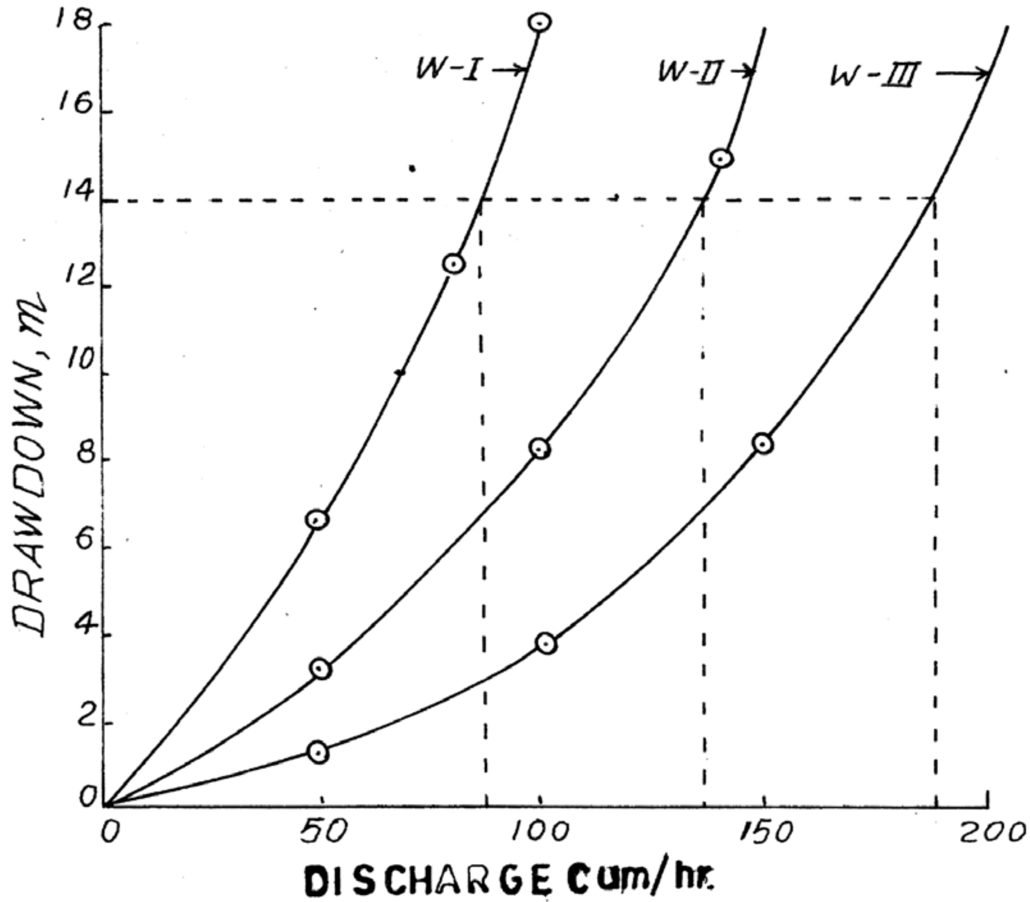

Fig. 2.2 Step drawdown data plot

The discharge for this particular drawdown of 1.8 m from a larger diameter well (800 mm) is computed by the same Eqn. 2.2 .

$$Q = \frac{4\,\pi\,Ts}{2.3} * \frac{1}{\log\ 2.25\ T\ t\ /r^2\ S}$$

$$= \frac{12.56 \times 250 \times 1.8}{2.3} * \frac{1}{\log\ (2.25 \times 250 \times 12)/0.16 \times 0.0001}$$

$$= \frac{5652}{2.3\ \times\ 8.625} = 285$$

The increment in discharge due to four times increase in diameter is 285 cubic m/hr. as against 250 cubic m/hr. which is an increment by 14 percent only.

Example 2.2

The step drawdown test conducted in a borehole yields the following data:

I Step : Q_1 = $80 m^3/hr$ s_1 = 4.80 m

II Step : Q_2 = $110\ m^3/hr.$ s_2 = 8.00 m

III Step : Q_3 = $150\ m^3/hr.$ s_3 = 15.2 m

Compute the drawdown if the well is pumped at a discharge rate of $200\ m^3/hr.$

Solution

Actual drawdown (s) is given by

$$s = \text{Theoretical drawdown} + \text{well loss} = BQ+CQ^2$$

Value of C is computed from Eqn. 2.7

Here,

$\Delta Q_1 = 30$; $\Delta Q_2 = 40$

and $\Delta s_1 = 3.2$; $\Delta s_2 = 7.2$

$$C = \frac{7.2\ /\ 40\ -\ 3.2\ /\ 30}{30\ +\ 40} = \frac{0.118\ -\ 0.106}{70}$$

$$= \frac{0.012}{70} = 0.00017$$

Value of B is computed from any of the three simultaneous equations of step drawdown test, i.e.

$$s = BQ + CQ^2$$

Considering the 3rd simultaneous equation, we have

$$15.2 = B \times 150 + C(150)^2$$

$$\text{Or} \quad 15.2 = 150\,B + 3.825$$

$$\text{Or} \quad B = \frac{11.375}{150} = 0.076$$

The drawdown for a discharge of 200 cu. m/hr. would be

$$s = 0.076 \times 200 + 0.00017\ (200)^2$$

$$= 15.2 + 6.8$$

$$= 22.00\ \text{m}$$

2.9 SURFACE WATER

The major source of all surface water is rainfall. The disposal of rain water after it falls on the land surface is run-off.Before run-off originates, the precipitation must satisfy the demands of evaporation, infiltration, surface storage and surface retention. The relaltionship between precipitation and other hydrologic parameters are given by the water balance equation i.e.

$$P = R+ET+SM+GS+U \qquad \ldots\ 2.8$$

where,

P = Precipitation DM = Soil Moisture

R = Run-off GS= Change in ground water storage

ET = Evapotranspiration U = Sub-surface flow

2.10 FACTORS AFFECTING RUN-OFF

Run-off is the surface water flow collected from a drainage basin (catchment) which moves out for disposal through a common outlet of the drainage basin.Factors that control the run-off from a catchment area are grouped as below.

Precipitation Characteristics

- Intensity of rainfall
- Duration of rainfall
- Distribution of rainfall

Catchment Characteristics

- Size of catchment
- Shape of catchment
- Slope of catchment
- Land use of catchment (vegetations, retention structures etc.)

Geological Characteristics

- Type of soil (Texture, permeability etc.)
- Physiographic controls

Metereological Characteristics

- Temperature
- Humidity
- Wind effect

2.11 ESTIMATION OF RUN-OFF

Quite a few emperical methods have been developed for the estimation of run off. Since characsteristics of each catchment varies widely, most of these indirect methods have applications limited to certain type of catchments only.

For small catchments, where adequate data is seldom available, the Burlow's method, Infiltration method and Strange's method have wide acceptance.

2.11.1 Barlow's Method

It is applicable for a particular rainfall and is given by

$$R = KP = K_1 K_2 P \qquad \ldots \quad 2.9$$

where,

R = Run-off, cm
P = Precipitation, cm
K = Run-off co-efficient for type of catchment
K_1 = Run-off co-efficient for type of rainfall
Values of K_1 and K_2 are obtained from the following tables.

Table 2.1 Barlow's Coefficients (K_1) on catchment type for average rainfall

Class	Description	Value of K_1
A	Flat, cultivated, Black Cotton soils	0.10
B	Flat, partly cultivated, various soils	0.15
C	Average catchment	0.20
D	Hills and plains with little cultivation	0.35
E	Very hilly and steep without any cultivation	0.45

Table 2.2 Barlow's Coefficient (K_2) as modified for type of rainfall

Nature of season	Class of catchment				
	A	B	C	D	E
Light rain	0.7	0.8	0.8	0.8	0.8
Average rain	1.0	1.0	1.0	1.0	1.0
Continuous down pour	1.5	1.5	1.6	1.7	1.8

Values of Barlow's coefficient (K) computed for a few cases are given below as an example.

Average catchment and average rainfall: $K = K_1K_2 = 0.2x1 = 0.2$

Hilly catchment and continuous downpour: $K = K_1K_2 = 0.45x1.8 = 0.81$

Flat catchment and continous downpour: $K = K_1K_2 = 0.1x1.5 = 0.15$

Example 2.3

Estimate the total volume of run-off due to 20 cms. rainfall over a period of 2 days in a partly hilly and partly plain 2000 hectare catchment area with little cultivation.

Solution

The rainfall may be considered as continuous down pour.Thus, from Barlow's table 2.1 and 2.2, we have

$K_1 = 0.35$ and $k_2 = 1.7$

$R = KP = K_1 K_2 P$

$= 0.35 \times 1.7 \times 20$

$= 12.75$ cm $= 0.1275$ m

Voulme of total run-off is therefore estimated as

$V = 2000 \times 0.1275$

$= 255$ ha.m.

$= 2550000$ cubic metre

2.11.2 Infiltration Method

Infiltration rate is the average rate of loss by infiltration into the soil. The volume of rainfall in excess of infiltration loss is taken equal to the direct run-off when evaporation loss is either insignificant or ignored.Rate of infiltration in cm/hr for different soils in different hyderometereological regions may be taken as follows.

Table : 2.3 Rate of infiltration loss in cm/hr.

Type of soil	Arid Area			Semi-arid/ sub Humid Area			Humid area		
	Forest	Farmland	Fallow	Forest	Farmland	Fallow	Forest	Farmland	Fallow
Clayey soil	0.09	0.07	0.05	0.07	0.05	0.04	0.05	0.04	0.05
Clayey Loam	0.17	0.13	0.09	0.13	0.10	0.07	0.09	0.07	0.05
Silty loam	0.26	0.20	0.14	0.20	0.15	0.11	0.14	0.11	0.08
Sandy loam	0.55	0.50	0.20	0.30	0.22	0.15	0.20	0.15	0.12
Sandy soil	0.50	0.40	0.30	0.40	0.30	0.20	0.25	0.20	0.15

This method can however be used only at instances where,

- The run-off for a known rainfall and known duration is to be computed
- The soil is reasonably dry prior to the rainfall under con sideration

Example 2.4

Estimate the total run-off from a catchment area of 1500 ha. comprising predominently of farmlands with sandy loam soil for a rainfall of 10 cm.in 20 hours using infiltration method.

Solution

As per table 2.3 , the rate of infiltration loss for farmland in sandy loam soil in semi arid region is 0.22.

Total infiltration in 20 hr = 20X0.22 = 4.4 cm

Total run-off after 20 hr = 10 - 4.4 = 5.6 cm = 0.056 m

Total volume of run-off = 1500 x 0.056 = 84 ha-m

= 840000 cubic metre

2.11.3 Strange's Method

Strange gave a table (also available as curves) for estimation of daily yield of run-off corresponding to daily rainfall for different ground conditions prior to the rainfall viz. dry, damp and wet. This table was prepared based on data from Maharshtra and other black cotton soil regions of central India for three types of catchments.A good catchment is one which has got good run-off qualities for a given rainfall giving more run-off in contradiction to bad catchment which gives less run-off for the same rainfall.

Table 2.4 Strange's table for estimation of daily run-off for an average catchment

Daily rainfall	DRY		Damp		Wet	
mm	Percent %	Yield mm	Percent %	Yield mm	Percent %	Yield mm
5.00	-	-	4	0.2	7.0	0.35
20.00	2	0.40	9	1.8	15.0	3.0
25.0	3	0.75	11.0	2.75	18.0	4.5
37.5	6	2.25	16.0	6.0	25.0	9.37
50.0	10	5.00	22.0	11.0	34.0	17.0
75.0	20	15.00	37.0	27.75	55.0	41.25
100.0	30	30.00	50.0	50.0	70.0	70.0

2.11.4 Direct Method

Run-off at a given location(cross-section) of a stream is best estimated by direct measurement of the flow. Actual quantity of flow is obtained by using reservoir monitoring, gauge-post, flow meter, area-velocity method etc. Gauge-post and flow meter are normally used for large rivers where flow is measured over a number of years. Area-velocity method is widely applied for small streams.

In the reservoir monitoring method, at first the reservoir is subjected to minute contour survey and then the area and the storage volume are calculated for various depths using volumetric formula. A depth-volume graph is prepared which would indicate the storage volume of the reservoir for any depth. A vertical post with depth marking from zero to maximum water level is erected at the deepest point. Observing the water level variation at fixed time interval would help in computation of run-off. If rainfall and catchment charecteristics are known then the method can also be used to calibrate a catchment and compute run-off co-efficient in Barlow's equation.

In area-velocity method, a comparatively straight portion of the stream is selected. Depths of water at regular intervals (say 1 m) are measnured along a cross-section by using a dip stick. Cross-sectional area(A) along the line of measurements is then drawn by plotting the vertical depth section to scale in a transparent sheet (Fig.2.3). The cross-sectional area is then computed by placing the transparent sheet over a graph sheet and counting the number of full squares plus portion of part squares falling within the cross-sectional area.

Velocity (V) of the stream is obtained by allowing a float (light object)to flow along the stream. Velocity of the flow is given by time taken for the float to travel a known distance along the stream. Average surface velocity is obtained by running a number of floats along different longitudinal sections of the stream. Since the velocity of flow in a stream varies both borizontally and vertically, the mean velocity is taken as 0.7 to 0.85 times the average surface velocity. The rate of discharge (run-off) is obtained by multiplying the Area (A) with the Velocity (V).

2.12 ESTIMATION OF HIGH FLOOD DISCHARGE

The maximum rate at which water flows down a catchment is called maximum rate of run-off or high flood discharge or intensity of maximum flood or peak rate of run-off. Sufficient estimate of high flood discharge is essential for the safe design of water retention structures (bunds, weirs etc.). To estimate the high flood discharge, a number of approaches are available. But for small catchments , for which systematic data collection is not practicable,the most popularly used approach is the application of the rational method in which

$$Q = \frac{CIA}{360} \qquad \ldots \quad 2.10$$

where,

Q = high flood discharge, cu.m/sec
C = Run-off co-efficient
A = Catchment area, ha
I = Intensity of rainfall in mm per hour for a period equal to the time of concentration of the catchment area for a designed frequency interval.

The following values of C are assumed for different types of soils.

Sandy soil	-	0.29
Sandy loam	-	0.40
Clayed soil	-	0.50

Time of concentration is the period taken for the run-off to travel from the furthest part of the catchment to the point of measurement and is given by

$$tc = (0.87 \times L/H)^{0.385} \qquad \ldots \quad 2.11$$

where,

tc = Time of concentration, hr
L = Distance from critical point to proposed site, Km
H = Fall of level (Vertical height) from critical point to proposed site, m

Number of years in which an exceptional high flood discharge is likely to occur is the recurrence period (T) Since such a high flood discharge occurs once in T years, the frequency of flood is 1/T. It is recommended that intensity of rainfall be taken from 25 years rainfall data. Amount of rainfall occuring per unit time is rainfall intensity which is normally expressed in mm/hour.

The time of concentration of a normal drainage area may be assessed as shown in the example below.

Example 2.5

Compute the time of concentration of a site whose distance from the critical point(ridge line) is 500 m.and fall of level is 8 m. What would be the peak rate of run-off at the site if highest rainfall of the area as obtained from 25 years rainfall frequency chart is 80 mm/hr, run-off co-efficient C is assumed as 0.35 and the size of catchment area is 50 hectares.

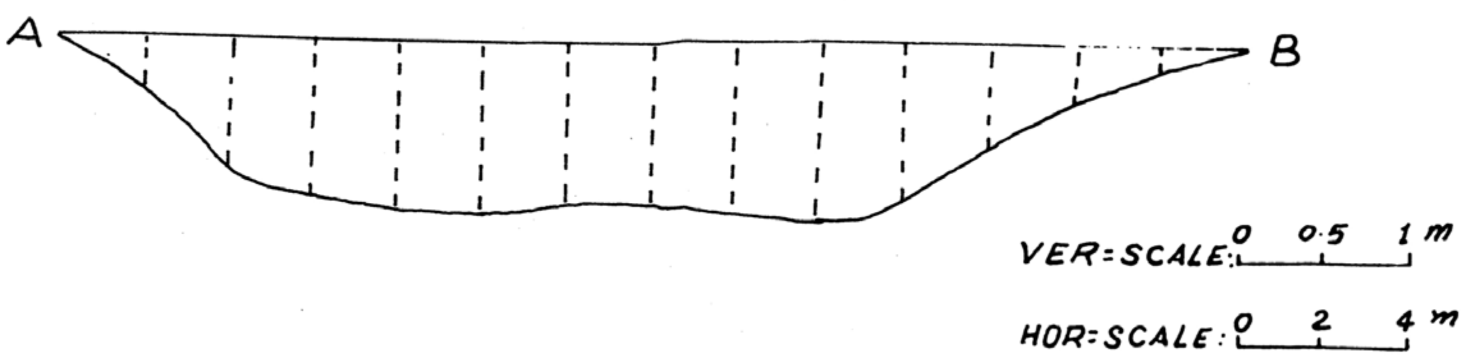

Fig. 2.3 Cross-sectional area of a stream

Solution

From Eqn. 2.11, we have

$$tc = (0.87 \times 0.5/8.0)^{0.385} = 0.20 \text{ hr} = 12 \text{ min.}$$

Highest rainfall from 25 years frequency chart is found to be 80 mm/hr.Combining the maximum rainfall with rainfall intensity duration curve the rainfall intensity (I) is computed as 200 mm/hr.for a time of 12 minutes (0.20 hr.)

The peak rate of run-off is obtained from Eqn. 2.10

$$Q = \frac{0.35 \times 200 \times 50}{360} = 9.72 \text{ cu.m/sec}$$

2.13 DEVELOPMENT OF SURFACE WATER

It is obvious that the foremost requirement for a lift irrigation scheme based on surface water is that the water be available for a larger part of the year. Stored water bodies like tanks and lakes should be considered as a reliable source only after a proper assessment of available water is made. A tank or a lake may appear to have sufficient water but this may not serve as a reliable source if the same do not receive adequate recharge either from the catchment area or through effluent seepage.

For example, a tank having the size of 100 m length, 100 m breadth and 1 m depth has a total volume of 10,000 cubic metre of water which is equal to 1 hectare-metre.Using 5 cm depth per application with 10 applications per hectares, this water is capable of irrigating just 2 hectares. Moreover a tank should never be allowed to be dewatered completely for the sake of irrigation alone.

Surface water sources, particularly seasonal streams,could be developed into a reliable source of water by constructing impounding structures like earthern bunds or overflow weirs.

2.14 EARTHEN BUNDS

An earthen bund is formed by constructing an embankment with locally available material like soil,morrum etc. across a natural stream to impound water. Earthen bunds require one escape outlet for excess water (spill way) but do not require solid foundation for its stability.

2.14.1 Selection of Site for Earthen Bunds

- The size of the catchment in the upstream of the site should be such that adequate water be available from the given rainfall in the area for the required storage.

- Storage depends upon the slope of the storage area and height of the bund. The site should ensure adequate storage capacity without undue extension of the submergence area. The storage area should be deepened if necessary to increase storage capacity.

- The soil permeability of the storage area be such that the quantum of subsurface infiltration is within the acceptable limit.

- The downstream area should permit natural way for safe and economic disposal of the surplus water from the stream.

- The site should be such that the length of the bund do not increase unnecessarily due to unfavourable topography.

2.14.2 Design Criteria

The cross-section of an earthen bund is shown in Fig. 2.4.The important features of an earthen bund are its top width, side slope and height. Once the base width and side slopes are fixed, top width of a bund is determined by its height. Top width should normally be kept 1/2 to 1/3 of base width. It is a good practice to maintain the top width uniform throughout the length.

The core of the bund should be made with impervious clayey soil with adequate compaction.Massonary or concrete core or cut-off walls are also provided. The core wall is normally kept embeded to a depth roughly equal to 1/3 to 1/4 of the water column and the top level is kept about 30 -50 cms.above the high flood level. The purpose of the core wall is to add strength to the bund and also to prevent infiltration (seepage).

Side slopes of a bund depends upon the nature of the material to be used. The side slopes should be such that the soil do not show the tendency of slipping. Upstream slope is kept flatter than the downstream slope.

Height of a bund is determined based on the following factors:

- High Flood Level (H.F.L.) of the reservoir

- Amount of free board to be provided

- Amount of foundation clearance. Deeper the foundation, below ground level, the higher would be the bund

- Amount of allowance for settlement

- The extent of height (h) should be 3 times the bottom width

In order to increase the stability of a bund, the following precautions should be taken:

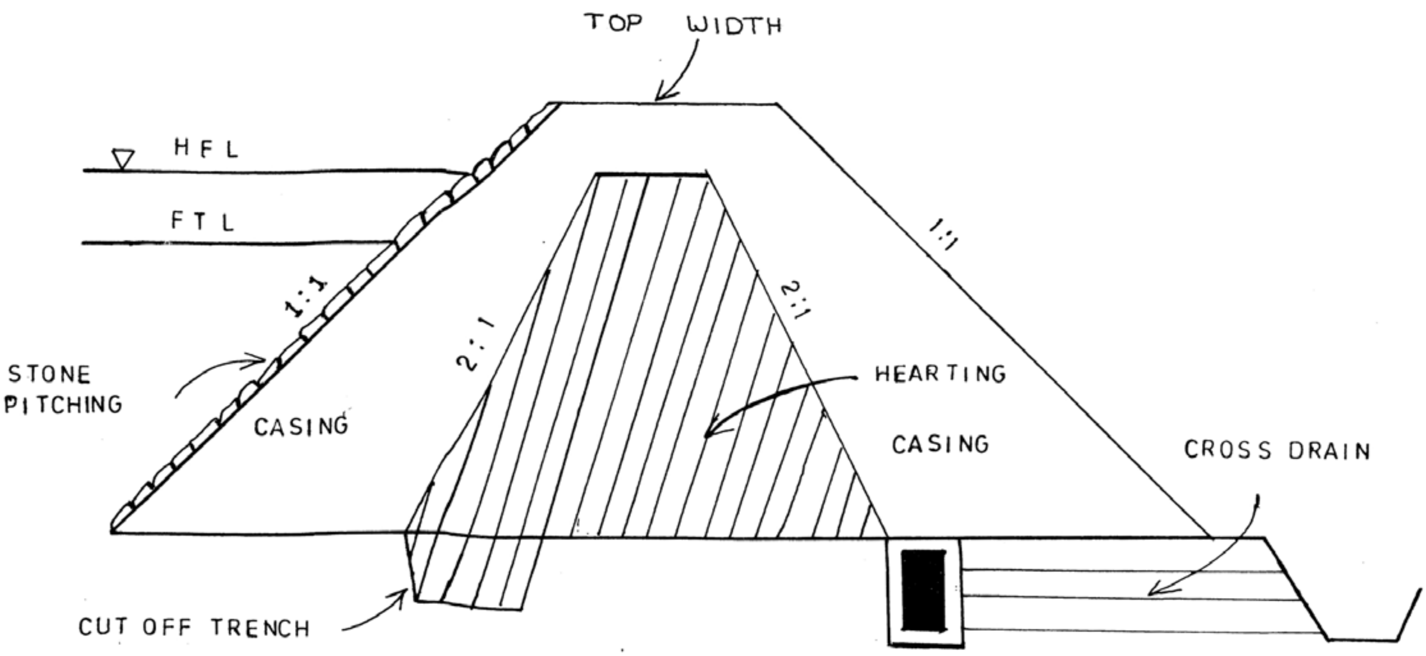

Fig. 2.4 Cross-section of an earthen bund

- To prevent overlapping, sufficient free board and proper escape weir (spill way) be provided.

- The upstream slope should be protected by stone pitching or adequate margin of slope to prevent scouring by wave action.

- Toe erosion should be prevented by providing riprap on the downstream slope.

- The seepage line should remain well within the downstream face.Cut-off wall makes percolation gradient flatter.A longitudinal drain parallel to the cut-off trench (wall) and cross drains at intervals help in providing good drainage to any water that may enter by percolation through the heart of the dam or from rainfall.

- The upstream face slope should be stable against sudden drawdown. Minimum inside slope should be 1:3 and outside slope be 1:2.

- The up and down stream slope of the earthen bunds should be flat enough so that the shear stress produced in the foundation is less than the shear strength of the material in the foundation.

- The seepage flow passing under the bund when it reaches the discharge surface should have pressure and velocity so small that it is incapable of moving material from the foundation of the bund.

2.15 OVERFLOW WEIRS

Weir is a continuous solid structure built across a stream over which water may flow. Weir impounds and raises water in the up stream side.

2.15.1 Selection of site

The criteria for selection of site for construction of a weir is listed below.

- The stream should be as straight as possible to permit maximum storage and allow minimum side scouring (erosion).

- Stream should have good depth and smaller width so that the length of the weir remains small.

- The upstream side should be reasonably broad so as to permit greater average volume of storage per unit height and length of weir.

- Presence of rock formation at shallow depth of the stream bed provides a stable foundation for the weir.Site with thick Clayey formation in the stream bed should be avoided as far as possible from foundation point of view.

- The longitudinal slope of the stream bed in the upstream side should be gentle so that larger volume of water is stored mostly as channel storage with minimum submergence of the surrounding area.

2.15.2 Design Criteria

A weir is basically a gravity dam. The stresses acting on a weir are:

- water pressure in the upstream face
- weight of the weir to the foundation
- uplift pressure, (bouyancy)
- expansion and contraction due to temperature variation
- crushing effect on the weir

The cross-section of a weir is shown in Fig. 2.5 The important design parameters are:

2.15.3 Crest Width

The crest should be strong enough to cope with the impacts of floating materials, logs, debrises etc. which are to be displaced over the weir. The crest width is usually determined by the following formula

$$a = \frac{3\ H}{2\ S} \qquad \ldots\ 2.12$$

where,

a = Width ,m
H = Flood depth over the crest,m
s = Specific gravity of construction material

Crest width however should not be lesser than 0.5m.

2.15.4 Bottom Width

Bottom width is given by

$$B = \frac{1.5\ h + H}{S^{0.5}} \qquad \ldots\ 2.13$$

where,

B = Bottom width of weir , m
h = Height of body wall (weir) above bed level, m

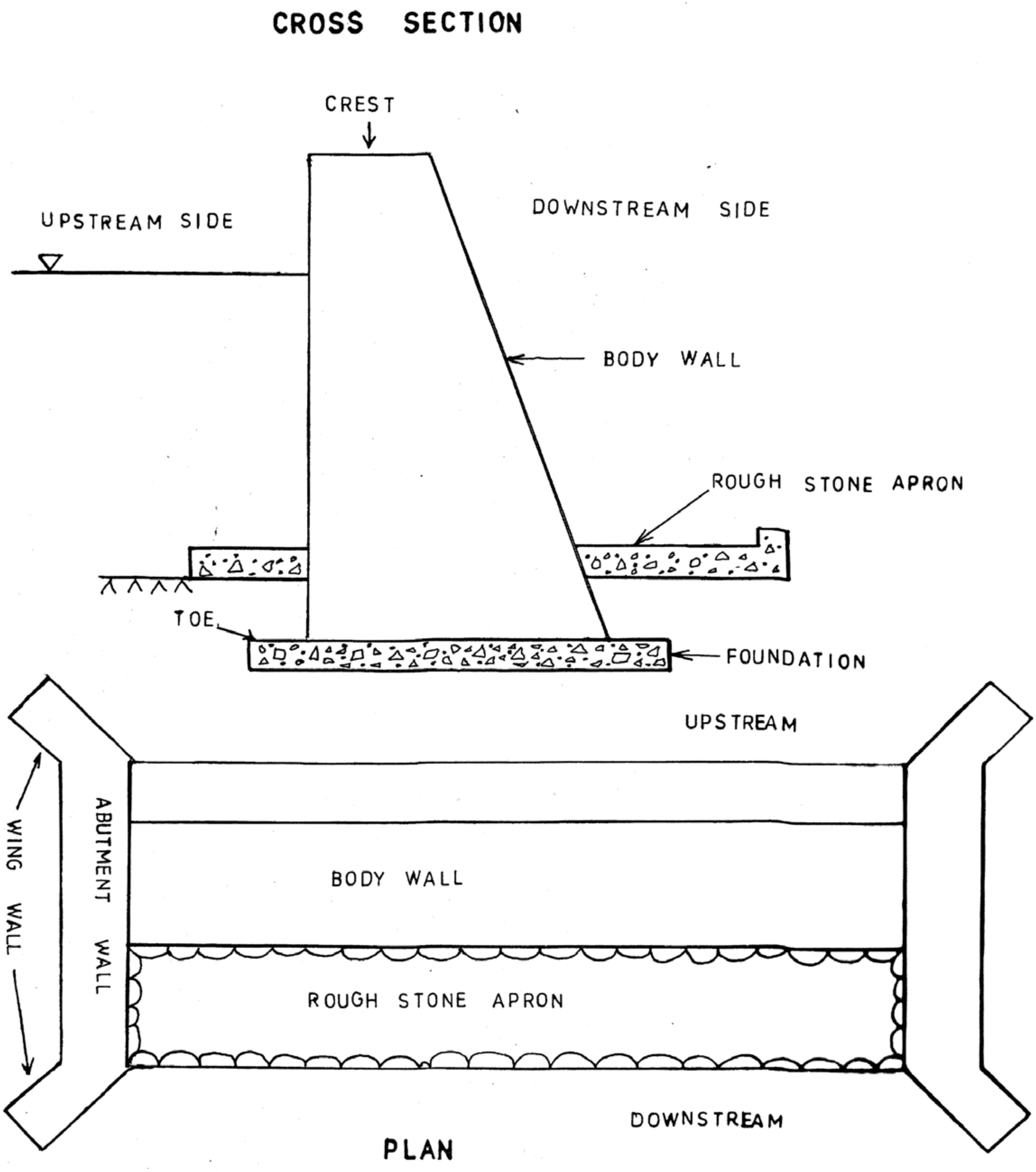

Fig. 2.5 Cross-section of a masonary weir

H = Flood depth, m
S = Specific weight of construction material
Cement concrete = 2.4
Stone masonary = 2.1
Brick masonary = 1.9

2.15.5 Length of Weir(Waterways)

For broad crested weir length can be obtained from the emperical formula

$$Q = 2.4\ LH^{1.5} \qquad \ldots\ 2.14$$

where,

Q = Peak discharge, cu.m/sec
L = Length of weir, m
H = Flood depth, m

2.15.6 Stability of Weir

- The theoritical profile of an overflow weir is a trapezium with water face vertical, top width equal to $H/S^{0.5}$ and bottom width equal to $h/S^{0.5}$.

- There should be no tension induced at any point on a horizontal plane through the body of the weir. The line of resultant force when the reservoir is empty or full should fall within the middle third zone of the profile of the dam.

- Foundation bed should neither settle nor fail by shear under the load of the weir.

- The material used should be strong enough to prevent crusing of the body nor should any part slide under the pressure of water.

- The uplift, if any should be considered because uplift causes overturning moment.

- The weir should not crack due to the expansion and contraction caused by the temperature variation.

- The induced shear stress at the toe and heal of the weir should be less than permissible shear stress of the masonary weir.

Example 2.6

Given the following basic data, design a suitable overflow weir

Maximum permissible submergence level (MPSL)	:	139.15 m
Bed level	:	136.52 m

Catchment area (A)	:	344.00 ha
Intensity of rainfall (I)	:	135 mm/hour
Run-off co-efficient (C)	:	0.40
Silt factor (F)	:	1
Depth of hard strata below bed level	:	1.5 m

Solution

i) Maximum flood discharge (Q)

$$Q = CIA$$
$$= \frac{0.4 \times 135 \times 344}{360} = 51.6 \ m^3/sec$$

ii) Length of water way (L)

$$Q = 2.4 x L x d^{3/2}$$

d = 1.35 m (assusmed)

$$51.6 = 2.4 x L x 1.35^{3/2}$$

$$L = \frac{51.6}{2.4 x 1.35^{3/2}} = \frac{51.6}{3.764} = 13.7 \ m$$

iii) Full tank level (FTL) = MPSL-d

$$= 139.15 - 1.35 = 137.80 \ m$$

iv) Height of body wall (H) = FTL-Bed level

$$= 137.80 - 136.52 = 1.28 \ m$$
$$= 1.3 \ m$$

v) Top width of body wall

$$a = \frac{3d}{2S} \text{ but not less than } 0.5 \ m$$
$$= \frac{3 \times 1.35}{2 \times 2.4} = 0.85 \ m \text{ (emperical)}$$

Say 0.9 m

vi) Bottom width (b)

$$b = \frac{1.5H + d}{S}$$

$$= \frac{1.5 \times 1.3 + 1.35}{2.4} = 2.13 \text{ m (approx)}$$

vii) Discharge per unit width of stream (q)

$$q = \frac{Q}{L} = \frac{51.6}{13.7} = 3.766 \text{ cumec/m}$$

viii) Scour depth (R)

$$R = 1.35 \left(\frac{V}{F}\right)^{1/3} = \frac{1.35 \times (3.76)^{1/3}}{1} = 3.26 \text{ m}$$

ix) Foundation depth

Foundation depth is 1.5 R from MPSL but not less than 1 m below GL or upto the hard strata

$$1.5R = 1.5 \times 3.26 = 4.89 \text{ m}$$

Since hard foundation is expected at 1.5 m, total foundation depth including footing may be kept as 1.8 m.

Design of apron

Length of aparon required in the downstream is X = 2 H(d+h) but not less than X = 2 (H+d+h)

$$X = 2 \times 1.3(1.35+0)$$
$$= 2.65 \text{ m}$$

But not less than $X = 2\ (1.3+1.35+0)$

$$= 5.3 \text{ m}$$

2.16 INTAKE STRUCTURE

Intake structures are used for pumping large quantity of water from a surface water body where direct pumping from the source is not convenient. Intake structures can also be used under favourable conditions as the "pump house". A pump house consists of pumping units, control valves, electrical switch boards, etc. installed within a stable structure for their protection. Location and design of the pump house depends largely upon the site condition and the type of pump used. Location of the pump house must satisfy the following needs.

- It should be located at a stable ground

- It should be free from the actions of active silting or scouring.

- It should be easily accessible for operation, inspection and maintenance.

- It should be located above the high flood level to prevent flooding of the installation.

- It should permit installation of the pumps as close to the water body as possible to reduce the suction lift particu larly when of centrifugal pump is used.

2.16.1 Dry Pit Pump House

In this arrangement (Fig. 2.6) the pump is located in a dry sump where water is not permitted to enter. Such pump houses are constructed at a stable ground adjacent to the source. The sump should be made strong and water proof with re-inforced concrete walls and flooring. Parapet walls are to be raised above water level to prevent innundation. Horizontal centrifugal pumps directly coupled with electric motors are commonly installed.

Diesel engines can also be installed but they shall require larger space inside the chamber and special care has to be taken for the disposal of smoke and noise. Suction pipes from dry pit pump house should reach the water body directly running horizontal as far as possible. Further extension of suction pipe can be made by using obtuse angle bends or flexible hoses. The area of penetration of the suction pipe through the pit wall should be made completely water-tight by applying thick cement pastes all around.

Both the pumps and motors in dry pit installation should be mounted securedly on high platforms. This protects the installation from partial flooding of the sump due to seepage, leakage of pipes and rain water inflow. The accumulated water at the bottom of the pit should be dewatered as early as possible. Provision of a mild slope to the floor towards a collection pit facilitate accumulation of such water to a corner which could conveniently be scooped up by rope and bucket. Mobile diesel pump may have to be used for dewatering during emergency arising out of flooding due to excessive rainfall.

2.16.2 Wet Pit Pumps House

In wet pit installation the sump acts as the collector well. The pump is mounted above the water level supported by suitable structure. Vertical turbine pumps and submersible pumps are ideal for this kind of installations. In case, centrifugal pump is to be used, vertical installation of the same has greater advantage.

The advantage of wet pit installation is that the pumping element

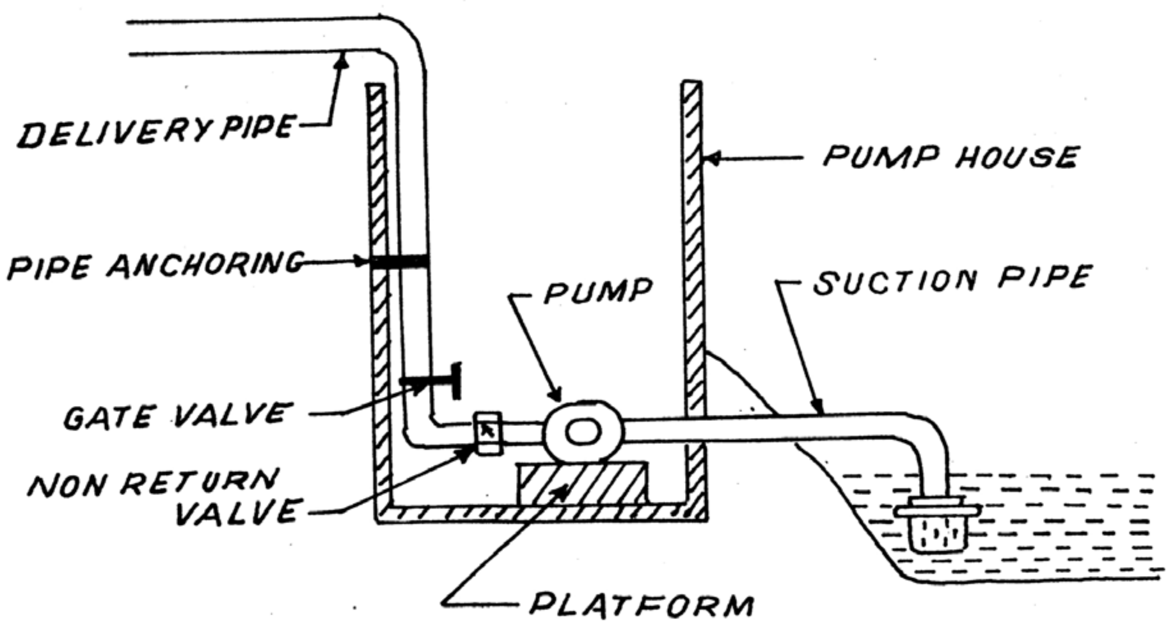

Fig. 2.6 Dry pit pump house

remain immersed within the water while the drive is mounted safely above the highest water level. This eliminates completely the limitation of installation as may be imposed due to the limiting suction lift and problem of cavitation in the use of a centrifugal pump. Certain loss on drive efficiency however takes place in such installation depending upon the nature and length of the drive arrangements.

2.16.3 Composit Pump House

Centrifugal pumps are most conveniently installed in this arrangement (Fig. 2.7). The pump is installed inside the pit but at a step like platform located higher than the water level in the pit. Further protection from inundation of the pump is done by providing an outlet pipe below the pump level so that excess water can drain out of the pit.

2.16.4 Jackwell

Seasonal streams during summer may not have an appreciable flow of water. Some streams however still can yield considerable quantity of "groundwater" when the stream bed is comprised of thick fluvial deposite.Jackwells (percolation wells) are constructed in the stream bed. The wells are properly lined with stone slabs, bricks, concrete rings etc. Provision of small "windows" in the lining covered with hard metal mesh helps in receiving water through direct percolation. The parapet wells should be made higher than the high flood level so that flood water do not enter into the well directly during the full flow season. Flood water may be allowed to enter through inlet pipes covered with wire mesh or gravel when water requirement is high.

Jackwells or intake wells can also be constructed adjacent to the stream bed in a stable ground. Water from the stream in such cases are made to flow in the well through open channels, large diameter pipes, small diameter perforated system of burried pipes etc. Pumps are installed close to these collector wells. When direct flood water is allowed to enter into the jackwells, special protective wire mesh should be placed around the foot value of the pump to prevent entry of floating materials like leaves, cloths, dead fish etc. Periodical desilting of these wells are necessary.

2.16.5 Floating Pumps

In a situation where construction of a puamp house is not feasible, pumps may be installed on a floating platform. The delivery pipe should necessarily be flexible so as to cope with the movements of the pump. Open well where water level is very deep but have an adequate yield, installation of floating pump could be useful. Horizontal directly coupled small centrifugal pumps is mounted over a float of suitable material and lowered into the well. This allows the pump to operate at a zero suction lift. The pump moves up or down with the float as water level may fluctuate. A counter weight attached with strings over a pully may be used to facilitate the up and down movement and provide stability to the float.

In case of river lift schemes, pumps could be mounted over raft, boat, burge etc. for pumping large quantity of water.These are normally done under the following site conditions.

- Seasonal fluctuation of the water level is too high.
- Water receeds far away from the bank during the lean season.
- Scouring or silting of the bank is too prominent.
- Banks are innundated widely during monsoon.
- Bank level is too high from the water level.

2.16.6 Direct Pumping

Situations where construction of inlet structures are not necessary pumping is done directly by lowering the suction pipe into the water body. Water should however be clean.

2.17 QUALITY OF IRRIGATION WATER

Plants can withstand irrigation with water having adverse qualities to a varying extent. Special attention to be given to study the quality of irrigation water are

- Total concentration of salt i.e. total dissolved solids (TDS)
- The proportion of sodium (Na) to other cations
- Presence of toxic ions

2.17.1 Total Dissolved Solids (TDS)

TDS is estimated by measuring electrical conductivity (EC) of water.

$$TDS = 0.64 \times EC \quad \text{at } 25^{o}C$$

EC is normally measured in micro-mho per centimeter at the room temperature which is later reduced to 25 degree C by using standard chart. TDS of water for most crops should not exceed 1000 parts per million (ppm).

2.17.2 Sodium

Sodium ions are normally estimated by using flame photometer. Sodium ions have a tendency to get absorbed in soil. If the irrigation water contain high amount of sodium it is likely to replace the calcium and magnesium present in the soil by a process known as base exchange. As a result soil becomes alkaline converting permeable soil to sticky clay of low permeability which when dries up forms hard lumps (Kankar pan) rendering ploughing difficult.

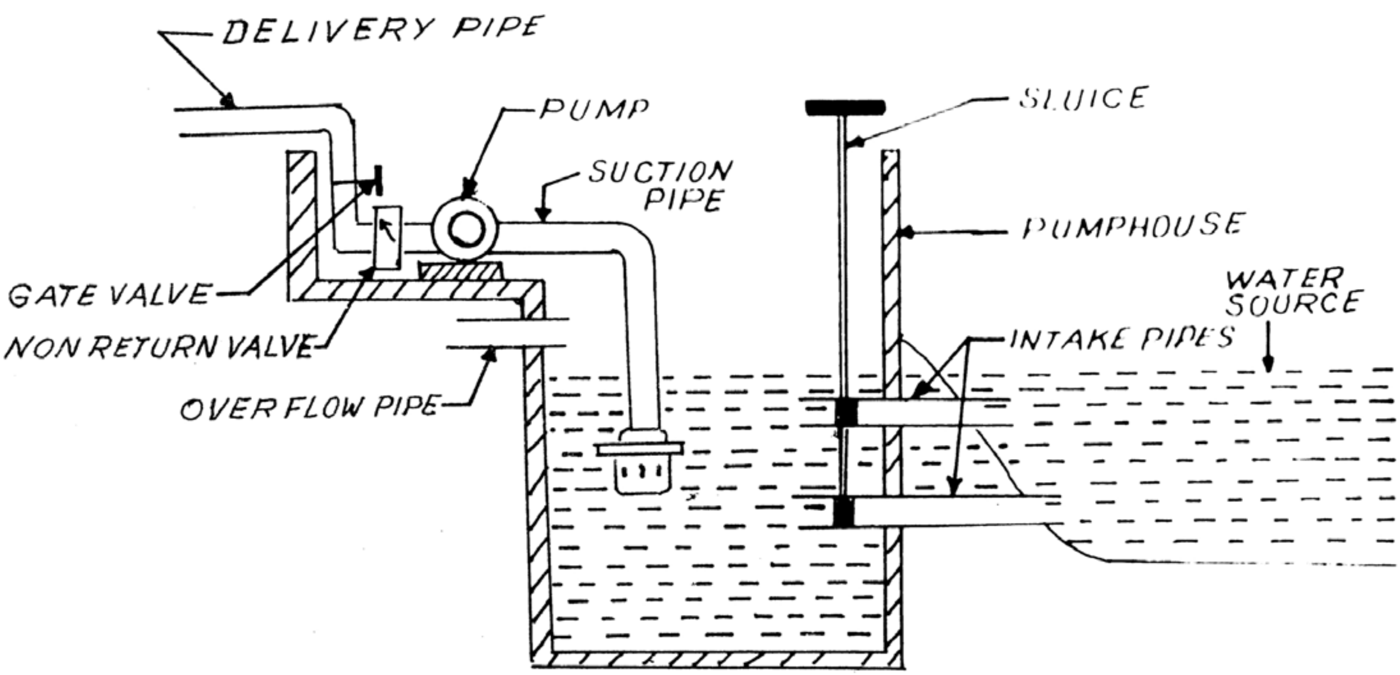

Fig. 2.7 Composit pit pupm house

The most commonly used indicator of measuring presence of sodium in soil is the sodium absorbtion ratio (SAR) which is given by

$$SAR = \frac{Na^{+}}{(Ca^{++} + Mg^{++})}$$

The SAR Level is rated as follows

0-10 : Low
10-20 : Medium
20-30 : High

Water having higher sodium content can be made useful for irrigation by adding calcium ions. Addition of gypsum to alkaline soil also helps in reducing the process of base exchange. Poor quality water can be mixed with quality water in proper proportion if available at site, for irrigation use.

2.17.3 Toxic ions

Toxic ions like Boron has an adverse effect on crops. For some crops excess Chloride, Sodium, Bicarbonate and Sulphate could also be harmful. Sulphides as such is not damagaing to crops but it has corrosive effect in the cement-concrete pipes, Sodium Chloride is however more harmful than other salts of Carbonates and Sulphates.

The recommended standards for irrigation water is presented below:

Class	Sodium	Boron ppm	TDS ppm	EC mmoh/cm	Rating
I	0-60	0 - 0.5	0- 700	0-1000	Good
II	60-75	0.5-2.0	700- 2000	1000-3000	Medium
III	Above 75	Above 2.0	Above 2000	Above 3000	Bad

For most crops Class I water is suitable, Class II water are not suitable for sensitive crops and Class III water is not suitable for irrigation at all.

CHAPTER 3

IRRIGATION REQUIREMENT OF CROPS

3.1 FUNCTION OF WATER

Irrigation is the practice of applying water by artificial means to the soil for sustaining plant growth. The function of irrigation water are many.

- Water acts as a solvent to dissolve the soil nutrients which the plants absorb through their roots.
- It replenishes the moisture lost by plants through the process of evapotranspiration, necessary for their growth.
- It aids in the growth of certain microbes in the soil beneficial to the plant growth.
- It provides the necessary cooling effect during the period of high ambient temperature.
- Extra water is required for preparation of land and raising of nursery.
- Extra water is required at times for removal of excess salts from soil by leaching.

3.2 CONSUMPTIVE USE

The quantum of water actually consumed by plants for their metabolic activities and through the process of evapotranspiration(ET) at any given time is known as consumptive use(CU). Since the water used in metabolic process is negligible, CU in practice is taken equal to ET. Consumptive use of crops actually include all water consumed by plants plus the water evaporated from the bare soil and water surface from the area under crop. In fact, direct evaportion from the cropped area cools plant body. If there is a decrease in evaporation, transpiration is increased correspondingly.

Temperature, humidity, wind condition and incoming solar radiation are some of the climatological factors on which CU depends largely. There is also a close relationship between sunshine hours and ET. For a crop at a given location the CU varies not only throughout the crop season but also from day to day. Averagae monthly CU is considered for determining water requirement of crops.

3.3 IRRIGATION REQUIREMENT OF CROPS

The quantity of water required by a crop during the entire crop period of time for its normal growth under the field condition is known as water requirement(WR) of crop. This includes the losses during Irrigation application, subsequent evaporation and water used for special needs e.g land preparation, leeching, transplantation etc. The water requirement (WR) is therefore given by

$$WR = ET\ (CU) + \text{Application losses} + \text{Special needs.}$$

The above WR can be obtained from sources like irrigation water(IR), effective rainfall(EF) and soil water contribution(S).

Therefore,

$$WR = IR + ER + S$$

or

$$IR = WR - (ER + S)$$

i.e. Irrigation water requirement (IR) is equal to water requirement(WR) less effective rainfall (ER) less available water in the soil (S).

Factors controlling irrigation requirement therefore depend largely upon the climatological conditions, soil conditions and irrigation management practices. A subjective treatment elaborating the factors in crop production is presented below :

3.4 RAINFALL

Plants meet most of their water requirement from rainfall. The need of irrigation however arises when

- There is no rainfall.
- Water available from rain is not adequate.
- Rainfall during the season may be sufficient but its distribution does not coincide with the entire need of the crop.

Rainfall in India is seasonal. It varies widely in its intensity and distribution over the country. Although, the rainfall in India follows a pattern but it is with sufficient deviation to make it unpredictable. The average annual rainfall of the country is about 120 cm. which takes place on an average of 130 rainy days.

The annual rainfall in some parts of Rajasthan is as low as 20 cm while that in some North-Eastern parts is as high as 1090 cm. The annual rainfall in areas like Amritsar, Delhi, Jaipur, Gwalior, Udaipur, Ahmedabad in the West and Aurangabad, Hyderabad, Bangalore, Pune in the central region is within 40-80 cm. Gangetic plain, Chandbali, Machhlipatnam, Nagpur and Bhopal region fall within the range of 100-200 cm. West coast extending from Bombay to Trivandrum receives about 200-400 cm. per year.

South West(SW) monsoon occuring during mid-June to mid-September is fairly wide spread throughout the country. About 80-90% of the total rainfall takes place during this period. North-east monsoon occuring during November-January is a typical feature of coastal South India.

Rainfall may also take place at any time as a result of depressions, thunder storms, cyclonic storms etc. A light winter shower is a typical feature of North India sometimes caused due to western disturbances. In India drought is said to have taken place when annual rainfall is less than the 50% of the normal. The situation is described as severe drought when annual rainfall is less than 75% of the normal rainfall.

3.4.1 Effective rainfall

When the rainfall is very little say to the order of 2-5 mm it barely moistens the soil and subsequently gets evaporated. Also in case of concentratred heavy rainfall major part of the water goes away as surface run-off after the soil is fully saturated. Excess water which flows away is not available to crops for their use. The useful part of the rainfall available insitu for crop production is the effective rainfall. It does not include water lost due to surface run-off, deep percolation below root zone and the moisture remaining in the soil after harvesting.

The effective rainfall is governed by numerous factors like distribution and intensity of rainfall, characteristics of soil and crops, land slope, carry over soil moisture, ground water contribution, surface and subsurface inflow and outflow etc.

Estimation of effective rainfall can be made easily by using standard tables when the mean monthly rainfall and mean monthly consumptive use(or pan evapometer data) are available. While determining irrigation requirement of crops, the effective rainfall, should be substracted from the total water requirement.

3.5 CROP SEASONS

Crops may be seasonal or perennial. The three major crop seasons in India are Kharif, Rabi and Summer. Crops taking more than 12 months to mature are perennial crops. Depending upon the rainfall pattern and crop variety, these seasons may vary slightly from one State to other. Months covering these seasons are:-

A. In North India:

Kharif	:	June-September
Rabi	:	October-March
Zaid Kharif	:	March-June
Zaid Rabi (Late winter crop)	:	January-April

B. In Deccan area:

Kharif (Monsoon)	:	Mid June-Mid October
Rabi (Winter)	:	Mid October-Mid Feb.
Summer	:	Mid Feb- Mid June.

In South India Kharif and Rabi seasons are not very distinguishable due to the total rain being distributed between these two seasons.

It is obvious that summer crops depend completely on Irrigation and Rabi crops on part rainfall and part irrigation. Kharif crops take full advantage of rainfall. Kharif crops however require Irrigation when monsoon fails. Water requirement of crops depend largely upon the season, crop period and their stages of growth. Crops like paddy can be grown in all seasons if Irrigation facilities are made available. Paddy consumes maximum quantity of water not only for its own use but also for the preparation of land and nursery.

As paddy and wheat are the main crops , the following cropping pattern are common in India which has been evolved considering primarily the rainfall.

Kharif	Rabi	Summer
Paddy	Wheat/maize	Pulses
Paddy	pulses	Paddy
Paddy/maize	Pulses/vegetables	--
Jawar/Bazra	Wheat/vegetables	--

Cash crops like sugarcane, cotton, tobacco, jute,oil seeds etc., are taken as per local practices in areas where supplimentary irrigation is available.

3.6 SOIL CHARACTERISTICS

Soil is basically a composite mixture of sand, silt, clay and organic matters. It may be divided into several groups based on their mode of classifications. The criteria of classification include age, occurence, geological factors, land use feasibility, chemical composition, structure, texture, chemical reactions etc. Study of soil in terms of its texture and chemical reactions have more relevance in determining crop water requirements.

3.6.1 Texture of soil

The texture of soil is determined by the percentage composition of sand, silt and clay, the constituent mineral matter of the soil. Sand, silt and clay which are determined by their particle size occuring in various proportions lead to the broad classification as detailed below:-

Table 3.1 Broad classification of soil texture

Diameter of particles	Texture of soil	Composition of typical soil		
		Sandy loam	Clay loam	Clay
mm		%	%	%
2.0 upwards	Gravel	-	-	-
2.0 - 0.2	Course sand	65	30	1
0.2- 0.02	Fine sand	20	30	9
0.02 - 0.002	Silt	5	20	25
Below 0.002	Clay	10	20	65

A more detailed classification of soil texture is however available covering all classes like sandy, sandy loam, silty loam, silty clay, loam, sandy loam, clay loam, silty clay, sandy clay and clayey. The broad classification presented above however suffice for common field practices.

3.6.2 Chemical reaction of soil

Common soil is a composite mixture consisting of mineral(inorganic)matter, organic matter, water and air.

The general chemical composition of a standard soil is as follows:

Radicals	Percent
Total carbon	0.6
Total nitrogen	0.06
Total phosphate	0.02
Total potasium (K_2O)	0.22
Total lime	0.1
Silica (SiO_2)	75.0
Iron and Allumina (Fe_2O_3+Al_2O_3)	15.0
Moisture etc.	9.0

The various chemical reactions of soil normally studied in the laboratory for the purpose of crop production are their nutrient contents, negative log of hydrogen iron concentration(pH) and total dissolved salts (TDS).

The nutrient content studies are restricted mostly to the available N, P, K and organic carobon. An indication on logarithmic scale of the degree of ionizaation (PH) indicates the degree of acidity or alkalinity of the soil. The scale is taken from 0-14. A neutral soil has PH value equal to 7. Below 7 the soil is acidic and above, it is alkaline. The acidity or alkalinity of soil is also a function of exchangable sodium percentage. Neutral soil has exchangable sodium percentage to the order of 15. Above 15 soil is alkaline(sodic). Total soluble salts on the other hand control the acidity. Total soluble salt in soil is obtained by measuring its electrical conductivity(EC) in millimhos/cm. The EC value of normal soil is below 1 and abnormal soil is above 4. The rating chart of soil test data is presented below for ready reference.

Nutrients	Low Kg/Ha	Medium Kg/ha	High Kg/ha
Available Nitrogen(N)	Below 280	280-560	Above 560
Available Phosphorus(P)	Below 10	10-25	Above 25
Available Potasium (K)	Below 110	110-280	Above 280

PH	Remarks	EC	Remarks
Below 6	Acidic	Below 1	Normal
6-8.5	Normal to saline	1-2	Critical for germination
8.5-9	Tending to become alkaline	2-4	Criticl for growth
Above 9	Alkaline	Above 4	Injurious to most crops

3.7 SOIL WATER

Subsurface water can broadly be divided into two parts namely the zone of aeration and the zone of saturation which are separated by the water table. Soil water is confined to the zone of aeration where both water and air occurs together. It includes gravitational water, capillary water and hygroscopic water.The water in the zone of saturation is called groundwater. Soil water is finally discharged into the atmosphere by evaporation from the soil or by transpiration through plants.

Gravitational water though readily available to the plants normally do not remain in place for long. It drains out by the force of gravity from the soil under favourable drainage condition. However, if it remains for long period in the soil, serious damage to the plants may take place due to lack of oxygen and accummulation of carbon dioxide in the soil.

Capillary water is retained in the soil by the force of surface tension after draining out of the gravitational water. It is held in the pore space as a continuous film around the soil particles which can be removed by applying force higher than the capillary force. Part of this water is available to plants.

Hygroscopic water is that amount of water which a dry soil will tend to absorb in order to reach to a staticstate with the atmospheric water at a particular ambient temperature. It is held by the force of molecular attraction and can be removed only by heating. This water is not available to plants.

Water holding capacity (a function of porosity) of different soils are different. The amount of water retained by coarse sand is around 5% of its dry weight while a loamy soil may retain 35% or more. Clay retains still a higher quantity which is about 45% of its dry weight.

3.7.1 Field capacity

It is also known as moisture equivalent or the water holding capacity of the soil. It is the moisture content that will be held in pores after excess water is drained out by gravity. The capillary and hygroscopic water together represent the field capacity of the soil. When expressed in percentage it is equal to specific retention i.e the ratio of weight of water retained in certain volume of soil after drainage by gravity has taken place and the weight of the same volume of soil multiplied by 100.

3.7.2 Wilting point

It is the moisture content of soil at which plant no longer obtain sufficient moisture to survive. If moisture content goes beyond this point, plants wither permanently and cannot be brought back to life even if water is added afterwards. The amount of moisture left in the soil after a plant has permanently wilted is called the wilting co-efficient of the soil or the permanent wilting point. Wilting point in most soil is in the region of 50% of the field capacity.

3.7.3 Available moisture content

The difference in the moisture contents between field capacity and wilting point is known as Available Moisture Content. It is therefore the moisture content upto the wilting point which is meaningful to plants. Readily available soil moisture is that portion of available soil moisture which is most easily extracted by plants. It normally varies from 75-80% of Available Moisture.

It may be noted that as the moisture content in soil decreases, the tension with which water is held in soil increases. Consequently plants cannot extract adequate moisture for maxium growth. The relationship between percentage of available moisture and soil moisture tension for three standard soils are shown in Fig. 3.1.

In this moisture release figure, the moisture content is expressed as a percentage of available moisture. The field capacity is taken 100% of available moisture, while permanent wilting is taken as 0 percent (15 atmosphere). For most crops except paddy maximum allowable depletion of moisture is taken as 50 percent. Depletion of moisture beyond this level should be replenished immediately.

Crops however require different level of moisture at their different stages of growth. It is therefore desirable to monitor the moisture level and apply water as per need. As the objective of irrigation is to eliminate the limitation to crop production from lack of moisture, in practice a supply of moisture at a sufficiently low tension is made available to the root zone which is most readily available to the plant.

3.8 METHODS OF IRRIGATION

Irrigation methods from water application point of view can broadly be divided into three categories namely surface irrigation, sub-surface irrigtion and overhead irrigation.

Surface irrigation is the most common practice of irrigation where water is applied to the field by simply flooding the plots. This methods of irrigation is resorted to when sufficient water is available at a low cost. In wild flooding methods there is a little application control while some controls are feasible in border flooding, check flooding and basin flooding applications. In "furrow method" water is made to flow through suitably sized and spaced furrows from a trunk channel travelling across the furrows from a higher to lower elevation. Crops are planted in the ridges in between the furrows. In hilly region water can be made to flow from plot to plot which are closely terraced and are collected by a common drainage channel for disposal.

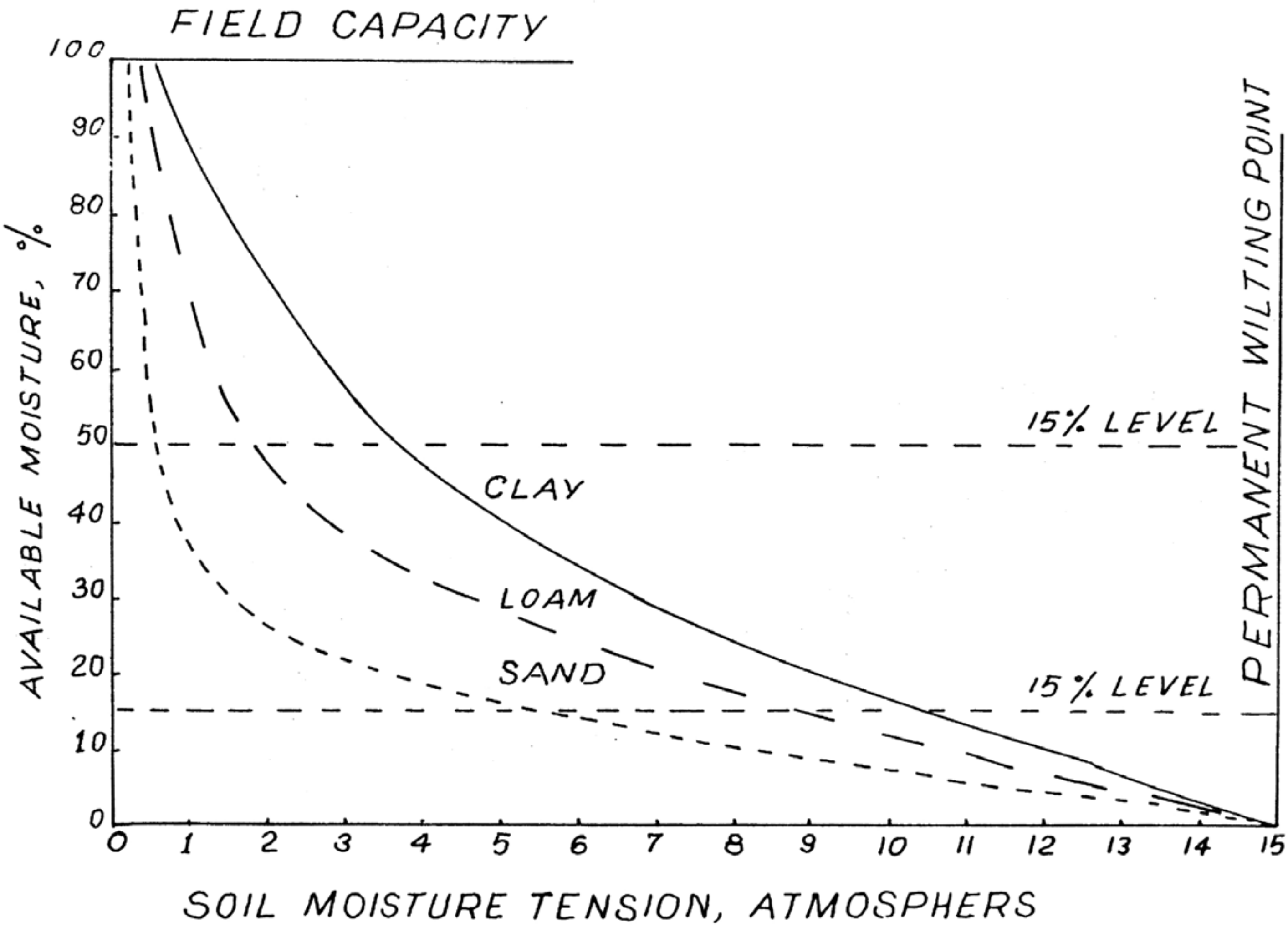

Fig. 3.1 Soil moisture tention in different soils

In sub-surface irrigation water is supplied directly to the root zone of the plants. Normally water is applied to a depth of 30-75 cm below ground level. Moisture moves upward by capillary action to meet the requirements in the plant root. Water is applied in drips at a controlled rate through buried pipe system or by other improvished means thus saving a considerable amount of water.

In overhead irrigation, water is applied by sprinkling the same over the crops to stimulate a condition similar to rainfall. Sprinkler system of irrigation is useful in saving considerable amount of water and in irrigating undulating land uniformly with less labour. It is not very suitable for use when water is unclean , infilteration rate of the soil is low and wind velocity is very high. The initial investment of a commercial sprinkler system is high but can be proved economical in the long run if managed properly.

3.9 IRRIGATION EFFECIENCY

It is the ratio of the actual amount of water available to the crop and the total quantity of water applied for irrigation expressed in percentage i.e. total irrigation efficiency is the combined result of both conveyance efficiency and water application efficienty.

$$Ei = (Wc/Wr) \times 100$$

Where,

Wc =Irrigation water consumed by crop during its growth period
Wr =Irrigation water supplied at source during the growth period

3.9.1 Net Irrigation Requirement

The amount of irrigation water required to be applied to the root zone depth of the plants so as to bring the soil moisture content back to its field capacity is Net Irrigation Requirement (NIR).

3.9.2.Gross Irrigation Requirement

While applying water to the root zone, certain amount of water is lost during conveyance and application. The gross quantity of water to be supplied at the source to meet the net irrigation requirements of crops taking into account the intervening lossses is Gross Irrigation Requirements (GIR).

3.10 IRRIGATION PRACTICES OF SOME PRINCIPAL INDIAN CROPS

3.10.1 Paddy

It consumes water many time more than any other crops. It is grown during all seasons in our country. Clay-loam to loam soils are ideal. Maturity period of paddy ranges between 100-180 days.

Some high yielding varties even matures in 90 days. The paddy varities of different seasons are known as:

March - June	Aus
July - November	Aman
December - March	Boro.

Nearly 3.5 cm water is required for seedlings and nursery. Pudling and transplantation require another 20-30 cm of water. Total water requirement varies between 185-240 cm. depending upon the area and desired depth of submergence. Yield of paddy has been found to vary depending upon submergence of the water applied. Submergence of 3-7cm throughout the growth period is conductive to higher yields. Direct seeded paddy has low yeild and also consumes about 30% more water. The nursery takes about 15-21 days to form the seedling which is transplanted as crop.

The various stages of growth after transplantation are as below:

Stages	**Days after Sowing**
Initial tillering	10-20 days.
Maximum tillering	20-30 days.
Jointing	30-50 days
Boot	50-60 days.
Flowering	60-70 days.
Grain development	70-90 days

3.10.2 Wheat

This crop is grown in quantity next to paddy. Well drained clay loam, loam and sandy loam soils are suitable for growing wheat. The crop season is normally Oct/Nov. to Jan/Feb. About 4-9 irrigations are necessary with about 4.5-7.5 cm of water in each application.

The various stages of growth are:

Stages	**Days after sowing**
Crown root initiation	20-25 days.
Tillering	40-45 days.
Jointing	55-60 days.
Boot/flag leaf	70-75 days
Flowering	85-90 days
Milk	100-105 days
Dough	115-120 days

3.10.3 Maize

Maize can grow through out the year in favourable climatic conditions in fertile and well drained loamy soil. While Kharif maize is rainfed, maize in other seasons require irrigation.Growing period is normally 100-120 days.

The states of growth are:

Stages	Days after sowing
Vegetative stage	20-40 days
Tasselling and silking	45-60 days

Maize is sentitive to excess water. Irrigation schedule of maize is as below:

Kharif	2-6 irrigation	36-10 cm each
Rabi	5 irrigation	30-35 cm each
Summer	2-25 irrigation	125-10 cm each

3.10.4 Vegetables

From season point of view, vegetables can broadly be classified into two groups namely winter vegetables and summer vegetables. In each season a numerous types of vegetables can be grown. The season wise description of common vegetable types are as follows:

Type of winter vegetables	Examples
Cole crops	Cabbage, cauliflower
Root crops	Radish, Beet root,carrot.
Bulb crops	Onion
Tuber crops	Pottato
Leafy vegetables	Spinach, palak
Legume vegetables	Peas.

These vegetable have a shallow root system and are sensitive to moisture stress. They normally extract water from the upper 15-30 cm of the soil profile. It is essential for these vegetables to maintain soil moisture level near field capicity. The soil moisture at about 15 cm depth should not be allowed to drop below 70 percent of available moisture. In case of onion, irrigations should be stopped when plants begin to mature.

Summer vegetable

Crops	Examples
Lagume vegetables	French beans, cow peas
Fruit vegetables	Tomato, Brinjal, chilli, Bhindi.
Cucurbitaceous crops	Cucumbers,melons,pumpkins,gourds.
Tuber crops	Sweet potato.

These vegitables have moderate to deep root system. They can effectively extract moisture from lower part (60-120cm) of the root zone even when upper part is near the witing point.Moisture requirement of these crops can be met if average soil moisture level to the root zone is maintained to 50-80 percent.

3.11 IRRIGATION OF FIELD CROPS

The earlier discussions in this chapter presented an overview of the factors affecting water requirement of crops and soil-palnt-water relationaship. Once the water requirement of plants is understood, it becomes an easy task to schedule the irrigation supply. Field scheduling of irrigation for crops can be achieved by various approaches taking into consideration the Duty, the Dalta, the Consumptive use and Soil moisture content of the crop. The various approaches followed in practice are discussed below.

3.12 DUTY APPROACH

The capacity to sufficiently irrigate any cropped area by a given quantity of water is the duty of the water. Duty therefore denotes the irrigating capacity of an unit volume of water. Water requirement of crops is not uniform throughout its period of gowth. It is therefore customary to express duty as the extent of cropped area that can be matured by an unit volume of water supplied althroughout the base period of the crop. Base period (B) of a crop is the period for which water is to be supplied to the crop during the crop season. Therefore, if 100 Ha of cropped area is irrigated by 0.2 cumec of water supplied althroughout the base period B, then the duty of this irrigation water will be 100/0.2 = 500 Ha/cumec for a base period B. i.e.

$$\text{Duty} = \frac{\text{Area irrigated during a base period B}}{\text{Mean supply utilized}}$$

Duty is said to be high when small volume of water irrigates a large area and vice versa. Duty approach is normally used in canal irrigation system to determine required discharge of a distributory. Similarly when irrigation is done from a reservoir, the duty of water is expressed in hectare per million cubic meter. In Lift schemes, hectares per cubic meter per hour can be conveniently used as the unit of water duty.

Duty of water will depend largely upon the following factors.

- Nature of crop
- Climate and season
- Type of soil
- Useful rainfall
- Method of irrigation

Attempts can be made to determine duty of various crops under the given set of conditions. But the duty is normally considered based on established values obtained from field experience for long years. The stardard duty of some Indian crops are presented in table 3.2.

Table 3.2 Duty of some Indian crops:

Sl.No.	Type of crop	Average duty ha/cumec
1.	Rice	600-1800
2.	Maize	500-1800
3.	Sugar cane	1000-2500
4.	Rabi crops	1500-3000
5.	Wheat	1900-3500
6.	Vegetables	600-700

Duty of well water can be established only after the well is used extensively for cultivation by utilizing maximum available water keeping its safe yield in mind. The duty will be determined by dividing the area under crop with discharge of the well in cubic meter/hour.Duty of water made available under LIS is higher than that of canal water. This is because water is applied to crops only when necessary by taking advantage of rainfall and higher irrigation effieciency. Duty of water from LIS for different crops and regions are yet to be properly established but for rough estimate, the established duty value of canal water can be extended suitably.

Example 3.1

The cropped area and duty of water inclusive of rainfall under a LIS is given below. Calculates the discharge requirement of the scheme when the pump is operated for an average of 12 hr/day and average irrigation effieciency is 80%.

	Crop	**Area** Ha	**Duty Ha/cumec**
1.	Paddy	12	600
2.	Maize	10	500
3.	Rabi crops	45	1500
4.	Sugarcane	10	1000

Solution:

The discharge requirement to mature the different crops are given by area divided by duty. Thus

Paddy = 12/600 = 0.02 cumec

Maize = 10/500 = 0.02 cumec

Rabi crops = 45/1500 = 0.03 cumec

Sugarcane = 10/1000 = 0.01 cumec

Maximum discharge requirement during different crop seasons are:

Crop	Kharif	Rabi	Summer
Paddy	0.02	-	-
Maize	0.02	-	-
Rabi crops	-	0.03	
Sugarcane	0.01	0.01	0.01
Total	0.05	0.04	0.01

Maximum discharge requirement for the crops is 0.05 cumec.Considering an averge pumping of 12 hrs/day i.e 1/2 times a day, the discharge should be modified to 0.05 x 2 = 0.10 cumec.Assuming average irrigation efficency as 80% the maximum workable discharge should be 0.1/0.8 i.e. 0.125 cumec or 125 lps.

Example 3.2

Calculate the Duty of crops under a LIS supplying 0.145 cumec of water for an average of 10 hr/day during Kharif and Rabi seasons to irrigate the following

Crops	Area in Ha
Paddy	35
Wheat	15
Winter Vegetables	20

Solution

Duty of a crop is obtained by dividing the cropped area by discharge (cumec), thus

Duty of paddy = 35/0.145 = 241.37 Ha/cumec.

Duty of Wheat = 15/0.145 = 103.45 Ha/cumec

Duty of Vegetable = 20/0.145 = 137.93 Ha/cumec

Since the area is covered by 10 hrs of pumping i.e. 10/24 = 0.416 part of a day. The above duty values should be modified accordingly thus,

Duty of paddy = 241.39/0.416 = 580 Ha/cumec

Duty of Wheat = 103.44/0.416 = 248 Ha/cumec

Duty of Vegetable = 137.93/0.416 = 331 Ha/cumec.

These values are however inclusive of effective rainfall and at the existing irrigation efficiency.

3.13 DELTA APPROACH

Every crop requires certain amount of water at a fixed interval of time during its period of growth. The amount of water applied each time multiplied by the number of applications will be the total amount consumed by a crop. The volume of this water applied to a given area can be expressed by its depth (Delta) if it is imagined to stand on the field without evaporation and percolation. Thus Delta is the total depth of water applied to a cropped area during the entire period of its growth. It is normally expressed in cm. The averge value of Delta of some Indian crops are presented below.

Table 3.3 value of Delta of some Indian crops

Sl.No.	Crops	Delta(approx) cm
1.	Paddy	120
2.	Wheat	40
3.	Maize	25
4.	Vegetables	45
5.	Gram	30
6.	Fodder	22.5
7.	Sugarcane	120
8.	Tabaco	75
9.	Cotton	50
10.	Garden fruits	60

Example 3.3

Wheat requires 6 cm of water after every 20 days. If the base period of wheat is 120 days, determine its Delta value.

Solution

Base period = 120 days
Irrigation interval = 20 days
Therefore number of watering = 120/20 = 6 Nos
Depth of water required each time = 6 cm
Therefore value of Delta = 6x6 = 36 cm

3.14 ESTIMATION OF DELTA (SOIL MOISTURE APPROACH)

Soil moisture is more readily available to plants when it is at low tension i.e. near field capacity. The water content at field capacity is considered 100 percent available for crop growth and that at permanent wilting point as zero percent. Soil moisture should never be permitted to fall below the permanent wilting point. The safe limit for soil moisture depletion of various crops have been found from field trials. For most crops depletion of soil moisture at the root zone is not permitted to fall below 50 percent from its field capacity.

Consumption of water by plants are however not uniform throughout its period of growth. There is a specific critical period when the crops consume maximum amount of water (Kor period). During this period of peak moisture use, high moisture level must be maintained for high yields. It varies with the growing season depending upon crop charateristics. Averge peak moisture use for some common crops under different clmatic conditions are presented below.

Table 3.4 Average peak moisture use in cm per day

	Cool climate		Moderate climate		Hot climate		
	Dry	Humid	Dry	Humid	Dry	Humid	Periods
Wheat	.38	.30	.56	.38	.63	.56	crown root
Bajra	.38	.30	.56	.38	.63	.56	
Maize	.50	.38	.63	.40	.89	.76	
Cotton	.40	.30	-	.40	.58	.50	flowering
Groundnut	.38	.30	.53	.40	-	-	flowering
Jowar.	.30	.30	.50	.38	.58	.50	foliage
Vegetables	.38	.30	.48	.30	.58	.50	fruit
Potato	.40	.38	.50	.50	.60	.35	tuber dev.
Sugarcane	.50	-	.63	-	.89	.76	

The soil profile from which plants can extract moisture is controlled by rooting characteristics of the plants. Depth of root zone for normal crops may range between 0.5m to 2 m. The normal depth of roots of some matured irrigated crops are presented below.

Table 3.5 Depth of roots of some common crops

Shallow rooted 60cm	Moderately deep rooted 90cm	Deep rooted 120cm	Very deep rooted 180 cm
Rice	Wheat	Bajra	Citrus
Onion	Tobacco	Jawar	Grape
Cabbage	Groundnut	Cotton	Sunflower
Cauliflower	Chillies	Maize	Coffee
Lettuce	Patato	Sugercane	Lucerne

Most of the feeder roots are located near the upper part of the root zone usually within the top 45 cm. Plants do not extract moisture equally from the entire root zone. The usual extraction pattern is that about 40 percent of the extracted moisture is from the top quarter of the root zone,30 percent from the second quarter, 20 percent from the third and 10 percent from the fourth quarter of the root zone.

Application of water will also depend upon the moisture holding capacity of the soil for example in sandy and loamy soil water would tend to move downwards beyond the root zone as deep percolation. The clay soil will retain such water for a longer period. The averge moisture holding capacity for normal soils are presented below.

Table 3.6 Averge available moisture holding capacity

Soil Texture	Available moisture cm/M
Very coarse sand	4.16
Coarse sand with fine and loamy sand	6.66
Sandy loam and fine sandy laom	10.40
Sandy loam with loam, sandy clay loam	15.83
Clay loam with silty clay loam	17.49
Sandy clay with silty clay and clay	16.66
Clay	23.00

3.14.1 Irrigation Requirement

The amount of irrigation water required to bring soil moisture in the root zone depth just to its field capacity is the net irrigation requirement. Thus if the initial moisture level is at 60 percent it is necessary to add an amount of water equal to 40 percent of available moiture the soil can hold in order to bring the root zone back to field capacity.Net depth of water application in cm is obtained by using the following simplified formula.

$$Dw = Ap\ D\ \frac{(Fc-Wp)}{100} \qquad \ldots \quad 3.1$$

Where,

Ap = Apparent density of soil
D = Depth of root zone
Fc = Field capacity
Wp = Wilting point or moisture before irrigation is started

Net depth of application can be worked out if available moisture holding capacity of soil and moisture depletion level at the time of irrigation are known.

3.15 IRRIGATION FREQUENCY AND IRRIGATION PERIOD

Irrigation frequency refers to the number of irrigations applied throughout the crop period. It depends on the consumptive use rate of the crop and on the amount of available moisture in the root zone between field capacity and the starting moisture level before irrigation. It is a function of both soil and crop. Irrigation period refers to the number of days within which irrigation must be given to a crop.

It is important that irrigation period is determined specially during the peak-use period. Irrigation period should be kept less or equal to irrigation frequency. The concept of Irrigation period and frequency is useful in designing the pumping capacity for irrigation systems like sprinkler, drip etc.

Design Irrigation frequency = (Fc - Mc)/(Cup) . . . 3.2

Where,

Fc = Field capacity of soil in the effective crop root zone

Mc = Moisture content of the same at the time of starting of irrigation

Cup = Peak period moisture use rate

$$\text{Irrigation period} = \frac{\text{Net depth of water application}}{\text{Peak moisture use rate}}$$

Example 3.3

Determine the irrigation period from the given set of data

Field capacity = 30 percent
Permanent wilting point = 15 percent
Apparent density of soil = 1.3
Effective depth of root zone = 120 cm
Daily consumptive use = 1.5 cm

Solution

$$Dw = Ap\ \frac{D(Fc-Wp)}{100}$$

$$= \frac{1.3x120}{100}\ x\ (30 - 15) = 23.4\ cm$$

$$\text{Irrigation period} = \frac{23.4}{1.5} = 15\ \text{days}$$

Example 3.4

Determine discharge requirement of a pump for the following set of data

Area = 10 ha
Pumping hours = 8 hr
Irrigation efficency = 70 %
AMc of soil = 1.6 cm/m
Peak rate of moisture use = 0.4 cm
Irrigation to be applied at 50 percent of (Available Moisture) at root zone

Solution

Net depth of water per application = (16 x 50)/100 = 8.0 cm
Irrigation period =8.0/0.4 = 20 days
Depth of water to be pumped per application(GIR) = 8/0.7
= 11.4 cm
Required capacity of the pump =(11.4x10)/20 = 5.7 Ha cm/day

$$= \frac{5.7x\ 10.000x1000}{100x8x60x60} = 19.8\ lps$$

Example 3.5

Determine the discharge requirement of a pump for 8 hrs pumping a day at Purulia,W.B, for irrigating 20 Hectares of land of wheat crop in loam soil. Assume others data as practically as possible.

Solution

Area = 20 Ha

From table 3.6, AMc of loam soil is 15.83 cm/M = 16 cm/M (say)
From table 3.4, peak rate of moisture use for wheat in an area of moderate dry climate is 0.56 cm/day.

Considering soil, conveyance distance , and irrigation methods, irrigation efficiency is considered 70 percent. Water is planned to be applied at 50 percent AM at root zone.

Net depth of application = (16x50)/100 = 8 cm

Irrigation period = 8/0.56 = 14.3 day = 15 days (say)

Water to be pumped per irriigation = 8/0.7 = 11.4 cm

Pump capacity = (11.4x20)/15 = 15.2 ha-cm/day

$$= \frac{15.2 \times 10000 \times 1000}{100 \times 8 \times 60 \times 60}$$

= 52.7 lps

3.16 RELATIONSHIP BETWEEN DUTY AND DELTA

Delta is related to Duty to the extent that it represents the corresponding depth of water for a given duty. For example, let the base period of a crop be B days during which 1 cumec water is applied to the field throughout.

Volume of this water = 1x60x60x24xB = 86400B cu.m as D = ha/Cumec

Total depth of water supplied = Volume/Area = 86400B/10000D

Therefore Delta = (8.64 B)/D

Examples 3.6

An area of 20 Ha is to be irrigated by pumping water from a surface tank. Calculate the minimum storage required in the tank,when the following data are available.

Crop	**Base period days**	**Duty ha/cumec**	**Intensity of irrigation %**
Paddy	120	600	40
Wheat	150	1500	20
Sugarcane	330	1000	10

Solution

$$\text{Delta for Paddy} = \frac{8.64xB}{D} = \frac{8.64x120}{600} = 1.73 \text{ m}$$

$$\text{Delta for Wheat} = \frac{8.64xB}{D} = \frac{8.64x120}{1500} = 1.73 \text{ m}$$

$$\text{Delta for Sugarcane} = \frac{8.64xB}{D} = \frac{8.64x330}{1000} = 2.85 \text{ m}$$

The area under each crop:

Area under Paddy = (20x40)/100 = 8 ha

Area under Wheat = (20x20)/100 = 4 ha

Area under Sugarcane = (20x10)/100 = 2 ha

Volume of water required for each crop.

Paddy = 8x1.73 = 13.84 ha-m
Wheat = 4x.69 = 2.76 ha-m
Sugar cane = 2x2.85 = 5.70 ha-m

Total = 22.30 ha-m say, 23 ha-m

It may be noted that the reservoir must have minimum of following quantity of water during different seasons.

Kharif : Paddy+Sugercane = 13.84 + 5.70 = 19.54 ha-m
Rabi : Wheat + Sugarcane = 2.76 + 5.70 = 8.46 ha-m
Summer : Sugarcane = 5.70 ha-m

The conveyance, application, evaporation, percolation and other losses are not included in the above estimation.

3.17 IRRIGATION SCHEDULING

Irrigation Scheduling for field crops can be properly planed when irrigation intervals and depth of water in each application are known.

3.17.1 Delta Method

The range of water requirement of some Indian crops is furnished in the table below.

Table 3.7 Water requirement of various crops

Crop	Duration day	No. of watering recommended No.	Irrigation interval day	Depth of water in each application cm
Paddy	100-150	8-10	10-15	10
Wheat	150-180	4-6	15-20	7
Maize	120-135	8-10	20-30	5
Vegetables	120-150	10-12	10-15	5
Bajra	120-150	3-4	20-25	7.5
Gram	150-195	2-3	20-25	5
Fodder	150-180	5-6	20-30	5
Sugarcane	330-540	30-40	10-15	10
Tobacco	150-180	6-7	10-15	5
Cotton	120-165	6-7	10-15	7.5
Groundnut	180-225	30-35	10-15	6

Scheduling can be done on daily basis, monthly basis and emperically.

Example 3.7

Calculate discharge requirements during Rabi season if a pump is to run (a) 8 hr/day and (b) 12 hr/day for irrigating the following field crops.

Wheat = 40 ha.
Vegetables = 20 ha.
Sugarcane = 10 Ha

Solution

Referring to table 3.7 and considering other local conditions the following values are noted.

Wheat

Irrigation interval = 15 day
Depth of each water = 7 cm

Vegetables

Irrigation interval = 12 day
Depth of each water = 5 cm

Sugercane

Irrigation interval = 12 day
Depth of each water = 10 cm

Area that can be irrigated per day with the above schedule are

Wheat 40/15 = 2.66 ha/day

Vegetable 20/12 = 1.66 ha/day

Sugarcane 10/12 = 0.833 ha/day

Quantity of water required per day are

Wheat = 2.66x7 = 18.62 ha-cm
Vegetable = 1.66x5 = 8.30 ha-cm
Sugarcane = 0.833x10 = 8.33 ha-cm

Total 35.25 ha-cm /day

Required discharge capacity of pump for

a) 8 hrs pumping $= \frac{35.25 \times 10.000}{8 \times 60 \times 60 \times 100} = 0.122$ cumec = 122 lps

b) 12 hrs pumping $= \frac{35.25 \times 10000}{12 \times 60 \times 60 \times 100} = 0.0815$ cumec = 81.5 lps

3.17.2 Emperical method

The discharge requirement of a pump can also be estimated quickly by using the following emperical relationship.

$$Q = \frac{28}{R} \frac{A}{T} I \quad \ldots \quad 3.3$$

Where,

Q= Required discharge, lps
A= Cropped area, Ha
I= Depth of irrigation, cm
R= Rotation of irrigation, days
T= Working period of pump, hr

Example 3.8

Calculate the discharge requirements of a pump operating 8 hrs/day to irrigate 20 Ha of wheat to apply 7 cm of water after every 15 days.

Solution

Here A=20, I=7cm, R=15, T= 8hr

Therefore, Q =(28x20x7)/(15x8) = 32.66 lps

Example 3.9

Compute discharge requirement of a pump operating for 8 hrs/day to irrigate the following crops using the emperical formula.

Solution

Total Area = 70 Ha

Wheat = 40 Ha

Vegetables = 20 Ha

Sugarcane = 10 Ha

From table 3.7 We have

Wheat :I = 7cm, R=15 days

Vegitables:I = 5cm, R=12 days

Sugarcane :I = 10cm, R=12 days

$$Q\ \text{Wheat} = \frac{28 \times 40 \times 7}{15 \times 8} = 65.33 \text{ lps}$$

$$Q\ \text{Vegetables} = \frac{28 \times 20 \times 5}{12 \times 8} = 29.16 \text{ lps}$$

$$Q\ \text{Sugarcane} = \frac{28 \times 10 \times 10}{12 \times 8} = 29.16 \text{ lps}$$

Total = 123.65 lps

Example 3.10

Compute the discharge requirement of pump on monthly water requirement basis for an cropped area of Wheat = 40 ha, vegetables = 20 ha and Sugercane = 10 ha

Solution

The pertinent data is presented in tabular form in table 3.8

Table 3.8 Total water requirement obtained from Delta approach taking into account effective rainfall

	Wheat 40 ha			Vegetables 20 ha			Sugercane 10 ha			
Mon	1*	2*	3*	1*	2*	3*	1*	2*	3*	Total ha-cm
JAN	7	2	560	5	2	200	10	3	300	1060
FEB	7	1	280	-	-	-	10	3	300	580
MA	-	-	-	-	-	-	10	3	300	300
APR	-	-	-	-	-	-	12	3	360	360
MAY	-	-	-	-	-	-	12	3	360	360
JUN	-	-	-	-	-	-	10	2	200	200
JUL	-	-	-	-	-	-	-	-	-	-
AUG	-	-	-	-	-	-	-	-	-	-
SEP	-	-	-	-	-	-	10	2	200	200
OCT	7	2	560	5	3	300	10	2	200	1060
NOV	7	2	560	5	3	300	10	3	300	1160
DEC	7	2	560	5	3	300	10	3	300	1160

1* = Depth of each irrigation, cm
2* = Number of irrigations, No.
3* = Volume of water (cm*No.*ha), ha-cm

The highest total annual water requirement is 1160 ha-cm during the months of NOV and DEC.

The discharge requirement of the pump operating :

(a) For 8 hr/day is $= \frac{1160 \times 10,000}{30 \times 8 \times 60 \times 60 \times 100} = 0.134$ cubic meter/sec =1341 lps

(b) For 12hr/day is $= \frac{1160 \times 010,000}{30 \times 12 \times 60 \times 60 \times 100} = 0.0895$ cub. meter/sec =89.51 lps

3.18 CONSUMPTIVE USE

Since the actual quantity of water consumed by plants for their metabolic activities are insignificant, evapotranspiration (ET) is taken equal to Consumptive use(CU).ET can be determined by installing in the field a standard USWB class A pan evaporameter. Installation of such evaporameter at a large number of sites are however not feasible. Also it is difficult at times to measure evapotranpiration directy under all field conditions.

For these reasons indirect estimation of ET has been presented by Blaney - Criddle, Thornthwaite, Penman, Christensen and others relating ET (CU) with climate. Major constraint in using these forumulea are however the nonavailability of reliable climatological data. Attempts have been made to use these formulea from climatological data collected for other purpose with necessary modifications. Blaney-criddle formula is however, the simplest for general application. It was observed by blaney-Criddle that CU of a growing crop is closely related to mean monthly temperature and day light hour. The relationship is given by

$$Cu = Kf = \frac{ktp}{100} = u \qquad \ldots \quad 3.4$$

Where,

- K = monthly crop co-efficient
- t = mean monthly temperature,degree ferinhit
- p = monthly daylight hours expressed as percentage of day light hour of the year
- u = monthly consumptive use,inches

The values of coefficient K for use in Blaney - Criddle formula as supplied by Dastane (1972) are presented in table 3.9 and the values of monthly percentage of the daylight hour (p) for the year at different lattitudes are presented in table 3.10.

The temperature is available from the records of local meteriological stations. With the known values of the above parameters, the monthly consumptive use CU can be computed from the above equation. Monthly CU multiplied with crop period gives the seasonal consumptive use.In metric system.

$$CU = K\ (p\ (0.46t + 8.13)\ ) \qquad \ldots \quad 3.5$$

Table 3.9 Monthly crop co-efficient (K) for use in Blaney - criddle formula

Crops / Months	Rice	Wheat	Maize	Sugercane
January	-	0.50	-	0.75
February	-	0.70	-	0.80
March	-	0.75	-	0.85
April	0.85	0.70	0.5	0.85
May	1.00	-	0.6	0.90
June	1.15	-	0.7	0.95
July	1.30	-	0.8	1.00
August	1.25	-	0.8	1.00
September	1.10	-	0.6	0.95
October	0.90	0.70	0.5	0.90
November	-	0.65	-	0.85
December	-	0.60	-	0.75

Table 3.10 Monthly percentage of day light hours for lattitude 8 degree to 36 degree covering India.

Lattitude N	JAN	FEB	MAR	APR	MAY	JUN	JLY	AUG	SEP	OCT	NOV	DEC
8	8.13	7.41	8.45	8.39	8.75	8.51	8.77	8.70	8.25	8.31	7.89	8.11
10	8.11	7.40	8.44	8.43	8.81	8.57	8.84	8.74	8.26	8.29	7.89	8.08
12	8.08	7.40	8.44	8.43	8.84	8.64	8.90	8.78	8.27	8.28	7.85	8.05
14	7.98	7.39	8.43	8.44	8.90	8.73	8.99	8.79	8.28	8.28	7.85	8.04
16	7.94	7.30	8.42	8.45	8.98	8.98	9.07	8.80	8.28	8.24	7.72	7.90
18	7.88	7.26	8.40	8.46	9.06	8.99	9.20	8.81	8.29	8.24	7.67	7.89
20	7.73	7.26	8.20	8.52	9.14	9.22	9.25	8.95	8.30	8.19	7.58	7.88
22	7.76	7.22	8.41	8.57	9.22	9.12	9.31	9.00	8.30	8.13	7.50	7.56
24	7.58	7.17	8.40	8.60	9.30	9.13	9.41	9.05	8.31	8.10	7.43	7.46
26	7.49	7.12	8.40	8.64	9.37	9.30	9.49	9.10	8.32	8.06	7.36	7.35
28	7.40	7.02	8.39	8.68	9.46	9.38	9.58	9.56	8.32	8.02	7.27	7.27
30	7.30	7.03	8.38	8.72	9.53	9.49	9.67	9.22	8.34	7.99	7.19	7.14
32	7.20	9.67	8.37	8.72	9.63	9.60	9.77	9.28	8.34	7.93	7.11	7.05
34	7.10	6.91	8.36	8.80	9.72	9.70	9.88	9.33	8.36	7.90	7.02	6.92
36	6.99	6.86	8.35	8.85	9.31	9.83	9.99	9.40	8.36	7.85	6.92	6.79

Example 3.10

Compute the consuptive use (Cu) of Sugarcane crop at Ahmednagar district (18 degree North) using Blaney - Criddle formula.

Solution

Month	Mean monthly temperature degree F	Monthly crop coefficient K (Table 3.9)	Percentage daylight hr p (Table 3.10)	Monthly consuptive use CU inches (CU=Ktp /100)
January	70.43	0.75	7.88	4.16
Feburary	73.58	0.80	7.26	4.27
March	79.61	0.85	8.40	5.68
April	84.65	0.85	8.46	6.06
May	85.82	0.90	9.06	6.99
June	81.41	0.95	8.99	6.95
July	76.82	1.00	9.20	7.06
August	76.28	1.00	8.81	6.72
September	77.00	0.95	8.29	6.06
October	77.99	0.90	8.24	5.78
November	73.22	0.85	7.67	4.77
December	69.89	0.75	7.89	4.13

Total annual consuptive us = 68.65 inches
= 174.37 cms

Consuptive water use of crop is not uniform.It varies throughout the season , month and even day at a given location.Daily transpiration rate is lowest just before the sunrise which reaches to a maximum shortly before noon.Consuptive use is low at the start of growing season, increases as plant foliage develops and days become longer and warmer.It generally reaches a peak during fruiting period and then rapidly declines to the end of the growing season.

The average daily water use rate of the highest consuptive use of the season which occurs for a few days (6-10 days) is called peak period use rate.This is the design rate to be used while sceduling for irrigation water.In shallow soils or in soils with low water holding capacity or for plants with shallow root system, the peak use period ranges from 3-6 days. The peak use period for plants with moderately deep root system growing in deep soils with good water holding capacity may range from 8 to 15 days.

3.19 RELATIONSHIP BETWEEN EVAPOTRANSPIRATION AND EVAPORATION

Attempts have been made in the field by using standard class A pan evaporameter to estimate evapotranspiration of crops.The general relation is

$$ET = K \; EP \qquad \ldots \; 3.6$$

Evaporation can also be estimated by using various curves available for this purpose.Many research stations are now equiped with evaporameter.Consuptive use estimated by multiplying the pan evaporameter data with a consuptive use coefficient K as presented for certain crops from different parts of the world is given below

Table 3.11 Coefficient K to be multiplied with class A pan evaporameter datas to obtain consuptive use (Evapotran spiration)

% of crop growing season	Wheat Ludhiana	Wheat Poona	Cotton Poona	Maize Ludhiana	Vegetables
0	0.14	0.30	0.22	0.40	0.25
5	0.17	0.40	0.22	0.42	0.28
10	0.23	0.51	0.23	0.47	0.30
15	0.33	0.62	0.24	0.54	0.38
20	0.45	0.73	0.26	0.63	0.45
25	0.60	0.84	0.35	0.75	0.50
30	0.72	0.92	0.58	0.85	0.55
35	0.81	0.96	0.80	0.96	0.58
40	0.88	1.11	0.95	1.04	0.60
45	0.90	1.1	1.03	1.07	0.63
50	0.91	1.0	1.08	1.09	0.65
55	0.90	0.91	1.08	1.1	0.65
60	0.89	0.80	1.07	1.11	0.65
65	0.86	0.65	1.05	1.1	0.63
70	0.83	0.51	1.0	1.07	0.60
75	0.80	0.40	0.93	1.04	0.58
80	0.76	0.40	0.85	1.0	0.55
85	0.71	0.20	0.73	0.97	0.50
90	0.65	0.12	0.62	0.89	0.45
95	0.58	0.10	0.50	0.81	0.38
100	0.51	0.10	0.40	0.70	0.30

The total moisture content in a soil depends upon its depth and water holding capacity. The finer the texture greater is its water holding capacity.These properties govern the depth and interval of irrigation.

3.20 ESTIMATION OF EFFECTIVE RAINFALL (ER)

Method - I

In Arid and semi arid region rainfall is normally light and soil moisture level is low. A good amount of water thus can be held by soil profile to be consumptively used by crops. ER in such areas is therefore relatively high. Effective rainfall is relatively low in humid and heavy rainfall areas. The percentage of effective rainfall decreases with respect to the monthly rainfall by decerement in the following order taking 5 counsequitive dry years as base.

Monthly rainfall mm	25	50	75	100	125	150	>150
Effective Rainfall %	90	85	75	50	30	10	0

Method - II

When the rate of CU of a crop is high the available moisture in the soil gets rapidly depleted. This creates more storage capacity high rate for subsequent rain. A relationship amongst mean monthly rainfall, average monthly CU and monthly ER has been worked out by soil conservation service USA. Which is presented in table 3.12.

As the monthly effective rainfall can not exceed the consumtive use the table ends at points where they become equal to consumtive use.It should be noted that in this table, it is assumed that the soil water storage capacity at the root zone is equal to 75 mm. The correction factors to be multiplied for different other water storage capacity at the root zone are given below. this table. Thus from table 3.12, ER for 50 mm storage capacity would be;

(i) CU = 200, R = 150, ER = 120 x 0.93 = 112mm
(ii) CU = 225, R = 225, ER = 87.7x1.04 = 91.2mm

Table 3.12 Normal monthly effective rainfall as related to normal monthly rainfall and average monthly Consumptive use.

Monthly rainfall	Average Monthly Consumptive Use mm						
	50	100	150	200	250	300	350
	Normal Monthly Effective Rainfall mm						
25	17	18	20	22	25	25	25
50	33	36	40	44	50	50	50
75	47	54	58	65	74	75	75
100	50	69	75	83	95	100	100
125		83	91	102	116	125	125
150		97	106	120	136	150	150
175		100	120	136	154	172	175

200			131	148	169	191	200
225			142	162	189	210	225
250			148	175	206	226	245
275			150	188	223	242	265
300				195	235	258	288
325				199	242	275	304
350				200	245	285	320
375					248	292	328
400					250	296	335
425						298	340
450						300	343
475							346
500							349
525	50	100	150	200	250	300	350

* Based on 75 millimeters net depth of application. For other net depths, apply multiplication factors shown below.

Net Depth:	25	38	50	63	75	100	125	150	175
Factor :	0.77	0.86	0.93	0.99	1.00	1.02	1.04	1.06	1.07

3.21 SEASONAL WATER REQUIREMENT (CONSUMPTIVE USE APPROACH)

Scheduling of field crops can also be determined for the entire crop season by preparing the relevant data as shown in Example 3.11 below

Example 3.11

Compute gross irrigation requirement for Paddy

Solution

Scheduling of field crop (Paddy)

	Kharif Paddy (in cm)						
	June 1-30	July 1-12	July 13-31	Aug 1-31	Sep 1-30	Oct 1-31	Nov. 1-24
1.Mid point	15	36	52	77	107	138	165
2.GS	8.5	20.3	29.4	43.5	60.5	78.0	93.2
3.EP	28.8	38.18	12.95	19.43	17.45	21.79	15.06
4.K	1.03	1.07	1.11	1.17	1.22	1.17	1.00
5.CU	29.69	8.76	14.38	22.73	21.29	25.50	15.06
6.P	-	-	11.58	18.90	18.29	18.90	18.63
7.ER	10.31	6.99	15.80	24.79	22.63	5.08	1.75
8.NIR	19.38	1.79	10.16	16.84	16.94	39.32	27.94
9.GIR	25.84	2.39	13.55	22.45	22.59	52.43	37.25
			Total GIR = 176.5				

Where,

1. Number of days from planting to mid point of each monthly period.
2. Percentage of growing season (GS) to the mid point i.e. item 1 divided by number of days required by crop.
3. Monthly pan evaporameter (EP) values as computed.
4. K consumptive use coefficient as per table 3.9
5. Consumptive use (CU) item 3 x 4.
6. Standard percolation loss for sandy loam soil(0.0254 cm/hr).
7. Effective rainfall ER from table-3.11
8. Net irrigation requirement (NIR) item 5-7
9. Gross Irrigation Requirement (GIR) assuming 75 percent application effecienncy i.e. item 8 x 0.75

From example 3.11 the total Gross irrigation Requirement (GIR) for Paddy assuming 75% irrigation efficiency works out as 176.5 ha-cm or 17650 cu.m/ha during the entire crop season.

For a pump operating for 12 hr a day should have a discharge capacity of

$$(17650 * 1000) / (30*12*60*60) = 13.62 \text{ lps/ha}$$

Example 3.12

Compute Gross irrigational Requirement for Wheat.

Solution

Scheduling of field crop(Wheat)

cm	Nov. 14-30	Dec. 1-31	Jan. 1-31	Feb. 1-28	March. 1-16
1.Midpoint	8	31	62	92	114
2.GS	6	25	50	75	93
3.EP	6	10.7	0.9	13.5	21.4
4.K	0.40	0.84	1.00	0.40	0.11
5.Cu	2.4	8.98	9.9	5.4	2.35
6.P	6.5	-	-	-	-
7.ER	1.0	-	-	1.5	1.0
8.NIR	7.9	8.98	9.9	3.9	1.35
9.GIR(e=60%)	13.1	14.9	16.5	6.5	2.25
			Total GIR = 53.25 ha-cm		

From example 3.12 the total Gross irrigation requirement (Nov. - March) for wheat assuming 60% irrigation efficiency works out to be 53.25 ha-cm or 5325 cu.m/ha during the entire crop season.

For a pump operating for 8 hr a day should have a discharge capacity of

$$\frac{53.25 \times 10,000 \times 1000}{30 \times 8 \times 60 \times 60 \times 100} = 6.16 \text{ lps/ha.}$$

CHAPTER 4

PUMPING OF WATER

4.1 PUMPING

Pumping is the process by which extra energy is added to a fluid **to make it move** against gravity. Since energy is the capacity of **doing work**, pumping involves doing some work against gravity. **Pumping** creates a pressure head which makes water move from one point to the other.

Pumps are mechanical devices in which energy from an external source is converted into hydraulic energy. Such conversion is normally achieved by using some physical processes involving atmospheric pressure, centrifugal force, physical scooping,movement of liquid due to difference in specific gravity etc.. Power needed to drive a pump can be supplied by sources varying from low speed manual drive to high speed machinaries.

4.2 PUMPING HEADS

Head means a vertical height of water column.Since a pump has to raise water from one level to other, the various heads against which a pump has to perform (Fig.4.1)are described below.

Static Suction Head is the vertical distance between the water level (free surface) and the central line of the pump inlet(bc). When the water level is located below the suction inlet of a pump which is a common situation, the term actually used is static suction lift. Lift indicates negative head.

Static delivery head is the vertical distance between the central line of the pump and the central line of delivery outlet (cd).

Submergence is the vertical distance between the surface level of the water being pumped to the end of the suction pipe (ab). It may be noted that as pumping continues, the depth of submergence may continue to reduce thus increasing the suction lift.

Total Static head is the vertical distance between the pumping water level and discharging water level (bd). In other words, it is the sum of the Static suction lift and Static delivery Head and is the total height by which water is to be lifted.

4.3 HEAD LOSSES IN FLOW

The term " static" is used to indicate that water is not in motion. When water is made to flow through a pipe, a series of energy losses take place along the flow. The losses are normally due to the energy requirement to initiate the flow, friction met along the flow and other minor losses. Such losses take place in both the suction and delivery sides. The pumping head with the water in motion is known as dynamic head.

Dynamic suction Head is therefore the sum of static suction head (lift) and various other losses originated due to the velocity of flow, entry of water and all frictions met in the suction pipe when the water is in motion.

Dynamic delivery Head similarly is the sum of static delivery head plus all other losses encountered in the delivery pipe due to velocity, friction and exit of water.

Thus,

Total Head = Dynamic suction head + Dynamic delivery head

Various losses of energy that normally takes place in a pumping system are:

4.3.1 Velocity Head

It is obvious that to initiate the flow of water to a given velocity (V), a pump has to impart certain amount of energy to the water. This energy expressed in its equivalent height of a vertical water column (head) is known as velocity head and is given by

$$hv = V^2/2g \qquad \ldots \quad 4.1$$

4.3.2 Friction Head

As water flows through a pipe, it encounters certain resistance or friction along the inner surface of the pipe and also within the flow itself due to relative sliding motion of water layers. Friction head could therefore be defined as the energy required to overcome the frictional resistances met by water during its flow expressed in equivalent height of water column and is given by Dercy-Weisback as

$$hf = 4fL\, V^2/2gD \qquad \ldots \quad 4.2$$

where,

hf = Head loss due to friction, m
f = Friction coefficient
L = Length of pipe, m
V = Velocity of water, sq.m
g = Accelation due to gravity (9.81 m/sec square)
D = Diameter of pipe, m

The above equation indicates that:

- For a flow through a given pipe, any little change in velocity brings about a correspondingly greater change in the friction head, ($hf \alpha v^2$)
- For a given velocity of flow, friction head decreases as the pipe diameter increases and vice versa ($hf \alpha 1/D$)

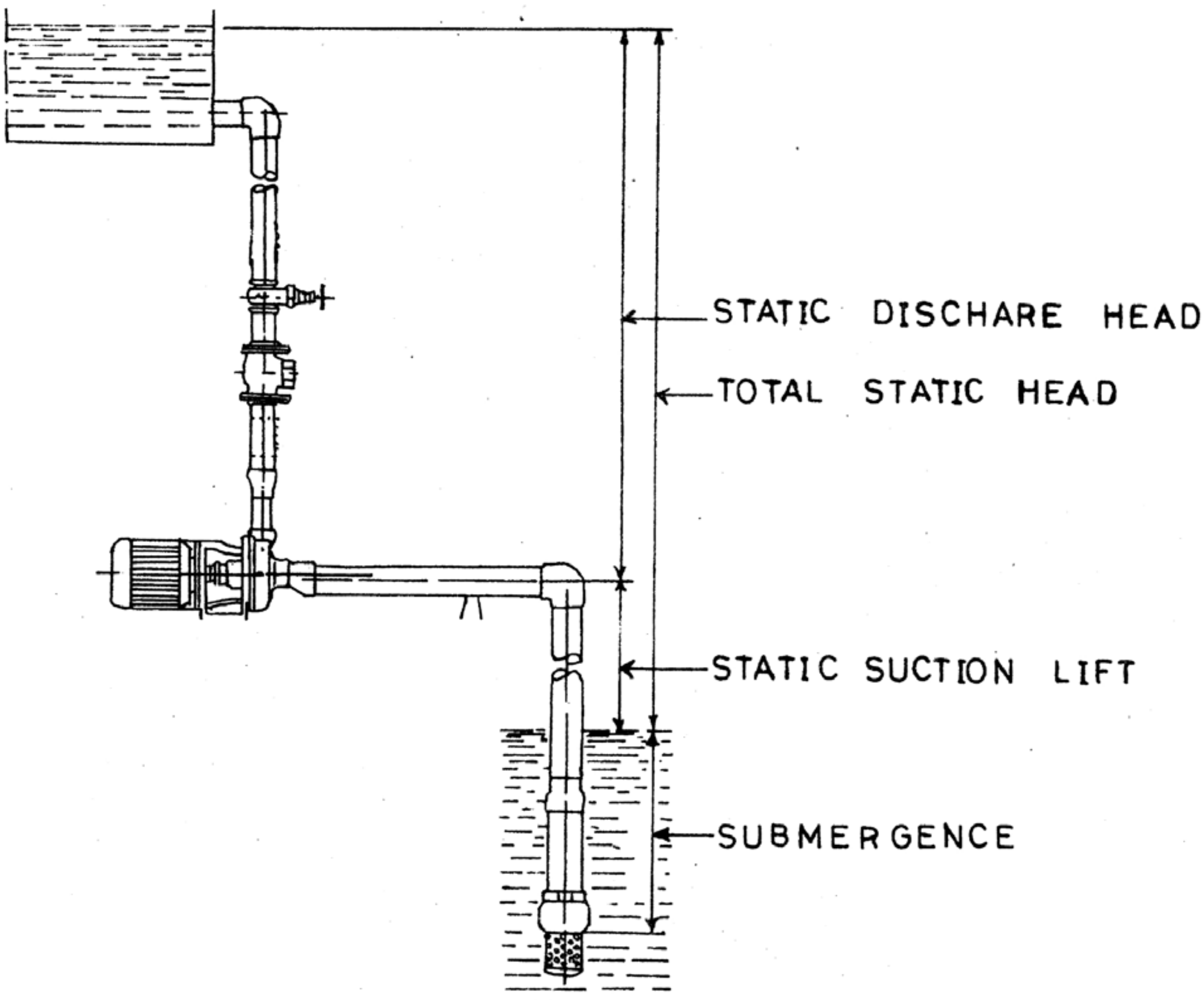

Fig. 4.1 Heads in pumping

- For a given velocity of flow through a pipe of given diameter, more the roughness of the inner surface of pipe, more is the frictional loss and vice versa ($hf \alpha f$)

- Longer the length of a pipe higher will be the frictional loss and vice versa ($hf \alpha L$)

- Frictional loss is not affected whether the pipe is laid horizontal, vertical or inclined

- Frictional loss is not affected by the pressure of water inside the pipe.

4.4 ESTIMATION OF FRICTION LOSS

When the value of all the factors in the right hand side of Eqn. 4.2 is known, the value of friction loss due to flow in a pipe (hf) can be computed . Standard value of friction co-efficient (f) adopted for pipes of different materials are:

Galvanised Iron (GI) pipe	:	0.005
Cast Iron (CI) pipe	:	0.008
Rigid Polyvynile Chloride (RPVC) pipe	:	0.001

The friction coefficient for old pipes with necessary safe margin for design purpose is taken as 0.01

The main objection in computing friction loss mathematically by using Eqn. 4.2 is in the selection of proper value of friction coefficient (f). Since the friction loss depends not only upon the pipe material but also upon the flow condition i.e. whether the flow is laminar, turbulent or transitional. For this reason frictional loss tables have been prepared by various workers from actual trials conducted in laboratories. Friction loss due to flow of water in metre per 100 metre length of commonly used pipes at different discharge rates are presented in tables 4.1 - 4.6

Table 4.1 Friction head loss in metre per 100 metre length of Cast Iron pipe

Flow	Diameter of pipe,cm				
IPS	20	25	30	35	40
20	0.25				
22	0.3				
24	0.4				
26	0.5				
28	0.55				
30	0.6	0.15			
35	0.85	0.25			

40	1.1	0.35			
45	1.5	0.45			
50	1.7	0.55			
55	2.2	0.65	0.25		
60	2.5	0.8	0.30		
65	3.0	0.9	0.35		
70	3.5	1.0	0.42		
75	4.0	1.2	0.50		
80	4.5	1.4	0.55	0.25	
85	5.0	1.6	0.60	0.30	
90	5.6	1.8	0.65	0.33	
95	6.5	2.0	0.70	0.36	
100	7.0	2.2	0.80	0.40	
120	10.0	3.2	1.1	0.50	0.25
140	14.0	4.5	1.8	0.70	0.38
160	18.0	6.0	2.0	0.90	0.45
180	23.0	7.8	2.5	1.20	0.60
200	28.0	10.0	3.5	1.70	0.70
220			4.0	1.80	0.90
240			4.5	2.25	1.00
260			5.5	2.50	1.20
280			6.5	3.0	1.40
300			7.2	3.5	1.60
350			10.0	4.5	2.25
400			13.0	7.0	2.5
450.			16.00	7.5	3.5
500			20.00	9.0	4.5
550			25.0	11.0	5.25
600			30.0	14.0	6.5
650			35.0	16.0	7.0
700			40.0	18.0	9.0
750			45.0	20.0	10.0
800			50.0	25.0	11.0
850				28.0	13.0
900				30.0	14.0
950				34.0	16.0
1000				38.0	18.0
1200				43.0	25.0

Table 4.2 Friction head losses in metre per 100 metre lenth of Galvanised Iron pipe

Flow	Diameter in cm					
lPS	5.0	10.0	15.0	20.0	25.0	30.0
1.0	1.1	-	-	-	-	-
1.2	1.6	-				
1.4	2.2					
1.6	2.8					
1.8	3.7					
2.0	4.5	-				
2.2	5.2					
2.4	6.4	0.16				
2.6	7.5	0.18				
2.8	8.7	0.22				
3.0	10.0	0.25				
3.5	13.5	0.33				
4.0	17.5	0.45				
4.5	22.5	0.55				
5.0	28.0	0.68				
5.5	33.0	0.83	0.095			
6.0	40.0	0.96	0.118			
6.5	47.0	1.15	0.140			
7.0	54.0	1.30	0.17			
7.5	62.0	1.50	0.18			
8.0		1.80	0.21			
8.5		2.00	0.23			
9.0		2.20	0.27			
9.5		2.50	0.29			
10.0		2.80	0.32			
12.0		4.00	0.47	0.10		
14.0		5.50	0.63	0.13		
16.0		7.20	0.83	0.17		
18.0		9.00	1.05	0.22		
20.0		11.00	1.30	0.27		
22.0		13.50	1.60	0.33	0.100	0.038
24		16.00	.90	0.40	0.120	0.045
26		18.70	2.20	0.47	0.145	0.055
28		22.00	2.50	0.55	0.165	0.060
30		25.00	2.80	0.63	0.19	0.070
35		33.00	4.00	0.85	0.27	0.095
40		45.00	5.20	1.10	0.33	0.128
45		56.00	6.50	1.40	0.43	0.16
50		70.00	8.30	1.70	0.55	0.22
60			11.50	2.50	0.75	0.28

65		14.00	2.90	0.90	0.34
70		16.30	3.50	1.08	0.40
75		18.50	3.90	1.20	0.46
80		22.00	4.30	1.40	0.52
85		23.00	5.00	1.50	0.58
90		27.00	5.60	1.70	0.65
95		30.00	6.30	1.90	0.72
100		33.00	7.00	2.20	0.80

Table 4.3 Friction head losses in metre per 100metre length of Asbestos Cement pipe

Flow lps	Flow lpm	Dia (O.D) in cm			
		10	15	20	25
2.5	150	0.11			
3.0	180	0.16			
3.5	210	0.21			
4.0	240	0.275			
4.5	270	0.350			
5.0	300	0.425			
5.5	330	0.510			
6.0	360	0.60			
6.5	390	0.71			
7.0	420	0.80			
7.5	450	0.915	0.13		
8.0	480	1.05	0.145		
8.5	510	1.15	0.170		
9.0	540	1.33	0.185		
9.5	570	1.45	0.205		
10.0	600	1.60	0.230		
10.5	630	1.75	0.250		
11.0	660	1.92	0.270		
11.5	690	2.10	0.295		
12.0	720	2.30	0.320		
12.5	750	2.50	0.345		
13.0	780	2.65	0.375		
13.5	810	2.90	0.405		
14.0	840	1.05	0.435		
14.5	870	3.35	0.470		
15.0	900	3.50	0.50	0.122	
16.0	960	3.60	0.535	0.188	
17.0	1020	4.30	0.640	0.155	
18.0	1080	4.85	0.710	0.175	
19.0	1140	5.20	0.800	0.195	
20.0	1200	6.00	0.880	0.210	

22.0	1320	7.25	1.05	0.25	
24.0	1440	8.60	1.25	0.30	
26.0	1560	10.00	1.45	0.35	
28.0	1680	12.10	1.65	0.395	
30	1800	13.25	1.90	0.450	0.138
32	1920	14.90	2.15	0.520	0.153
35	2100	18.00	2.60	0.605	0.185
38	2280	21.00	3.00	0.720	0.235
41	2460	-	3.55	0.820	0.250
44	2640	-	3.90	0.940	0.290
47	2820	-	4.55	1.05	0.325
50	3000	-	5.10	1.20	0.370
55	3300	-	6.10	1.44	0.440
60	3600	-	7.20	1.710	0.520
65	3900	-	8.90	2.00	0.615
70	4200	-	9.80	2.30	0.690
80	4800	-	12.70	2.95	0.880
90	5400	-	16.00	3.72	1.10
100	6000	-	19.40	4.55	1.38
110	6600	-		5.43	1.69
120	7200			6.30	1.93
140	8400			8.25	2.55
160	9600			10.50	3.40
180	10800			13.70	4.4

Table 4.4 Friction head losses in metre per 100 metre length of RCC pipe

Flow lps	Flow lpm	Dia. of pipe,cm					
		20 cm..	25 cm	30 cm	35 cm	40 cm	45 cm
6	360	0.037	0.013				
6.5	390	0.041	0.015				
7.0	420	0.048	0.017				
7.5	450	0.054	0.019				
8.0	480	0.062	0.022				
8.5	510	0.066	0.024				
9.0	540	0.078	0.027				
9.5	570	0.086	0.03				
10	600	0.096	0.033				
10.5	630	0.105	0.036				

11.0	660	0.115	0.039	0.0155			
11.5	690	0.125	0.043	0.017			
12.0	720	0.135	0.046	0.018			
12.5	750	0.145	0.049	0.02			
13.0	780	0.155	0.052	0.021			
13.5	810	0.165	0.056	0.022	0.011		
14.0	840	0.18	0.06	0.024	0.0115		
14.5	870	0.19	0.065	0.026	0.0125		
15	900	0.20	0.069	0.027	0.0132		
16	960	0.23	0.078	0.031	0.015		
17	1020	0.255	0.088	0.0035	0.017		
18	1080	0.285	0.098	0.0385	0.019		
19	1140	0.32	0.108	0.043	0.0205		
20	1200	0.35	0.118	0.047	0.023		
21	1320	0.42	0.14	0.053	0.027		
24	1440	0.49	0.165	0.066	0.032	0.016	
26	1560	0.565	0.195	0.076	0.037	0.019	
28	1680	0.65	0.22	0.088	0.043	0.021	
30	1800	0.74	0.25	0.10	0.048	0.024	
32	1920	0.84	0.28	0.0115	0.054	0.027	
35	2100	1.00	0.33	0.135	0.064	0.033	0.0195
38	2280	1.15	0.38	0.155	0.075	0.038	0.023
41	2460	1.32	0.45	0.18	0.088	0.044	0.026
44	2640	1.50	0.51	0.205	0.098	0.05	0.03
47	2820	1.72	0.58	0.235	0.115	0.056	0.034
50	3000	2.00	0.56	0.265	0.125	0.063	0.038
55	3300	2.25	0.77	0.31	0.15	0.75	0.044
60	3600	2.65	0.92	0.37	0.18	0.089	0.053
65	3900	3.05	1.05	0.425	0.205	0.105	0.061
70	4200	3.55	1.20	0.495	0.235	0.412	0.07
80	4800	4.50	1.55	0.63	0.305	0.155	.089
90	5400	5.60	1.95	0.80	0.38	0.19	0.115
100	6000	8.20	2.80	1.15	0.55	0.275	0.165
120	7200	9.60	3.30	1.35	0.64	0.325	0.195
130		11.0	3.8	1.55	0.74	0.36	.22
140		13.0	4.3	1.8	0.84	0.42	.25
150		15.0	5.0	2.05	0.98	0.48	.285
160		16.5	5.7	2.3	1.1	0.54	.32
170		18.0	6.4	2.6	1.22	0.61	.355
180		20.0	7.0	2.9	1.4	0.68	.04
190		22.0	7.6	3.2	1.5	0.76	.043
200		25.0	8.5	3.55	1.64	0.84	.485
210		27.0	9.0	3.9	1.82	0.91	.53
220		30.0	10.0	4.2	1.98	1.0	.58
230			10.8	4.5	2.12	1.10	.63
240			11.8	4.85	2.35	1.18	.68
250			12.5	5.3	2.50	1.28	.74

260	13.0	5.7	2.78	1.38	.79
270	14.0	6.2	2.9	1.45	.84
280	15.2	6.6	3.1	1.55	.90
300	18.0	7.5	3.45	1.78	1.02
350	23.0	9.6	4.5	2.3	1.35
400	30.0	12.8	6.0	3.0	1.80
450	37.0	15.8	7.3	3.7	2.1
500		19.0	9.0	4.4	2.6
550		22.5	10.8	5.3	3.1
600		26.5	12.8	6.3	3.6
650		31.0	14.8	7.3	4.2
700		36.0	16.5	8.3	4.8
750			18.5	9.4	5.4
800			21.0	10.6	6.0
850			23.5	11.8	6.7
900			26.5	13.0	7.5
950			28.5	14.0	8.4

Table 4.5 Friction head losses in metre per 100 metre length of HDP pipe

Flow lps	Flow lpm	Diameter of pipe, cm					
		5.0	7.5	10	15	20	25
1	60	0.65	0.095	0.02			
1.5	90	1.30	0.19	0.05			
2	120	2.20	0.33	0.082			
2.5	150	3.25	0.50	0.123			
3	180	4.40	0.67	0.168			
3.5	210	5.65	0.90	0.23			
4	240	6.80	1.10	0.27			
4.50	270	8.30	1.40	0.34			
5	300	11.50	1.65	0.45			
5.5	330	13.00	1.90	0.50			
6	360	14.60	2.25	0.58	0.08		
6.5	390	17.30	2.50	0.65	0.09		
7	420	20.2	2.90	0.75	0.10		
7.5	450	23.5	3.30	0.84	0.128		
8	480	27.0	3.75	0.95	0.135		
8.5	510	-	4.25	1.05	0.155	0.04	

9	540	-	4.50	1.20	0.16	0.043	
9.5	570	-	4.80	1.30	0.18	0.048	
10	600	-	5.75	1.40	0.20	0.057	
10.5	630	-	6.20	1.55	0.22	0.063	
11	660	-	6.30	1.70	0.24	0.070	
11.5	690	-	7.50	1.80	0.27	0.075	
12	720	-	8.50	2.0	0.29	0.081	
12.5	750	-	8.70	2.1	0.31	0.084	
13	780	-	9.00	2.35	0.33	0.090	
13.5	810	-	10.00	2.50	0.35	0.095	
14	840	-	11.00	2.75	0.375	0.10	
14.5	870	-	11.50	2.85	0.40	0.105	
15	900	-	12.00	3.00	0.425	0.11	
16	960	-	13.00	3.30	0.47	0.125	
17	1020	-	-	3.70	0.55	0.140	0.050
18	1080	-	-	4.15	0.60	0.155	0.055
19	1140	-	-	4.50	0.65	0.17	0.060
20	1200	-	-	5.00	0.70	0.195	0.065
22	1320			5.75	0.85	0.230	0.075
24	1446	-	-	6.9	0.95	0.27	0.09
26	1560	-	-	7.8	1.10	0.30	0.105
28	1680	-	-	9.0	1.20	0.35	0.115
30	1800	-	-	10.3	1.40	0.38	0.14
32	1920	-	-	11.2	1.50	0.42	0.15
35	2100	-	-	-	1.90	0.55	0.18
38	2280	-	-	-	2.20	0.60	0.21
41	2460	-	-	-	2.55	0.68	0.24
44	2640	-	-	-	2.80	0.78	0.27
47	2820	-	-	-	3.20	0.90	0.30
50	3000	-	-	-	3.70	1.0	0.35
55	3300	-	-	-	4.40	1.20	0.40
60	3600	-	-	-	5.25	1.40	0.30
65	3900	-	-	-	6.20	1.60	0.56
70	4200	-	-	-	7.00	1.80	0.70
80	4800	-	-	-	9.00	2.30	0.85
90	5400	-	-	-	11.00	2.75	1.00
100	6000	-	-	-	12.50	3.40	1.30
110	6600	-	-	-	15.00	4.00	1.40
120	7200	-	-	-	17.00	7.00	1.70

Table 4.6 Friction head losses in metre per 100 metre length of Aluminium pipe

Flow lps	Flow lpm	Dia of pipe, mm 75	100	125	150	175	200	250
2	120	0.042	0.10					
2.5	150	0.66	0.155					
3	180	0.90	0.22					
4	210	1.20	0.29					
5	240	1.60	0.38					
4.5	270	2.30	0.52	0.170				
5	300	2.50	0.60	0.190				
5.5	330	2.80	0.69	0.230				
6	360	3.60	0.820	0.270				
6.5	390	4.10	0.960	0.310				
7	420	4.80	1.10	0.360	0.150			
7.5	450	5.00	1.20	0.390	0.160			
8	480	6.40	1.50	0.49	0.180			
8.5	510	6.80	1.60	0.52	0.210			
9	540	7.60	1.80	0.59	0.240			
9.5	570	8.80	1.95	0.64	0.270	0.115		
10	600	9.45	2.15	0.70	0.30	0.130		
10.5	630	10.00	2.40	0.80	0.335	0.140		
11	660	11.00	2.65	0.84	0.350	0.155		
11.5	690	12.00	2.80	0.92	0.390	0.17		
12	720	13.50	3.10	1.00	0.420	0.18		
12.5	750	14.50	3.35	1.10	0.460	0.20	0.10	
13	780	15.50	3.55	1.15	0.48	0.21	0.105	
13.5	810	16.25	3.80	1.25	0.51	0.22	0.110	
14	840	17.00	4.00	1.35	0.54	0.245	0.125	
14.5	870	18.80	4.40	1.45	0.57	0.260	0.130	
15	900	20.00	4.70	1.55	0.62	0.280	0.140	
16	960	23.00	5.20	1.80	0.72	0.320	0.160	
17	1020	25.00	5.80	1.90	0.76	0.350	0.170	
18	1080		6.80	2.20	0.90	0.40	0.20	
19	1140		7.40	2.40	1.00	0.44	0.22	
20	1200		8.00	2.70	1.10	0.49	0.245	
22	1320		11.00	3.60	1.55	0.660	0.33	
24	1440		14.00	4.60	1.70	0.80	0.42	
26	1560		13.00	4.85	1.95	0.87	0.47	
28	1680		15.00	5.20	2.10	0.96	0.50	0.175
30	1800		17.00	5.80	2.40	1.10	0.525	0.195
32	1920		19.00	6.30	2.65	1.20	0.620	0.22
35	2100			7.70	3.15	1.45	0.725	0.26

38	2280	9.00	3.70	1.70	0.90	0.31
41	2460	10.00	4.35	1.95	1.0	0.35
	2640	12.00	5.0	2.30	1.2	0.42
	2820	13.5	5.5	2.60	1.35	0.48
	3000	15.0	6.20	2.90	1.55	0.50
	3300		7.50	3.40	1.80	0.62
	3600		8.90	4.1	2.1	0.74
	4200		12.00	5.6	2.85	0.90
	4800		15.00	7.2	3.70	1.25
	5400			9.0	4.75	1.60
	6000			11.0	5.80	1.95
	6600			13.0	7.00	2.30
	7200			15.5	8.20	2.80

4.5 OTHER MINOR LOSSES

When water flows through a pipe, in addition to the friction loss other minor losses also take place due to the fittings like bends, valves, joints etc. and also due to the entry and exit of water.

4.5.1 Head loss due to bends

When water flows through a a bend, it tends to move away towards the outer curve of the bend due to centrifugal force. This causes a turbulance which gradually returns to normal as the flow continues beyond the bend. Energy loss due to bend expressed in terms of equivalent head is given by:

$$h = Kb\ V^2/2g \qquad \ldots\ 4.3$$

Where Kb is the coefficient of head loss due to bend. Value of Kb will depend upon the ratio of radius of curvature (r) and diameter (D) of the bend.

Values of Kb for 90 degree smooth bend obtained from laboratory experiments are given below:

Table - 4.7 Values of Kb for 90 degree bends

r/D	1	2	4	6	10	15	20
Kb	0.335	0.19	0.16	0.22	0.32	0.38	0.42

For rough bends, value of Kb may be taken twice that of the above values. Losses in 45 degree bends are usually taken 0.5 times while losses in 180 degree bends are taken 1.25 times that of 90 degree bends.

4.5.2 Head loss due to Gate Valves

Losses due to gate valves for the different positions of valve opening will be different. Naturally loss is minimum when the gate valve is fully opened. Head loss due to gate valve is given by

$$h = Kg\ v^2/2g \qquad \ldots \quad 4.4$$

Where Kg is the coefficeint of head loss due to bend. Value of Kg decreases as the ratio d/D increases where d is the height of apparture opening and D is diameter of full valve opening. In other words Value of Kg reduces as the valve opening increases.

Table 4.8 presents values of Kg for valves of different diameter at various d/D positions.

Table - 4.8 Values of Kg for gate valves

Nominal diameter of valve mm	Ratio of height d of valve opening to diameter D of full valves opening (d/D)					
	1/8	1/4	3/8	1/2	3/4	1
25	310	32	9.0	4.2	0.90.23	
50	140	20	6.5	3.0	0.68	0.16
100	91	16	5.6	2.6	0.55	0.14
150	74	14	5.3	2.4	0.49	0.12
200	66	13	5.2	2.3	0.47	0.10
300	56	12	5.1	2.2	0.47	0.07

4.5.3 Head loss due to Foot Valve

Similarly head loss due to foot Valve (check valve) and its strainer can be computed as

$$h = Kf\ V^2/2g \qquad \ldots \quad 4.5$$

$$h = Ks\ V^2/2g \qquad \ldots \quad 4.6$$

Where Kf and Ks are coefficient of head loss due to foot valve and strainer respectively. While computing these losses, values of Kf is taken as 0.80 and that of Ks is taken as 0.95.

4.5.4 Friction loss due to Fittings

In practice however the friction loss due to pipe fittings is expressed in equivalent length of straight pipeline. For ready reckoning, equivalent length of straight pipe (in metres) for some commonly used pipe fittings are presented in table 4.9.

Table 4.9 Equivalent length of straight pipe line for different pipe fittings

Type of fittings		Pipe dia in m m						
		25 1"	50 2"	75 3"	100 4"	125 5"	150 6"	200 7"
Regular 90 degree Bend	screwed	1.6	2.6	3.3	4.0	-	-	-
	flanged	0.50	0.95	1.3	1.8	2.22	2.71	3.70
Long rad. 90 degree Bend	screwed	0.82	1.1	1.2	1.4	-	-	-
	flanged	0.49	0.82	1.03	1.3	1.52	1.74	2.13
Regular 45 degree Bend	screwed	0.4	0.82	1.22	1.67	-	-	-
	flanged	0.25	0.52	0.80	1.06	1.37	1.70	2.34
Regular 180 degree Bend	screwed	0.58	2.60	3.35	4.00	-	-	-
	flanged	0.49	0.95	1.34	1.80	2.22	2.71	3.66
Straight flow in Tee	screwed	0.97	2.35	3.66	5.20	-	-	-
	flanged	0.30	0.55	0.67	0.85	1.00	1.16	1.43
Branch flow in Tee	screwed	2.01	3.66	5.20	6.4	-	-	-
	flanged	1.00	2.01	2.86	3.66	4.57	5.49	7.31
Gate valve full open	screwed	8.84	16.46	24.08	33.53	-	-	-
	flanged	13.72	21.34	28.65	38.60	45.72	57.91	79.2
Angle valve	screwed	5.18	5.49	5.49	5.49	-	-	-
	flanged	5.18	6.40	8.53	11.58	15.24	19.20	27.4
Check valve	screwed	3.35	5.80	8.23	11.58	-	-	-
	flanged	2.20	5.18	8.23	11.58	15.24	19.20	27.4
Foot valve	screwed	3.35	5.80	8.23	11.58	15.24	19.20	27.4

Once the equivalent length of straight pipe for a particular fitting is obtained, the head loss due to friction can be further arrived at by using frictional loss tables (Tables 4.1 - 4.6) For example, to determine the friction loss due to full open 200 mm GI gate valve for a flow of 120 lps the equivalent length of pipe for gate valve as obtained from table 4.9 is 79.2m or say 80m. The head due to friction is then obtained by using table 4.1. The friction loss due to 80m length of straight pipe is (10/100)*80 = 8m.

Equivalent length of straight pipe for any pipe fitting is also obtained alternately through a standerd Nomogram available for this purpose (Fig. 4.2).To use the Nomogram , the points for different pipe fittings are joined with the corresponding values of pipe diameter by a straightline.The point of intersection of this line with the vertical line for the equivalent length of straight pipe is then read out. Once the equivalent length of straight is obtained, the corresponding friction loss is then obtained from the friction loss tables.

4.6 HEAD LOSS DUE TO SUDDEN CONTRACTION

When water is made to flow from a large diameter pipe to a many times smaller diameter pipe, a sudden contraction of flow takes place. Owing to this sudden contraction, the velocity of flow increases abruptly in the smaller diameter pipe creating a local turbulence. The energy lost is

$$h = Kc\ V^2/2g \qquad \ldots\ 4.7$$

Where Kc is the coefficient of head loss due to sudden contraction in flow. Values of Kc for a few standard combinations are presented below:

4.10 Values of Kc for sudden contraction

Velocity in Smaller pipe ft/sec	m/sec	Ratio of smaller to larger diameter								
		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
2	0.6	0.49	0.48	0.45	0.42	0.38	0.28	0.18	0.07	0.03
5	1.52	0.48	0.47	0.44	0.41	0.37	0.28	0.18	0.09	0.04
10	3.04	0.46	0.45	0.43	0.40	0.36	0.28	0.18	0.10	0.04
20	6.08	0.43	0.42	0.40	0.37	0.33	0.27	0.19	0.11	0.05
40	12.16	0.36	0.35	0.33	0.31	0.29	0.25	0.20	0.13	0.06

4.7 HEAD LOSS DUE TO SUDDEN ENLARGEMENT

When water flows from a small diameter pipe to a many times larger diameter pipe, the velocity of flow in the larger diameter pipe drops abruptly causing a local turbulence. The energy loss due to such sudden enlargement of flow is given by:

$$h = Ke\ V^2/2g \qquad \ldots \quad 4.8$$

Where Ke is the coefficient for sudden enlargement in pipes. Values of Ke are presented in table 4.11

Table 4.11 Values of Ke for sudden enlargement

Velocity in smaller pipe		Ratio of smaller to larger pipe dia.							
ft/sec	m/sec.	0.0	0.1	0.2	0.4	0.5	0.6	0.8	0.9
2	0.6	1.0	1.0	0.96	0.74	0.60	0.49	0.15	0.04
5	1.52	0.96	0.95	0.89	0.6999.5		0.41	0.14	0.04
10	3.04	0.91	0.89	0.84	0.65	0.52	0.39	0.13	0.04
20	6.08	0.86	0.84	0.80	0.62	0.50	0.37	0.12	0.04
40	12.16	0.81	0.80	0.75	0.58	0.47	0.35	0.11	0.04

4.8 ENTRANCE LOSS

This is a special case of sudden contraction. This occurs when water enters in a pipe from a large tank or reservoir. Water converges from all sides to enter the pipe. A turbulance is created at the mouth of the pipe due to its small size compared to the total water mass. Magnitude of such loss will depend upon the shape of the entrance of the pipe and its projection inside the water body.

Thus, in the following equation,

$$h = Ken\ V^2/2g \qquad \ldots \quad 4.9$$

the value of Ken is taken as 0.5. This value could be of higher order ranging from 0.5 to 0.9 when the projection of pipe in the water body is fairly long. The value will be small when the pipe mouth is bell shaped.

4.9 EXIT LOSS

This is a special case of sudden expansion. This occurs when a pipe discharges into a tank or a reservoir full of water. In this case all the velocity energy in the flow tend to come to

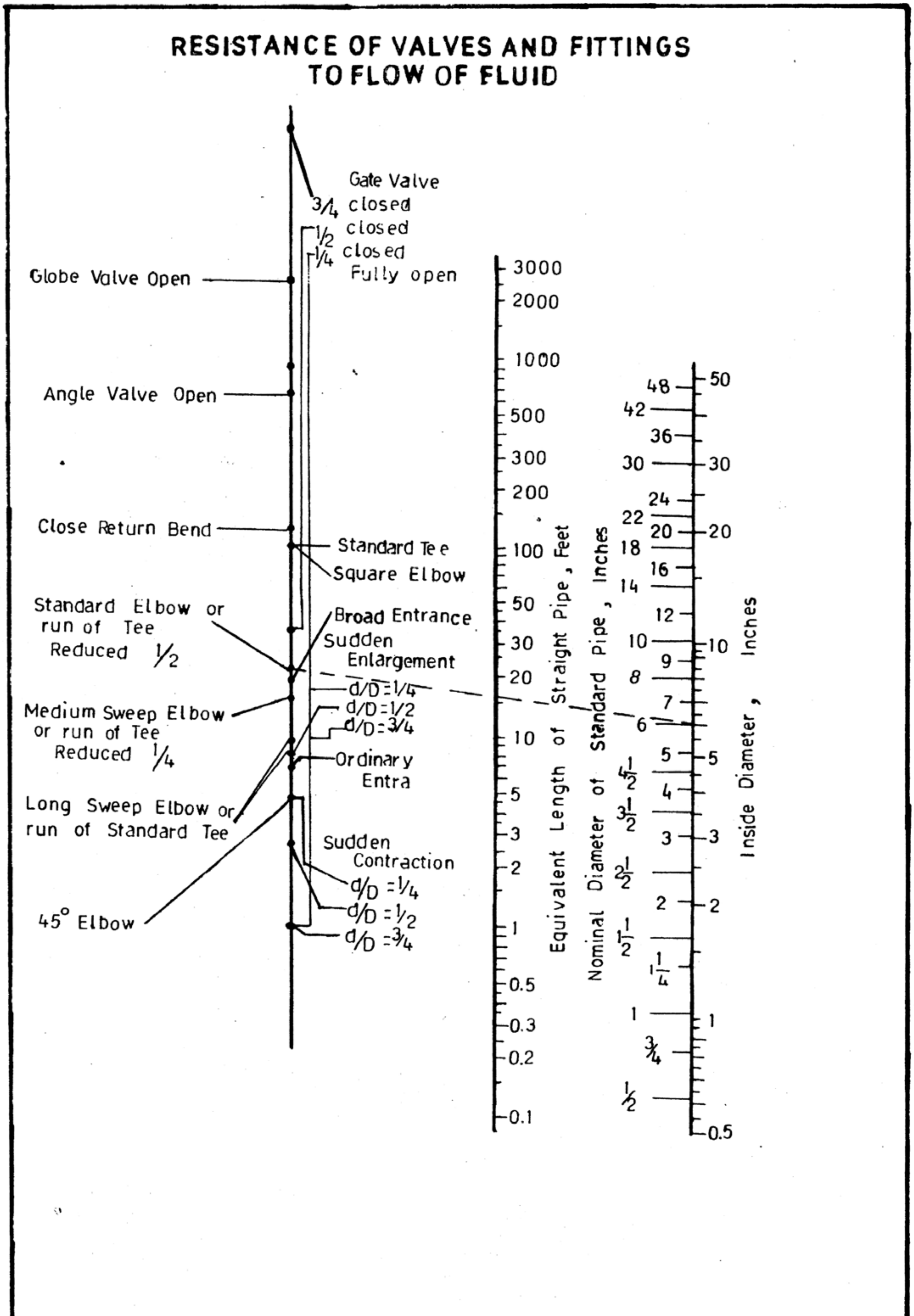

Fig. 4.2 Friction loss Nomogram for pipe fittings

zero velocity almost instantly thus causing a loss of energy.

Thus, in the equation

$$h = Kex\ V^2/2g \qquad \ldots \quad 4.10$$

Since the loss of energy is complete the values of co-efficient of exit loss is taken as 1. Exit loss is therefore equal to the velociety head in its magnitude.

All the energy losses as expressed in its equivalent height of water column (head) should therefore be added to the static head to obtain the total dynamic head under which a pump will have to work.

4.10 CAPACITY OF PUMP

Capacity of a pump is given by the volume of liquid delivered per unit time. This is also referred commonly as "discharge of pump". While a low capacity is normally expressed in litre/hr,the unit used for a high is cubic metre/sec. Other suitable units like l/min. cubic metre/hr etc. are used for intermediate range of capacity. Cusec and Gph are the widely used units of pump capacity under FPS system.

Capacity of a pump is independent of the diameter of its discharge pipe. Capacity however depends upon the design of its pumping element (impeller), suction condition,dynamic head and other operating conditions. Capacity of a given pump vary within a limit from one location to other due to varying operating conditions.

4.11 HORSE POWER

Power is the rate of doing work i.e.work divided by time by which it is done. It may be difficult to understand the concept of power without a clear understanding of work. Work (W) is given by the Force (F) multiplied by Displacement (S) i.e. W = FS.

In absolute unit therefore if one dyne of force moves a mass to the direction of Force by a distance of 1 cm, the work done is 1 dyne-cm or 1 erg. Similarly in FPS system if one pound of force moves a body by one foot, the work done is one Foot-Poundal. In gravitational unit, one gram - cm Work is done when a mass of 1 gr is lifted to a height of 1 cm. Similarly in FPS, one foot-pound work is done by raising a mass of 1 pound to a height of 1 foot.

Power is the work done per unit time i.e. FS/t. Unit time may be second, minute and hour depending upon the unit of power chosen. 550 foot-pounds of work done per second is called Horse power (H.P.) The term Horse Power was initially used by James watt in lifting water from a well using horses. In metric system however the term Horse power has been retained by giving it an equivalent value. 75kg-m of work is the nearest round figure to 550 foot pounds. Hence 75kg-m of work done per second is equal to Horse power (metric Horse power).

This 75 Kg-m of work can however be performed by a pump in a wide range of combinations e.g.

75 kg of water lifted to 1 m in one sec
37.5 kg of water lifted to 2m in one sec
7.5 kg of water lifted to 10 m . in one sec
150 kg of water lifted to 0.5 m in one sec
750 kg of water lifted to 0.1 m. in one sec etc.

It should be remembered that the term Horse power is used when mechanical work is done. Similarly power may be defined suitably when other types of work is considered. It is worth while to note the following relationships.

1 metric H.P. = 0.986 British Horse power
= 0.746 Kilo-watt

4.11.1 Water Horse Power

Let us assume that a pump is lifting Q cubic meter of water against a head of H meter. As weight **W** of One cubic meter water is 1000 Kg, the weight of Q cubic meter of water is 1000 X Q KG.

Therefore work done by the pump in the above case is 1000 x Q X H Kg-m. Now since 1 H.P. = 75 KG-m/second.

We may write

Water Horse Power = (1000 Q x H)/75 = (WQH)/75

The theoretical horse power that will be required to raise a given volume of water to a given height per second is known as Water Horse Power (WHP).

4.11.2 Brake Horse Power

In actual pumping, it is electric motor or the diesel engine that is the prime mover which supplies the WHP to a pump. There will be some power loss between the power input to the mover to its output. Similarly power loss will take place in transmitting power from the output of mover to the pump input through the transmitting system. Some power will also be lost in the pump itself between its input and output.

Brake Horse Power is therefore the actual Horsepower to be applied to the prime mover so that after accounting for all the losses, the power used by pump becomes equal to its requirement for Water Horse Power. Magnitude of BHP will naturally be slightly larger than that of WHP. The magnitude of power loss will depend upon the overall efficiency of the system i.e. Brake Horse Power = Horse power/Efficiency

or, BHP = (WQH)/75e . . . 4.11

4.12 EFFICIENCY

Efficiency is given by the ratio of output to the input of any system expressed normally in percentage. If there is no loss in a given system then the Input and Output will be the same and the system may be said to be working at 100 percent efficiency. But, it is well known that no system works with 100 percent efficiency. The input to output loss is therefore a measure of efficiency and higher the efficiency of a system lower would be such loss. Therefore:

Efficiency = Output/Input

The over all efficiency of a "pumping system" is the combination of the efficiency of the following sub systems.

Mover efficiency
Transmission or Drive efficiency
Pump efficiency

4.12.1 Mover Efficiency

For electric motor input is electircal power while output is mechanical power: Thus,

Mover Efficiency = (BHP X 100)/ KWH

For oil engines, liquid fuel is the input while mechanical power is the output.Thus,

Engine Efficiency = (BHP X 100)/Fuel consumed in lph

4.12.2 Drive Efficiency

Drive efficiency will depend upon the type of the drive used which are normally

Direct coupled drive
Flexible coupled drive
Belt drive
Gear drive

Since, for direct coupled drive there is no loss in transmission, the efficiency is 100% With other drives, the efficiency level is reduced as per the nature of the drive. Belt drive has the lowest transmission efficiency.

4.12.3 Pump Efficiency

Some minor losses also take place within the pump itself. Such losses may be categorised as:

- Hydraulic loss
- Mechanical loss
- Leakage loss

Hydraulic loss takes place due to shock or turbulence at the entrance and exit of the impeller and friction loss in the impeller, diffusers etc. Mechanical loss takes place due to the main bearing, glands and friction between impeller and water. Some leakage may take place if the packings and seals are not perfect.

The efficiency of a centrifugal pump can be expressed in any one of the following forms.

4.12.3.1 Manometric efficiency

Manometric efficiency is defined as the ratio of the total delivery head developed by the pump (manometric head) to the head imparted by the impeller to the water. This indicates the efficiency of energy conversion within the casing of the pump.

4.12.3.2 Mechanical efficiency

The mechanical efficiency is defined as the ratio of power actually delivered to water by the impeller and the power supplied to the pump shaft. This indicates losses due to coupling, pump bearing, windage etc. i.e. all losses from driving end upto impeller.

4.12.3.3 Volumetric efficiency

The volumetric efficiency is defined as the ratio of the quantity of water discharged per second from the pump to the quantity passing through the impeller per second. This accounts for all losses due to construction of the components of the pump including leakages.

Example 4.1

Determine head loss due to friction, using friction loss tables for the following flow of water.

i. 60 lps through 20 cm dia 250 m long CI pipe
ii. 600 lps through 40 cm dia 550m long RCC pipe
iii. 100 lps through 20 cm dia 180 m long HDP pipe
iv. 20 lps through 10 cm dia 60 m long alluminium pipe

Solution

i) As per table 4.1, the head loss due to friction for 60 lps through 20 cm dia CI pipe is 2.5m per 100 meters. The total friction loss through 250 m long pipe is therefore:

2.5 X (250/100) = 2.5 X 2.5 = 6.25 m

ii) As per friction loss Table 4.4 head loss due to 600 lps of flow through 40 cm dia RCC pipe is 6.3 m per 100 m per 100 m.

The total frictional loss is 550 m long pipe will therefore be

6.3 X (550/100) = 6.3 x 5.5 = 34.65 m

iii) As per Table 4.5, the frictional loss for a flow of 100 lps through 20 cm dia. HDP pipe is 3.4 m per 100 m.

The total frictional loss in 180 m long pipe will therefore be:

3.4 X (180/100) = 3.4 X 1.8 = 6.12m.

iv) As per table 4.6 the friction head loss in 10 cm dia allu-minium pipe for a discharge of 20 lps, is 8 m per 100 m. The total friction loss in a 80 m long pipe will therefore be:

8X (80/100) = 8 X 0.8 = 6.4 m.

Example 4.2

A centrifugal pump will have to work to discharge 60 lps against a static suction lift of 2.5 m and static delivery head of 20 m. The suction pipe used is a 12m long and 200 m GI pipe with one foot-valve and one long radius 90 degree bend. The delivery pipe is 150 mm dia, 125m long GI pipe attached with one check valve and one glove valve (full open) with flaged joint.Estimate the total dynamic head.

A. Friction head in suction side

Length of suction pipe	:	20.00 m
Equivalent length of one foot valve (from table 4.9)	:	27.40 m
Equivalent length of one long radius 90 degree bend (from table 4.9)	:	12.13 m
Total equivalent length	:	59.53 m

Say 60 m

From table 4.2 of frictional loss through GI Pipes at 60 lps is 2.5m per 100 meters.

Friction head loss in 60 m long pipe will therefore be

2.5 X (60/100) = 2.5 X0.6 = 1.5 m

B. Friction loss on Delivery side

Length of delivery pipe	:	125.00 m
Equivalent length of 2 nos 90 degree bends (from table 4.9 @ 2.71 m each)	:	5.42 m
Equivalent length of one check valve	:	19.20 m
Equivalent length of one glove valve	:	57.91 m
Total equivalent length	:	207.53 m

As per table 4.2 friction loss in 15 cm GI pipe for 60 lps is 7.75m/100 m.

Total head loss in 207.53 length of pipe will therefore be

7.75 X (207.53/100) = 7.75 X 2.0753 = 16 m

C. Velocity Head

Q = AV or Q = (π r2/4) v

Here,

Q = 60 lps = 0.06 cu.m/Sec and D = 150 mm = 0.15m

or, 0.06 X 4 = (3.14 X $(0.15)^2$)/4

or, V = 0.2/0.07 = 2.85 m/sec

Velocity head = $V^2/2g$

= (2.85) / 2 * 9.81 = 8.122/19.62 = 0.414 m

D. Exit loss

= Ke/2g = 0.414 m

Therefore total dynamic head:

Static suction lift	2.50 m
Static delivery head	20.00 m
Friction head on suction side	4.71 m
Friction head on delivery side	16.00 m
Velocity head	0.41 m
Exit loss	0.41 m
Total	44.03 m

Say 44.0 m

Percentage of head loss in the pipe system :

= (22.5/44.0)X 100 = 51.13%

Example 4.3

A centrifugal pump has to deliver 80 lps to a total height of 22 m.If the total friction loss in pipes from all sources be 4.2 m, calculate the Horse power requirement of the prime mover assuming that the pump operates at 70% of its efficiency.

Solution

Total Head H = 22 + 4.2 = 26.2 m
Capacity Q = 80 lps = 0.08 cu.m/sec

WHP = (WQH)/75 = (1000x.080x26.2)/75 = 27.95

Say 28.0

BHP = (WHP)/e = 28/0.7 = 40 HP

CHAPTER 5

IRRIGATION PUMPS

5.1 HANDLING OF IRRIGATION WATER

Irrigation pumps are used for delivering clear, cold and fresh water in appreciable quantity from its level of occurence to a higher level. Maximum permissible qualitative limits for such water as per Indian Standard (I.S.I) are given below :-

Turbidity	500 PPM (Silica scale)
Chlorides	500 PPM
Total dissolved solids	3000 PPM
Temperature	33 degrtee Centigrade
PH	6.5 - 8.5
Specific gravity	1.004

5.2 TYPE OF PUMPS

All pumps can be classified under two broad categories namely :

Constant Displacement Pumps
Variable displacement Pumps

5.3 CONSTANT DISPLACEMENT PUMPS

Constant displacement pumps are those which deliver water at a constant quantity as per their capacity irrespective of the head at which they operate. Rotary pump and Reciprocating (Piston) pump are the two major types that fall under this category. Since these are low capacity pumps commensurate to their sizes, they have limited use in agriculture.Variable displace pumps are those which deliver water in a variable quantity as a function of the head at which they operate. centrifugal pump, jet pump, hydraulic ram etc. are variable displacement pumps.

5.4 ROTARY PUMP

The design (Fig. 5.1) essentially comprises of two gear type impellers fitted within a housing with a close clearence for rotation. The pump housing is provided with an inlet and an outlet placed opposite to each other and an opening for the shaft at a right angle. The shaft carries drive-gear intermeshed with an idler gear (impeller).

As the shaft with the drive-gear is rotated by the prime mover in any one direction, the idler gear moves in the opposite direction. As a result the fluid is pushed up through the gap between the gears and the housing and is thrown out through the outlet. The fluid in this pump is thus physically pushed up by the continuous scooping process. As the pumping continues more fluid moves through the inlet to fill in the vaccum created due to the pumping process.

Advanced design of rotary pumps use vane type or piston type rotary elements. They are normally used in precision fluid delivery system and also in pressure application like in 'hydraulic pumps'. Rotary pump can handle water with high temperature or high vapour pressure. It is also used as booster pump in sprinkler application.

5.4.1 Features of Rotary Pump

- Operation is very simple.
- Uniform discharge is obtained.
- Liquid of any viscosity and temperature can be handled.
- No priming is required to intitiate pumping.
- Fluid should be free from sands, grits etc. so as to prevent damage of the rotars.
- Initial cost is relatively high.
- Regular maintenance is needed due to its having high speed moving parts.

5.5 RECIPROCATING PUMP

In reciprocating pump pistons are moved in and out (reciprocated) inside a closely fitting cylinder. By this reciprocating process the necessary pressure difference is created. This lead to drawing of water along the suction pipe through an inlet valve to the cylinder from which water is subsequently forced out through an outlet valve to the delivery pipe.

A reciprocating pump is necessarily comprised of the following components:-

- Piston or a plunger
- Pump cylinder
- Suction pipe
- Discharge pipe
- Inlet valve
- Outlet valve

The piston is closely fitted into the cylinder and can be moved in and out at any desired speed. Both the inlet and outlet valves open in one direction only to permit entry and exit of water from the cylinder respectively.

Thus in Fig. 5.2 when the plunger is moved from point A to point B, a vaccum is created within the cylinder. As a result, some water will rush into the cylinder due to the atmospheric pressure acting on the free water surface. When the plunger is moved again back to point A, the one way inlet valve (I) remaihs closed while the outlet valve (O) opens allowing water to move in the discharge pipe. Water from the discharge pipe cannot return to the cylinder due to the one way operation of the outlet valve.

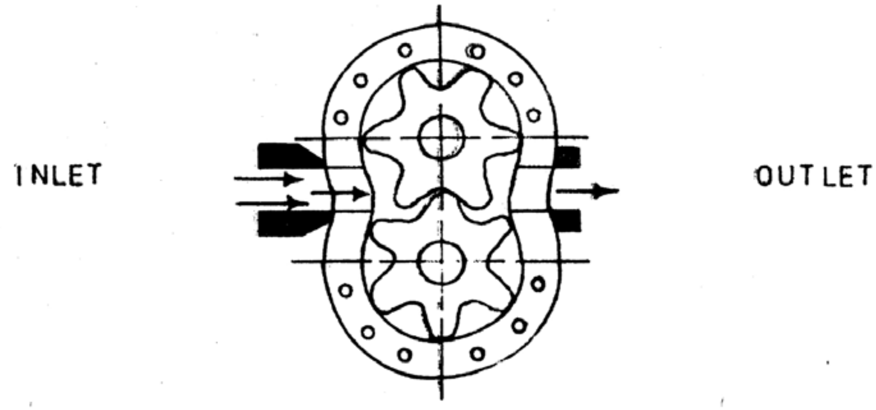

Fig. 5.1 Schematic section of a rotary pump

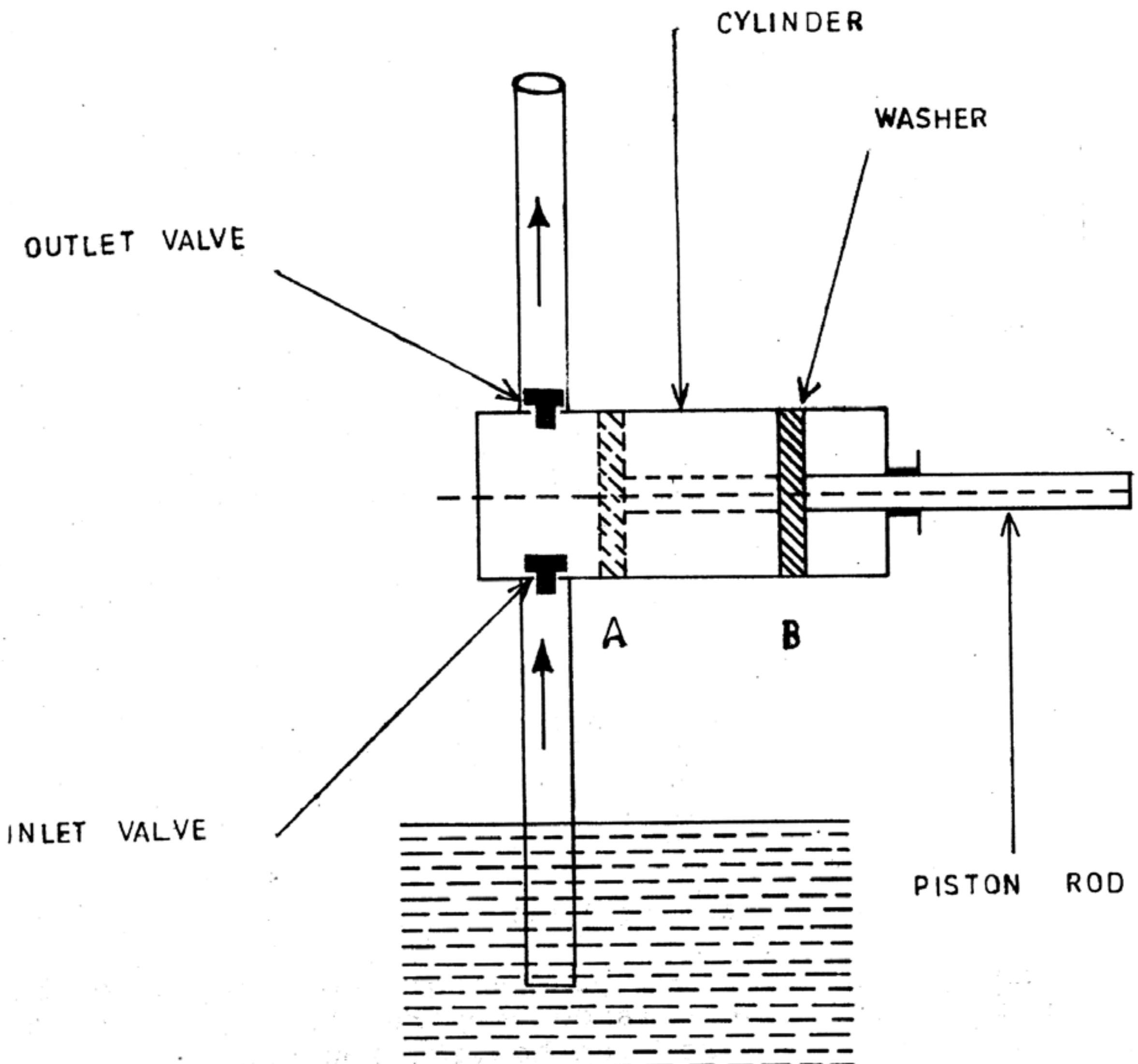

Fig. 5.2 Operation of a piston pump

Fig. 5.3 illustrates modifications made in the cylinder assembly for use in deep well operations. The inlet valve is fixed at the bottom of the cylinder while the plunger itself act as the outlet valve. The valves are thick circular body with vertical passage for water with rubber seats placed above. Circular rubber flaps placed horizontally above these holes permit water movement only in the upward direction. The rubber flap seals the holes preventing return of water. Thus when the plunger is moved from point B to A, water is sucked in the cylinder through the foot valve.

During the downward movement of the plunger, water moves past the plunger by pushing up the rubber flap. This water is prevented from coming back to the cylinder as the rubber flaps gets pressed by the pressure of water column above,thus closing the holes.

Water can be pumped up to any practical height by this process. Since two strokes are necessary in a single acting cylinder to lift the water, such reciprocating pumps are known as single acting double stroke pumps. similarly four strokes pumps are possible. Reciprocating pumps can be double acting (duplex) and Triple acting (triplex) depending upon the number of cylinders used.

5.5.2. Power Requirements

Let,

A = Area of cross section of piston,sq.m
L = Stroke length,m
Hs = Suction head,m
N = Number of strokes per minute
Hd = Delivery head,

Neglecting leakage and other minor losses,

Volume of water delivered per stroke = AL
Weight of water per stroke = WAL
Weight of water delivered per sec = (WALN)/60 Kg-Sec

Now as

Total Head = Hs + Hd
and
Horse power =(WQH)/75, we have

$$HP = (WALN\ (Hs + Hd))/4500 = 0.22ALNH$$

Since, H = Total head
W = 1000 Kg

5.5.3 Features of Reciprocating Pump

- Size and weight of a reciprocating pump for a particular capacity is relatively large compared to other pumps thus requiring more floor area and heavier foundation.

- Dirty or viscous water cannot be handled adequately.

- Delivery is not continuous but pulsating. Continuous flow however is possible in double acting pump.

- Although capacity is directly proportional to the speed but it cannot be run at a very high speed without the problem of cavitation.

- Maintenance cost is relatively high due to wear and tear of large number of parts and high speed operation.

- Although the pump is self priming but suction lift is limited. However development of high head or pressure in the delivery side is possible.

- Higher initial cost.

- Reciprocating pumps can be driven by Internal combustion (IC) engines, electric motors, lever(handpump) and wind mills.

5.6 WIND MILL

Wind mills are wind powered rotors (prime mover) which are used for lifting of water from shallow depths using reciprocating pumps. Wind mills may be deployed for lifting water in areas where minimum wind velocity of 3 m/sec is available throughout the year.

Windmill consists of the following parts:

- Tower stand
- Rotor with blades
- Tail for guiding the rotor
- Crank disc
- Connecting rod
- Reciprocating pump

As strong wind blows, the rotor moves at a high speed due to wind hitting the blades. The tail attached behind the rotor guides the blades towards the direction of wind. This happens as the tail gets pushed away by wind turning the rotor face the wind direction.

An automatic device unlocks the rotor from the main tail when the wind speed is very high (above 10 m/sec). With fall of wind speed the rotor regains its position in steps and gets attached. A standard wind mill rotor is mounted at the top of a tower made up of iron angles and is placed at about 6.5-7 m height. The rotor is mounted at the top of the tower. Number of blades varing from 4-12 are attached to the rotor hubs with blade supporters. More than one crank positions helps in matching the stroke length of the pump used.

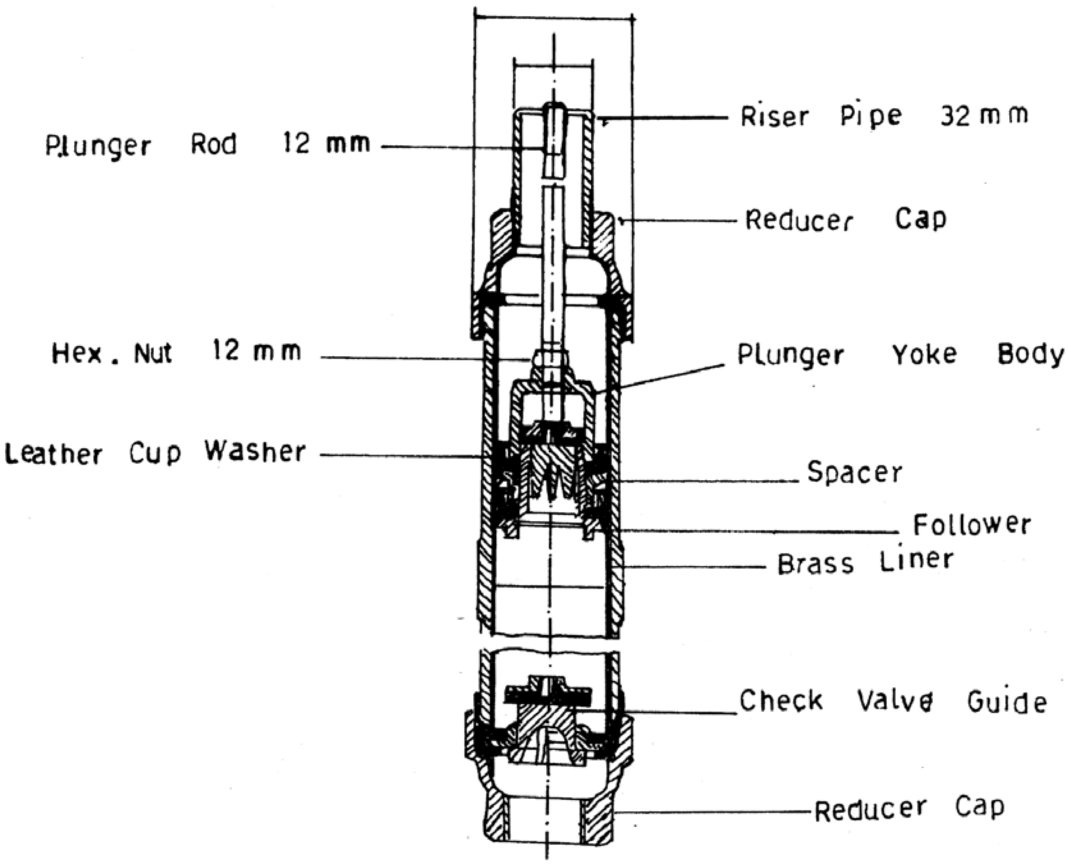

Fig. 5.3 Cylinder assembly of a deep well hand pump

The pump used is necessarily piston type(reciprocating) with wooden piston placed in a steel cylinder. The circular wooden piston have a few vertical holes above which is fitted a circular leather disc. This allows water to move upwards through the piston while the leather disc prevents downward movement of water. Delivery pipe is normally left to the ground level so as to reduce the delivery head. Wind mills are suitable for open-wells, sumps and borewells for a low lift and low capacity but continuous operation.

Discharge from a wind mill is the function of;

- Wind speed or rotational speed
- Head (lift)
- Tower height
- Piston size
- Piston stroke length

A wind mill could be either of the following designs.

- Horizontal or vertical Axis
- High speed or low speed

5.6.1 Theory of Wind Power

Wind power = Kinetic energyxflow of air per unit area.

$$= 1/2\ MV^2 xVA = 2\ m\ V^3\ A = 0.\ 014\ A\ V^3$$

Where,

M = Density of wind per unit volume
V = Velocity of wind
A = An area of cross-section perpendicular to which flow of wind takes place or swept area

Power generated by an wind mill therefore is directly proportional to the cube of velocity (or the speed) of the wind. This means power increases eight folds when velocity doubles. Similarly wind power is also proportional to the density of air and area swept by the rotor ($?R^2$)

5.6.2 Performance of Wind Mill

The standard wind mill design developed in the country with adequate field trials is 12 UP 500. Sample performance of this wind mill as obtained from field trials indicate its usefulness as an irrigation pump.

Field trials of 12 UP 500 wind mills in U.P. recorded following average discharge per month against a pumping head of 6 m.

TABLE 5.1 Average monthly discharge from 12 UP 500 wind mill

Month	Discharge cubic meter	Month	Discharge cubic meter
January	710	July	1440
February	845	August	1485
March	1295	September	1300
April	1705	October	585
May	2185	November	335
June	2060	December	550

The experimental discharge from these wind mills as a function of wind speed for various stroke lenghts (crank positions) for 6.5 metre high tower and 15 cm dia. piston pump is presented below :

Table 5.2 Discharge from 12 PU 500 wind mill for diferent wind speeds and stroke lenghts.

Wind speed m/sec	Flow in litres/second		
	R1	R2	R3
2.5	0.6	-	-
3	0.8	1.2	1.4
3.5	1.0	1.6	2.1
4	1.3	2.0	2.5
5	1.5	2.5	3.0
6	1.6	2.9	3.5
7	1.8	3.3	4.1
8	2.1	3.7	4.6
9	2.3	4.0	5.2
10	2.6	4.5	5.7

The discharge performance of wind mill as a function of wind speed applicable to a 12 UP 500 model with total head of 6m and a piston pump of 15 cm diameter and 24 cm stroke lenght with Rotor of 5m diameter, air density of 0.29 kg/sq.m and over all efficiency of 18% have been experimentaly obtained as below.

TABLE 5.3 Discharge performance of a 12 UP 500 wind mill as a function of wind speed

Velocity of wind m/s	Km/hr	Rotation	Discharge Lps	m^3/h
2-3	7-11	21	1	3.6
3-4	11-14	31	1.5	5.4
4-5	14-18	42	2	7.2
5-6	18-22	55	3	10.8
6-7	22-25	70	4	14.4
7-8	25-29	86	5	18.8
8-9	29-32	105	6	21.6

5.6.3 Features of Wind Mill

- Any tree or other structure which may obstruct the wind speed will hamper the wind mill performance.
- When wind falls below a particular velocity, pumping of water stops automatically.
- Water level should not be too deep. Wind mill can normally operate upto a total head of 20m. This limitation is due to available wind power.
- No fuel is required and hence no power cost.
- Although discharge is low but it is substantial over a long period when run continuously. There may be a need to store the pumped water for its proper application.
- Low initial cost as indegenous materials and local talents are used.
- Breakdown rates are high particularly when the materials used are not of good quality.

5.7 VARIABLE DISPLACEMENT PUMP

Variable displacement pumps deliver water in quantity varing inversely with the head against which they operate. Pumps like Centrifugal pump, Ejecto pump, Air lift pump, Hydraulic Ram etc. fall under this category.

5.8 CENTRIFUGAL PUMP

These are most widely used irrigation pumps for handling large quantities of water. These are pumps where rotation of water is used as pumping force and hence are also known as Rotodynamic pumps. It is well known that when a mass of water is rotated in a very high speed along an axis, it is thrown out from the axis of rotation due a force developed by the rotation called centrifugal force. This centrifugal head will continue to push water to a higher level as long as additional water is supplied to the centre of rotation.

5.8.1 Construction of Centrifugal Pump

Centrifugal pump is comprised of two basic elements, the rotating element or impeller and the static element or casing. The impeller is rotated at a high speed inside a closely fitted casing at a high speed.

The impeller, a circular disc like structure having numbers of vanes (blades) is centrally mounted at one end of the shaft. Other end of the shaft is connected to the prime move either directly or through a coupling. The casing generally designed in two parts are bolted around the impeller. The casing has an inlet opening in front of the impeller and an outlet opening for delivery.

The staffing box is attached in the opposite end of casing which houses the shaft bearing that holds the shaft. The shaft is so designed that it dynamically balances the heavy impeller placed on its end. A screw fitted air vent or priming cap is attached to the top part of the casing for releasing trapped air from within the system. The entire unit is firmly mounted over a baseplate.

The basic components of a centrifugal pump can therefore be listed as below :-

- Impeller	- Casing
- Shaft	- Stuffing box
- Bearing & seals	- Wearing ring
- Coupling	- Base plate.

5.8.2 Operation of Centrifugal Pump

In practice a centrifugal pump is required to be filled with water prior to its operation which is known as priming. Water can be poured through the end of delivery pipe while suction end is kept closed. A check valve (foot valve) provided at the bottom end of the suction pipe keeps it filled with water. Trapped air, if any can be bled out by unscrewing the prime cock attached to the casing and closed tight after the expulsion of air.

As the impeller is moved at a high speed throwing water by centrifugal force in an outward direction, a partial vaccum is created due to the upward movement of water. This vaccum gets automatically filled up by water drawn through the suction pipe. Through this process water continues to flow to the pump through the suction pipe due to the atmostpheric pressure operating on the free water surface. The high velocity of water in the casing is gradually reduced converting the velocity energy into pressure energy which pushes water upwards.

Conversion of velocity energy into pressure energy is attained either by using a specially designed tappered casing known as volute casing or by using diffuser vanes provided inside the casing as an integral part of the pump (Fig. 5.4).

5.4.8.3 Classification of Centrifugal Pump

A centrifugal pump can be categorised in a number of ways depending upon the particular aspect considered for the purpose. Various criteria for classification of a centrifugal pump are described below :

5.8.3.1 Energy conversion

A pump can be identified by the way it converts the velocity energy into the pressure energy. In a centrifugal pump the conversion of velocity energy into pressure energy is performed either by volute casing or by diffuser casing.

Volute Casing

In volute type of pump the impeller is surrounded by the casing which is flared in shape known as volute chamber. In otherwords, the space between impeller and casing is narrow in one end which enlarges gradually towards the outlet . The narrow end of the volute casing is known as tongue of the volute. Pumps with volute casings are generally called volute pumps.

Diffuser casing

In this case, the impeller is surrounded by a number of guide vanes (blades). The guide vanes are so shaped that they gradually increase in cross-sectional area with the direction of flow of water. Since this diffuser blades resembles a turbine in its construction, pumps using such casing used to be known as turbine pumps. With the advent of designs, the term turbine pumps are now more specially used for deep well centrifugal diffuser type pumps known also as vertical turbine pumps.

5.5.8.3.2 Impeller type

Centrifugal pumps can be classified in relation to the shape and number of impeller vanes controlling the direction of flow with reference to the axis of rotation. Number of vanes in an impeller may vary from one to eight. The shape and form of these vanes also guides the type of flow within the pump housing. Classification of pumps as per vane shape may not be easy since impeller shape may overlap.

It is therefore customary to classify pumps as per direction of water flow leaving the impeller which however depends upon shape of the impeller vane.

In radial flow impeller,water after entering through the suction eye are thrown out radially with reference to the axis of rotation.

In axial flow pump water moves parallaly after it is pushed by the impeller which looks like a propeller. Pumps using such impeller are also called propeller pumps.Propeller-type of impellers are used to handle large quantity of water at no lift or low head services. These pumps must be submerged in water. These are not suitable at all in situation where suction lift is involved.

Head of propeller pumps can be marginally increased maximum two to three stages or by using mixed flow impellers. these pumps are exclusively used for handling a large quantity of water at a low to medium head.In some particular application of low head and high capacity installation, mixed flow impellers are preferred than propeller pumps so as to reduce the pump size as well as increase the efficiency. Low suction head ensures low Net Positive Suction Head (NPSH) requirement which allows the use of high (specific) speed.

5.8.3.3 Impeller opening

Open impellers

Open impeller is comprised of vanes attached to a central hub that is fixed on the shaft. Open impellers do not have any side wall or shroud and therefore is made to rotate between two side plates. Open impellers must be made structurally strong particularly when the vanes are long. They have initial low cost. The clearence between the impeller and sidewall cause some water slipage which may increase substantially the wear and tear requiring replacement of side plates and impellers to restore the

CENTRIFUGAL PUMP

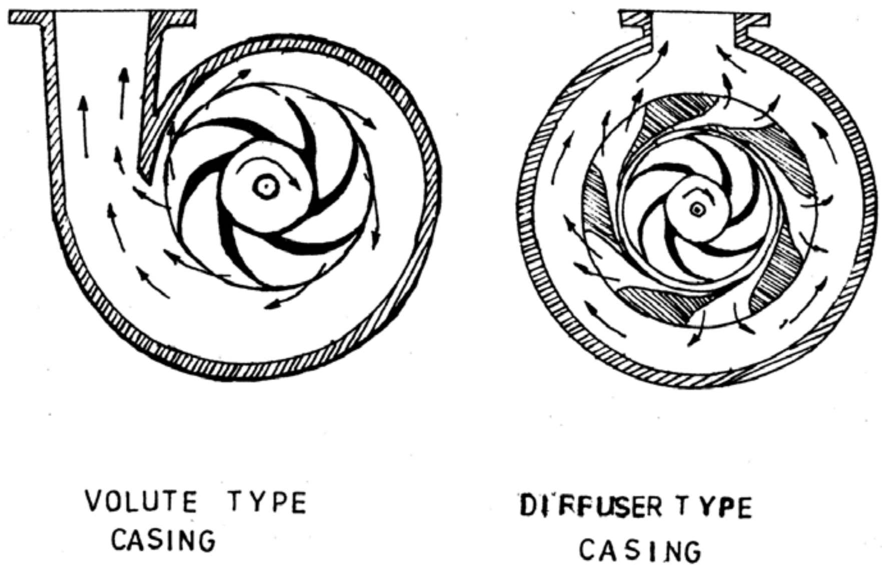

Fig. 5.4 Type of casings in Centrifugal pump

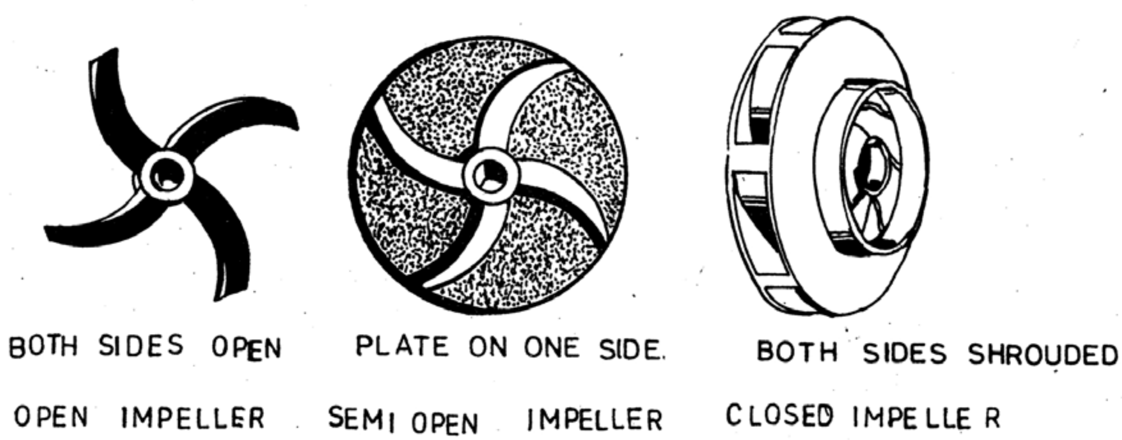

Fig. 5.5 Type of impellers in Centrifugal pump

original efficiency.

Open impellers are used in pumps handling water with abressive materials and limited quantity of sand, grit etc. Water having fibrous material which may lodge itself in the impeller and the stationary side plates, are to be avoided.

Semi open impellers

In semiopen impellers the backwall or shroud is casted as an integral part of the vanes. The function of the back shroud is to strengthen the impellers and also to prevent lodging of foreign matters behind the impeller. To pump water containing fibrous material, semi solids, sediments and foreign materials, a single vane semiopen impeller is used. These are used where non-clogging is the prime requirement than the efficiency.

Closed impellers

Closed impellers have shrouds on both sides of the vanes. The shape and number of vanes are decided by the type of service required. They have high initial cost but assure full capacity operation over a long period. These impellers are used for handling clear water and are extentively used in irrigation applications. Figure 5.5 shows differences amongst the three types of impellers.

5.8.3.4 Number of suction

Single suction

Single suction pumps are those which have only one suction inlet. These are general purpose pumps most commonly used in irrigation.

Double suction

Double suction pumps are those where water is sucked in by two different inlets. The impellers are generally mounted over a common shaft in a back to back position with the suction and delivery ends in opposite diretions (180 degree from each other). Impellers may be back to back single admission or double admission naturally balanced type.

Water flows to the impellers symetrically from both sides, and is pushed out together through a common outlet. These are suitable for high capacity, low head installation and have wide application in lift irrigation schemes.

5.4.1.8 Suction position

As per location of the suction opening, pumps can be classified as end suction, top suction, bottom suction and side suction pumps. Similarly delivery openings may be at top or side. Bottom horizontal discharge is however not possible. Generally, all double suction axially split pumps have side discharge opening with either a side or bottom suction opening.

5.8.3.6 Number of stages

Single stage

Single stage pumps are those which have a single impeller. Single stage pumps are simple, less expensive and used in general purpose services. Head developed cannot be too high since a single impeller is used.

Multi stage

In multi stage or multi impeller pumps, more than one impellers are placed in series within the same casing. In multistage pump all the impellers are placed under a single casing each taking its suction from the discharge of the preceeding impeller.

Normally as many as 8 stages are feasible. Multistage pumps are necessarily used for high head or high pressure installations with low to moderate capacity. For a high capacity multistage pump, it will be desirable for the first stage to have double suction impeller so as to reduce the NPSH for the given capacity. The arrangement of impellers are however made as per design requirements satisfying the balancing, shaft length and casing size.

Pumps are said to be running in series when the discharge of one pump is fed to the suction of the other pump. The total head of pumps in series are the algebric sum of head developed by each individual pump. Two single stage pumps with a common shaft can also be run by a single central drive with the delivery of both the pumps joined with each other.

5.4.1.10 Axis of Rotation

Based on the axis of rotation, pumps may be classified as vertical and horizontal. The prime criterion for selection of either type is the type of installation. In horizontal pump the shaft is mounted horizontally which means the impeller moves in a vertical plane.

Most centrifugal pumps are horizontal type. They are installed in dry pits with their suction pipe extended to the pumping water. Horizontal pumps are therefore prone to suffer from the problems of suction limitation. In some installations however the pump itself can be placed near the water surface while the drive can be placed at a different location and a belt-drive is used. End suction are common for horizontal pumps.

In vertical pumps, the shaft is mounted vertically which means the impeller rotates in a horizontal plain. Vertical pumps may be used in both dry pit and wet pit type of installations without any limitations. Dry pit vertical,pumps are however basically horizontal pumps with slight modifiations to adopt to the vertical drive.

Vertical pumps may be of bottom suction, or side suction type while the motor canbe connected right on top of pump casing or through a long vertical shaft. In vertical wet pit installation pumps are kept submerged while the driver is mounted above the water level connected with a vertical shaft.

Vertical turbine, propeller, and sump pumps fall under this category. Pumps having water tight motors directly coupled with the impellers ment for underwater installation are known as submersible pumps.

5.8.3.8 Direction of rotation

The direction of rotation of a pump may be either clockwise or anti clockwise,The direction of rotation is generally noted while looking at the pump from the suction end. To avoid confusion the manufacture will sometimes provide an arrow marking indicating the direction of rotation. it is obvious that the direction of rotation should match with that of the drive. Manufactures can supply most pumps with any given direction of rotation as per the choice of users. For this the whole unit like impeller curvature, casing design etc will be symmetrically opposite.

5.8.3.9 Speed of pump

Based on the speed of rotation, pumps may be classified as low speed or high speed pumps. The required speed of rotation of a pump for a given performance varies inversly with the diameter of the impeller. In otherwords to obtain the same performance from a given pump,if the diameter of the impeller is reduced the speed should be increased or vice versa. The speed should however be contained within the practical limits otherwise there would be high wear and tear.

The speed of a pump is decided by its total "specific speed" requirement which is a function of Head and capacity. Speed of a pump is also designed to match the speed of he prime mover.

5.8.3.10 Casing divide

Casings are usually made in two parts. Depending upon the plane at which the casing are split, pumps can be classified as:

- Axially (horizontally) split casing
- Radially (vertically) split casing
- Angular (diagonally) split casing

The design of casing is largly controlled by the location of suction and delivery opennings. It also depends whether the pump is single or multistage and the impleller is single or double suction type.Axially split means the casing is made in two halves in a plane parallel to the axis of rotation. The term axially split is more explanatory than horizontal split specially when a horizontal pump is used as vertical pump. Similarly the term radially split is more appropriate than the term vertically split.

Therefore in most of the casing designs, either the suction nozzle and in some designs both the nozzles are made into two halves. Casing halves are usually bolted with each other surrounding the impeller. In modern pumps, casing spliting however can be so made that none of the nozzles are splited.

5.8.3.11 Drive connection

To rotate the impeller of a pump at the desired speed, the shaft is connected with the prime mover through various arrangements such as :

Direct coupled (monoblock)

In this case, the pump and the prime mover are manufactured as a composite unit wherein the shaft of the mover and the pump is a single common unit. Direct coupled or monoblock pumps are usually used for common and simple installations. Their adoptability is naturally limited.

Flexible coupled

In flexible couple arrangement, the drive shaft and the pump shaft is bolted together with integrally casted or threaded trangular or round plates. Proper allignment of these joints are necessary for smooth functioning of the pump.

Belt drive is given when the pump and the mover is placed at two different locations. Belt drive is generally preferred in installation where the mover has multipurpose use or more than one pump is run by a single mover.Gear drive is also possible by driving a pump through two intermeshing gears, one connected to the pump shaft while the other with the shaft of the prime mover.

5.8.3.12 Type of mover

Pumps can be classified depending upon the type of mover used:-

Electric pumps

In this case, pumps are run by electronic motors. These are high efficiency compact, low noise, trouble free units. The cost is relatively less with universal application in all types of pumps. Stable supply of electricity at the location of use is however essential.

Engine powered pumps

Engine drives are be steam engines or internal combustion (I-C) engines. Steam engines are less used and are of low efficiency.

Diesel engines are in common use. These are extensively used at location where electric power is not available or as "stand by" arrangements in areas with unreliable eletric supply. Both the initial and operating cost of diesel engines are comparatively higher than that of electric motors.

5.8.4 Features of Centrifugal Pump

- Simple and compact in construction
- Steady in discharge
- Easy to install and requires less floor space
- Easy and simple in operation
- Lower initial cost and also less maintenance cost
- Long dependable and durable service
- Adoptable to various prime movers
- Water of moderate viscosity and with some sand and grit can be handled
- Priming is necessary
- Suction lift is limited
- Suitable for both low head high capacity and high head low capacity installations
- Not suitable for water at high temperature and high pressure operations

5.8.5 Net Positive Suction Head (NPSH)

NPSH is defined as the total available head in meter of liquid (absolute) measured at the suction opening of the pump minus vapour pressure of the liquid in meter (absolute) at the pumping temperature. In other words NPSH is the pressure head available at the suction inlet minus the vapour pressure. While pumping, the pressure at any point in the suction pipe must never be less than the vapour pressure of the water otherwise cavitation will take place.The pressures are however measured in equivalent height of water column in vaccum (absolute pressure).

The concept of NPSH is useful in understanding the suction limitation of centrifugal pumps. It is the atmospheric pressure which forces water into a centrifugal pump when a partial vaccum is created by the pump impeller. Total available pressure head at the pump inlet is therefore the atmospheric pressure head minus intervening losses.

Thus mathematically

$$P/W = (Pa/W) - (V^2/2g) - hf - H \qquad \ldots \quad 5.2$$

Where,

Pa/W = Atmospheric pressure head
P/W = pressure head at the suction inlet
$V^2/2g$ = Velocity head in suction pipe
hf = Friction head in suction pipe including entry loss
H = Static suction lift

As per definition:

$$NPSH = P/W - (Pvap)/W$$

$$= Pa/W - H - (V^2)/2g - H_f - (Pvap)/W \quad . . . 5.3$$

The above equation can be written as

$$NPSH = Ha + Hs - Hvap$$

where,

Ha = Atmospheric pressure head in meter of water absolute
Hs = Total suction head(+)or total suction lift(-)in meter of water including velocity head and friction heads
Hvap = Vapour pressure of water in meter absolute at the pumping temperature of water

5.8.6 Cavitation in Centrifugal Pumps

Cavitation in a centrifugal pump occurs if the pressure at any point within the flowing water drops below the vapour pressure of water at the existing temperature. Since the pressure head available is limited to the suction side, cavitation is prone to occur in the suction pipe.

Boubles formed due to the cavitation moves along the water through the pump towards the delivery side. At one point of their movement due to increasing pressure the boubles burst creating pressure transients. Cavitation may therefore result in vibration of pump and can cause damage if the process is continued.

Cavitation also affects efficiency as some energy loss takes place to the water due to acceleration in filling up the cavities.Cavitation is likely to take place when either the suction lift or the pump speed is too high. Cavitation will also take place when available NPSH is less or equal to the required NPSH.

Chances of cavitation is reduced when

- Required NPSH is low
- Velocity of flow is low
- Operating speed is low
- Suction lift is less
- Number of venes in the impeller is more which reduces turbulence

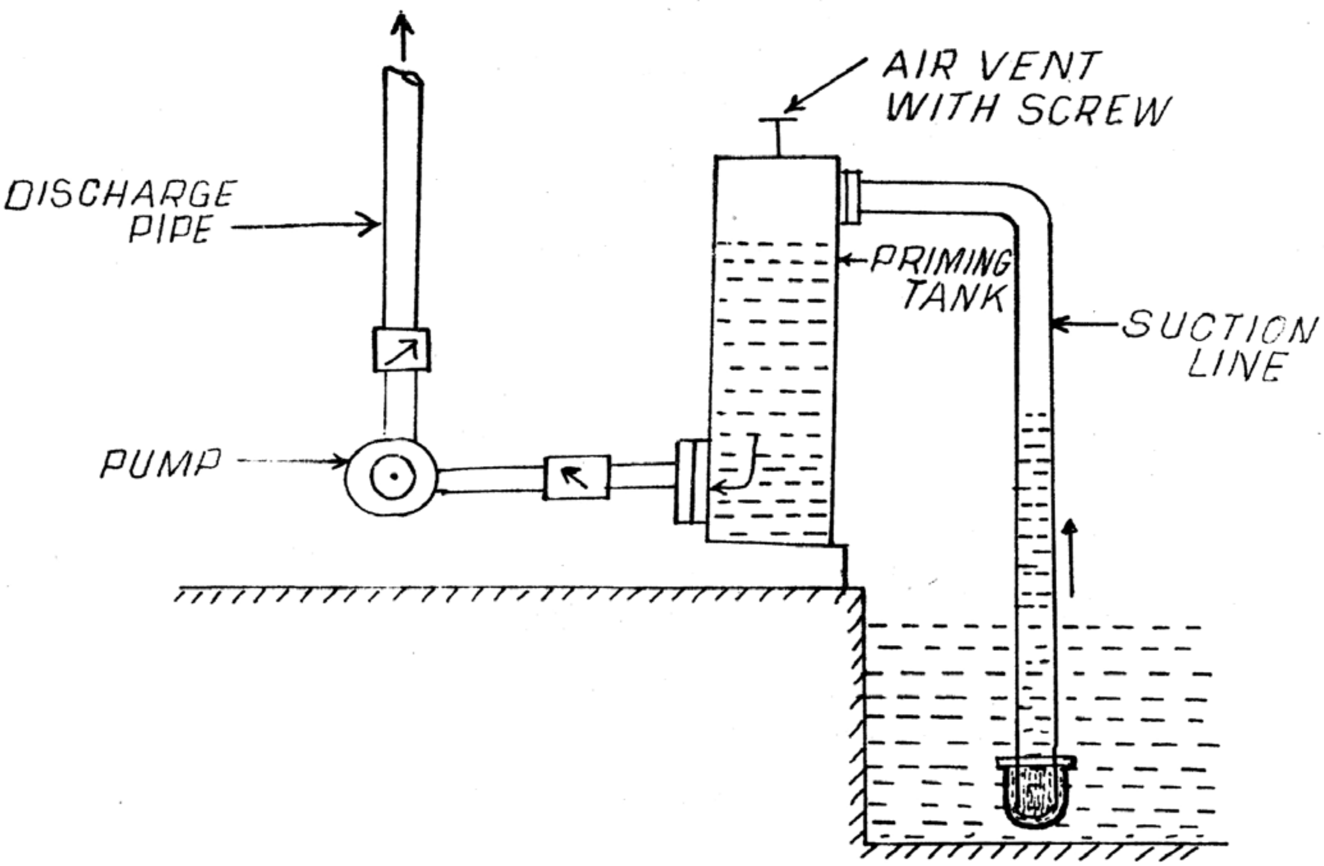

Fig. 5.6 **Self priming of a Centrifugal pump**

5.8.7 Priming of Centrifugal Pump

Priming of a centrifugal pump is done by filling completely the pump casing and the suction pipe with water. This expels the air completely from the system providing the needed vaccum in the system for the pump to function.Priming can be done manually or automatically.

5.8.7.1 Manual Priming

In manual priming foot valves are used. A foot valve is basically a non-return valve encased by a strainer to be fitted at the end of the suction pipe. Water can enter the pipe from outside only through the one way valve which closes tightly in opposite direction due to the weight of the water column. This prevents draining of water when the pump is not working thus maintaining the priming. Water is filled in through the delivery pipe or through the pump casing. Once a pump is stopped it may loose its priming. Water stored in the delivery pipe can be allowed to flow into the suction pipe through a small bye pass fitted with a hand cock for priming before starting the pump once again.

5.8.7.2 Priming without foot valve

Use of priming chamber is the simplest method. In this system (Fig. 5.6) the priming tank is connected at the top with the suction pipe. The outlet pipe connected in opposite direction at the bottom feeds water to the pump. The tank is filled with water through an inlet pipe fitted at the top of the tank by operating the hand cock. An outlet vent with lock is also provided for releasing air. Size of the tank between the inlet and outlet is about three times the volume of suction pipe.

For priming, the air vent is opened and the tank is first filled with water from external supply through the priming cock and the air vent is then closed tightly. As the pump is started it initially takes its supply from the tank. The level of water falls in the tank. The air inside is stretched and a partial vaccum is created. As a result water starts rising through the suction pipe from the source and pumping continues.

In large installations, priming can be achieved by using vaccum producer or ejector pipes attached to the casing. Stand by pump attached to the main pump can sometimes be used for priming. Some self priming pumps are so designed that initially during its rotation the vanes purge most air out from the casing. The air water mixture progressively gets free of air and water finally moves out the delivery pipe when the pump is fully primed.

5.9 TURBINE PUMP

These are basically multistage difusser type of pumps. A typical deep well vertical turbine pump Fig.5.7 consists of:-

- Vertical motor

- Column assembly with discharge bend
- Vertical drive shaft
- Bowl assembly
- Thurst/shaft bearing with holders

The motor is placed at the ground level over the column pipe which extends in to the well. Column pipe also has a ground level discharge outlet near the base of the motor. In side the column pipe, a vertical shaft passes downward through thurst bearings supported by bearing retainers. Bearings may be water lubricated or oil lubricated. The shaft is connected with the motor at its upper end while lower end contains the bowl assembly. Bowl assembly is the pumping element of a vertical turbine pump. It consists of multistage closed or semi open impellers extending upwards within the pump bowl. Each impeller developes certain pressure depending upon their diameter and speed of rotation. Required head is obtained by connecting number of impellers or stages in series.

5.10 SUBMERSIBLE PUMP

These are basically multistage centrifugal pumps where both the motor and pump assembly closely coupled are submerged into the water and hence called submersible pumps(Fig. 5.8)

The inherent suction limitation of centrifugal pumps are thus eleminated in these pumps. Submersible pumps are available mostly for borewell use. The basic components are :

- Motor
- Pump assembly
- Discharge pipe
- Power extension cables

Narrow cylindrical high speed motors suitable for insertion in borewells are directly coupled below the pump assembly of similar shape. Number of impellers connected in series (stages) is decided by the head requirements of the pump.

As the motor drives the impellers, water enters the pump assembly through the opening between motor and pump covered usually with a strainer. The pumped water is carried upwards through a riser pipe. The riser pipe which takes the complete load of the pump and motor is clamped firmly at ground level.

The power supply to the motor is made by intregally attached, specially made flat cable. Motors are necessarily water proof using either oil or water lubricated bearings.

Annular space between the motor and borewell should be sufficient for a steady circulation of water so as dissipate the heat and to keep the motor cool. Starting of subersible pumps upto 10 H.P. are done by Direct on line (DOL) switches while star-Delta connections or auto start circuitry are recommended for pumps of higher horsepowers. Submersible pumps are also available for use

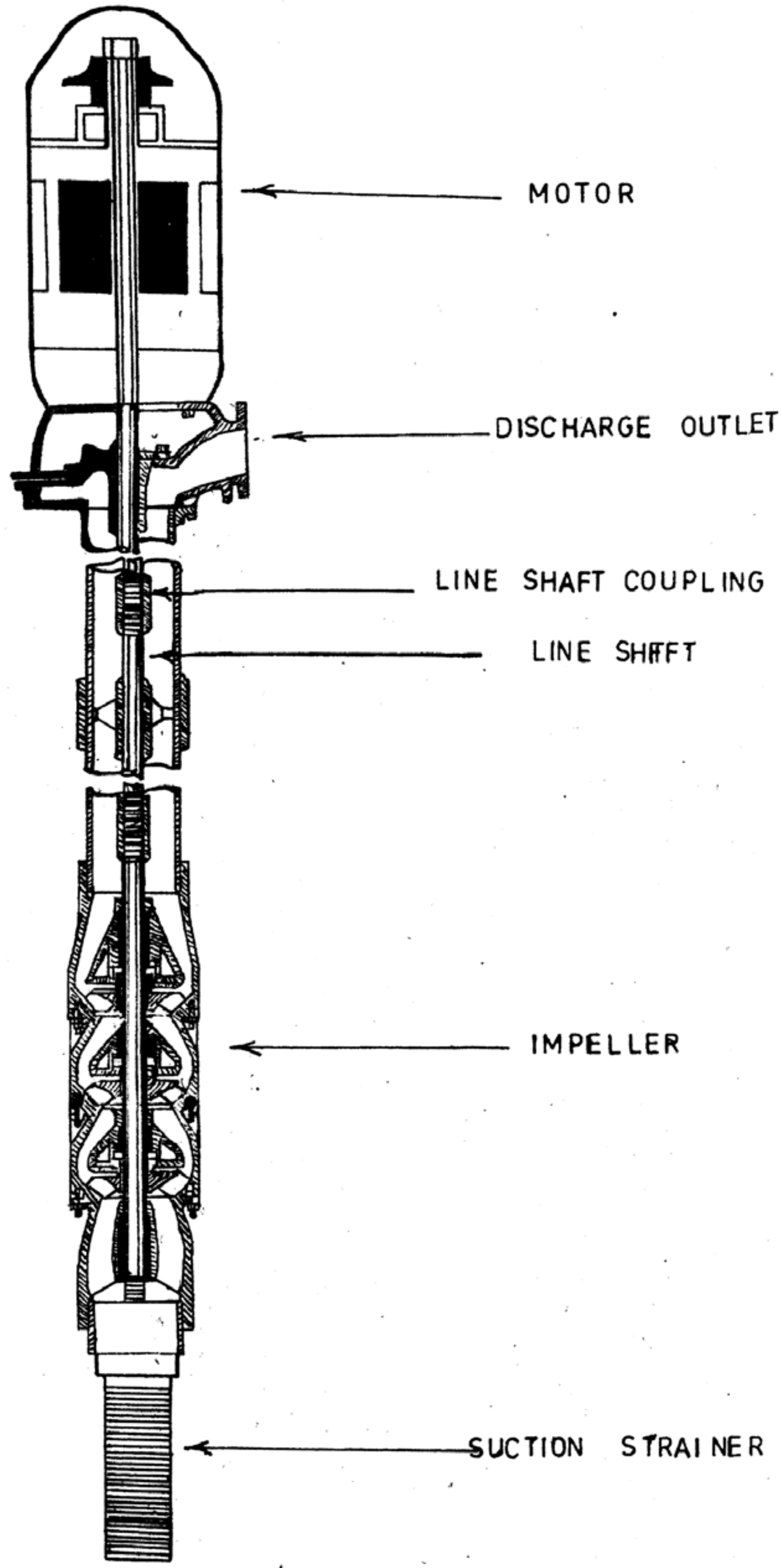

Fig. 5.7 Sectional view of a vertical Turbine pump

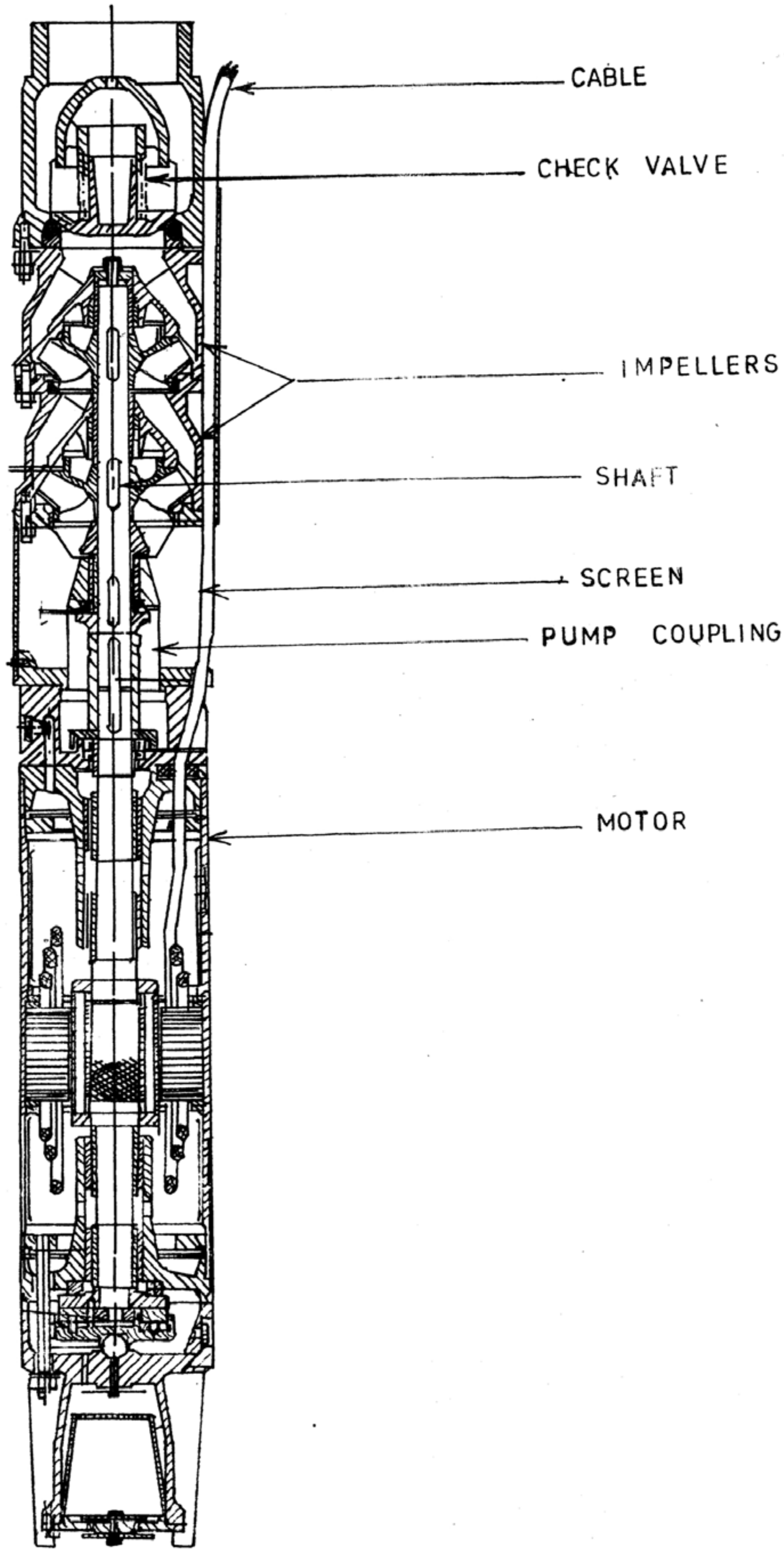

Fig. 5.8 **Sectional view of a submersible pump**

in openwells.

5.11 COMPARISION BETWEEN VERTICAL TURBINE PUMP AND SUBMERSIBLE PUMP

It is a difficult task to indicate off-hand the superiority of one pump over the other. Both have certain merits and demerits and are suitable for use as per the purpose. A comparative note is however presented below for understanding their utilities.

- Turbine pumps are driven by line shaft.A very long line will naturally entail high power loss. Other advantages of these pumps are therefore nullified when water is to be pumped from a deeper level say beyond 30 metres.

- Submersible pumps are lowered with pipes together with the attached power cable. Such pumps can perform well at any practical depth.

- Turbine pumps can be driven by electric power as well as oil engines. Submersible pumps run only on electric power.

- Turbine pumps are run by low speed motors (1450 rpm) and hence are subject to less wear and tear. Submersible pumps due to its size constrain are run by high speed motors. Moreover due to less clearence between the stator and the rotor, water suspended particles like sand etc can easily damage the motor.

- Vertical turbine motors are sturdy in construction and can take high voltage fluctuations.Submersible motors are under an inherent load since it is submereged in water,they are prone to damage due to overloading,power fluctuation and overheating.

- Since motors of vertical turbine pumps are located at ground level they are subject to theft or damages.Submersible pumps are beyound the reach from such possibilities.

- Turbine pumps can be fitted in the borehole which is fairly vertical and relatively large in diameter. Submersible pump can be installed even into a borehole which is slightly inclinded. The main bearings of the motor will however be subject to differential wearing 1when installed in an in clinded bore.

- Transporting and fitting of turbine pumps from factory to the site or from one site to the other require extra care. Slight bending of the shaft will reduce performance of the pump to a great extent. Submersible pumps can be lowered pipe by pipe even by unskilled personnel.

- Initial costs of a turbine pump is higher than that of a submersible pump of the same power rating but is likely to be cheaper in the long run if fitted properly since overall efficiency of the former (75-77%) is higher than that of the later (32-60%).

5.12 EJECTO OR JET PUMP

Jet or ejecto pump is one in which high speed jet is used to draw water from a lower level. Theoretically, water is lifted by converting pressure head partialy into the kinetic energy.

The suction side comprises of twin pipes attached with an U-joint at the bottom(Fig. 5.9). Water is circulated through one of the pipes in high pressure. As water makes its upwards journey in the other pipe, it passes through the tappered U-joint which contains a ventury nozzle. The nozzle is so shaped that the cross-sectional area reduces smoothly but very sharply. As water passes through this nozzle the velocity of water increases abruptly causing a drop in the pressure energy, as a result of which water is sucked in from below through a drop pipe.

This sucked in water coming up through the intake pipe attached below the nozzle joins the flowing water to move up towards the pump. Velocity of the water comes back to normal in the pipe after it moved past the nozzle. The extra water lifted is passed through a control valve to the delivery pipe, while the remaining water goes back into the circulation.

Efficiency of Jet pump can be expressed as;

$$E = (Qs\ (Hd + Hs))/(Qj\ (He - Hd)) \qquad \ldots\ 5.4$$

where,

- Qs = Quantity of water flowing through the suction pipe (cu.m/sec)
- Qj = Quantity of water supplied throough the Jet (cu.m /sec)
- He = Supply head above the Jet, m
- Hs = Height of Jet above pumping water level, m
- Hd = Delivery head above the jet, m

The efficiency of jet pump is relatively low and it ranges between 20-25 percent.

5.12.1 Features of Jet Pump

- Simple in design.

- Not many moving parts and hence less wear and tear.

- Initial and maintenance cost is relatively low.

- Drive pump can be adopted to both electric motor and diesel engine with belt drives. The drive pump can be placed near or far away from the water body or borehole.

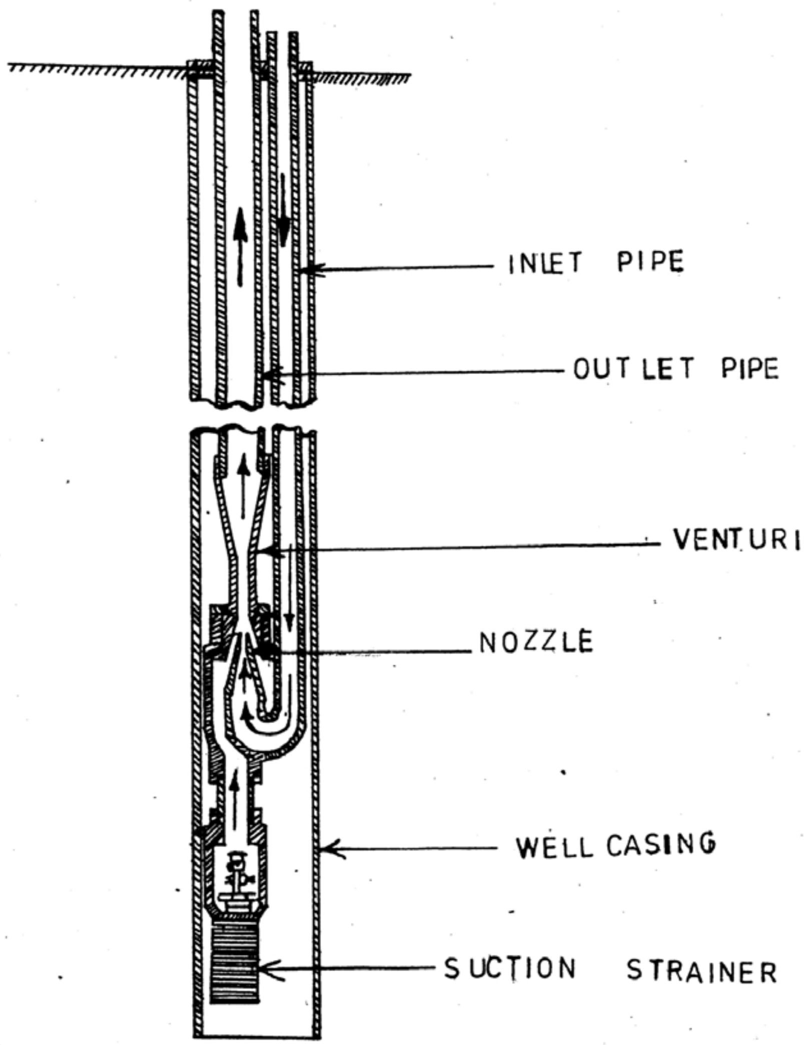

Fig. 5.9 Sectional view of the U-joint of a Jet pump

- There is no limiting suction depth as the jet itself is inserted into the bore. These are more suitable for low capacity deep lifting installations particu larly in boreholes.

- With pressure tank arrangements automatic starting and stopping of the pump is possible.

5.13 AIR LIFT PUMP

Compressed Air is used to lift water in this system.(Fig. 5.10) Two concentric pipes are lowered into the water which is to be lifted. The inner pipe through which compressed air is passed is known as air line while the outer pipe which carries lifted water up is known as lift pipe or riser pipe.

The compressed air being released into the water forms a mixture of air and water which has much lower specific gravity than the water itself. Since the atmostpheric pressure can support a much longer column of such mixture, the water air mixture flows up the riser pipe.

Total lift by such pump depends upon the submergence of air pipe within the water. Normally the required depth of submergence of airline is to be 1.5 to 2 times that of the lift. Air consumption increases as the depth of submergence is increased. Therefore efficiency falls with the depth of submergence which is a function of water to be lifted.

The efficency of an Air Lift pump is

$$W = \frac{\text{Work done in lifting water}}{\text{Workdone on the a1ir assuming isothermal compression}}$$

$$= \frac{Q * W * Hd}{P_a * V_a * \log_e (P1/Pa)} \quad \ldots \quad 5.5$$

Equating available pressure head to cause the flow with total lift and friction and velocity head in pipe we have.

$$E = (Hd/Hd + Hf) + (V^2)/2g) \quad \ldots \quad 5.6$$

where,

Q = Quantity of water lifted, cu.m /sec
Hf = Head loss due to friction,m
Hd = Height of lift (delivery), m
w = Specific weight of water, Kg/sq.m
V^2 = velocity of water, m/sec
P1 = Pressure at the base of lift pipe,Kg/sq.cm
Pa = Atmostpheric pressure,Kg/sq.cm

Actual efficiency of air lift pump varies between 25-30 percent but at times it may be as high as 45 percent.

5.13.1 Features of Air Lift Pump

- It can lift moderate quantity of water from narrow wells.
- It can handle sandy, gritty, muddy, alkaline and acidic water. It also can handle water at most tem peratures.
- Capacity of the pump can be varied within a range as per the demand.
- Efficiency is low. Efficiency increases as the lift decreases.
- Lift requires adequate submergence. So the depth below water level must permit the necessary submergence.
- Water level or drawdown cannot be measured when the pump is in operation.
- As compressors (prime mover) are noisy equipments,these pumps are normally used in field operation as temporary installation.Small capacity ellectric compressors are used in some permanant installations.

5.14 HYDRAULIC RAM OR HYDRAM

This is a pumping device which uses the momentum of the moving column of water to lift a small part of the same water to a height above its original supply head. In fact, the principle of water hammering is used as the source of the driving force. It uses the force created due to sudden stopping of a large quantity of falling water to lift a part of this water while the remaining of water gets releases for disposal.

A Hydraulic Ram consists of an inclined supply pipe, Ram assembly and delivery pipe (Fig. 5.11). The Ram assembly comprises of an air chamber, air valve, waste water outlet and the delivery outlet.As water moves down the inclinded drive pipe it forces an automatic valve to close after it has gained certain momentum. The closing of the valve stops the flow of water suddenly in the drive pipe. The weight of the moving water thus creats a surge pressure as it is stopped suddenly. This surge pressure forces some water past a non-return delivery valve into an air chamber.

As the pressure drops in the drive pipe, the main valve opens automatically to allow the flow to begin and closes again when water flow develops the closing pressure. Thus due to alternate closing and opening of the drive pipe certain amount of water continues to be forced into the air chamber. The compressed air in the air chamber pushes water up continuously along the delivery pipe and the larger part of water in the drive pipe escapes through the waste valve.

Quantity of water lifted by hydram is given by:

$$Q = WHe/h \qquad \ldots \; 5.7$$

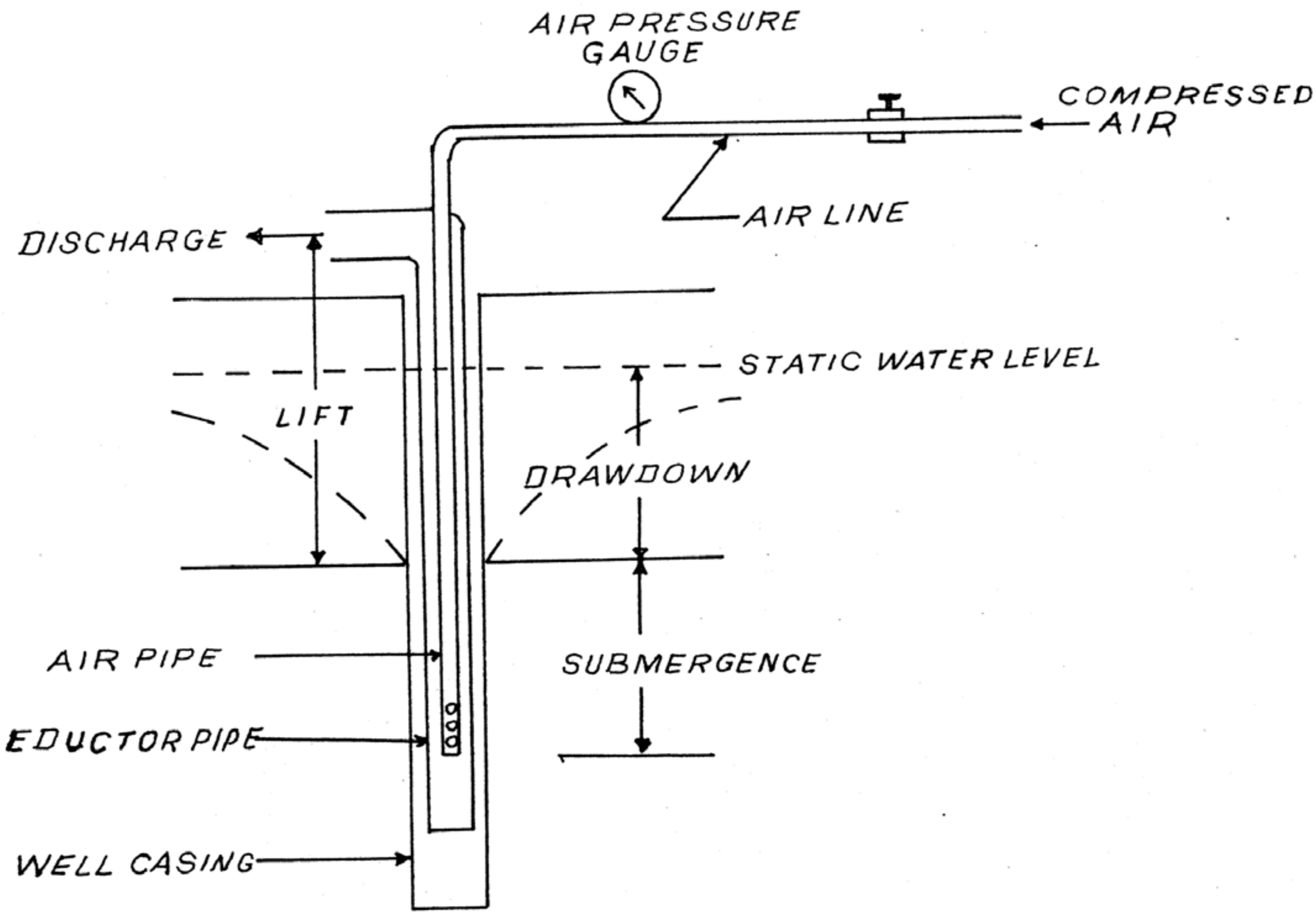

Fig. 5.10 Schematic diagram of Air Lift pump

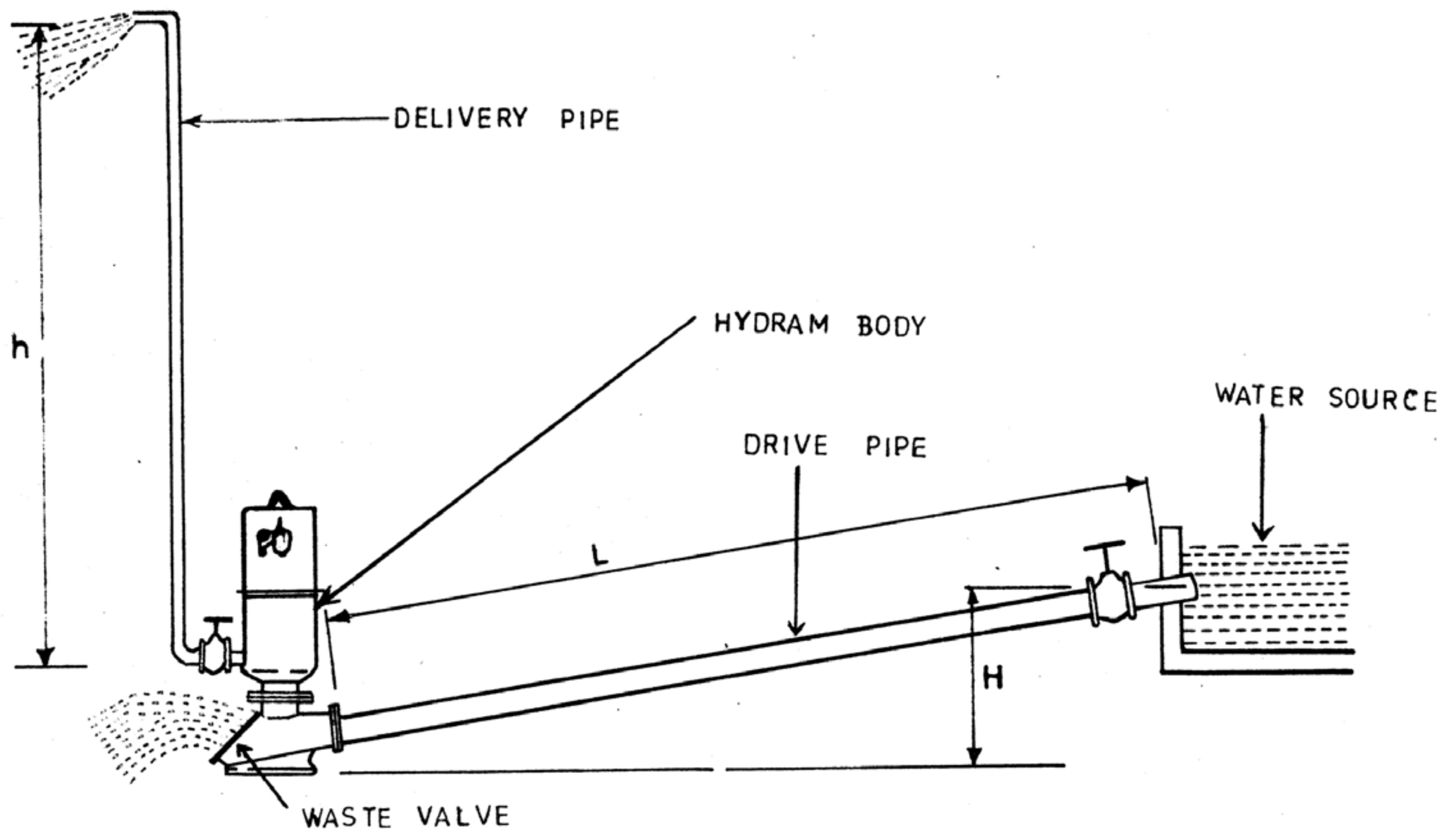

Fig. 5.11 Hydraulic Ram (Hydram)

Where,

Q = Rate of flow, cu.m
h = Vertical height of delivery, m
H = Vertical fall of water in drive pipe, m
W = Quantity of drive water, cu.m
e = Efficiency of the hydram.

Efficiency of a hydram depends upon the ratio of vertical height of delivery to the vertical fall of water in drive pipe i.e. h/H. The efficiency level for different h/H ratio is presented in table below :-

Table 5.4 Efficiency of Hydram

h/H %	e	h/H	e
2	0.85	9	0.65
3	0.85	10	0.60
4	0.80	12	0.60
5	0.75	15	0.55
6	0.75	18	0.45
7	0.70	20	0.40
8	0.65	25	0.40

5.4.6.1 FEATURES OF HYDRAM

- No fuel is required and hence running cost is very low.
- It is durable and reliable except the valve ele ment which requires frequent replacement.Maintenance cost is also less as it has only two moving parts.
- Although capacity is comparatively low and discharge is pulsating but once a hydram is set in motion it runs continuously.
- Suitable for remote hilly areas where source of conventional power is limited.
- It is efficient for lift upto 30 m.
- Adequate, water supply for drive pipe should be available round the year and the location should also be favourable for such installation.
- Water should be clean. Presence of sand, grit or floating matter in the water may hamper normal fuctioning.
- Operation is slightly nosiy.

CHAPTER 6

SELECTION OF IRRIGATION PUMPS

6.1 SELECTION CRITERIA

A wide range of pumps are available for use in irrigation. Selection of the right pump for the right use depends upon a series of inter-related factors. To a common user it is not only difficult at times to choose a particular type of pump but it may be equally difficult to choose the specific size(model) from the type under consideration. These difficulties are further accentuated when a pump suitable for a particular operation is supplied by more than one manufacturers.

The centrifugal group of pumps have the largest application in irrigation as compared to other type of pumps. For example, Rotary pumps have their use in sprinkler irrigation where working pressure is more important than the discharge. Also reciprocating pumps have their use in wind mill operations. Pumps like Hydram, Jet pump and other types have limited application in large scale irrigation. Most commonly used centrifugal types of pumps are horizontal end suction, vertical submersible and vertical tubine pumps. Mixed or axial flow impellers are sometime used in low lift, high capacity operations like lifting of water from a canal.

The factors that affect the decision one way or the other to select a particilar centrifugal pump are listed below :

- Capacity
- Head
- Working pressure
- Efficiency range
- Nature of water to be handled
- Fluctuation of pumping water level
- Suction conditions
- Shape and size of the water extraction structure
- Quality of water
- Choice of driving power
- Fluctuation of loads
- Floor space available
- Priming requirements
- Accessibility and protection of the pump
- Initial cost
- Operating cost
- Reliability
- Flexibility
- Availablity
- Acceptability

In order to select a suitable pump it is important not only to have a comprehensive knowledge about how a pump functions under different working conditions but also to take into account economical considerations. All the factors listed above can be grouped togaether to the following set of criteria.

- Pump performance
- Design characteristics
- Operating condition
- Type of installation
- Economic considerations

6.2 PUMP PERFORMANCE:

One of the most important features of a centrifugal pump is that its capacity varies inversely with the head. Owing to such an inverse relationship between head and capacity, the efficiency and the power consumption also changes in accordance with the operating Head and capacity i.e the duty point at which a pump works.

In fact, capacity of a centrifugal pump varies with a number of parameters like Head, power input, efficiency, speed of rotation, NPSH, quality of water, design of the impellers and specific speed. Relationships of capacity with any of these factors when represented in a graphical form, are known as characteristic curves. These characteristic curves of a given pump is prepared based on actual measurements by test pumpingconducted in the laboratory.Characteristic curves are generally grouped into the following categories.

Constant speed characteristics

- Main characteristics
- Operating characteristics
- Iso-efficiency curve

Constant head-constant capacity curves(variable speed curves)

6.2.1 Main Characteristics

Centrifugal pumps are designed for operation at a constant speed. This is so, because for a given Head-capacity combination normally there will be only one speed at which the pump will work at its highest efficiency. For a centrifugal pump running at a constant speed, the graphical relationships between capacity with Head, water horse power and efficiency are knows as main characteristics. These curves generally are presented together to indicate the characteristics of a pump.

It is clear from the following expressions that,

$$WHP = (WQH)/75$$

$$Q = (75\ WHP)/(WH) \quad \ldots \quad 6.1$$

and $$e = (WHP)/(BHP) \quad \ldots \quad 6.2$$

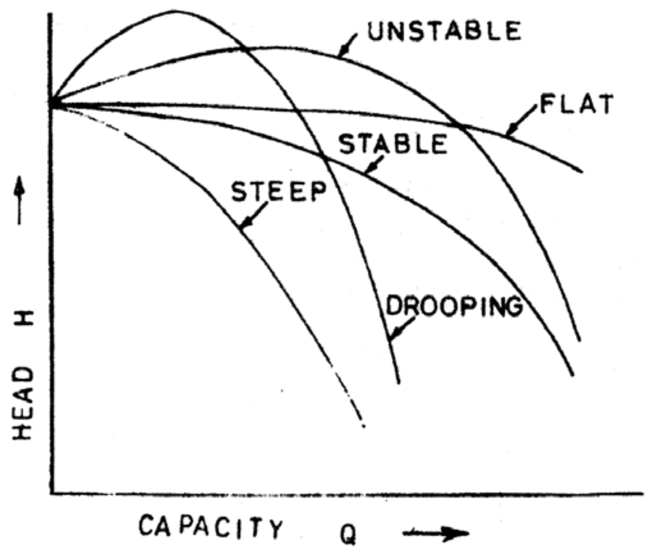

Fig. 6.1 Types of Head-Capacity curves

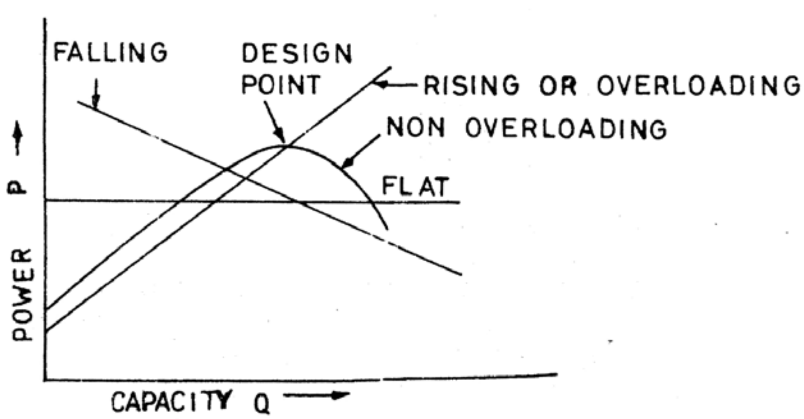

Fig. 6.2 Types of Power-Capacity curves

Capacity varies inversely with head when WHP is constant.

- Capacity varies directly with WHP when Head is constant.
- Efficiency varies directly with water horsepower for a given BHP.

6.2.1.1 Capacity-Head Curve

This is a curve which represents the relationship between capacity and Head. The shape of Head-capacity curves varies depending upon the design of the pump(impellers). A few typical shape of such curves are presented in Fig. 6.1.

Pumps sometimes are also classified on the basis of the shape of its capacity-Head curve. Thus stable or rising curve is one in which head rises steadily as capacity is decreased. In other words capacity inreases steadily as head is reduced. In drooping curve head increases initially to some extent with the increase in capacity but after a point reduces abruptly as capacity continue to increase. Unstable curve is one where the same head is developed at two or more capacities. Flat curve is the one where rate of change of head is too little with the changing capacity. In steep curve rate of change of head is too high with changing capacity.

A rising curve therefore is the most preferred characteristics of any given pump. The points at which the Head-capacity curves intersects the Head(ordinate) and capacity (abcsissa) are known as shut-off head and shut-off capacity respectively.

6.2.1.2 Capacity-efficiency curve

Since energy loss in a pump due to leakage,friction, cavitation etc. remains the same irrespective of the flow,(capacity), efficiency will therefore be low at a low capacity and inreases as capacity is increased upto the point of highest efficiency. Beyond this point however the efficiency starts declining, as capacity is further increased. Efficiency is zero at zero capacity. Hence efficiency curve starts from zero capacity, rises to a peak then falls as capacity is increased.

6.2.1.3 Capacity-power (BHP) curve

Theoretically capacity varies directly with power when head is constant. But in practice, head does not remain constant but it reduces as capacity is increased. Thus power curve rises intially with the increase in capacity but the rate in the rise is reduced due to simultaneous decrease in head. Shape of capacity-power curve is therefore controlled by Head-capacity relatioship. Power curve begins from the ordinate at a cut-off head equal to H/e max.

The capacity-power(BHP) curve(Fig. 6.2) which falls in a straight line with the increase of capacity is a falling curve. The flat curve is one where the BHP varies in a straight horizontal line irrespective of capacity. Rising or overloading curve is one where BHP increases steadily with the increase in capacity.

Overloading curve for a pump is not preferred as it is related to the instable Head-capacity curve. Non-overloading characteristic curve is one where BHP rises upto a designated point (highest efficiency point) with the increase in capacity and then falls as capacity is further increased. The shape of power curve also varies with the specific speed. As a result of which power may have low, medium or high value at shut-off head.

Pumps with non-overloading power curves are preferred since the mover do not get overloaded under an unsuitable working condition. But however they are not available in all specific speed types. In such case, the size of mover should be selected so that it can take load of the pump within its normal range of operation.

6.2.1.4 Capacity - NPSH curve

This curve represents the variation of required NPSH with that of capacity. Required NPSH is a function of pump design. However sometimes it becomes necessary to plot this curve to learn the range of required NPSH. While installing a pump, the use of such curve may be made to make sure that the available NPSH is higher than the required NPSH at the given duty point.

6.2.2 Operating Characteristics

All the main characteristics curves of a pump when placed together in a single graph, the group of curves are known as the operating characteristic curves of the pump.It may be noted that while plotting all the main characteristic curves in a single graph, the unit of capacity in the abcsissa remains the same while the units in ordinate are taken differently as per the parameters under consideration. Operating characteristic curves therefore help in selecting the working condition at which the pumps should operate at its best.

A typical operating characteristic curve of a centrifugal pump is shown in Fig. 6.3. It is clear from this figure that at Head H and capacity Q, the pump will operate at its highest efficieny.The required BHP at this duty point will be P while required NPSH will be h. Therefore for a pump running at a constant speed there will be only one Head-capacity combination at which the pump will oprate with its highest efficiency. In practice however, operation within a range of efficiency level extending in either side of the maximum efficiency point is permitted.

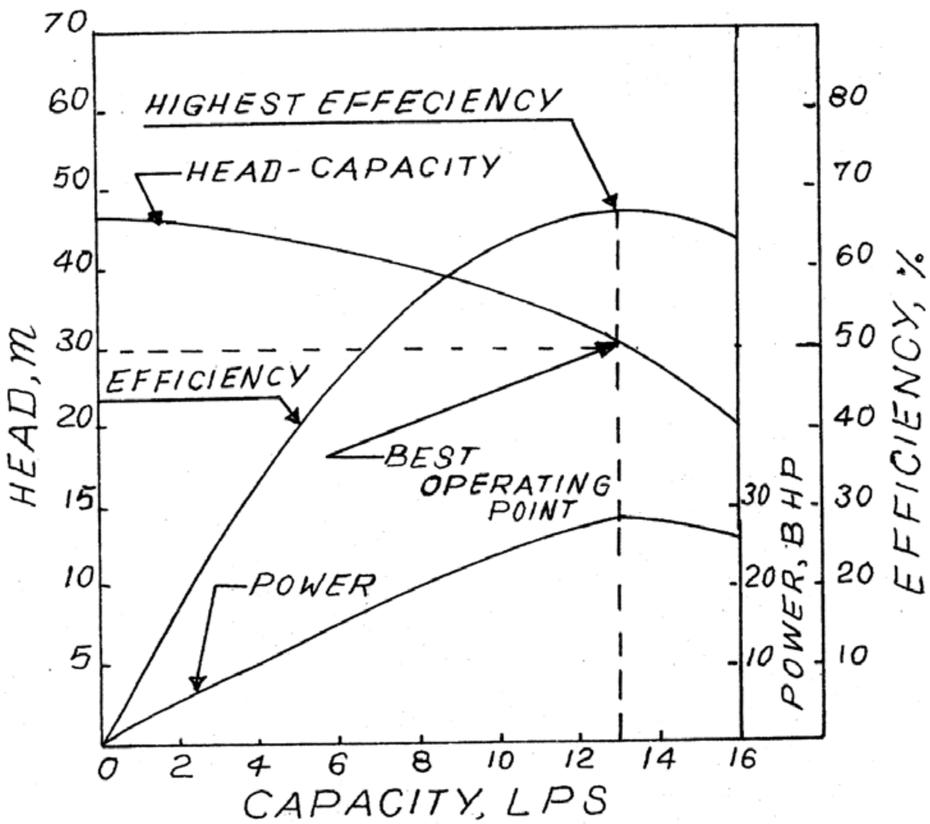

Fig. 6.3 Operating Characteristics of a Centrifugal pump

6.2.3 Iso-Efficiency Curves

These curves permit finding the required speed and efficiency for any given opeating (Head-capacity) condition Fig.6.4 and Fig. 6.5 show iso-efficiency curves for different speeds and for different impeller diameters respectively.

To plot such curves, the capacity-efficiency curves for different speed of a pump is first prepared. A series of horizontal lines (lines of equal efficiency) are then drawn to intersect the efficiency curves. Points at which each of these lines intersect the efficiency curves are transferred to a set of Head-capacity curves at their corresponding capacity and head values.By joining these points of equal efficiency by a smooth line the iso-efficiency curves are obtained. Figure 6.4 represents standard/iso-efficiency curves at different speeds.

With the help of these curves it is possible to determine the change in efficiency levels along a particular Head-capacity curve for a given operating speed. Similarly the iso-efficiency curves(Fig. 6.5) for different impeller diameters permit finding the change in effificency along a Head-capacity curve for a particular impeller diameter (size) of a pump.

6.2.4 Variable speed charateristics

Although pumps are normally designed for constant speed operation, it may however sometimes become necessary to operate a pump at a speed other than its design speed. The change in performance of a pump due to change in operating speed can be predicted from the following relationships.

- Capacity varies directly with speed

  i.e $Q/N = K = Q1/N1$

  or $N/N1 = Q/Q1$ . . . 6.3

- Head varies directly with the square of speed

  i.e $H/N^2 = K = H1/N1^2$ . . . 6.4

- Power varies directly as the cube of speed

  i.e $P/N^3 = K = P1/N1^3$ . . . 6.5

Where,

$N =$	Speed at which performance is known
$N1=$	New speed at which performance is to be obtained
$Q =$	Capacity at speed N
$Q1=$	Capacity at changed speed N1
$H =$	Head at capacity Q and speed N
$H1=$	Head at changed capacity Q1 at the changed speed N1
$P =$	Water Horse power at speed N, capacity Q and Head H
$P1=$	Changed Horse power at speed N1, capacity Q1 and Head H1

The rate of change in Head or capacity due to the change in speed vary from pump to pump based on its design specific speed. If the change in Head or capacity for an installation due to a small change in speed is too large then another pump should be used. But if however the above change due to the change in speed is not very significant then the same pump can be used.

6.3 DESIGN CHARACTERISTICS

While performance characteristics deals with the response of a pump over a range, due to changes in the operating conditions, the design characteristics represent the response in the performance of a pump due to the changes in its design.

6.3.1 Effect of Impellers

6.3.1.1 Impellers in series operation:

For a pump running at a constant speed, head varies directly as the number of impellers in series. In other words the head developed by a pump is increased as the number of impellers in series(stages) are increased and vice versa. In series operation, impellers are mounted on a common shaft all facing towards the suction side.

The delivery of water from the first impeller feeds as suction to the second and subsequently cumulative head is built up in this process. Total head developed in a multistage pump is the algebric sum of heads developed by each impeller separately. Capacity of a multistage pump however do not change appreciably. Multistage pumps available in both vertically and horizontally split casing design are therefore suitable for low to medium capacity and high head operations.

6.3.1.2 Impellers in parallel operation

Capacity increases in direct proportion as the number of impellers in parallel operation. In double suction pumps, two impellers are used for parallel operation. The impellers are placed back to back mounted in the same shaft facing in opposite direction. Both the impellers take their suction from a common suction pipe through which water enters in a direction lateral to the impellers.

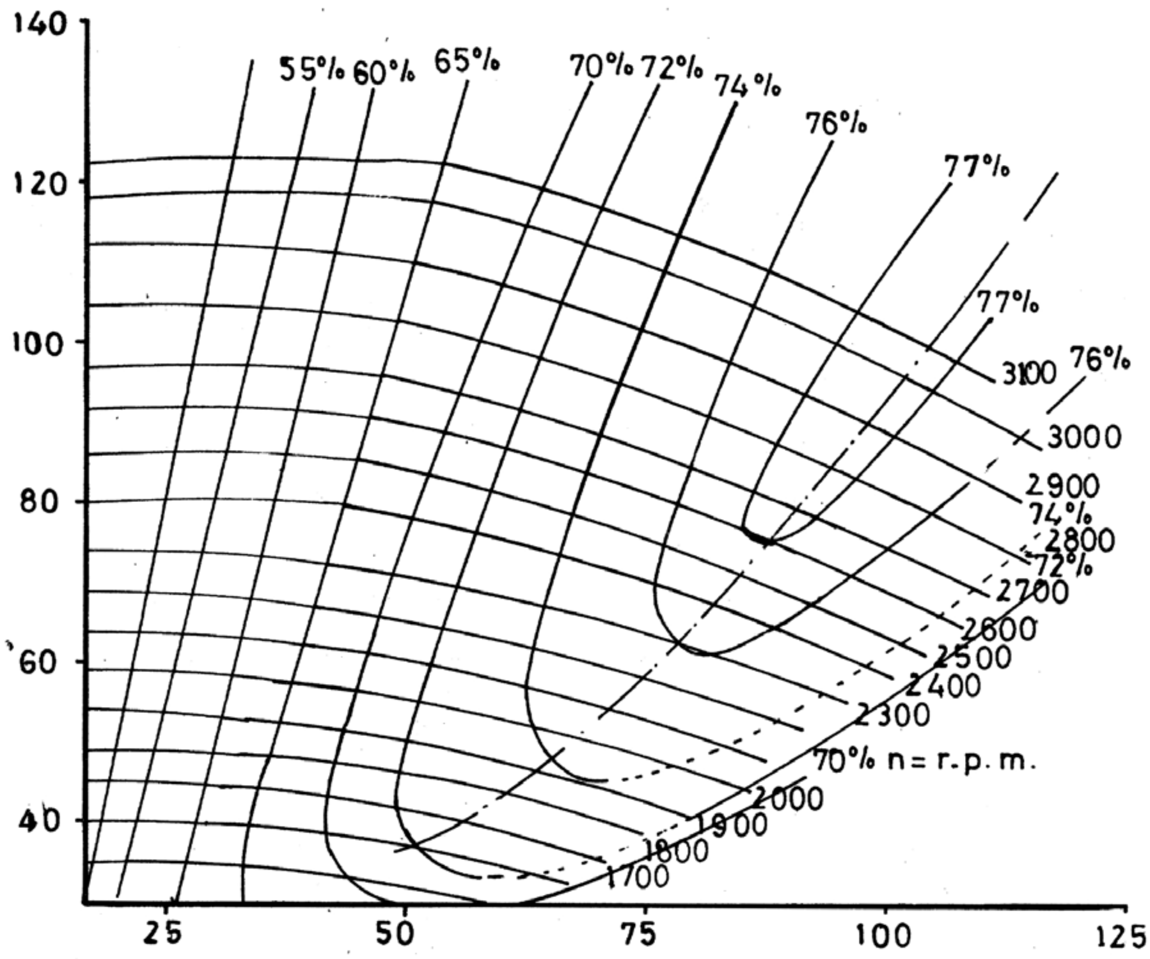

Fig. 6.4 Iso-effeciency curves of Centrifugal pump at different speeds

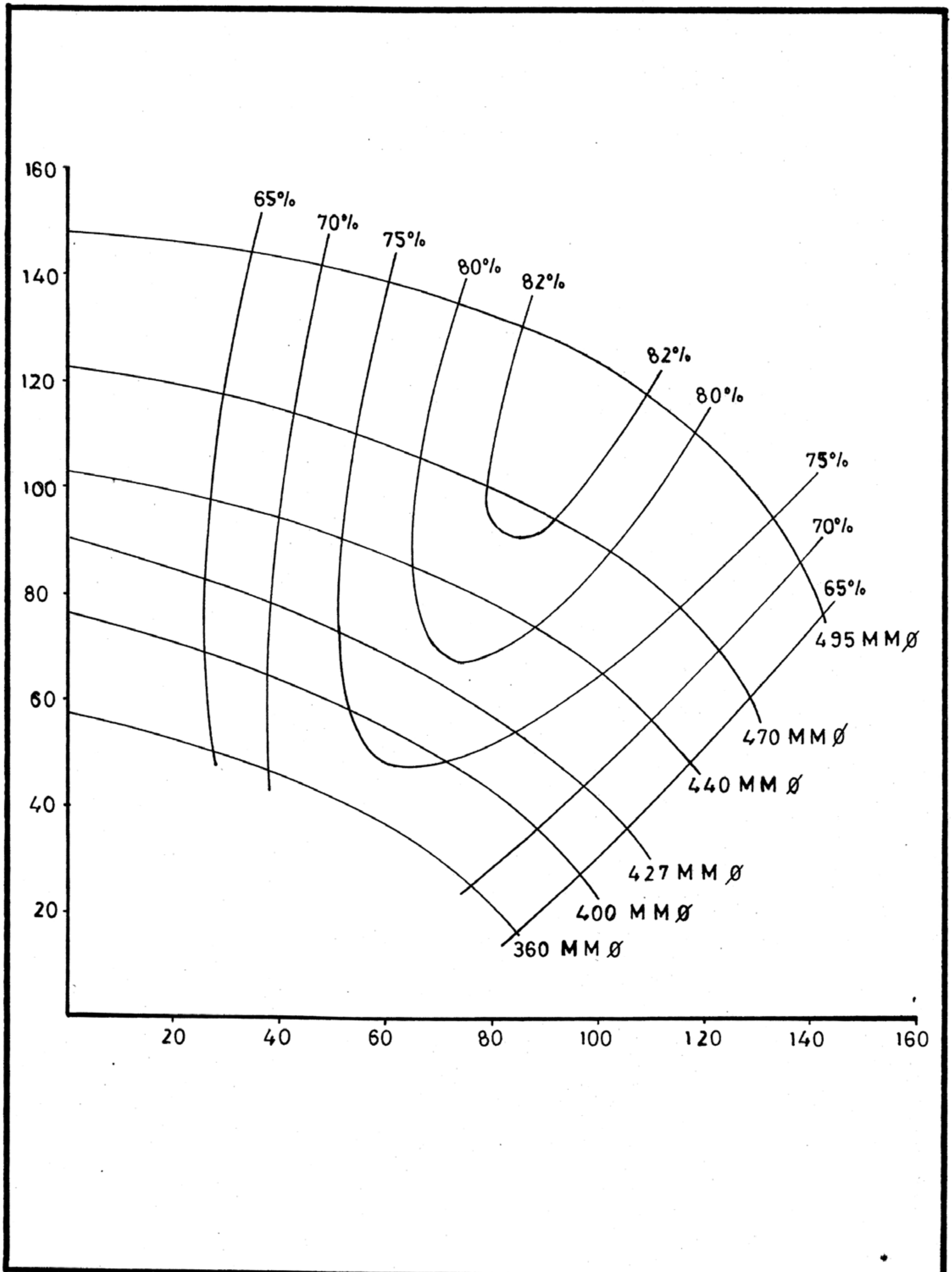

Fig. 6.5 Iso-effeciency curves of Centrifugal pump at different diameters

Discharges from both the impellers joins a common delivery pipe through two short bends curved into the casing. From design point of view it is not normally preferred to use more than two impellers in parallel operation. Double suction pumps are therefore suitable for low head and high capacity installations.

6.3.1.3 Effect of change in impeller diameter

The changes in capacity, Head and power consumption with respect to impeller diameter are as follows.

- Capacity varies directly as the impeller diameter.
- Head varies directly as the square of impeller diameter.
- Water Horse Power varies directly as the cube of impeller diameter.

Mathematically thus :

$$D/D1 = Q/Q1 = \sqrt{H/H1} = \sqrt[3]{P/P1} \qquad \ldots \quad 6.6$$

Where,

D = Original diameter
D1 = Changed diameter
Q = Capacity at orignial diameter
Q1 = Capacity at changed diameter D1
H = Head at original diameter and capacity
H1 = Head due to changed diameter and capacity
P = WHP with original impeller at corresponding capacity and Head
P1 = Changed WHP at changed capacity and Head corresponding to the changed diameter

The above relationship therefore can be used in predicting the change in performance of a pump due to a change in its impeller diameter.

In practice, since measurement of changes in operating speed of an electric motors is difficult, it is practical to modify the diameter of the impeller to obtain a particular performance at a constant speed.

It is obvious that increasing the diameter is not possible due to limited casing size. Only reduction of diameter is possible by grinding it in a leathe. Impellers can be actually grinded down upto about 20% without significantly affecting the efficiency. In propeller pump sometimes a set of interchangeable impellers of different diameters are used to obtain the required performance.

Problem 6.1

At 1800 rpm, a 20 HP centrifugal pump with impeller diameter of 15 cm discharges 100 cu.m/hr at a head of 30 m predict the changes when,

a. the pump is run at 1600 rpm

b. the impeller diameter is reduced to 12 cm.

Solution

The capacity(Q), head(H) and power(P) decreases in the following proportions.

a. $Q = 100(1600/1800) = 88.88$ cu.m/hr

$H = 30(1600/1800)^2 = 23.70$ m

$P = 20(1600/1800)^3 = 14.04$ HP

b. $Q = 100(12/15) = 80$ cu.m/hr

$H = 30(12/15)^2 = 19.2$ m

$P = 20(12/15)^3 = 10.24$ HP

6.3.1.4 Impeller (Vane thickness)

Capacity of a pump varies directly with the diameter of the impeller, thickness(breadth) of the impeller, curvature of the vane and number of vanes in the impeller. Thus mathematically

$$Q = Cf(D\ b\ Vf) \quad \ldots \quad 6.7$$

Where,

D = Diameter of impeller

b = Breadth of impeller at inlet

Vf = Velocity of flow at inlet

Cf = Contraction factor due to reduction in the area of flow due to vane thickness which normally varies between 0.8 to 0.9

6.3.1.5 Number of vanes

The number of vanes should be adequate so that they can guide the water properly. However, if the number of vanes are too may then the surface area will increase resulting more frictional loss. The number of vanes to be used therefore depend upon the shape of the vanes. In actual contruction the number of vane is decided based on experience. With the number of vanes increasing(within a practical limit), the Head-capacity curve becomes flat. Increase in number of vanes beyond a practical limit may increase the frictional loss so high that the Head-capacity curve becomes steeply rising.

6.3.1.6 Vane angle at inlet

Entry of water at the inlet is normally radial.
Hence when ? = 90 degree, tan ? = Vf

where, Vf = velocity of a vane.

Since inlet diameter, speed and velocity of flow at inlet (Vf) are known, ? can be calculated. The vane angle normally is made between 10 degree to 25 degree.

6.3.1.7 Vane angle at outlet

Normally vane angles at the outlet are made curve backwards. This is so because it gives a rising Head-capacity curve. Absolute velocity of water leaving a vane is minimum when it is curved backward so that the remaining energy can be used up in building the pressure head. Outlet vane angles are normally kept between 15 degree to 35 degree.

6.3.1.8 Shape of vane

Vanes should be so shaped that the water passage is not unnecessarily long. Otherwise frictional loss would increase. Also there should be gradual change in area of flow to avoid creation of turbulence.

6.3.1.9 Shaft diameter

Shaft diameter should be of sufficient dimension so that it can take the necessary torque and bending moment at the designed speed. To prevent bending of the shaft its diameter should be a function of shear stress and tensile stress of the metal used for the shaft. Pump shafts are normally made of mild steel. The weight of impeller is made such that it balances its radial thrust. Unbalanced radial thrust should not be such that the shear stress and tensile (bending) stress for a steel shaft exceed 280 kg/sq.cm and 420 kg/sq.cm respectively.

6.3.1.10 Hub diameter

It is normally made 1.5 cm larger than the shaft diameter depending upon the shaft diameter.

Thus, $Dh = Ds + 1.5$ cm . . . 6.8

6.3.1.11 Eye diameter

The velocity of flow at the eye of the impeller is normally slightly larger than the velocity of flow in the suction pipe. Eye diameter should be designed to an optimum size. Velocity of water moving into the pump should be low enough so that it does not cause high frictional loss due to turbulene. On the other hand a low velocity of flow at the impeller will require a large eye diameter thus increasing impeller diameter beyond proportion. It is therefore necessary to maintain a optimum eye dia so that water enters with adequate velocity but do not have too much turbulence. Normally 3 m/sec is considered to be optimum velocity at the suction eye.Diameter of impeller at the inlet is normally made equal to the eye diameter so as to ensure smooth flow particularly in radial flow impellers.

6.3.1.12 Specific speed

It is a term used to classify a pump in a standardised but theoretical form. Specific speed is defined as the speed in rpm at which a pump impeller wil rotate if it is reduced to such a size that it delivers 1 cu.m/sec of water against 1 meter head.

The term specific speed involves relating Head, capacity and speed into a single term for use as an index to compare one pump with another. From the expression of specific head,

$$N = 3.65\ n\ \ Q/H^{3/4} \qquad \ldots \quad 6.9$$

It is clear that

Between two pumps having same speed n and capacity Q, the pump with higher specific speed will have lower head.

Between two pumps of same speed and head, the pump having higher specific speed will have higher capacity.

Centrifugal pumps are also classified in term of its specific speed. Radial flow pumps which are normally used for high head installations have therefore lower specific speed (90-220). Mixed flow pumps preferred for medium head and medium capacity installations have medium specific speed (300-600). Propeller pumps which are exclusively used for low head and high capacity installations have higher specific speed (800-1300).

Higher specific speed means smaller impeller diameter leading to less friction and higher efficiency. Higher specific speed for a given Head-capacity point will mean increase in speed which may cause cavitation. Again higher specific speed means less head, requiring more stages with longer shaft causing greater shaft deflection and hence less reliability. Selection of pump specific speed therefore will determine the suitability of that pump for the required performance.

6.4 OPERATING CONDITION

Performance of a centrifugal type of pump also depends largely upon its operating condition. Factors which affect the performance of a pump considerably are discussed below:

6.4.1 Nature of fluid

The nature of fluid to be handled by an irrigation pump is water. Though change in property of water within a permissible ambient condition do not affect the pump performance in a large way, effect of the following changes in pumping are however worth mentioning.

6.4.1.1 Effect of specific gravity

A pump will operate to its designed capacity and Head irrespective of specific gravity (density) of water. But due to change in specific gravity, specific weight of the raised water column will change. This will cause a change in the water Horse Power requirement. Therefore a pump will consume higher power when it has to handle water with higher specific gravity.

Change in specific gravity will also affect the suction condition. The column of water raised by the atmospheric pressure is inversely proportional to the specific gravity of water. The NPSH available for denser (heavier) water will therefore be slightly less than standard water.

6.4.1.2 Effect of temperature

Temperature affects specific gravity of water. Specific gravity of water is unity at the temperature of 4 degree centigrade. Specific gravity reduces exponentially as temperature is increased.

Temperature also affects the vapour pressure which in turn affects the NPSH at the suction inlet. Viscosity of water is also affected by temperature. An increase in temperature reduces the viscosity and vice versa.

6.4.1.3 Effect of viscosity

Frictional loss due to flow increases with the viscosity. As a result, when viscosity increases, head and capacity reduces thus lowering the efficiency and increasing the power consumption. Increase in viscosity will also lower the available NPSH at the suction end.

6.4.1.4 Effect of turbidity

The impeller design is modified depending upon the amount and size of suspended material in water. For example, a closed impeller handles clear water while semi-open or open impellers are necessary for handling dirty water. The suspended material sometimes gets lodged in the impeller or in between the impellers in multistage pump.

Periodical cleaning of such impellers are possible in case of horizontal split casing pump by just removing the top half of the casing while leaving the rest of the pump undisturbed. Impellers of different materials are used while handling sandy and gritty water of abrassive nature or acidic water of corrosive nature.

6.4.1.5 Fluctuation in power input

Pumps are normally designed for constant speed operation. But in practice,both electric motor and oil engines may suffer from variation in rpm due to fluctuation in power input. Both the Head and capacity fall resulting in a fall of efficiency when power input is reduced and vice versa.

6.4.1.6 System Head

As a pump normally delivers water through a network of pipelines with various fittings, the resulting frictioal losses are added up with the static delivery head against which the pump has to work. The curve showing the variation in friction head loss due to the pipe system over a wide range of duty is known as system head curve.

The system head curve do not begin from the origin as the existing static head is added with the friction head and the curve increases exponentially with capacity. There may a be situation where static head also changes with time. This normally happens due to fall in pumping water level(drawdown). In such cases it is necessary to plot different system head curves to obtain the whole range of pump performance (Fig. 6.6).

6.4.1.7 Suction condition

This is the single most important condition to be considered when the pump operates with a suction lift.·Favourable suction condition for a horizontal centrifugal pump is absolutely essential. It is necessary to contain the working suction lift within the permissible suction lift of the pump, otherwise cavitation will result and finally the pump may stop. Suction lift is developed due to atmospheric pressure.

It is well known that theoreotically, at mean sea level, the atmospheric pressure is equal to 10.34 m of water column at 15 degree centigrade . But in practice, the total suction lift is reduced due to various losses depending upon the working condition. To obtain the permissible suction lift of a pump it is therefore necessary to take these loses into account. The corrections which should be applied are:

6.4.1.8 Friction loss

Friction losses due to the suction pipe, bends, foot valve, entry loss and velocity head, all put together may contribute to a considerable extent to the reduction of permissible suction lift. This total amount of head loss due to friction should therefore be substracted from the theoretical permissible sucution lift.

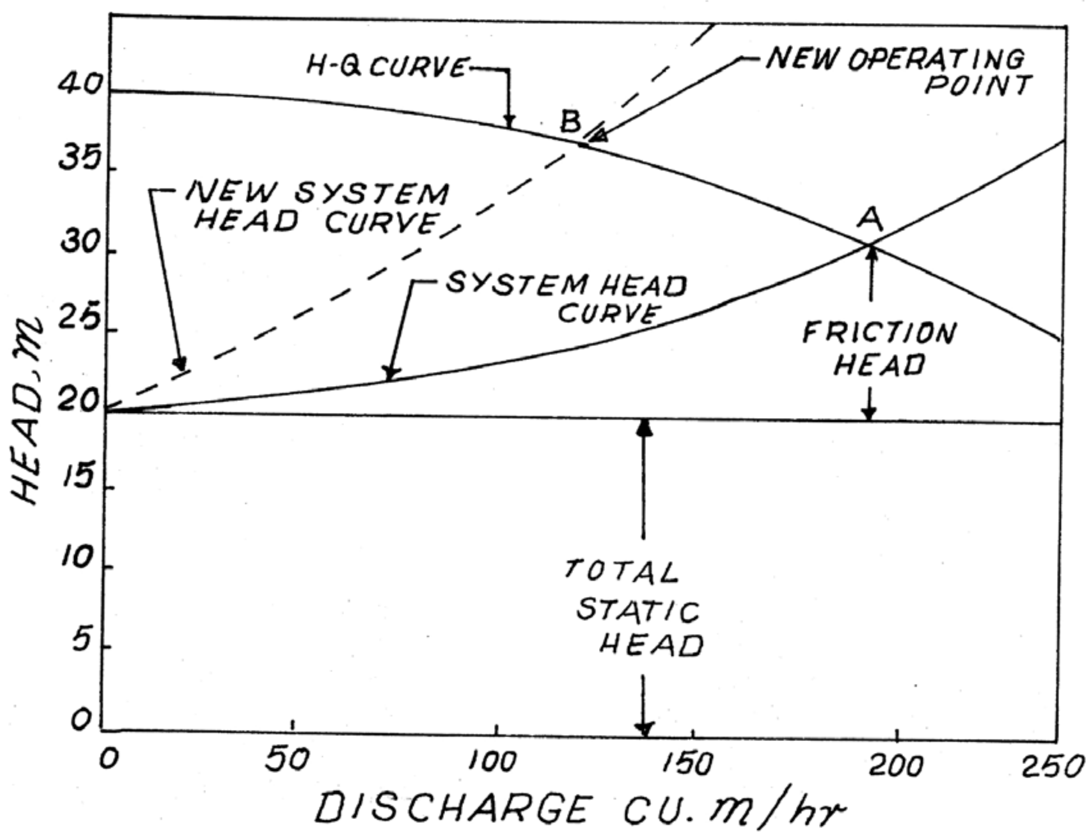

Fig. 6.6 Dynamic system friction head curve

6.4.1.9 Vapour pressure:

Table 6.1 Variation of vapour pressure with temperature at mean sea level

Temperature degree F	Temperature degree C	Vapour Pressure (absolute) Metre	Temperature degree F	Temperature degree C	Vapour Pressure (absolute) Metre
50	10.0	0.125	100	37.7	0.670
60	15.5	0.180	110	43.3	0.902
70	21.1	0.256	120	48.8	1.204
80	26.6	0.356	130	54.8	1.585
90	32.2	0.500	140	60.0	2.066

It can be noted from the above table that at 100 degree centigrade the vapour pressure at MSL is equal to 10.34m. The theoretical suction lift at 100 degree centigrade is therefore zero. In practice however pumping will stop even at a lower temperature(160 degree F) due to extensive cavitation.Vapour pressure affects NPSH which is clear from the following expression:

Avail.NPSH = Pressure head due to atmospheric pressure - all friction in suction pipe - static suction lift - vapour pressure

or NPSH = Ha-Hs-Hvp

Higher the vapour pressure at pumping temperature lower will be the available NPSH. Other factors remaining unchanged, for a given installation, the available NPSH will be determined by the vapour pressure of water at the pumping temperature.

For a pump to run smoothly, it requires certain minimum NPSH below which cavitation starts. Required NPSH is therefore that NPSH a pump will require to operate without cavitation. Required NPSH is a function of pump design and the capacity. It varies directly as the square of capacity. Design parameters like suction eye diameter, number and shape of vanes, impeller diameter and specific speed etc. have a bearing on required NPSH.

It is therefore obvious that if at any stage the required NPSH becomes higher than the available NPSH, the pump will stop functioning. For smooth and cavitation less operation. NPSH available should be greter than NPSH required.Water to be pumped at a higher temperature should therefore be supplied under a positive suction head.

6.4.1.10 Altitude

The atmospheric pressure changes with altitude. It is equal to 10.34m of water colum at 15 degree Centigrade when measured at the mean sea level.It is well known that atmospheric pressure decreases with increase in altitude. Rate of such variation is presented in the table below.

Table 6.2 Variation atmospheric pressure with altitude

Altitude above MSL (m)	0	500	1000	1500	2000	3000	4000	5000
Atmospheric pressure in water column (m)	10.34	9.8	9.2	8.6	8.1	7.2	6.3	5.6

A correction is normally applied to the permissible suction lift due to the reduction of atmospheric pressure at a rate of about 0.36 m for increase of every 300 m of elevation above M.S.L.

6.4.1.11 Seasonal barometric pressure

Even at a particular site, atmospheric pressure may change from season to season. There may be a period of Barometic depression due to weather condition. Some margin for such variation to the suction lift should also be provided for. Normally a margin of about 0.35 m to take care for such variation is considered adequate.

6.4.1.12 Drawdown

Drawdown is the extent of fall in pumping water level. If the water surface is expected to have a drawdown, a margin equal to the expected maximum drawdown should be included while deciding the permissible suction lift.

6.5 TYPE OF INSTALLATION

Type of installation may also play a great role in deciding the type of pump to be selected. Type of installation is decided by the shape and size of the intake structure, cost of such structure and construction facilities. Water intake structures may be:

- Open well/collector well
- Tube well
- Open water body

Both horizontal and vertical centrifugal pumps can be used in open well depending upon whether it is a dry pit or a wet pit installation. For tubewells, either submersible or vertical turbine pumps are conveniently used. For tubewells with very shallow water level horizontal centrifugal pump can be used either by directly coupling the suction pipe with the tubewell casing or lowering the suction pipe with a foot valve into the well.

6.6 OTHER CONSIDERATIONS

A normal user while selecting a pump also looks for the following advantages in a pump.

Highest efficiency

The present day pumps are of quite advanced design. It is however not possible to go on increasing the efficiency of a pump by merely changing the design. Pumps of lower efficiency due to bad material however should be avoided. Highest efficiency from a particular pump is obtained only by its proper installation.

Lowest NPSH

To make NPSH to the lowest, several considerations are to be made. Lowest NPSH is desired because it gives highest permissible suction lift. This can be achieved by higher specific speed but higher specific speed means lower head and higher vulnarability to cavitation.

Greatest reliability

This is very much possible and should be adopted looked for.

Widest range of operating capacity

This however is not desirable. Pump should preferably be selected for that particular duty point (Head-capacity values), at which it is expected to perform.

Longest life

There is no reason why this requirement cannot be satisfied. There cannot be any compromise on this point.

Lowest price

It should not be the sole criteria. It is the performance/cost ratio that should be considered than the cost alone.

Availability

Although one may arrive at one's requirement of the most ideal pump but may have to finally choose the nearest suitable pump which is readily supplied by the manufacturer or the local dealer.

Choice of pump may also be affected by the factors like supply of power, initial cost, operating cost,versality,noise level,maintenance facilities etc.

Fortunately a wide range of pump suiting to all irrigational purposes are available in our country.Beginning with only three manufacturers in 1920 India has today more than 500 registered manufacturers. The various range of centrifugal pumps available can be listed below :

Table 6.3 Range of pumps available in the country

Type of pump	Maximum discharge lpm	Maximum Head m
Multistage horizontal	420	1300
Single end suction with radial and mixed flow impellers	6500	165
Single stage non-clog	1000	100
Horizontal split casing	13,400	400
Horizontal self priming	165	80
Turbine with radially mixed and axial flow impellers	7,700	500
Submersible 4" - 12" dia. 1.5 -115 HP	280	225
Reciprocating	58	4200
Rotary	97	200

For ready selection of a pump of a particular series, the family curves or performance table are supplied by the manufacturers. Some manufacturers supply the test performance curves for proper selection, while others provide published tables giving the entire range of capacity at different heads.

Purchase data:

It is therefore recommended that the following inventory be first prepared which shall help in making the correct selection of a pump.

- Number of pumps required
- Type of drive
- Capacity required
- Total head

- Length and size of suction and delivery pipes
- Number and type of pipe fittings
- Altitude of the operating site
- Maximum and minimum temperature round the year
- Maximum and minimum depth of water
- Humidity and weather
- Quality of water
- Lay-out plan showing the pump and pipe lines with fittings.

Problem 6.2

A centrifugal pump is to be installed at an altitude of 500 m for handling water at 32 degree centigrade. If the suction lift is 4.0 m and other estimated friction losses in the suction pipe, bends, fooot valve, strainer etc., are 0.5 m, what would be the available NPSH .

Solution

NPSH = Ha - Hs - Hvp - Hf

Here,

Ha = 9.8 m (From table 6.2)
Hs = 4.0 m
Hf = 0.5 m
Hvp = 0.49 (From table 6.1)

NPSH = 9.8 - 4.0 - 0.5 - 0.49
= 4.59 m, say 4.6 m

In selecting a pump for the above condition, the required NPSH as can be obtained from the characteristic curve at the pumping capacity should be 4.6 m or less(a margin of safety upto 0.6 m) is however advisable.

Problem 6.3

At 5 lps a Centrifugal pump requires 3 m NPSH. If this pump is installed at an altitude of 1000 m to pump water at 26 degree centigrade, what will be the maximum practical suction lift at which the pump can operate satisfactorily.

Solution

NPSH = Ha - Hs - Hvp

From table 6.1 and 6.2,We have

Ha = 9.2 and Hvp = 0.336
or 3.0 = 9.2 - Hs - 0.336
or Hs = 5.864 m

The maximum suction lift with a margin of safety(0.64 m) is 6.5m.

CHAPTER 7

SELETION OF PIPE AND FITTINGS

7.1 SELECTION CRITERIA

Pipes are integrated part of a pumping system. In any lift scheme, selection of proper pipes are as important as that of the pump. Pipes not only comprise a major part of the initial investment but improper selection of these will also adversely affect the operating and maintenance cost. At times, complete failures of a scheme may result due to an improper selection of pipe.

Selection of pipe requires proper decision on its diameter, material and cost keeping in view the working condition and availability of such pipe in the market. The basic considerations that finally lead to the selection of pipe are discussed below.

7.2 WORKING PRESSURE

Pipe material and its thickness should be such that it can withstand the maximum pressure generated inside the pipe during its use. Working pressure is the pressure exerted to the internal surface of a pipe by the fluid during its flow. Total internal pressure however is the sum of fluid pressure and water hammer. For safety of pipes, it is recommended that the test bursting pressure should be minimum of 3-5 times greater than the estimated working pressure.

The hydrostatic pressure in a pipeline can be determined from its pressure head i.e. p = hw. The dynamic pressure is however higher than static pressure since extra pressure builds up due to the momemtum of water(water hammer). Since 10 m head of water will exert 1 kg/sq.cm of pressure at its bottom, in other words a pipe rated for 1 kg/sq.cm of working pressure can withstand static pressure equivalent to 10 m of water column. In the case of a constant head gravity flow, the pressure in a pipeline drops as it flows along the pipe, by an amount equal to the frictional loss. Conversely in case of pumped flow pressure at the pump impeller (also in the pipelines) increases as water is lifted higher and higher.

For a long pipeline, therefore, it is economical to use pipes having higher working pressure close to the pump and use pipes of lower working pressure as moved away from the pump provided the pressure due to water hammer is taken into consideration for the entire length.

7.3 WATER HAMMER

When flow of water through a pipe is stopped abruptly by sudden closure of a valve or stopping of the pump, the whole moving column of water tend to come at rest at once. As a result a series of expansions and contractions of the flow take place causing pulsating or serging pressure inside the pipe before the water comes to rest. Such effect is knows as water hammer and the resulting pressure as serge pressure. As such water hammer effect takes place due to any sudden change in the velocity of flow. Magnitude of such serge pressure could be as high as 15-20 times that of hydrostatic pressure.

Water hammer can mathematically be expressed as

$$P = M\ dv/dt \qquad \ldots\ 7.1$$

However Maximum pressure developed due to water hammer is given by

$$P = \frac{14.76\ V}{[(1+KD)/t]^{0.5}} \qquad \ldots\ 7.2$$

Where,

V = Velocity before obstruction,m/sec
t = Thickness of pipe,m
D = Diameter of pipe,m

$$K = \frac{\text{Modulus of eleasticity of pipe material}}{\text{Bulk modulus of elasticity of water}}$$

K = 0.01 for steel pipes
= 0.02 for cast iron pipes
= 0.1 for cement concrete pipes

Value of P is high in small diameter pipes which diminishes with the increasing diameter of the same.

Following are the recommended values of water hammer pressure in pipes of various diameters which should be provided for while assessing the working pressure.

Table 7.1 Recommended value of water hammer pressure in pipes of different diameter

Pipe dia cm	7.5 to 25	30 to 40	50	60	75	90	Above 100
Water hammer Kg/cm^2	8.4	7.7	6.3	6.0	5.6	4.9	4.9

Effect of water hammer however could be reduced by using reflux of non-return valves in the pressure line. In normal operation, valves should therefore be opened or closed gradually. In gravity mains effect of water hammer can be reduced by introducing a "break pressure tank" or a "serge tank" in the pipe line. Simplest type of serge tank is a long cylindrical chamber connected with the pipeline at a suitable distsance from the pump. Water within the serge tanks remain open to atmosphere and moves up or down as per the pressure fluction within the pipeline.

While selecting pipe material, it is therefore necessary to provide not only for the static pressure but also for the water hammer and other dynamic forces coming into play.

7.4 INTERNAL PRESSURE

The pressure exerted on the walls of the pipe by flowing water when expressed in the form of Hoope's tension is given by

$$\text{Circumferential stress} = \frac{pD}{2t} \qquad \ldots \quad 7.3$$

p = Internal static pressure, kg/sq.cm
D = Diameter of pipe, m
t = Thickness of the pipe, m

The circumferential tensile stress is counteracted by providing hoop's reinsforcement which is a function of thickness of the pipe . The pipe thickness should therefore be adequate.

7.5 EXTERNAL PRESSURE

Application of external pressure to a pipe line may be either due to the vertical stress originating from the weight of the pipe itself in between the two support pillers or external load of earth fill when burried underground. When a large diameter pipeline is burried underground the weight of the earth fill produces some stress in the pipe material.

Stress due to earth fill load is given by

$$S = \frac{0.227 \quad hD^2}{t} \qquad \ldots \quad 7.4$$

Where,

S = External stress produced by earth fill, kg/sq.cm
h = Depth of burrial i.e. heigh of earth fill above the pipe surface, m (Weight of earth is assumed as about 1840 kg/cu.m)
D = Diameter of the pipe, m
t = Thickness of the pipe, m

For buried pipelines, precautions against other source of stresses like load due to traffic, expansion and contraction of earth etc. should also be considered.

7.6 AMBIENT TEMPERATURE

Variation of atmospheric temperature should also be considered while selecting pipe material for use in a particular location. Pipeline when laid over ground is subject to expansion and contraction due to the increase and decrease in atmospheric temperature respectively.

The pipe material chosen should therefore be such that its coefficient of linear expansion within the lowest and the highest temperature range should not be too large to cause damage to the pipe joints and support pillers. For large diameter overground steel pipes, expansion joints are provided. These are flexible joints which can accomodate certain amount of expansion or contration. Simplest type of expansion joint is made by welding two pipes whose faces are connected with flexible and flared steel.

Very low ambient temperature also assumes great importance in cold region. Frozen water may burst a pipe line since it expands in its volume in sub-zero condition.

7.7 FRICTION IN PIPE

It is well known that smoother the internal surface of a pipe lesser would be the frictional loss offered by the pipe. Thus power consumption for a given system would be lower when a smmother pipe line is used. Smoothness depend upon the material used for the pipe. Having other criteria like durability, cost, availability, working pressure, satisfied, selection of pipe should be such that friction factor be as small as possible.

7.8 GROUND CONDITION

When pipelines are laid undergound, attention should be paid on the ground conditiion. Pipe material should be innert against any possible reaction with the chemicals that may be dominantly present in the soil. such reaction may be mild in nature but nevertheless can in the long run cause rusting, corrosion or deteriotion of the pipeline.Certain clay soil expands considerably when soacked with water. Heavy expansion and contraction of such soil may damage underground pipes if the material choosen is not strong enough to withstand the resulting stresses.

7.9 HANDLING OF PIPES

Even if most other considerations are similar, sometimes pipes of one material may be preferred over another on the ground of easy handling, transportation and storing. Factors like method of jointing, weight of each pipe length, chances of damages in handling and easy laying etc. affect the choice of pipe of one material to another.

7.10 QUALITY OF WATER

Attention should be paid in the selection of pipe material if the water to be transmitted has adverse chemical quality.Water may be highly acidic or alkaline. It may contain disolved gasses or corrosive chemicals. Such water damages pipes normally by corrosion or by scale formation inside the pipe (incrustation).

7.11 DISCHARGE REQUIREMENT

It is well known that discharge through a pipe of a given diameter is directly proportiosnal to the velocity of flow. In practice, however, velocity can not be permitted to increase without any reservation. Increase of velocity beyond a particular level will increase turbulence causing high frictional loss.

Th diameter of pipe should be such that the total pressure head is adequate to overcome frictional resistance and maintains a good flow at the same time the resulting velocity should not be too large to cause excessive loss of head due to friction. Normally head loss due to friction is kept contained within 10 percent of the total pressure head.The required diameter for a given discharge is obtained by using a number of approximate formulae. Some of such useful formulae are:

7.11.1 Lea's formula

$$D = 0.97 \text{ to } 1.22 \quad Q^{0.5} \qquad \ldots \quad 7.5$$

Where,

D = Diameter, m
Q = Discharge, cu.m/sec

The above formula will result in optimum velocity of water between 0.8 to 1.35 m/sec.

7.11.2 Hazzen-William's formula

$$Q = 990 \times C \times S^{0.54} \times D^{2.63} \qquad \ldots \quad 7.6$$

Where,

C = coeffecient which reanges from 100-150 for common pipes
S = Hydraulic gradient

7.11.3 Scobby's modiefied formula

$$D = 0.29 \times S^{-0.1875} \times Q^{0.375} \qquad \ldots \quad 7.7$$

Where,

D = Diameter,m
S = Hydraulic gradient
Q = Discharge, cu.m/sec

7.11.4 Emperical formula

$$D = [Q/(0.06)]^{0.5} \qquad \ldots \quad 7.8$$

Where,

D = Diameter, mm
Q = Discharge, lpm

7.12 COST OF PIPE

Primary consideration against using pipes with best of material and adequately large diameter is of course the cost factor. As the cost of pipe made from diferent materials veries so does its life and hydraulic performance. There is no such pipe which is of low cost and at the same time fulfill all the requirements under most situations. The relationship between cost and diameter of pipe is given by $c = Ad^B$ where A and B are coefficients. This means that cost of a pipe depending upon the values of A and B increases in a geometric progression with the increase in its diameter. An optimization is therefore necessary to keep the cost factor low.

7.13 AVAILABILITY

Sometimes the right type of pipe may not be easily available in the market and even to the manufacturer. At times the carrying cost of certain pipes may become as high as the cost of the pipe it self. Similarly pipes of a prefered thickness, diameter, length, and jointing facility may not be readily available. Under all such circumstances a working compromise may have to be made for the next alternative of pipe available in the market.

As pipe is a high cost item it should be available within a reasonable time frame from the time of placement of orders. It is therefore important to have adequate knowledge about the types of pipe available in the market and variations in their costs.

7.14 TYPE OF PIPES

A wide range of pipes of different materials, dimensions and pressure requirements are available in the market. Pipes having frequent use in irrigation are discussed below:

7.14.1 Vitrefied clay pipes

These pipes are made from special type of clay. The clay mixture is moulded into pipe shape which is later backed to the required hardness. Both the interior and exterior of these pipes are made highly glazed. This is done to prevent absorption of water by clay. Standard sizes are in 0.6 - 1.2 m length with 10-45 cm diameter. Jointing of socket and spigot ends are done by rich cement morter. They are non-corrosive but are brittle thus requiring careful handling. Once laid properly lasts long. vitrefied clay pipes are used mostly for underground sewage line or low pressure distribution lines. They are never used as mainline.

7.14.2 Asbestos Cement Pipe

These are made from asbestos fibre mixed with cement. The mixture is first rolled into lamenated material and then made into pipes. They are normally available in 3 m length upto a diameter of 10 cm and in 4 m length above 10 cm upto 20 cm. Jointing are done by detachable couplers, asbestos cement coupler, concrete collars and compressed rubber ring joints. Other technical data are presented in the tables below

Table 7.2 Pressure rating of asbestos cement pipes

Class of Ac pipes	Hydraulic test Pressure	
	N/m m^2	Kg/cm^2
5	0.5	5
10	1.0	10
15	1.5	15
20	2.0	20
25	2.5	25

Table 7.3 Ratio of Bursting pressure (BP) to working pressure(WP) and test pressure (TP) for asbestos cement pipe

Nominal dia cm	BP/WP	BP/TP
5.0 - 10.0	4	2
12.5 - 20.0	3.5	1.5
25.0 - 60.0	3.0	1.0

Note:
Working pressure should not be more than 50% of test pressure in pumping mains and 67% for gravity mains.

Table 7.4 Thickness of abestos cement pressure pipe for different diameters and class of pipes

	Thickness of pipe in m.m.				
Nominal dia cm	class 5	class 10	class 15	class 20	class 25
8	9.5	9.5	9.5	11.0	13.5
10	9.5	9.5	10	13.5	16.5
12.5	9.5	9.5	11	14	17.5
15	9.5	9.5	13	16.5	21
20	9.5	11.5	16.5	2	27.5
25	9.5	12	17	23	28.5
30	9.5	14	20	27	34.5
35	14.5	14.5	21	27.5	35
40	16.0	16	24	32	39.5
45	17.5	17.5	26.5	35.5	44
50	19.5	19.5	29	39	48.5
60	23.5	23.5	35	46	58

7.14.2.1 Applicability

- Smooth internal surface having low friction
- Immune from the attack of corrosive soil and water
- Light in Weight
- Fragile and soft and hence can not take much load. It needs to be protected from surface impact and careful handling.The pipe becomes useless once it develops a crack or breaks at ends.
- Leaking of joints and of pipes should be checked before laying.
- Suitable for low-medium underground pressure use. These are not preferred as main lines.

7.14.3 Cement Concrete Pipe

These are the most commonly used pipes for sewage, drainage and irrigation. Non reinforced cement concrete (NRCC) pipes are made by plain moulding of suitable aggregates. They are normally, hand made, casted in about 1 m length of required diameter. Care should be taken to make good quality pipes by using good quality cement in a homogeneous mixture of aggregates. These are low cost pipes with limited application where working pressure do not normally exceed 0.6 KG/Cm .

Reinforced cement concrete (R.C.C.) pipes are moulded with suitable reinforcement of woven steel wire, rods or plates to provide greater strength and hence increased applicability. They can be both hand casted or spun. Spun pipes are machine casted where the aggregates are homogeneously mixed by centrifugal motion to provide greater strength. Prestressed concrete pipes can be made with steel plate reinforcement through advanced technology. Premo pipes one example of this type.

Jointing is normally done by using cement morter over spigot-socket or tongue-groove ends. R.C.C. collars with cement morter properly placed can be used for jointing plain face to face or tonge and grove ends. Rubber gaskets are sometimes used in socket-spigot joint to make joints water tight.

R.C.C. pipes are usually made extending to 2-2.5 m in length with diameters ranging from 15 to 45 cm. Large diameter pipes with requisite reinforcement can be moulded as per need.

From working pressure point of view, these pipes are manufactured into two categories namely presure pipes (p class) and non-pressure (NP class) as per specifications laid down by Indian standard institute(ISI).

The pipe and requisite coller thickness of Non pressure and pressure pipes in relation to their diameters are furnished in tables below:

Table 7.5 Recommended barrel thickness for Non-pressure cement concrete pipes

Internal dia mm	NP1 NRCC	NP2 R.C.C	NP3 R.C.C	NP4 R.C.C
80	25	25		
100	25	25		
150	25	25		
250	25	25		
300	30	30		
350	32	32	75	
400	32	32	75	75
450	35	35	75	75
500		35	75	85
600		40	80	85

700	40	80	95
800	45	90	100
900	50	100	115
1000	55	100	115
1100	60	115	120
1200	65	115	135
1400	75		140
1600	80		150
1800	90		

Table 7.6 Barrel thickness and coller specifications of RCC pres sure pipes

Internal dia mm	Min. barrel thickness mm			Min. Collar thickness mm			Min. Collar length mm		
	P1	P2	P3	P1	P2	P3	P1	P2	P3
80	25	25	25	25	25	25	150	150	150
100	25	25	25	25	25	25	150	150	150
150	25	25	25	25	25	25	150	150	150
200	25	30	35	25	30	35	150	150	150
225	25	30	35	25	30	35	150	150	250
250	25	30	35	25	30	35	150	150	150
300	30	40	45	30	40	45	150	150	150
350	32	45	55	32	45	55	150	150	150
400	32	50	60	32	50	60	150	150	150
450	35	50		35	50		200	200	
500	35	55		35	55		200	200	
600	40	65		40	65		200	200	
700	40			40			200		
800	45			45			200		
900	50			50			200		
1000	55			55			200		
1100	60			60			200		
1200	65			65			200		

P1 = 2 kg/sq.cm

P2 = 4 kg/sq.cm

P3 = 6 kg/sq.cm

7.14.3.1 Applicability

- When made to specification P1, P2, and P3 pipes are safe for use upto working pressure 2,4 and 6 kg/sq.cm respectively.

- Rusting or incrustation do not take place. Hence flow condition remains unchanged over a long period.

- Cost is on the lower side. Moreover small workshops on order can manufacture the requisite pipes at a short notice.

- When properly laid, can last for a long period.

- They are not very suitable for highly acidic or alkaline water. Neither should such pipes be used for alkaline soil.

- For gravity main, working pressure should not exceed 2/3rd and for pumping main it should not exceed 1/2 of the burst ing pressure.

- Adequate margin of safety in working pressure should be provided when bursting pressure is not known and pipes used are not from a very reliable source.

- Normal pressure pipes are suitable as low pressure distribu tion lines. Proper selection and adequate care should be taken while using as main line. Hume steel pipes which are manufactured by lining thin steel cylindrical shell with cement morter from both inside and out side are suitable for most pressure applications.

- The RCC pipes are available in almost any required range of diameter.

7.14.4 Poly Vynile Chloride (PVC) Pipe

These are relatively new introduction reffered commonly as PVC or plastic pipes. These are extruded from a chemical commound containing polyethelene with carbon back, special resin and non-toxic anti-oxidant mixed homogeneously.

These pipes are normally differentiated into following groups.

7.14.4.1 Light density polyethelene (LDPE) pipe

These are highly flexible and ductile pipes. They can be made into coils and normally used in temprorary installations or extension of delivery lines. They are available in coils of

25,50, 100, 150, 200m lengths. Diameter ranges from 1-14 cm and pressure rating from 2.5 to 10 kg/sq.cm. Jointing are telescopic joint with clamps which are rarely leack proof. These pipes are also subject to easy damage developing cracks and holes. They have a relatively low cost and less life.

7.14.5 High density polyethelene (HDPE) pipe

These are less flexible than LDPE pipes but can still made to bend. They come in long lengths with diameter ranging from 2 cm to 50 cm. These pipes are classified into five classes based upon their working pressure rating.

Table 7.7 Class and pressure rating of HDPE pipe

Class	1	2	3	4	5
Working pressure. kg/sq.cm	2	2.5	4	6	10

Bursting pressure of these pipes are normally 8 times the recommended working pressure. The nominal thickness and averge weight of HDPE pipe for different diameters at different pressure ratings are presented in table below.

Table 7.8 Thickness and weight variation of different classes of HDPE pipes with its diameter

Dia	Nominal thickness mm					Average weight kg				
cm.	Class					Class				
	1	2	3	4	5	1	2	3	4	5
9.0	2.2	2.8	3.5	5.1	8.2	0.64	0.79	0.98	1.39	2.12
11.0	2.7	3.5	4.3	6.3	10.0	0.94	1.20	1.46	2.08	3.14
12.5	3.1	3.9	4.9	7.1	11.4	1.23	1.51	1.88	2.66	4.08
14.0	3.5	4.4	5.4	8.0	12.8	1.54	1.92	2.32	3.34	5.11
16.0	3.9	5.0	6.2	9.1	14.6	1.95	2.47	3.04	4.35	6.67
18.0	4.4	5.6	7.0	10.2	16.4	2.48	3.12	3.84	5.48	8.92
20.0	4.9	6.2	7.7	11.4	18.2	3.05	3.84	4.69	6.79	10.40
22.5	5.5	7.0	8.7	12.8	20.5	3.86	4.84	5.96	8.55	13.10
25.0	6.1	7.8	9.7	14.2	22.8	4.76	5.99	7.37	10.60	16.20
28.0	6.9	8.7	10.8	15.9	25.5	5.98	7.47	9.18	13.20	20.30
31.5	7.7	9.8	12.2	17.9	28.7	7.51	9.45	11.70	16.70	25.70
35.5	8.7	11.1	13.7	20.1	32.3	9.54	12.10	14.70	21.20	32.60
40.0	9.8	12.4	15.4	22.7	36.4	12.10	15.20	18.70	26.90	41.40
45.0	11.0	14.0	17.4	25.5	41.0	15.20	19.20	23.70	43.00	52.40
50.0	12.2	15.5	19.3	28.3	45.5	18.80	23.7	29.20	41.90	64.60

Jointing of HDPE pipes are made by

- Push fit
- Flange fit

- Hot plate or fushion welding

Collars are made by injection moulding. Flanges are made by injection moulding with ms sheet insert for imparting strength. Compression moulded flanges use ms backing plates for strength.

7.14.6 Unplasticised or Rigid PVC pipe

These pipes are not flexible and are quite rigid. They are available in 2-50 cm diameters with lengths of 3.5 m and 6 m as per choice. They are normally made in 4 pressure classes as shown in then table below.

Table 7.9 Pressure rating for various classes of RPVC pipe

Class	Pressure kg/sq.cm
Class 1	2.5
Class 2	4
Class 3	6
Class 4	10

Wall thickness of different classes against their diameters and pressure ratings are presented in the table below.

Table 7.10 Average wall thickness of RPVC pipe

Out side diameter cm	Wall thickness			
	Class1	Class2	Class3	Class4
9.0	1.3	2.1	3.1	05.0
11.0	1.6	2.5	3.7	6.1
12.5	1.8	2.9	4.3	7.7
14.0	2.0	3.2	4.8	7.7
16.0	2.3	3.7	5.4	8.8
18.0	2.6	4.2	6.1	9.9
20.0	2.9	4.6	6.8	11.0
22.5	3.3	5.2	7.6	12.4
28.0	4.1	6.4	9.5	15.4
31.5	4.6	7.2	10.7	17.3
35.5	5.1	8.1	12.0	19.6
40.0	5.8	9.1	13.5	22.0
45.0	6.5	10.3	15.2	24.8
50.0	7.2	11.4	16.9	27.5

Jointings of RPVC pipes are done by

- Solvent welded joint
- Flanged joint
- Screw or threaded joint
- Rubber ring joint

Other fitting are normally moulded or fabricated. RPVC pipes can therefore be easily connected with pipes of other material through a wide range of connector devices.

7.14.6.1 Applicability of PVC pipes

- Resistant to corrosion and most of the chemicals.
- They do not rot rust or incrustate over a long period. The smooth bore offers relatively less friction.
- They are flexible and very light weight.
- Handling, transporting, storing and laying are relatively easier.
- They can withstand heavy soil movementnts when laid under ground.
- They do not normally break when pipe water is frozen and self extinguishing when exposed to fire.
- They are subject to physical damage and deteroriate due temperature increase and exposure to sunlight when laid overground.
- Can take high impact load and floats in water due to low density.
- They should not be used for water with very high temperature.
- Specific gravity ranges from 0.9 to 1.4.

7.14.7 Cast Iron pipe

These are most extensively used pipes for irrigation and other water works. These pipes are so called as they are casted in vertical moulds. Further development has been made in spun pipes where casting is done by fast rotating horizontal mould using molten iron material normally mixed with magnesium.

The centrifugal force due to the rotation ensures higher density, lesser flaws, uniform texture, close grain size, smoother inner surface and greater tensile strength. Horizontally centrifugual cast iron pipes have therefore thinner wall diameter and lesser weight than the vertically cast iron pipe. Cast iron(CI) pipes normally are available in 3.7-5.5 m in length ranging in diameter from 5-75 cm. The diameter of CI pipe however can be made as high 150 cm as per order. However pipes above 75 cm diameter requires special handling arrangements due to their weight.

Vertically cast iron pipes are manufactured as class A and B while class 1A, A and B are made in centrifugally cast (spun) pipes. Class B pipes have 10% greater thickness than class A. The hydrostatic test pressure for various types of standard CI pipes are given in tables below.

Table 7.11 Test pressure for standard Vertically cast Iron pipe

Dia	Test pressure Kg/cm^2			
cm	Socket & spigot joints		Flanged pipe joints	
	Class A	Class B	Class A	Class B
Upto 30	20	25	20	25
30-60	20	25	15	20
60-100	15	20	10	15
100-1500	10	15	10	10

Table 7.12 Hydrostatic test pressure for centrifugally spun pipe

Class	Hydrostatic test pressure at works Kg/ square cm	Max. hydrostatic test pressure after installation Kg/ square cm	Type of joints
1A	35	12	Sockets spigot
A	35	18	
B	35	24	
A	35	18	Flanged
B	35	24	

Jointing of CI pipes are done by

- Spigot-socket ends with lead coalking. Quantity of lead varies from 3.5-4 kg for 15 cm diameter to about 40 - 50 kg per joint for 1.2 m dia pipes.
- Flanged joints using nut-bolt with intregally casted flanges.

7.14.7.1 Applicability

- CI pipes are strong and durable.
- Resistant to corrosion when laid properly with bituminous protective coating. They can last as long as 50-100 years.
- Suitable for most high working pressure applications.
- High initial and laying cost and low maintenance cost.
- Relatively heavy in weight and are prone to cheaping of the socket end or cracking in transportation, if not handled properly.
- Low tensile strength and hence prone to breaking due to impact.

- They are also prone to defect like not having a very smooth interior surface.

7.14.8 Steel pipe

These are made from mild steel. Small diameter pipes can be made from solid bar sections by hot or cold drawing process in which case the pipes are known as seamless pipes. Large diameter pipes are made by welding plates curved into the shape to form a pipe. Standard length is 5.5 m with diameter ranging from 10 - 180 cm. Larger diameter pipes can be made to any size as per the need.

Thickness of steel pipe is lesser than equivalent size of CI pipe. Steel pipes also can be made twice as long. Thus considerable saving is possible in transportation and jointing. The thickness of steel pipe should be adequate to take working pressure, external loads and keep the pipe in circular shape in transportation, storing, laying etc. The thickness is given by

$$t = \frac{P\ D}{2\ S\ F} \qquad \ldots\ 7.9$$

Where,

P = Internal water pressure (Kg/sq.cm) which causes the hoop stress
S = Safe working pressure,sq.cm =(1100 for steel pipe)
F = Factor for efficiency of the longitudinal joint
= 0.6 - 0.8 for welded pipes
= 0.9 for seamless pipe

An excess of 3 mm is provided as margin of safty to the theoretical thickness value to provide for corrosion and water hammer effects.

Jointing can be thread joint with spun yarn and white lead. Jointing can also be done by face to face welding for straight pipes. Welded flanges are convenient for small diameter pipes. Fillet welds (segmented joints) are made by using short sleeves to give a deflection in the pipeline wherever necessary.

7.14.8.1 Adoptability

- Steel pipes are preferred for high pressure requirements.

- The pipes have good strength, less weight, more tensile stress and are moderately ductile.

- They can be laid both under and overground with as much advantages.The pipes can adopt to some amount of ground movements and subsidence

- They have high initial cost but also have a long life. Repairing and maintenance are also relatively easy.

- They are prone to corrosion particularly with soft acidic water. Longer life can however be easily assured by applying suitable corrosion resistant coating. Advanced technique of cathotic protection is applied for long and important pipe lines.

- They are not adopted to withstand a very high external pressure.

- They have high adoptability in handling and laying. They can be cut, shaped and welded at the site itself.

- For long and large diameter pipelines, it is necessary to provide expansion joints to take care of linear expansion and contraction of the pipe line. Such provision may not normally be required for burried pipes.

7.14.9 Galvanised Iron pipe

These are normally wrought iron pipes which are galvanized for longer life. Wrought iron pipes are slightly lighter than C.I. pipe . As wrought iron pipes are highly prone to quick corrosion, a lead zinc galvanised coating is applied in both inner and outer surface to render longer life. GI pipes are suitable for most uses.

They are available in 5.5 m length with diameter ranging from 0.8 to 10cm. pipes with 15cm and 20cm diameters are also available at times while pipes with higher diameter are rarely available.

Jointing is either thread joint with coupler or flanged joints. Face to face welding is also possible. The adoptability of GI pipe is about the same as that of steel pipe. The only restraining factor for general use of GI pipe is its high cost. For larger diameter application steel pipe are preferred than GI pipes.

7.14.10 Alluminium pipe

These are made from ductile alluminium. In welded tube,alluminium sheet is rolled and welded to make the pipes. Solid bar section drawn into pipe is known as extruded tube or seamless pipe. Extruded tube naturally causes lesser frictional loss than welded tube.Normal pipe lengths are 6 m with diameter ranging from 5-15 cm.Working pressure is rated maximum upto 30Kg/ sq. cm.

Jointings are done by inserting one plain end (socket) to the other flared end and providing support by bolts or specially designed clamps attached with the pipe. This type of joints are known as bolted harmetic joints. Jointings are also made using reinforced synthetic rubber hose. In ball and bell coupler, ball end is inserted in the bell end with gaskets and springs in between.

7.14.10.1 Applicability

- They have smooth bore and hence offer lesser frictional loss.
- The pipes are light weight having high tensile strength and ductility.
- They get rapidly corroded when laid underground.
- The pipes are suitable for water of different qualities.
- They are suitable for high pressure operation and in tempo rary surface application like sprinkler and flow irrigation.

7.15 GATE VALVE

Valve which are used for controlling the flow of water is known as gate Valve or glove valve or sluice valve or stop valve or shut off valve etc. Function wise they are the same. They are installed for stopping the flow partly or completely when closed partly or fully.They are necessarily used to cut-off or allow flow into the mainline or selected secondary lines from the main-line. In long pipe line they are installed for isolaling pipe sections to facilitate repair etc.

A single valve may be used upto pipe dia. of 35 cm. For larger dia installations, a small diameter bye-pass line is attached to the valve to equalize pressure on both sides of the valve. Turning of the valve lever otherwise will be difficult due to water pressure operating in one side of the valve,if such bye-pass line is not provided.

7.16 AIR RELEASE VALVE

A pipeline should be so laid that no air enters the line. But despite of this, some air mixed with the water always enters the line. Air release valve permits escape of trapped air from the line.It consists of a floating ball in a vertical chamber which keeps the exit point sealed due to water pressure. During fall in water pressure, the moving ball drops down permiting air to excape through air vent. The ball will move up again to seal the vent when water pressure is built up suficiently. Some water may also escape with the air during this process.In a long pipeline, it is necessary to place air release valve at given intervals and at change of grades. Air valves are usually bolted by using standard flanges. Tee joints are preferred for easy removal and maintenance.

7.17 SCOUR VALVE

For flushing out the deposits settled within a pipeline, scour valve or drain valve may be installed at the lower end of the pipeline. Lowest points in a pipeline are the area where the water borne material settle down. Scour valves are therefore placed at the bottom of all depressions and dead ends. Deposited sediments can be removed from time to time by removing valve window. They are also sometimes called blow-off valves. From design point of view, the scour valves are similar to gate valves.

7.18 REFLUX VALVE

They allow flow of water to move in only one direction. The valve flap closes when the flow direction is reversed thus preventing any return flow. These are also known as non-return valve or check valve or flap valve. They are generally installed immediately after the pump in its delivery line. When flow of water stops due to sudden stopage of pump or closure of a gate valve in the delivery line, reflux valve prevents water from rushing back to the pump which could otherwise cause damage to the pump.

7.19 FOOT VALVE

Foot valves are similar to reflux valves installed necessarily at the water entry point in the suction pipe. These are used primarily to maintain priming of a centrifugal pump by holding the water column in the suction pipe.Good quality foot valves are necessary to reduce entrance loss. The free open cross-section should be at least 1.5 times that of the suction pipe and the open cross sectional area of strainer should be atleast 4 times that of the suction pipe.

7.20 RELIEF VALVE

In long pipelines where the effect of water hammer may be high, relief valves or safety valves are placed in the lower end. These are spring loaded valves which are ment for reducing the serge pressure of water as and when it takes place.

Less expensive devices like surge tanks or break pressure tanks can be used in gravity mains. Surge tank is a air tight steel tank having one inlet and one or more outlets located at various heights. The trapped air acts as spring while the flow continues through the outlets. In order to limit the maximum pressure in a pipeline it should be laid into seggments separted by break pressure tanks. The function of such tank is to limit the static pressure by providing open water surface at certain places along the pipeline for free flow. Use of serge tank allows using of pipes with lower pressure ratings.

Riser pipes also can be provided at places mainly from concreat, asbestos and plastic pipes laid underground from where water may be taken out in the future for distribution. A riser valve is placed on top of the riser pipe. Water from riser pipes are not used simultaneously. Riser valve is a circular plate raised or lowered by a vertical hand operated spindle screw. When open, water gushes out through the opening and when closed,the cap fits tightly over the circular edge of the riser pipe through rubber seal to make it water tight. Hydrants may be used for taking out water from a raising main by placing the hydrant over the raising valve.

CHAPTER 8

INSTALLATION, MAINTENANCE AND OPERATION

8.1 SIZE OF SCHEME

As such there is no clear cut criteria based on which a scheme may be called large or small. Normally, the total area brought under irrigation or the total Horse power of the installed pump set are considered while qualifying the size. Again, since the same quantity of water can cover different size of areas depending upon a number of factors, the classification from area covered point of view becomes less acceptable. A lift scheme therefore can be termed large, medium or small depending upon the Horse power of the pump installed. Again till today, in our country there is no accepted norm for classifying the size of a LIS. Considering the present day trend, it can be mentioned that for LIS using electric pumps upto 10 HP may be called small, 10-50 HP medium and over 50 HP large. Similarly for installations using diesel engine upto 7.5 HP may be called small, 7.5-20 HP medium and above 20 HP large.

8.2 INSTALLATION OF PUMPSET

Whether a scheme is large or small, proper installation of pump and pipes are absolutely necessary. A correct installation aids in longer life and lower maintenance cost. Installations may again be temporary or permanent. For medium to large schemes the installation should necessarily be of permanent type. Temporary installations are applicable commonly to small horizontal centrifugal pumps directly coupled with internal combustion engines. The whole unit is mounted on wheels so that it can be moved from site to site.

Tractor power take-off is also used to run a pump in a temporary installation. When properly planned it is possible to use the same electric pump at different sites by providing sockets in electric poles for power connection. For large to medium size schemes, the pumpsets installed are however of permanent nature. The following guidelines should be followed, to the extent possible, in the placement of pumpsets in permament installations.

8.2.1. Location

- Pumping sets shold be so located that they are easily accessible during all seasons for inspection, operation and maintenance.

- It should be protected from weather and malicious damage.A permanent pump house is preferred.

- It has to be protected from flooding. Such protection will however not be necessary for wet pit installations.

- It should be located as close to the water body as possible so as to keep the suction lift and the length of suction pipe to the minimum. A horizontal centrifugal pump operates to its highest efficency when suction lift is minimum.

- There should be enough head space and working space around the pump for its repair, maintenance and removal whenever necessary.

- It should be so located that noise level or smoke exhaust do not disturb the nearby inhabitants, if any.

8.2.2 Foundation

A pump set should be rigidly mounted over a horizontal structure. This is to prevent straming of the casing and excess shock (vibration) to the pump. Pump foundation could be a simple masonary or R.C.C. structures.The height of the platform should conform to the suction opening in the pump house so that the suction pipe can run straight out of the pumphouse. Vertical pumps are normally clamped with strong iron girders placed across the pump house to serve as a foundation. Normally, pump set complete with its base plate is placed over the platform. The base plate is secured at place with Nut-bolts or cementing. For large units, the pump and its mover may be mounted seperately and alligned afterwards. Grouting of the foundation is necessary for heavy installations.

8.2.3 Alignment

Proper alignment of the prime mover and the pump is absolutely essential. Alignment between the two is necessary for smooth, effecient and long trouble free service. For direct coupled horizontal pump sets such problems do not occur. However even if the pump and its mover are alighned at the factory and are mounted over a base plate there is always a chance of the alignment being disturbed in transportation. Besides, when pipes are fitted with the pump chances of further misalignment may occur.

However for small units, it may not be necessary to remove factory adjusted alignment. The baseplate is simply mounted over the platform. Some metal wedges may be placed between the baseplate and foundation till the proper levelling is achieved. Baseplate is then either securely bolted or grouted.

For large units, the coupling bolts are first removed. The pump and the mover are then levelled seperately. Provision for little adjustment for final levelling is kept on either side. For alignment, the couplings are brought face to face after levelling has been completed.

Both the pump and mover shaft are then checked for bending. The faces of the coupling halves are checked with tapered thickness guage or feeler guage to check that they move freely indicating that the shafts are not disturbed. Since the coupling halves are normally true circles of equal diameter and their faces are flat, exact alignment is achieved when the two faces match completely. In a misaligned system, the pump will work at a lesser efficiency, the bearings will wear out early, the drive will tend to get overheated and the shafts may finally be damaged. Use of flexible coupling somehow compensates partly for misalignment, if any.

8.3 TROUBLE SHOOTING OF CENTRIFUGAL PUMP

A centrifugal pump installed properly gives long,trouble free survice. But at times due to minor installion faults, or pump defects a centrifugal pump may either operate unsatisfactorily or may stop functioning all together. Minor adjustments or corrections applied properly can revive the operation. A set of su(symptoms and their likely causes are listed in below:-

Table 8.1 Trouble Shooting Of Centrifugal Pump

Symptom/Possible Causes	Key
Pump does not deliver water 1,2,3,4,6,11,14,16,17,22,23	1.Pump not primed 2.Pump or suction pipe not filled completely with water 3.Suction lift too high 4.Insufficient margin between suction pressure and vapor pressure
Insufficient capacity delivered 2,3,4,5,6,7,8,9,10,11,14,17,20,22,23,29,30,31	5.Excessive amount of air or gas in liquid 6.Air pocket in suction line 7.Air leakage into suction line 8.Air leakage into pump through stuffing boxes 9.Foot valve too small 10.Foot valve partially clogged 11.Inlet of suction pipe insufficiently submerged
Insufficient pressure developed 5,14,16,17,20,22,29,30,31	12.Water seal pipe plugged 13.Seal cage improperly located in stuffing box, preventing sealing fluid entering space to form the seal 14.Speed too low 15.Speed too high 16.Wrong direction of rotation
Pump loses prime after starting 2,3,5,6,7,8,11,12,13	17.Total head of system higher than pump design head 18.Total head of system lower than pump design head 19.Specific gravity of liquid different than design

Pump requires excessive power

15,16,17,18,19,20,23,24, 26,27,29,33,34,37

Stuffing box leaks excessively

13,24,26,32,33,34,35,36, 38,39,40

Packing has short life

12,13,24,26,28,32,33,34, 35,36,37,38,39,40

Pump vibrates or is noisy

2,3,4,9,10,11,21,23,24, 26,27,28,30,35,36,41,42, 43,44,45,46,47

Bearing have short life

24,26,27,28,35,36,41,42, 43,44,45,46,47

20. Viscosity of liquid differs from that for which designed
21. Operation at a very low capacity
22. Parallel operation of pumps unsuitable for such opera tion
23. Foreign matter in impeller
24. Misallignment
25. Foudations not rigid
26. Shaft bent
27. Rotating part rubbing on stationary part
28. Bearigs worn
29. Wearing rings worn
30. Impeller damaged
31. Casing gasket defective, permitting internal leakage
32. Shaft or shaft sleeves worn or scored off the packing
33. Packing improperly installed
34. Incorrect type of packing for operating conditions
35. Shaft running off-center due to worn bearings or misa lignment
36. Rotor out of balance result ing in vibration
37. Gland too tight, resulting in no flow of liquid to lubricate packing
38. Failure to provide cooling liquid to water-cooled stuffing boxes
39. Excessive clearance at bottom of stuffing box between shaft and casing, causing packing to be forced into pump interior
40. Dirt or grit in in sealing liquid, leading to scoring of shaft or shaft sleeve
41. Excessive thrust caused by a mechanical failure inside the pump or by the failure of the hydraulic balancing devise, if any
42. Excessive amount of grease or oil in the housing of an antifriction bearing or lack of cooling, causing exces- sive bearing temperature

Pump overheats and seizes 1,4,21,22,24,27,28,35,36,41	43.Lack of lubrication 44.Improper installation of antifriction bearings (damage during assembly of stacked bearings, use of unmatched bearings as a pair) 45.Dirt getting into bearings 46.Rusting of bearings due to water getting into housing 47.Excessive of cooling of water-cooled bearing resulting in condensation in the bearing housing of moisture from the atmosphere

8.4 INSTALLATION OF PIPES

Basic guidelines for instalation of pipes are listed below:

- Piping and pump casing should be so joined that no strain acts between them.

- Pipe flanges should squarely be brought together face to face and nut-bolts are tightened. Tightening of bolts should not be used to force match the faces.

- Both the suction and delivery pipes should be properly anchored by permanent support. They should not exert any load on the pump casing.

- Small pieces of pipes should be provided at both the suction and delivery ends. This permits easy removal of the pump for repairs etc. without disturbing the rest of the pipes. Only the small pipes are disjointed when the pump needs removal.

8.4.1 Suction Pipe

Other than the misalignment, major causes of troubles in a centrifugal pump arise from its suction line. Suction pipes must therefore be laid with proper care keeping the following in mind.

- Suction pipes should preferably be as short and as direct as possible.

- When use of long suction line becomes unavoidable it is recommended that the diameter of the pipe is increased by one or two commercial size larger than the diameter of the pump inlet. This will help in containing the frictional loss within the permissible limit.

- When larger diameter of pipe is used, it is essential to in stall an eccentric reducer between the suction inlet of the pump and the suction pipe. The eccentric side of the reducer is kept facing down thus eliminating any chance of air being trapped in the upper half of the reducer.

- Gate valve should not be placed in the suction line. Should it be placd due to some special reason, the valve handle should point horizontal or doownward to prevent formation of air pocket at the upper half of the valve. Suction pipe should never be throttled when the pump is in operation.

- All joints in the suction line have necessarily to be air-tight and leak proof.

- Foot valve should be of standard design. It should have two times entry area and four times screen area Compared to the diameter of the suction pipe.

- The end of the Suction pipe should be submerged into water minimum by 4 times the pipe diameter.

- Total suction lift without friction should normally be kept within 4.5-6.0 meters.

- The suction pipe should be laid with an uniform rise towards the pump. There should not be any high spot where air pockets are likely to form.

- When use of bends in the suction line becomes essential, only wide angle (long radius) bends should be used. Should any ob struction needs to be crossed, bends in the suction line should cross from under such obstruction so as to prevent formation of air pocket in the line.

8.4.2 Delivery Pipe

- Delivery pipes should be laid as straight as possible.

- A check valve and a gate valve should necessarily be used in the delivery line. Check valve which is placed between the pump and gate valve prevents back flow of water into the pump in case pumping is stopped suddenly. Gate valve which is placed after the check valve, is used to controll the flow in the delivery line.

- In a long delivery line, air release valves are placed at high points. Similarly scour valves are placed at appropriate locations. Air pipes or Air vents are provided in gravity mains.

- A gate valve each should be placed in the lateral pipe which takes off from the main delivery line. Flow into the lateral lines are controlled by these gate valves.

- In long and overground delivery line where expansion or contraction of pipe may occur due to the variation in ambient temperature, expansion joints, loops, bends, flexible steel hose etc. should be used to compensate such expansion or contraction.

- Necessary supports at suitable intervals should be provided to firmly anchor the pipe in place.

- Provision of flow measuring devices viz.pressure gauge,menometer, V-notch etc at the delivery end, enable estimation of discharge readily. Callibrated pressure gauge may also be fitted in the delivery line to obtain working pressure.

8.4.3 Diameter of Suction and Delivery Pipes

In any large scheme it is most crucial to correctly arrive at the optimum diameter of pipes. Even in small pumps the energy loss due to faulty pipe size could be enormous in the long run. It has been observed that use of a slightly higher diameter pipes than the pump inlet and outlet diameters are more advantageous. Though the initial cost of larger diameter pipe is higher but it more than comprensates in the long run in terms of savings in energy and thus the operating cost.

The recommended suction and delivery pipe diameters for optimum operation are presented in table below

Table 8.2 Optimal size of suction and delivery pipes

Flow lps.	Nominal pipe diameter	
	Suction pipe mm	Delivery pipe mm
0.5	20	20
1.0	30	25
1.25	40	32
1.6	40	40
2.0	40	40
2.5	50	40
3.2	65	50
4.0	65	65
5.0	65	65
6.0	80	80
10.0	100	80
12.5	100	100
16	125	100
20	125	125
25	150	125
32	150	150
40	200	150
50	200	150
60	250	200

80	250	250
100	300	250
125	350	300

The above values have been arrived at keeping in view that velocity of flow remains 1.5 m/sec in the suction pipe and 2.0 m/sec in the delivery pipe.

As a general guideline the following fromula can be used to obtain the required diameter of pipes.

Suction pipe diameter $D = 29 \sqrt{Q}$

Delivery pipe diameter $D = 25 \sqrt{Q}$

where,

D = Recommended diameter in mm
Q = Design discharge in lps

8.5 LAYING OF PIPES

Pipes are laid either underground or overground. Some basic considerations should be observed while laying the pipes.

8.5.1 Underground Laying

- They should be laid sufficiently deep so as to protect them from damage against moving traffic.

- The trench dug should have uniform grade all along its length. Grade of laying is to be changed only when absolutely necessary.Trench bed should be smooth and compact.

- Care must be taken against uneven settlement of pipe and sur rounding soil. Placement of gravel, Khoya or compaction of the loose area will be helpful.

- RCC pipes require extra care in jointing while being laid. Since the pipes get heated during the day time, jointing might crack when the pipes contract at night. Pipes should be kept filled with water till the joints harden.

- Metal pipes which are subject to easy corrosion will require adequate protection. Coating of tar, bitumen, resins, paints etc. both inside and outside the pipe ensures longer life. Care should also be taken to apply at the last moment, the protective coating near the joints and other places where the paint might have come off. These spots, if left unattended become the week spot at which rusting sets in and sprads quickly.

- Care should be taken to remeove sharp stones and cheaps from the trench while laying plastic pipes. Such edge in contact with the pipe when subject to heavy loads, may cause damage to the pipe.

- All underground pipes should be hydrostatically tested for leakage prior to back filling of the soil.

8.5.1.1.1 Advantages

- They are protected from ambient temperature variation,physical and malicious damage,unauthorised tapping etc.

- Pipes being located deep underground,no surface area is wasted for normal use.

- Similarly,they do not cause any interuption to ploughing activities,movements of personnel and other farm machinaries.

8.5.1.2 Disadvantages

- Repairing will require exposing the damage portion, involving extra labour and cost.

- Detection of faulty portion may not be easy. However in case of large leakage the wet portion in the ground will give the indication.

- Corrosion is more intensive.

- In claye soil, due to heavy expansion and contraction, pipe joints may get easily damaged if not laid properly.

- Riser pipe should be provided for surface discharge at suitable locations.

8.5.2 Overground Laying

- Pipes should have suitable supports at suitable spacings. Support spacing is decided in accordance with the thickness and diameter of the pipe and the topography.

- Provision of expansion joints should be made where necessary.

- Anchor blocks should be constructed at every change of grade and direction.

- It is advisable to lay pipes by shortest route even if the grade of the route is high.

8.5.2.1 Advantages

- Less suceptable to corrosion
- Easy to repair and maintain
- Branch lines can be taken with less problems
- Damaged part can be identified and subsequently repaired easily

8.5.2.2 Disadvantages

- They occupy useful surface area which hinder movements
- Subject to damages from various physical sources

8.6 PRIME MOVER

It is essential that BHP supplied by the mover should match the WHP of the pump. If the mover supplies power either higher or lower than the actual requirment of the pump, the overall efficiency suffers. Choice of mover is practically restricted to diesel engines and electric motors. Other non conventinal prime movers are seldom used for various operational reasons. Biogas, wind and solar power are yet to be properly harvested for running medium and large scale schemes on long term basis.

8.6.1 Diesel Engine

Diesel engines for pumps were introduced into the country around 1930.The common type of diesel engines are:

- Horizontal engines: These are less in use in the present days.
- Lister type: These are slow speed vertical engines.
- High speed vertical engine: These are most commonly used these days.
- Diesel engines for Agriculture purpose are available in 5,6,7,8,9,10,12,14 HP range. Diesel engines may be either air cooled or water cooled. Air cooled engines are available normally upto 25 HP. They are improvisation over water cooled engines. They are less messy than the water cooled engines where the coolant (water) is to be properly disposed off to prevent the site turning mucky. Water cooled engines however are made in all sizes.

8.6.1.1 Performance of Diesel Engine

Performance of diesel engine depends upon a number of factors involving its design and quality of various components used.
The characteristic of an engine is normally studied by comparing its output with respect to the rpm. Figure 8.1 shows general characteristics of a diesel engine i.e. the variation of power with respect to rpm and specific fuel consumption with respect to load. Though, Horspower is directly proportion to the rpm, but due to design contraints, rpm can not be increased without reservation. The design rpm for diesel engines are normally kept at 1500 rpm.

Figure 8.2 shows the test curves for specific fuel consumption (SFC) and torque variation with respect to rpm. It is obvious that when the engine runs at or near its 100 percent loading, the specific fuel consumption becomes low. The same may be very high when the engine runs at too less a load.While assessing the performance of a diesel engine following points should be borne in mind.

- Attention should be paid while noting the HP rating whether it is with or without the accessry loads. A diesel engine when operated with loads Viz. fan, dynamo, water pump etc. may loose as much as 10 percent of Horse power output of the engine operating without these loads.

- An internal combustion engine looses power at the rate of 4 percent for every 30 meter of altitude above mean sea level.

- The continuous load rating of HP should be reduced by 1% for every 3 degree centridrade of temperature rise after 30 degree centrigrde.

8.6.1.2 Fuel consumption

Fuel consumption of Internal Combustion(IC) engine is given by specific fuel consumption (SFC). The approved SFC for the standard diesel engines are 175 - 260 grm/BHP-hr. Engine consuming more than 200 grm/BHP-hr should be avoided. Consumption of lubrication oil should normally be 1 percent of diesel consumption. It may however vary for different engines and may go as high as 3-4 percent.

Exhaust smoke is an indication of high or normal consumption . In an engine of good working condition, the exhaust smoke should barely be visible.

Table 8.3 Recommend SFC for diesel engines as per Indian Standards Institution(ISI)

Rated engine	S F C (maximum)	
speed (RPM)	grm/K W -hr	grm/BHP-hr
Upto 500	332.5	245
500 - 1000	275.5	203

It is clear from the above table that optimum SFC value lies somewhere between 1000-2000 RPM. Ideally, a diesel engine should consume about 250 milli-litre per unit (100 KG-m). Consumption of a 5 HP diesel engine should be about 1.2 ltr/hr.

8.6.1.3 Common causes of high diesel consumption

- Wrong selection, installation and operation of pumps, diesel engines, pipes, foot valves etc.
- Installation of a diesel engine irrespective of the HP requirement of the pump. The diesel engine will then operate in a situation of either over or under load.
- Temperature of the coolant (water) is very low. Normally the temperature of coolant should be around 6 degree centrigrade.
- Efficiency of the pump for the given duty is sufficiently low
- Under size suction and delivery pipes are used.
- Use of unnecessarily long or high suction and delivery pipes

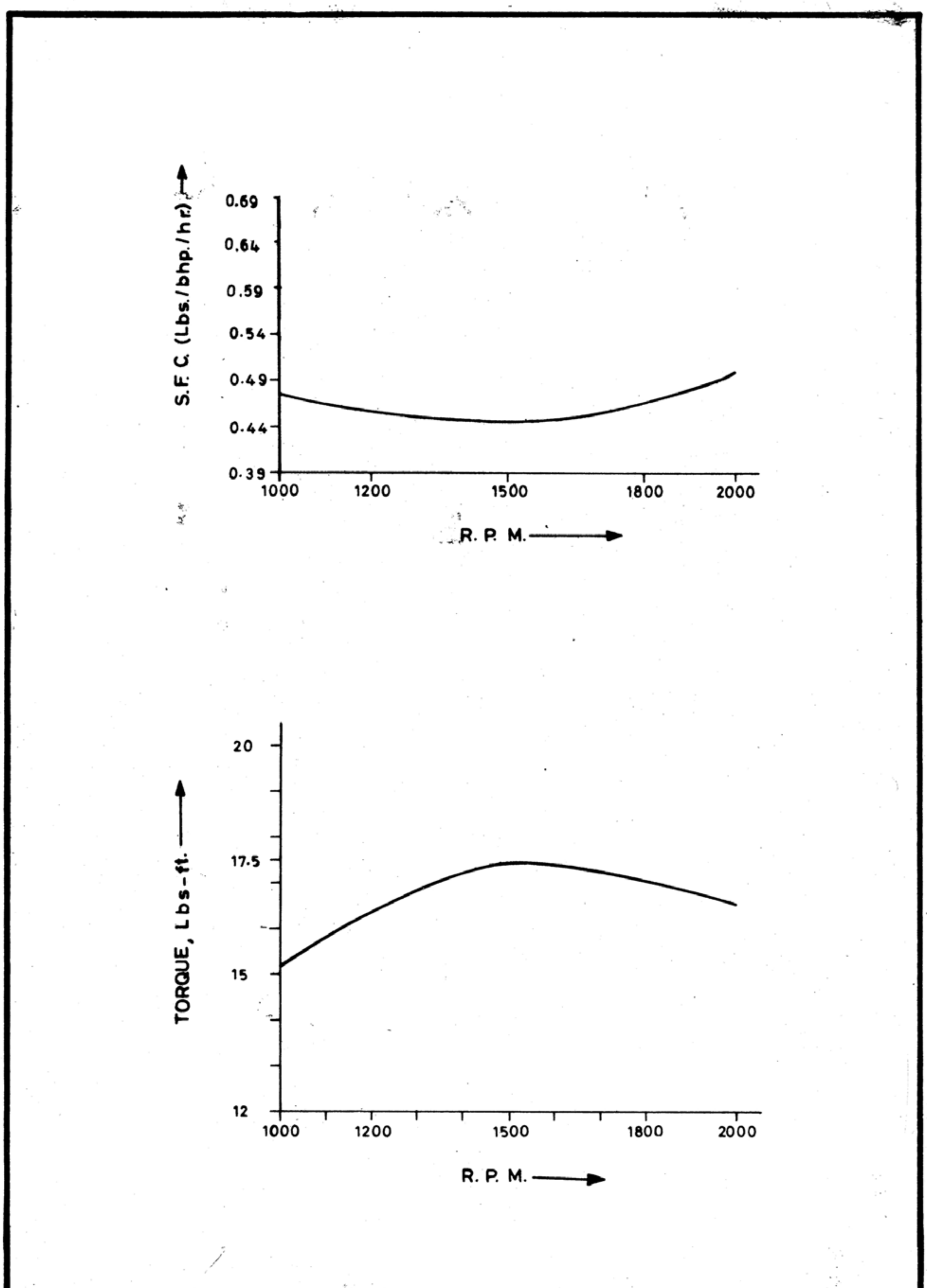

Fig. 8.1 Operating characteristics of a diesel engine

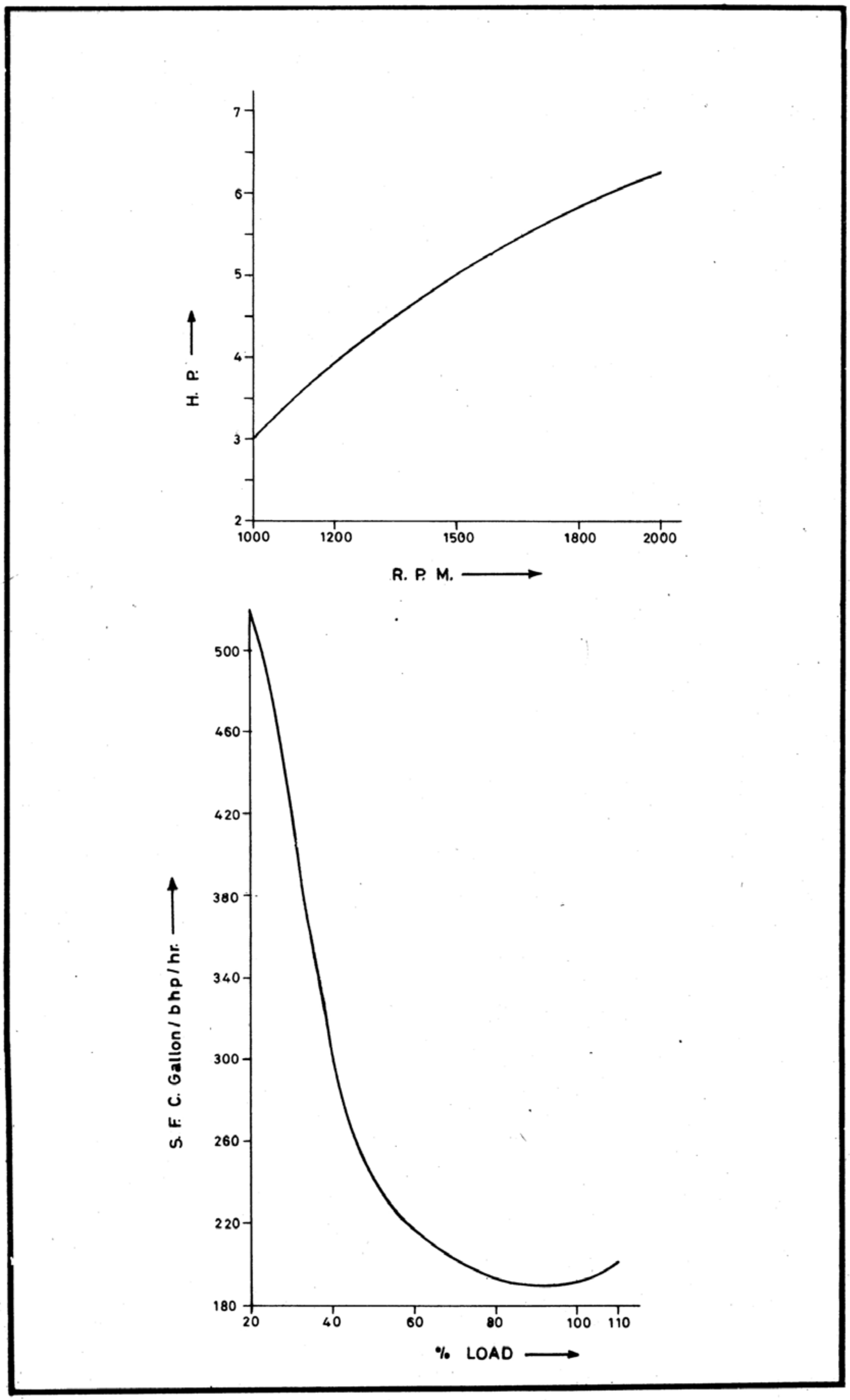

Fig. 8.2 Design characteristics of a diesel engine

increasing the total Head.
- Use of poor quality foot valve.

8.6.1.4 Measures to reduce diesel consumption

- Use good quality foot valve which causes least frictional resistance.
- Replace undersized suction and delivery pipes with those recommended in table 8.2.
- Select pump which will operate at a high efficiency level or minimum 50-60% at the given duty.
- The pump should be operated at the correct (designed) speed.
- Select diesel engine which matches properly with the pump.
- Avoid over cooling of the diesel engine.
- Ensure an efficient and dependable transmission system.
- Ensure proper maintenance and care of the engine.
- Purchase good quality pump and engine which conforms to the ISI standards.

8.6.1.5 Features of disel engine

- Operating cost of pumping by diesel engine is more than twice than that of electric motor.

- Diesel engines are dependable but require more attention than electric motors. Diesel engine is therefore considered as an alternative when electric power is either not available or not dependable.

- Horse power produced is proportional to the speed and hence weight of diesel engine do not increase propotionally with per unit increase in Horse power.

- Diesel engines are normally heavy and they do not operate satisfactorily at a very high speed.

- Engines less than 5 HP are normally not used. Less HP engines are neither smaller in size nor are proportionately fuel effective.

- Speed of diesel engines however can be varied considerably without seriously affecting its efficiency. Speed can there fore be adjusted within limit to suit the pump requirement.

- More versatile and flexible than electric motors.

- Wear and tear are more partcularly when operates under over loading.

- Diesel engines are harder to start, take more floor space and generate noise and smoke.

8.6.1.7 Common protections

Diesel engines should normally be fitted with oil pressure and temperature guages. For large, unattended engines, various protective switches are provided. Ignition cut-off switch automatically shuts-off an engine when fuel pressure drops beyond a permissible level or water temprerature is excessive. Similarly water pressure switch protects the engine against loss of priming or drop in discharge.

8.6.1.7 Operation of diesel engine

Initial checking

- Level of oil in the engine is checked. Oil of recommended grade is filled into the rocker box by opening its lid, if oil level is low.

- The air vent screw on the fuel pump body is slightly lossened so that air bubbles are removed from the pipeline through which oil passes. The engine is cranked a few times for complete bleeding.

- Position of diesel in the tank, oil in the air cleaner (wet type) etc. are checked and topped up if necessary.

Starting the engine

- Fly wheel is rotated at a high speed using the handle while the decompression lever is kept lifted by other hand.

- While removing the handle care should be taken not to oppose the movement of the fly wheel. The handle should rather be turned one or two turn extra with the moving engine and then slowly removed.

- It should be essured that the connecting pipe from the pump to the engine head provided for cooling is intact. This applies for water cooled engines only.

Stopping the engine

The stop lever is pressed in the direction of fuel pump till the engine stops completely. The engine should not be stopped by using the decompression lever. This may result in damage of major parts.

8.6.1.8 Maintenance

A diesel engine has to perform the following functions in correct sequences.

- It takes in measured quantity of air through the inlet valve.
- Air is compressed and air-fuel mixture is ignited at the

designed compression within the cylinder.
- The expanded gas push the piston which supplies power to the engine.
- Exhaust gas is allowed to ascape through outlet valve.

The above cycle is completed either in two strokes or in four strokes. Accordingly the engine is known as two or four stroke engine.

The entire maintenance of diesel engine therefore is centered around.

- Clean fuel
- Clean air
- Clean lubricating oil
- Clean engine

Daily checks

- Oil level is checked and topped up if necessary. Oil level should never be permitted to fall below the lower mark in the dip stick.

- Diesel tank is checked for enough fuel. Tank should be filled up with clean diesel at the end of day's operation.

- Engine is wiped clean after days operation is over. Observed leakage of oil from any area should be attended to by tightening nuts-bolts or changing gasket or parts etc. to the earliest.

Every 100 hours

- Air filter element is cleaned. In case of dry type air cleaner diesel or water are not to be used for cleaning.

- In case of oil bath type air cleaner, the cleaning element can be cleaned with diesel. The element is than placed back in fresh oil bath upto the mark.

- All nut -bolts are checked and tightened if necessary.

Every 250 hrs

- Used oil is drained out. Crank case is then washed with flus ing oil. Oil chamber is then filled up with recommended grade of oil preferably from a sealed tin.

- Oil filter is cleaned in diesel and replaced.

- All the carbon shoot is cleaned from the exhaust pipe.

Every 500 hrs

- The diesel filter element is replaced by new one. Quality of filter should be ensured.

- Oil filter element is replaced by a new one.

- Air filter element, in case of dry type air cleaner, is re placed by a new one.

- Diesel tank is cleaned. The ceramic filter is also cleaned in diesel.

Every 800 hrs

- The valve clearence and their setting is checked.

- Injector spray and compression are checked. Any correction of the above two items must be carried out by authorised mechanic.

After 1000hrs. which normally conforms to one agricultural year,the engine requires servicing and overhouling by experienced machanics.

8.6.1.9 Trouble shooting

Complaints	Causes
Engine does not start	Dirty/clogged air cleaner No fuel Chocked fuel line Chocked fuel injector holes Wrongly adjusted tappets Valve leakages Incorrect valve and fuel timings
Engine has starting difficulty	Poor quality of fuel Broken, seized/worn out piston rings Worn out liner/piston Incorrect valve and fuel timings High exhaust back pressure Damaged or dribbling nozzle Air in fuel line
Engine starts but stops after some time	Dirty/clogged air cleaner No fuel Chocked fuel line Faulty fuel pump Water mixed with fuel Damaged main and connecting bearing One or more cyllinders not working

Engine lacks power	Dirty/clogged air cleaner High exhaust back pressure Derating due to temperature Poor quality of diesel Faulty relief valve setting Derating due to altitude
Engine speed does not remain constant	Chocked fuel injector holes Dirty/chocked fuel filter Faulty fuel pump One or more cyllinders not working Faulty governor setting Damaged or dribbling nozzle
Excessive smoke at no load	Dirty/clogged air cleaner Damaged/dribbling nozzle Engine used after a long time Faulty fuel pump Wrongly adjusted tappets Incorrect valve and fuel timing Worn out liner/piston
Engine gives out white smoke	Wrong grade of lub oil used Defective bye-pass valve Engine used after a long time Broken/seized/worn out piston ring Worn out liner/piston Worn out valve and valve guides
Engine gets over-heated	High exhaust back pressure Wrong grade of lub oil used Dirty/clogged oil filter Faulty relief valve setting Dirty air cooling fans Engine oil not changed Engine overload
Execessive smoke at full load	Dirty/clogged air cleaner Derating due to altitude Excessive end play in crankshaft Worn out liner/piston Faulty fuel pump Wrongly adjusted tappets Engine overload
Engine gives out blue smoke	Wrong grade of lub oil used Wrong oil level Engine used after a long time Broken/seized worn out piston ring Worn out piston
Excessive consumption of diesel	Dirty/clogged aircleaner Wrong grade of lub oil used faulty fuel pump Wrongly adjusted tappets

	Valve leakage Worn out liner and piston Damaged main and connecting bearing
Mixing of diesel with lub oil	Damaged/dribbling nozzle Faulty fuel pump External and internal leakages
Excessive lub oil consumption	Wrong grade of lub oil used Wrong oil level External/Internal fuel leakage Broken seized/worn out piston rings Worn out liner/piston Incorrect bearing clearance Worn out valve and valve guide
Low lub oil pressure	Wrong grade of lub oil used Dirty/chocked suction tube strainer Dirty/chocked oil filter Chocked fuel pump Faulty fuel pump Engine oil not changed
Bearing wear	Wrong grade of lub oil used Dirty/chocked suction tube Dirty/chocked oil filter Engine oil not changed Engine over load Faulty oil pump

8.6.2 Electric Motor

If the brake horsepower of a centrifugal pump exceeds the safe operating load of the motor, the motor may be damaged or burned out. The shape of the power charecteristiccurve and the system head charecteristics will determine if pump operation may exceed the safe loading of its electric motor. Attension must also be paid to the shape of the speed-torque curve of the motor and the voltage supply of the power system. Any supply voltage lower than the rated voltage would be unable to take the load. the motor should be able to take the loads over the entire range of operating conditions.

8.6.2.1 Direct Current Motor

Direct current(dc) motors are available in three types viz. shunt wound, series wound and compound wound. But dc motors are not very suitable for centrifugal pumps as the full load speed of dc motors may vary by 5 to ~~7.5% from the rated speed.~~ A dc motor to be used for a centrifugal pump has therefore to be specially ordered to match the pump speed. Such use is possible only for special requirements. A dc motor is useful in situation where duty of the pump changes periodically.

8.6.2.2 Alternating Current Motor

Single phase motors are normally used as directly coupled with small centrifugal pumps. Use of single phase motors are restricted upto 7.5 HP applications. Motors above 7.5 HP are 440 Volts, 3 phase 50 - 60 cycles/sec.

8.6.2.3 Three Phase Motors

Squirrel cage motors are simplest type of polyphase motors and are most widely used for irrigation pumps. These motors have a primary winding (stator) and a secondary squirrel cage winding (rotor) which takes power from the primary winding by transformer action. Centrifugal pumps do not require motors with high strarting torque. Hence normal torque, Normal starting current (NEMA class A) or Normal torque low-staring current (NEMA class B) motors are adequate. Ac motors run at a constant speed. Standard rpm is 1450 or 1760 but also available at other speeds of 1160, 3475. Other motors are wound rotor motors and synchronous motors.

Motors can also classified into horizontal or vertical depending upon their housing design, slow speed or high speed, constant speed or Multispeed motors depending upon their speed characteristics.Motors can be run in either direction by simply changing any two leads of a 3 phase motor.

8.6.2.4 Features of Electric Motors

- Long life
- Low operating and maintenance cost
- Dependable service
- Easy operation
- Practically automatic
- Ineffective when power supply is not dependable.
- Extension of power lines and installation of transformers etc for a new scheme cost extra and cause inordinate delay
- Standard motor sizes are 3.5,7.5,10,15,20,25,30,40,50,60 75, 100, 125, 150, 175, 200, 225, 250, 300 HP.
- Closed motors have advantage over open motors in the sense. that they are protected from dust, drip and rodents.
- Motors have a built in service factor of 10 to 15% at air temperature 40 degree cetigrade or less. Present day motors can withstand upto 70 degree centigrade without much damage.
- Present day motors are smaller in size than earlier models due to the use of high quality insulations.
- Sometimes Voltages in all the three phase may not be equal. A 3.5% imbalance may cause about 25% increase in temperature.
- Similarly a reduction of 10% voltage in each line will cause temperature increase of about 16%.
- Motors are air cooled by the surrounding air and are safe upto an altitude of 1000 m above msl.For higher altitude operation special motor is necessary.

- Standard motors are designed to operate satisfactorily at 5% reduction of rated frequency.
- The normal efficiency of an electric motor ranges from 70% to 90%.

8.6.2.5 Starting of motors

Motors require two to three times more current to start than when it is running on full load.Normally motors upto 7.5 HP are started by Direct on line(DOL) starter. Motors ranging from 7.5-50 HP are started on "Star - Delta" starter where the motor through a star shaped circuit draws more current while starting. As the motor picks up speed the starter connection is changed over to delta shaped circuit either automatically or manually. Star and delta means the shape in which the leads gets arranged within the starter by different shifting contacts and the circuit ressemble either a trangular star or a delta.

Motors of 50 HP and above are started by using an auto-transformer in line. Starter of electric motors also contain built in circuit breaker as a protective device. Good quality starter therefore ensures motor safety. Motor may also get damaged if there is no current in one of the three phases due to faulty connection. To overcome this single phasing preventor should be set. Controlls of motor prevent the motor from over-loading and power fluctuation. Controlls like time delay fuses are available to start pump automatically after power supply is resumed after a power failure. Suitable alarms, height indiçators etc. are available to indicate whether a motor is running a dry pump.

8.6.2.6 Installation

In a situation where it is difficult to calculate the exact head correctly, in order to avoid continuous overloading of electric motor, a safely margin over the calculated HP is recommended at the following rate:

Add 50% for pumps requiring upto 2 HP
Add 30% for pumps requiring 2-5 HP
Add 20% for pumps requiring 5-10 HP
Add 15% for pumps requiring 10-20 HP
Add 10% for pumps requiring over 20 HP

In large schemes, instead of having a single motor or having provision for standby motors, parallel operation is more advantageous. The recommended break up of motors are as below.

Upto 200 HP : 2 numbers

200-300 HP : 3 numbers

Above 300 HP : 4 numbers

Decision on the number of motors however is guided more by the required capacity of each pump, the capacity of power supply and availability of motors.

8.6.3 Electric connections

Load upto 100 KW can be handled by Low Tension(LT) lines (440 V). For higher load, Heigh Tension(HT) line is required. The state electricity boards normally follow a few stipulations in giving new connections from exisiting rural transformer sub-station or any other existing service line connection. About 300 meters of line is provided free of cost by the state electricity Board. When distance is more, the extra line is laid at the consumer's cost. Supply line however should terminate in a permanent structure or pole. For LT lines, the board seeks a gurantee of regular use for 3 years to the least. For HT seven years guarantee and annual payment of 15% of the cost of laying the line is stipulated.

For an isolated schemes , the board may extend new LT line for pump installations more than 15 HP with a guarantee for consumtion for 7 years and an annual charge of 15% of the laying cost are stipulated. For HT lines,transformer at consumer point is to be supplied by the consumer. The transformer may however be provided by board on hire. These are 11 KV transformers.The stipulations however vary from state to state.

CHAPTER 9

ECONOMICS OF LIFT IRRIGATION

9.1 INTRODUCTION

Basic laws of economics are as much applicable to LIS as in any other production activities. Production of a particular crop however depends upon a number of factors, the most important of which are soil condition, metereological condition and application of inputs. Other factors remaining unchanged at a given location, water is the single most important input for crop production. Economics of LIS therefore centres around studying the crop yield in relation to the application of water which is normally obtained at a substantial cost.

The direct benefit of a LIS is the crop yield which on sale gives the revenue. Cost is all expenses incurred directly and indirectly in applying water to the crop. In the context of LIS,Cost is of two types - one is the initial or capital cost and the other is operating cost which comprises of both fixed and variable cost.

9.2 CAPITAL COST

This include all costs involved in the implementation of the scheme. Capital cost could be ascertained very easily after the scheme is implemented. Once the technical design of a scheme is finalised, a rough estimate of the capital cost could also be obtained well in advance by applying standard methods of cost estimation done by experienced person.Capital cost in a LIS is generally incurred in the following items:

- Water intake structure (tube well, open well, jack well, intake well etc)
- Pump and prime mover with suction and delivery pipes
- Pump house
- Field level distribution system
- Electric installations

It is however important that realistic cost to the different components are assigned based basically on actual market situation.

9.3 FIXED COST

Operating cost has both the fixed and variable cost components. Once the capital investment is made, the scheme would entail certain fixed costs irrespective of whether the scheme is operational or not. The annual fixed costs are ;

- Interest
- Taxes and insurances
- Fixed payment for electric installation
- Depreciation of equipments

While, interest cost, taxes, insurances and fixed payment for electric installations etc. are easily ascertainable based on actual payments , the depreciation of equipments is caluclated on the basis of the effective life span of the equipments.

Annual depreciation in the simplest method (straightline depreciation) is obtained by dividing the cost of each equipment with its estimated life span. The useful life of different items used in a LIS are different. Depreciation of each item is therefore obtained separately as presented in table 9.1.

TABLE 9.1 Estimated useful life of pumping equipments

Item	Estimated useful life
Centrifugal pump	32000 hours or 16 years
Diesel engine	28000 hours or 14 years
Electric motor	50000 hours or 25 years
Turbine pump	
Bowl assembly	16000 hours or 8 years
Column assembly	32000 hours or 16 years
V-belt	6000 hours or 3 years
Mild steel pipe	25 years
Tube well casing	20 years

9.4 VARIABLE COST

The direct cost incurred in pumping is the variable cost. Variable costs is therefore a direct function of available water. Higher the quantity of water pumped higher would be the pumping cost(variable cost) and vice versa.

Pumping cost of a given installation would also vary depending upon the type of prime mover used e.g whether an electric motor or a diesel engine. The cost of pumping in either case could be obtained either theoretically or noting the expences actually incurred.

9.4.1 Electric Motor

Theoretical consumption of power for an electric motor is given by

$$\text{KW-hr} = (1000\ QH)^{*}\ (0.746T)/75\ e \qquad \ldots\ 9.1$$

where,

Q= Discharge of the pump, cu.m/sec
H= Total head, m
e= Overall efficiency
T= Time of consumption,hr

In practice, however, the amount of power consumed is read directly from the meter installed for this purpose. The amount payable is calculated based on the tariff rate per unit multiplied by total units (KW-hr) consumed plus fixed charges if any.

9.4.2 Diesel Engine

Fuel consumption of internal combustion engine is given by its Specific Fuel Consumption (SFC). Approved SFC for common diesel engine are to the order of 175-260 grm/BHP-hr. Engines consuming more than 200 grm/BHP-hr should normally be avoided. Consumption of lubricating oil should normally be 1% of diesel consumption but may however be accepted as much as 3-4% depending upon the type of engines. The recommended SFC for standard diesel engine at different speed is presented in Table 9.2 below. It should be remembered that HP generated by a diesel engine increases proportionally with its speed (rpm).

TABLE 9.2 Recommended Specific Fuel Consumption for diesel engine

Rated engine speed rpm	Maximum S F C grm/BHP-hr	Maximum S F C grm/KW-hr
upto 500	245	332.50
500 - 1000	203	275.50
1000 - 2000	185	251.75
above 2000	227	308.75

It is clear from the table above that optimum SFC value lies somewhere between the speed of 1000-2000 rpm. Ideally, a diesel engine should consume about 250 ml of diesel per unit (100 kg-m). Consumption of a 5 HP diesel engine should be around 1-1.2 ltr/hr.

Taking the specific gravity of diesel as 0.835, the fuel consumption in litre is thus obtained as:

$$\text{Fuel consumption} = \frac{\text{SFC} \times \text{BHP} \times \text{hours}}{0.835 \times 1000} \quad \ldots \; 9.2$$

Example 9.1

A pump discharging 20 lps against 25 m of total(dynamic) head is worked for 2000 hr a year. Compute the power cost when the pump is run by (a) a diesel engine (b) an electric motor. Both the set-up is assumed to have 60% overall efficiency. Cost of diesel is Rs .05/litre and that of electric power is Rs 0.50/unit. SFC of the engine is 214 grm/BHP-hr.

Solution

Horse power requirenment = (QH)/75e
= (20x22)/75x0.6 = 9.77 HP
= 10 HP (say)

a) For diesel engine :

Annual diesel consumption = $\frac{\text{HP x SFC x hours}}{\text{Sp.gravity x 1000}}$

$= \frac{10x214x2000}{0.835 \text{ x } 1000}$ = 5125.7 litres

Annual diesel cost = 5127.7x5.0 = Rs 25628.70

b) Electric motor:

Annual power consumption = 10x0.746x2000
= 14920.0 KW-hr

Annual power cost = 14920x0.5 = Rs 7460.00

For the same installation as is shown in the above example, the power cost for diesel engine works out to be nearly 3.5 times higher than the cost to be incurred by an electric motor. In practice, the pumping cost for a given installation may be 3 to 5 times higher when a diesel engine is used in place of an electric motor.

Moreover, the efficiency of electric motor which ranges between 60-80% is generally higher than that of diesel engine (50-60%). Therefore, an installation whether the prime mover is electric motor or diesel engine, the efficiency of the system selected should be as high as possible so that cost incurred is minimized.

Example 9.2

Given below are the pertinent details of two pumping installations discharging the same quantity of water. Compute the difference of annual pumping cost between the two.

	Case I	Case II
Discharge, lps	20	20
Head, m	25	25
Pump efficiency, %	75	50
Connected load, BHP	10	15
Motor efficiency, %	80	80
Land irrigated, ha	10	10

Solution

	Case I	Case II
BHP at pump shaft	$\frac{20 \times 25}{75 \times 0.75}$	$\frac{20 \times 25}{75 \times 0.50}$
	= 7.66	= 13.33
KW input to motor	$\frac{7.66 \times 0.756}{0.8}$	$\frac{13.3 \times 0.746}{0.8}$
	= 7.14	= 12.43
Time to irrigate each ha, hr	2	2
Time to irrigate 10 ha, hr	20	20
Number of irrigation in Rabi, nos	12	12
Total pumping time, hr	240	240
Total power consumed, Kw.hr	240x7.14	240x12.43
	= 1713.6	= 2983.2
Cost of power @ Rs 0.50/unit,Rs	856.80	1491.60

Net savings per season by case I over case II,Rs 1491.60 - 856.80 = Rs 634.80

Pumping cost could also be calculated in the following manner for the sake of obtaining diffrent unit costs.

- Cost per ha-m = $\dfrac{\text{Total yearly cost}}{\text{Total ha-m of water pumped}}$

- Cost per ha-m/m of lift = $\dfrac{\text{Cost per ha-m}}{\text{Total head}}$

- Operating cost/ha-m = $\dfrac{\text{Total pumping cost}}{\text{ha-m of water pumped}}$

- Irrigation cost/ha = $\dfrac{\text{Total yearly irrigation cost}}{\text{Total hectare irrigated}}$

9.5 PROFITABILITY

Since certain amount of capital is invested in a LIS as a means of production, it is important to study the profitability of such investment. Profitability study should be undertaken both at the time of planning the project with estimated cost and anticipated benefit and also verify the same after the actual production is started. If the rate of return received from a LIS is found to be less than the existing interest rate then naturally a thorough economical review of the entire activity is called for.

9.5.1 Present worth

The real value of certain amount of money invested today but receivable after a long time is not the same. Value of money reduces with time.

Thus, Rs 650/- deposited today in a bank @ 9% interest per year would fetch cummulatively Rs 1000/- at the end of 5 years. The formula used to compute the amount receivable at the end of certain period at a given interest rate is given by

$$S = P\ (1 + i)^n \qquad \ldots\ 9.3$$

Where,

S = Future value after n years
P = Amount invested today
i = Interest rate per year
n = Number of years of investment

Thus, $S = 650\ (1.09)^5$

$= 1000$

It is therefore evident looking conversely that Rs 1000 receivable after 5 years from today at the prevailing interest rate of 9% has the present value or present worth of Rs 650. The process of finding present worth of any single future payment is simply opposite of compounding and is known as discounting.

Discounting is done quickly by using the following formula.

$$P = S[\ i/(1+i)\ ]^n \qquad \ldots\ 9.4$$

Where,

$(i/1+i)^n$ is called discounting factor

Similarly, the discounting for a series of payments of the same amount every year (annuity) is obtained by using the following formula :

$$P_n = R\ [((1+i)^n-1)/i(1+i)^n] \qquad \ldots\ 9.5$$

Where,

P_n = Present value of an annual payment over n years
R = Constant rate of yearly payment
i = Interest (discount) rate

The present value of any payment receivable in the future (Eqn. 9.3) and the present value of any annunity (Eqn. 9.5) are also found in tabular form in many books published for this purpose. For ready reference a discount table ranging from 9% to 20% interest rate is presented below.

Table 9.3 Present value of Re payable or receivable at the end of each period

Yr.	9%	10%	11%	12%	14%	16%	18%	20%
1.	0.917	0.909	0.900	0.893	0.877	0.862	0.847	0.833
2.	0.841	0.826	0.811	0.797	0.759	0.743	0.718	0.694
3.	0.772	0.751	0.731	0.712	0.675	0.640	0.608	0.578
4.	0.708	0.683	0.658	0.635	0.592	0.552	0.516	0.482
5.	0.605	0.621	0.593	0.567	0.519	0.476	0.437	0.402
6.	0.596	0.564	0.534	0.506	0.455	0.410	0.370	0.334
7.	0.547	0.531	0.481	0.453	0.399	0.354	0.314	0.792
8.	0.502	0.466	0.434	0.403	0.350	0.305	0.266	0.232
9.	0.460	0.424	0.391	0.360	0.307	0.263	0.225	0.194
10.	0.422	0.385	0.352	0.322	0.269	0.226	0.191	0.161
11.	0.387	0.350	0.317	0.287	0.236	0.195	0.162	0.134
12.	0.355	0.318	0.286	0.256	0.207	0.168	0.137	0.112
13.	0.326	0.296	0.257	0.229	0.182	0.145	0.116	0.093
14.	0.299	0.263	0.232	0.204	0.159	0.125	0.098	0.078
15.	0.274	0.239	0.209	0.183	0.140	0.108	0.083	0.068
16.	0.252	0.217	0.188	0.163	0.123	0.093	0.070	0.054
17.	0.231	0.197	0.169	0.145	0.108	0.080	0.059	0.045
18.	0.212	0.179	0.152	0.130	0.094	0.069	0.051	0.037
19.	0.194	0.163	0.137	0.116	0.083	0.059	0.043	0.031
20.	0.178	0.148	0.124	0.103	0.073	0.051	0.036	0.026
21.	0.164	0.135	0.112	0.092	0.064	0.044	0.034	0.021
22.	0.150	0.123	0.100	0.082	0.056	0.038	0.026	0.018
23.	0.137	0.111	0.091	0.074	0.049	0.033	0.022	0.015
24.	0.126	0.101	0.081	0.065	0.043	0.028	0.188	0.012
25.	0.116	0.092	0.073	0.059	0.038	0.24	0.159	0.010

Example 9.3

What would be the present worth of Rs 6000 receivable

a) at the end of 5 years at a discount rate of 10%

b) at the end of 10 years at a discount rate of 10%

c) at the end of 7 years at a discount rate of 15%

Solution

$$P = S\ [\ 1/(1+i)\ ]^n$$

a) $P = 6000\ [1/1.1]^5$
$= 6000\ (0.909)^5 = 6000\ (0.6209)$

$= 3725.5$

The present worth of Rs 6000 is Rs 3725.50 and the discount factor is 0.6209.

b) $P = 6000\ (1/1.1)^{10} = 6000\ (0.3855)$
$= 2313.16$

c) $P = 6000\ (1/1.15)^7 = 6000\ (0.3759)$
$= 2255.62$

9.6 BENEFIT-COST ANALYSIS

In making the benefit-cost analysis one of the problems encountered is that while the cost is made at present, the actual benefits are obtained in the future. It is therefore important that while comparing the benefit with the cost, all the future benefits are first reduced to its present worth for a realistic comparison.

A project is said to be viable if the benefit-cost ratio is greater than unity. As such benefit-cost ratio depends upon the prevailing interest rate which is taken as the discounting rate. Higher the discounting rate, lower would be benefit-cost ratio.

To compare the benefit-cost ratio of a LIS the present worth of each anticipated net benefit to be accrued over the entire life span of the scheme is first brought down to its present worth value. The cumulative present worth of benefit is then compared with the cumulative present worth of all costs including, operation and maintenance costs. The method is enumerated in the example below

Example 9.4

An irrigation scheme based on tubewell is to be installed at a cost of Rs 3,50,000 out of which Rs 1,20,000 is to be spent in the first year and Rs 1,30,000 in the second year. Actual production would begin from the 3rd year giving a net benefit of Rs 90,000 per year. The prevailing interest rate is 15% per year. The affective life of the tubewell is taken as 12 years after which the pipes and pumps would be sold away as scrap.

Solution

in Rs '000 (thousands)

Yr	Capital cost	O&M cost	Total cost	Discount factor 15%	Present worth of cost	Net benefit	Discount factor 15%	Present benefit
1.	120.0	-	120.0	0.869	104.2800	-	-	-
2.	130.0	-	130.0	0.756	98.2800	-	-	-
3.	-	12.5	12.5	0.657	8.2125	90	0.657	59.13
4.	-	12.5	12.5	0.571	7.1375	90	0.571	51.39
5.	-	12.5	12.5	0.497	6.2125	90	0.497	44.73
6.	-	12.5	12.5	0.432	5.4000	90	0.432	38.88
7.	-	12.5	12.5	0.375	4.7000	90	0.376	33.84
8.	-	15.0	15.0	0.327	4.9000	90	0.327	29.43
9.	-	15.0	15.0	0.284	4.2600	90	0.284	25.56
10.	-	15.0	15.0	0.247	3.7050	90	0.247	22.23
11.	-	20.0	20.0	0.215	4.3000	90	0.215	19.35
12.	-	20.0	20.0	0.187	3.7400	90	0.187	16.83
	250.0	1475.0	3975.0		2551.275			5964.975

$$\text{Benefit/cost} = \frac{5964975}{2551275} = 2.338$$

9.7 INTERNAL RATE OF RETURN

One of the standard methods of finding out the worth of a project is to find its Internal Rate of Return(IRR). If the IRR is sufficiently higher than the prevailing discounting rate then the project makes a profit. In order to find the IRR, that particular discount rate is found out at which the present worth of total future income becomes equal to the present worth of total future costs. Since it is difficult to find the exact discount rate at which the present worth of all costs and the present worth of all future benefit becomes exactly equal, IRR is therefore determined by trial and error or mathematically by using the formula.

$$A_o = A_n/(1+r)^n \qquad \ldots \; 9.6$$

Where,

A_o = initial investment
$A_1, A_2 \ldots\ldots A_n$ = stream of future cash flow upto nth year
r = internal rate of return

This method is also useful in comparing the profitability between any two projects.

Example 9.5

Cost of an irrigation tube well complete with pump and distribution network is likely to be Rs 3,00,000. The life of the scheme is anticipated to be 20 years. The scheme is expected to give next profit of Rs 50,000 per year. The prevailing interesdt rate is 20%. Find if the proposal would be profitable.

Solution

Discount rate (%) (assumed)	Discount factor 20 years	Net earning per year	Present value of earning
10	8.5136	50000	425680
11.	7.9633	50000	398165
12.	7.4694	50000	373470
13.	7.0248	50000	351240
14.	6.6231	50000	331150
15.	6.2593	50000	312965
16.	5.9288	50000	296440

The IRR lies between 15% and 16% differing by 312965 - 296440 = 16525 for 1% change. At 15% rate the difference between the total present value and initial investment is 312965 - 300000 = 12965. To compensate for this difference an additional discount rate of 12965/16525 = 0.78 should be added. The IRR is therefore 15.78%. Since 15.78% is far below the prevailing interest rate of 20%, the proposed scheme is uneconomical.

9.8 PAY BACK PERIOD

Another simple method of estimating roughly the profitability is to work out the payback period. By this method the number of years in which the cost of initial investment would be recovered is studied. Pay back period is given by

$$\text{PBP} = \frac{\text{Cost of investment}}{\text{Net cash earning per year}}$$

Pay back profitability = earning per year (Total life - pay back period)

Example 9.6

Considering the data of example 9.5 find out the pay back period of the project.

Solution

$$PBP = \frac{300000}{50000} = 6 \text{ years}$$

Payback profitability = 50,000 (20-6)

= 50,000 x 14 = 7,00,000 .

If Rs 3,00,000 is kept in bank @ 20% interest per year it would fetch at the end of 20 years

$$P_n = R[(1+i)^n - 1)/i(1+i)^n]$$
$$= 300000\ (4.8696)$$
$$= 1460880$$

which is much higher than Rs 700000 to be obtained from the above scheme.

Hence the project is unviable.

9.9 BREAK EVEN ANALYSIS

This is an analytical technique to study the effect on profit due to the changed level of production or the price. Break even analysis is a device to determine the level of production at which the revenue earned just covers the cost. Figure 9.1 explains the method. The total cost (fixed cost + viable cost) line intersects the revenue line at point P at whcih the cost of production i.e Rs 11500/- is just balanced or made break even at a production level of 23 units. Any production above this level at the given cost would give profit and below would give loss.

Algebraically

$$PQ = F + VQ \qquad \ldots\ 9.7$$

Where,

P = Price per qunital
F = Fixed cost
Q = Quantity (quintal) at BEP
V = Variable cost/unit quintal

Example 9.7

If a farmer incures a fixed cost of Rs 2500 and variable cost of Rs 9000 to grow wheat in his land

a) What should be minimum yield of wheat to break even his cost if the value of wheat is Rs 500/ quintal ?

b) What should be the cost of wheat per quintal to break even his cost if the wheat production was 230/qunitals ?

Solution

a) $$Q = \frac{F}{P - V} = \frac{2500}{500 - 9000/Q}$$

$$= \frac{2500}{(500Q - 9000)/Q} = 23 \text{ qunitals}$$

b) $$P = F/Q + V$$

$$= 2500/23 + 9000/23 = 108.7 + 391.3$$

$$= 500 \text{ i.e Rs. } 500/ \text{ quintal}$$

Break even analysis has limited application in a production activity like irrigation. The main assumption that production varies directly with the variable cost is not fully applicable in case of irrigation. Yield of a crop infact increases linearly with respect to application of water only upto a certain level beyond which such break even analysis do not apply.

9.10 PRODUCTION FUNCTION

If the production of a crop against a single varying input i.e water is studied with the assumption that level of other inputs remain unchanged, the production curve would look somewhat as shown in Fig.9.2. The figure could be easily understood by studying the sample table presented below:

Unit of water applied X Unit	Total product Y Qnt	Average product Y/X	Marginal product
1	5	5	5
2	12	6	7
3	21	7	9
4	32	8	11
5	41	8.2	9
6	45.6	7.6	4.6
7	47.6	6.8	2.0
8	48.6	6.08	0.1
9	48.6	5.4	0
10	46.6	4.6	2

Where Average crop Product (AP) is obtained by dividing total product (output) with the number of variable inputs (Y/X) and Marginal Product (MP) is the additional output obtained due to

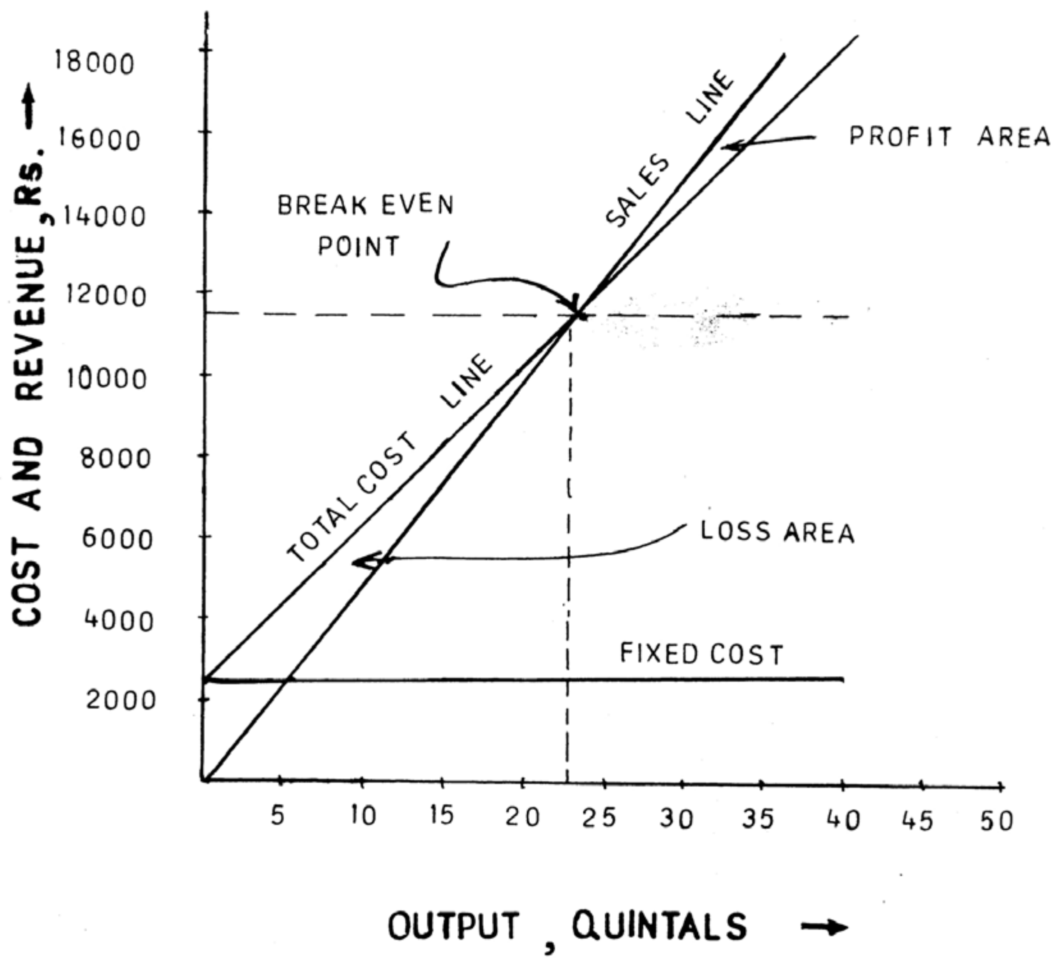

Fig. 9.1 Graphical presentation of Break even analysis

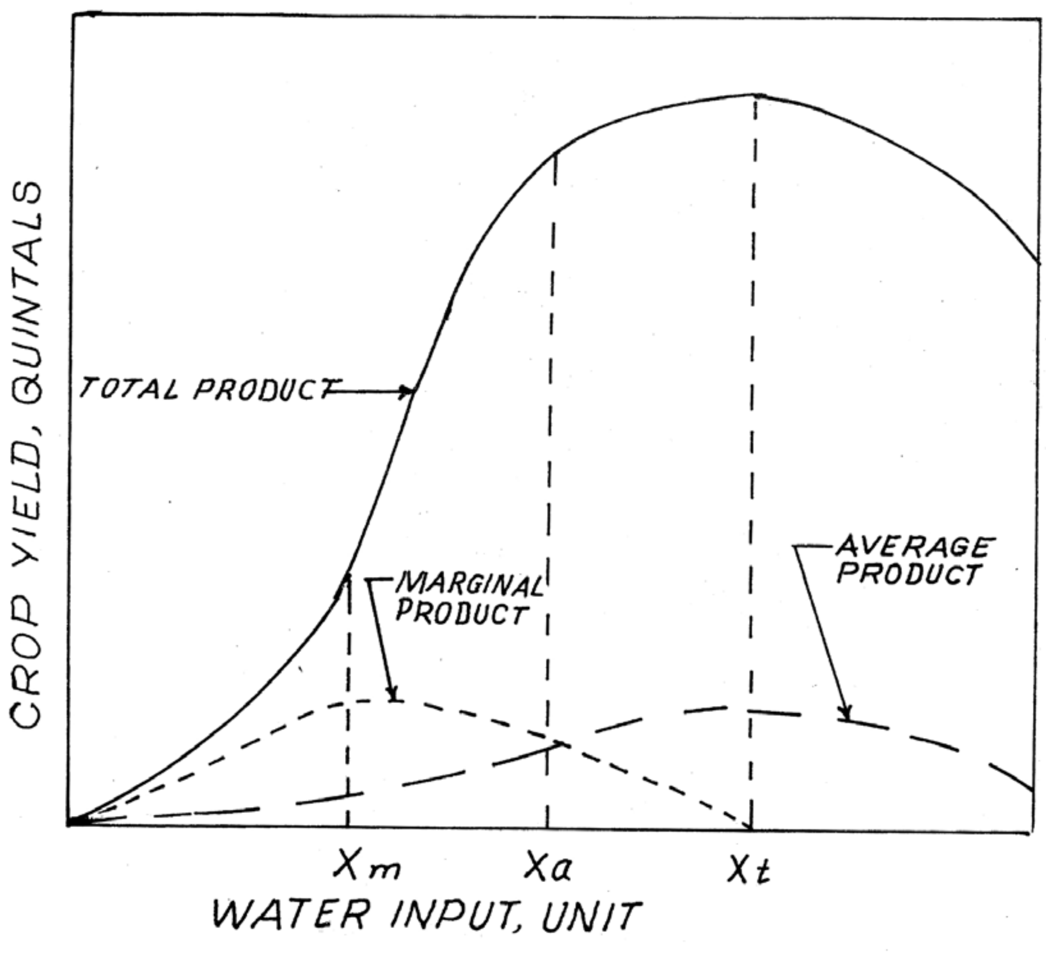

Fig. 9.2 Production function curve

application of one additional input.

The production function can be broken into 3 stages (Fig. 9.2)

STAGE I

Both the Total Product (TP) and Average Production (AP) per unit application of water increases steadily. The efficiency of water application is maximum at Xm i.e at the boundary line between Stage I and Stage II which is also indicated by Average Product (AP) per unit water being the highest.

STAGE II

Here, both the AP and MP decreases but however since MP is positive the TP continues to rise though at a smaller rate. At the boundary line between Stage II and III at Xa MP reduces to zero as TP attains the maximum value.

STAGE III

Here AP decreases and MP becomes negative. The TP also decreases. Stage II is therefore the unit beyond which a farmer should not produce (or apply any further water).Algebraically, a production function can be written as

$$Y = bX + cX^2 - dX^3 \quad \ldots \quad 9.8$$

Where,

Y = Output/ha
X = Water applied/ha
and b,c,d are co-efficents
Also AP = Y/X and MP = dY/dX

Example 9.8
If the production function equation of an irrigation system is given by $12X + 2.4X^2 - 0.2X^3$. At what level of water application the MP becomes zero i.e.The Total Product is highest.

Solution

$$\begin{aligned} MP &= dY/dX = 0 \\ &= 12 + 2*2.4X - 3*0.2X^2 = 0 \\ \text{or,} \quad & 0 = 6X^2 - 4.8X - 12 = 0 \\ \text{or,} \quad & 6X^2 - 48X - 120 = 0 \\ \text{or,} \quad & X^2 - 8X - 20 = 0 \\ \text{or,} \quad & X^2 - 10X + 2X - 20 = 0 \\ \text{or} \quad & X(X-10) + 2(X-10) = 0 \end{aligned}$$

Ignoring negative root of X, we have X =10 units of water.

Therefore Y = 12*10+2.4*100 - 0.2 *1000
= 160 quintals/hectare = Total output

To determine the level of water application at which AP attains maximum i.e at the line between stage I and stage II, the first derivative of AP is taken equal to zero.

$$AP = Y/X = 12+2.4X - 0.2X^2$$

$$d(AP)/dX = 2.4 - 0.4X = 0 \text{ or } X = 6 \text{ units}$$

The value of Y at Xa is

$$Y = 12*6 + 2.4*36 - 0.2* 216$$

$$= 115.2 \text{ quintals/hectare}$$

9.11 APPLICATION OF WATER AT PROFIT MAXIMIZATION LEVEL

Let the profit equation be

$$R = Py.Y - Px.X - B \qquad ... \quad 9.9$$

Where,

R = Profit per hectare of crops, Rs
Py= Price of crop per quintal, Rs
Y = Output level
Px= Price of water per unit, Rs
X = Units of water applied, ha-cm
B = Cost of inputs other than water

The equation represents a concave parabolla . The necessary condition for profit maximisation is the point where marginal production is zero. Mathematically this can be obtained by taking the first derivative of Eqn. 9.9 and equating it with zero.

$$dR/dX = Py\ dY/dX - Px = 0$$

$$\text{or} \quad Py\ dY/dX = Px$$

Example 9.9

A farmer has 5 ha of land . What would be the economic returns, if he applied

a) 20 units of water
b) 30 units of water
c) 35 units of water

Solution

a) 20 units of water means he gets 4 units of water per hectare. He can either apply

i) 4 units each to all the 5 ha
ii) 6 units each to only 3.33 ha

i) In the first case

$$\text{Yield} = 12X + 2.4X^2 - 0.2X^3$$
$$= 12 * 4 + 2.4 * 16 - 0.2 * 64$$
$$= 73.6 \text{ quintal}$$

In 5 ha, yield = 5*73.6 = 368 quintals

ii) In the second case

$$\text{Yield} = 12X + 2.4X^2 - 0.2x^3$$
$$= 12 * 6 + 2.4 * 36 - 0.2 * 216$$
$$= 115.2 \text{ quintal}$$

For 3.33 ha, yield = 3.33*115.2 = 383.6

Therefore he earns more money (crop) in the second case

b) 30 units of water means 6 units per hectare.
Application of 6 units of water in 5 ha would give him

$$Y = 115.2 \text{ quintal}$$

For 5 ha, yield = 115.2* 5 = 576.0 quintal

c) 35 units of water can be applied i) as 7 units per hectare equally or ii) 10 units of water to 3.5 ha of land

i) $$Y = 12 * 7 + 2.4 * 49 - 0.2 * 343$$
$$= 133 \text{ quintal}$$

For 5 ha, yield = 133*5 = 665 quintal.

ii) Suppose he applies 10 units of water to 3.5 ha of land only, then

$$Y = 12 * 10 + 2.4 * 100 - 0.2 * 1000$$
$$= 16.0 \text{ quintal}$$

For 3.5 ha, Yield = 560 quintal

In the second case the profit is less

Example 9.10

If price per quintal of wheat is Rs 100 and the cost of water per unit is Rs 660. Compute the application of water at profit maximization level. Cost of inputs other than water is Rs.1000/-.

Solution

$$R = Py\,[12X + 1.4X^2 - 0.2X^3] - PxX - 1000$$
$$dR/dX = 100[12 + 4.8X - 0.6X^2] - 660 = 0$$

or $1200 + 480X - 60X^2 - 660 = 0$

or $60X^2 - 480X - 540 = 0$

or $X^2 - 8X - 9 = 0$

or $X^2 - 9X + 1X - 9 = 0$

or $X(X-9) + 1(X-9) = 0$

Ignoring negative root of X we have X = 9 units which give maximum profit.

9.12 OPTIMUM YIELD FROM A TUBEWELL

Since it is difficult to ascertain the cost of water in the open market, optimum yield of a tubewell is obtained by comparing with another tubewell (least cost alternative). Benefit from a tube-well with respect to discharge has a straight line relationship meaning that more the discharge more the benefit. The cost on the other hand has a curvilinear (concave upwards) relationship with discharge meaning that pumping cost increases in some form of geometric progression with discharge. Optimization occurs at the point where benefit-cost is maximum. The concept is explained in Fig. 9.3.

Algebraically,
Benifit from water is given by:

$$B = QR1T1 \qquad \ldots \quad 9.10$$

Where,

B = Benefit from water, Rs
Q = Discharge from the well, cu.m/hr
R1= Value of one unit of water, Rs
T1= Time of pumping, hr

The cost of water would be :

$$C = R2(KW\text{-}hr)$$

$$= \frac{R2(1000\ HQ)(0.746)T2}{75 \times 0.6 \times 3600} \qquad \ldots \quad 9.11$$

$$= R2(QH)T2K$$

Where,

K = (0.746*1000)/75*0.6*3600) = 0.0046
R2= Cost of power per unit (KW-hr), Rs
T2= Time of pumping, hr

The drawdown in a well is given by:

$$s = BQ+CQ^2 \qquad \ldots \quad 9.12$$

and $$H = so + s \qquad \ldots \quad 9.13$$

Where,

so = Static water level
s = Drawdown
B = Aquifer loss coefficient
C = Well loss coefficient

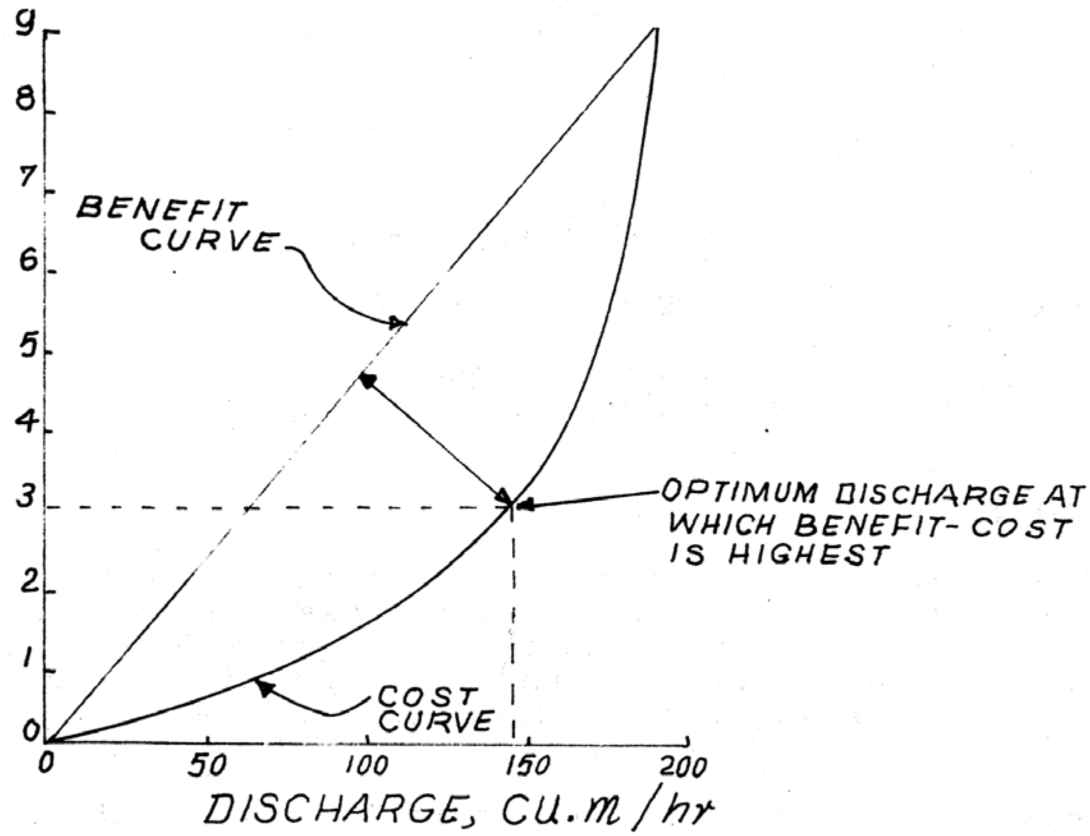

Fig. 9.3 Estimation of optimum yield from a tube well

Replacing Eqn. 9.13 into Eqn. 9.11, we have

$$C = R2Q(so+s)T2\ K$$

$$= R2Q(so+BQ+CQ^2)T2K$$

$$=R2T2KsoQ+R2T2KBQ^2+R2T2KCQ^3$$

For profit maximization

(Benefit - Cost) should be maximum

Taking $T1 = T2 = 1$,
Max(Benifit-Cost)

$$Max(QR1 - R2KsoQM - R2KBQ^2 - R2KCQ^3)$$

Differentiating the above equation w.r.t Q and making it equal to zero,we have

$$df/dQ = (R1 - R2Kso - 2R2KBQ - 3R2KCQ^2) = 0$$

$$or,\ 3R2KCQ^2 + 2R2KBQ + R2Kso - R1 = 0$$

The above equation is a quadratic equation from which the value of Q can be obtained after its solution.

9.13 OPTIMUM DIAMETER OF PIPE

For a given flow, as the diameter of a pipe is increased, the frictional loss decreases which means less pumping cost. But since the cost of pipe increases almost in geometric progression with the increase in diameter, use of larger diameter pipe increases the capital expenditure.There has to be an optimum size of pipe at which both these costs balances with each other.

In a simple method, the optimum diameter of pipe is obtained by studying the total annual cost due to the capital investment, interest,depreciation and cost of power to overcome the friction loss incurred in each size of pipe.The pipe diameter giving least of the total cost is taken as optimum diameter. The method is enumerated in Fig. 9.4.

Example 9.11

A pump has to supply 30 lps of water through a 500 m long RPVC pipe operating for 2500 hr annually. The overall efficiency of the pump and the motor is 70%. Select the economic diameter of the pipe from standard sizes available in the market. Prevailing interest rate on capital is 20% and cost of power is Rs.0.50 per unit.

Solution

The possible sizes considered are 5 cm, 10 cm, 15 cm, 20 cm and 25 cm. Total cost of pumping is computed as below.

1* cm	2* Rs.	3* Rs.	4* Rs.	5* m	6* KWhr	7* Rs.	8* Rs.
5	25000	5000	833.3	750.0	2664250	1332125.0	13,62,958.30
10	45000	9000	1500.0	51.5	183000	915600.0	1,47,000.00
15	70000	14000	2333.3	7.0	24850	12425.0	98,763.00
20	105000	21000	3500.0	1.90	6725	3362.5	1,32,882.50

Where,

1* = Size of pipe,cm
2* = Cost of 500 m pipe,Rs
3* = Cost of interest,Rs
4* = Depreciation,Rs
5* = Friction loss(From tables),m
6* = Power consumed to overcome friction,KWhr
7* = Cost of power to overcome additional friction,Rs
8* = Total cost,Rs

Lowest total annual cost of Rs 98,763.00 is incurred for pipe having 15 cm diameter and hence is considered the most economic diameter.

In another method, the economic diameter of pipe is obtained more accurately by amortizing the pipe cost instead of considering the depreciation.

Economic diameter of pumping main is a direct function of factors like cost of pipe, interest rate on capital, service life, friction lossssss and cost of power. The costs involved are capital cost and pumping cost.

Annual amortized cost of pipe (Cp) is given by :

$$Cp = PL \frac{i(1+i)^n}{(1+i)^n - 1} \quad \text{. . . 9.14}$$

Where,

P = cost of pipe per metre, Rs
L = acquired length of pipe, m
i = annual interest rate, %
n = expected life span, years

The relation between cost of pipe and diameter is given by:

$$P = A\, d^{B} \quad \text{. . . 9.15}$$

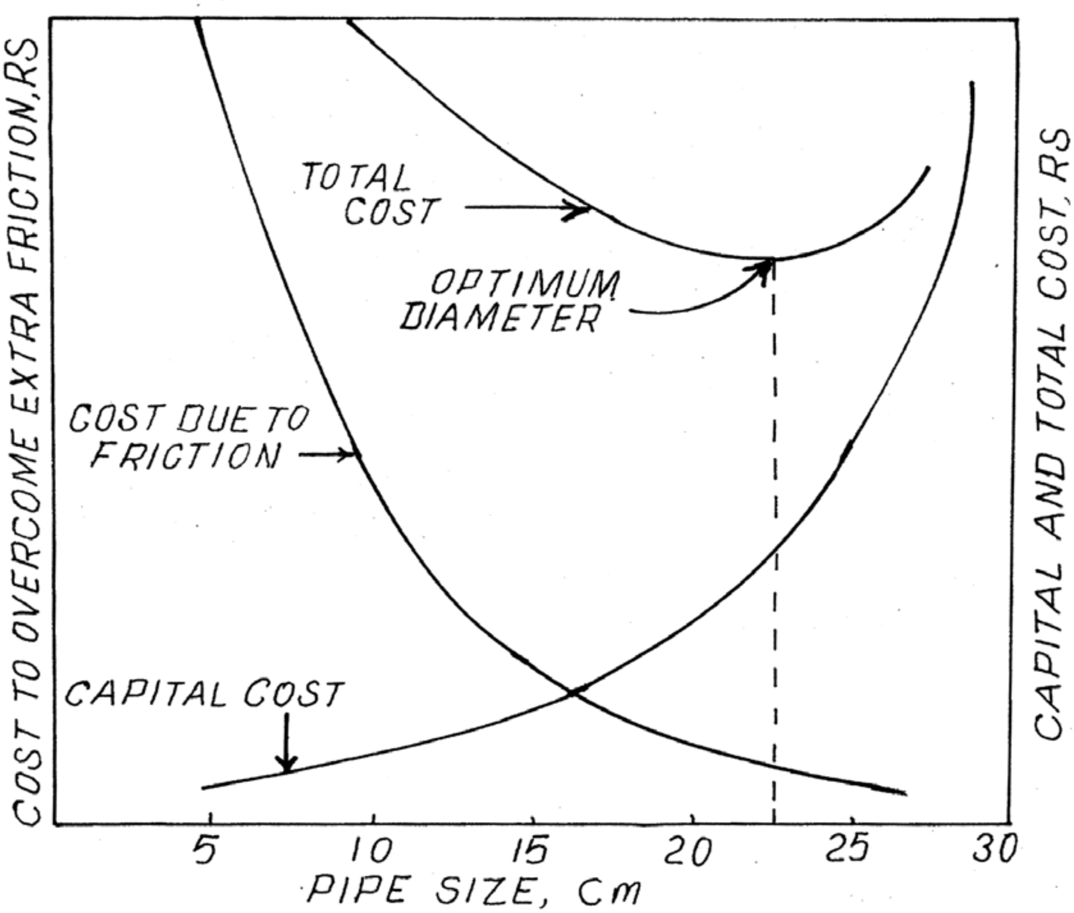

Fig. 9.4 Estimation of optimum diameter of pipes

Where,

d = Diameter of pipe, cm

A, B = Cost coefficients

The cost of power consumed to overcome the frictional loss is obtained from Dercy - Weisback formula :

$$hf = \frac{32fLQ^2}{gd^5} \qquad \ldots\ 9.16$$

Where,

f = Friction factor
L = Length of pipe, m
Q = Flow rate, cu.m/sec
d = Pipe diameter, m

The annual operating cost (C0) to overcome friction as obtained from Eqn. 9.1 and 9.16 is given by:

$$Co = \frac{3.244\ fL\ Q^3\ T\ M}{d^5 e} \qquad \ldots\ 9.17$$

Where,

T = Time of pumping, hr
M = Cost of energy, Rs./KW-hr
e = Efficiency of the pump

The total cost to overcome friction is therefore given by:

$$Tc = Cp + Co$$

$$Tc = (Ad^B)L\ \frac{i(1+i)^n}{(1+i)^n - 1} + \frac{3.244fLQ^3TM}{d^5 e}$$

For obtaining the minima, the above equation is differentiated w.r.t diameter d and is equated to zero.

Thus,

$$BAd^{B-1}\ L\ \frac{i(1+i)^n}{(1+i)^n - 1} = \frac{16.22fLQ^3TM}{d^6 e}$$

$$\text{or, } d^6 \, d^{B-1} = \frac{16.22 \, f \, Q^3 \, T \, M}{ABe(i(1+i)^n)/((1+i)^n-1)} \quad \ldots \quad 9.18$$

Value of d can be obtained easily once the value of the cost coefficients A and B are obtained by using the price list for different diameters of pipes since the values of other factors involved in the Eqn. 9.18 are known. The friction factor(f) of different pipes are give below :

GI Pipe	:	0.0027 to 0.0056 = 0.0005
CI Pipe	:	0.0031 to 0.0061 = 0.0006
Concrete pipe	:	0.003 to 0.0102 = 0.010
RPVC pipe	:	= 0.002

For old pipes f may be taken as 0.01

Example 9.12

Given below are the necessary data. Compute the most economic diameter of RPVC pipe for a flow requiremnent of 30 lps.

Q = 30 lps = 0.03 cu.m/sec
n = 30 years
i = 20% per year
e = 70%
M = Rs 0.50/KWH
T = 2500 hrs/year
f = 0.002
L = 500 m

Cost of Pipe

Diameter cm	Cost Rs.
5	25,000
10	45,000
15	70,000
20	1,05,000
25	1,50,000

Solution

$C = Ad^B$

or $\log C = \log A + B\log d$

or $\log 25000 = \log A + B \log 5$
or $4.398 = \log A + 0.699\,B$ . . . (1)

Again

$\log 150000 = \log A + B \log 25$
or $5.176 = \log A + 1.398\,B$ . . . (2)

Substracting 1 from 2

$0.778 = 0.699B$
or $B = 1.11$

Substituting value of B in equation (2)

$5.176 = \log A + 1.552$
or $\log A = 3.624$
or $A = 4207.26$

Substituting the known values in Eqn. 9.18

$$d^6 d^{\,0.11} = \frac{16.22 \times 0.002 \times (0.03)^3 \times 2500 \times 0.5}{4207.26 \times 1.11 [0.2 (1.2)^{30}] / [(1.2)^{30} - 1] 0.7}$$

$$= \frac{0.00109}{653.8}$$

$$= 0.00000167$$

or $6.11 * \log d = -5.776$
$\log d = -0.945$
$d = 0.113 \text{ m} = 11.3 \text{ cm}$

REFERENCE

AFPRO, 1982, Technical manual on water supply in the planta tion for UPASI, St. John's Hospital, Bangalore.

A.R.D.C., 1980, Pilot project studies for quality control of agricultural pumpsets, Bombay.

A.R.D.C., 1981, Guideline for selection of agricultural pumpsets, Bombay.

Garg, S.P., 1982, Groundwater and tubewells. Oxford & IBH Pub. Co. New Delhi.

IARI, 1970, Hand Book of farm irrigation structures, New Delhi

Indian Pump Manufacturers' Association, 1983., Directory of Indian pump Industry, Bombay.

Johnson Division, 1983, Ground water and wells, Jain Brothers, Udaipur.

Karassik, I. and Carter, R., 1960, Centrifugal pumps selection, operation and maintenance, McGraw-Hill Inc. U.S.A.

Kirloskar Brothers Limited, 1976, Pump engineering data, Poona.

Michael, A.N., 1978, Irrigation-Theory and practices, Vikas Publishing House, Bombay.

WATER SUPPLY SPECIALISTS PRIVATE LTD. (WASP), Useful information on pumps and pumping, Bombay.

W. M. Division, 1971, Hand book on irrigation water management. Dept. of Agriculture, Ministry of Agriculture, New Delhi.