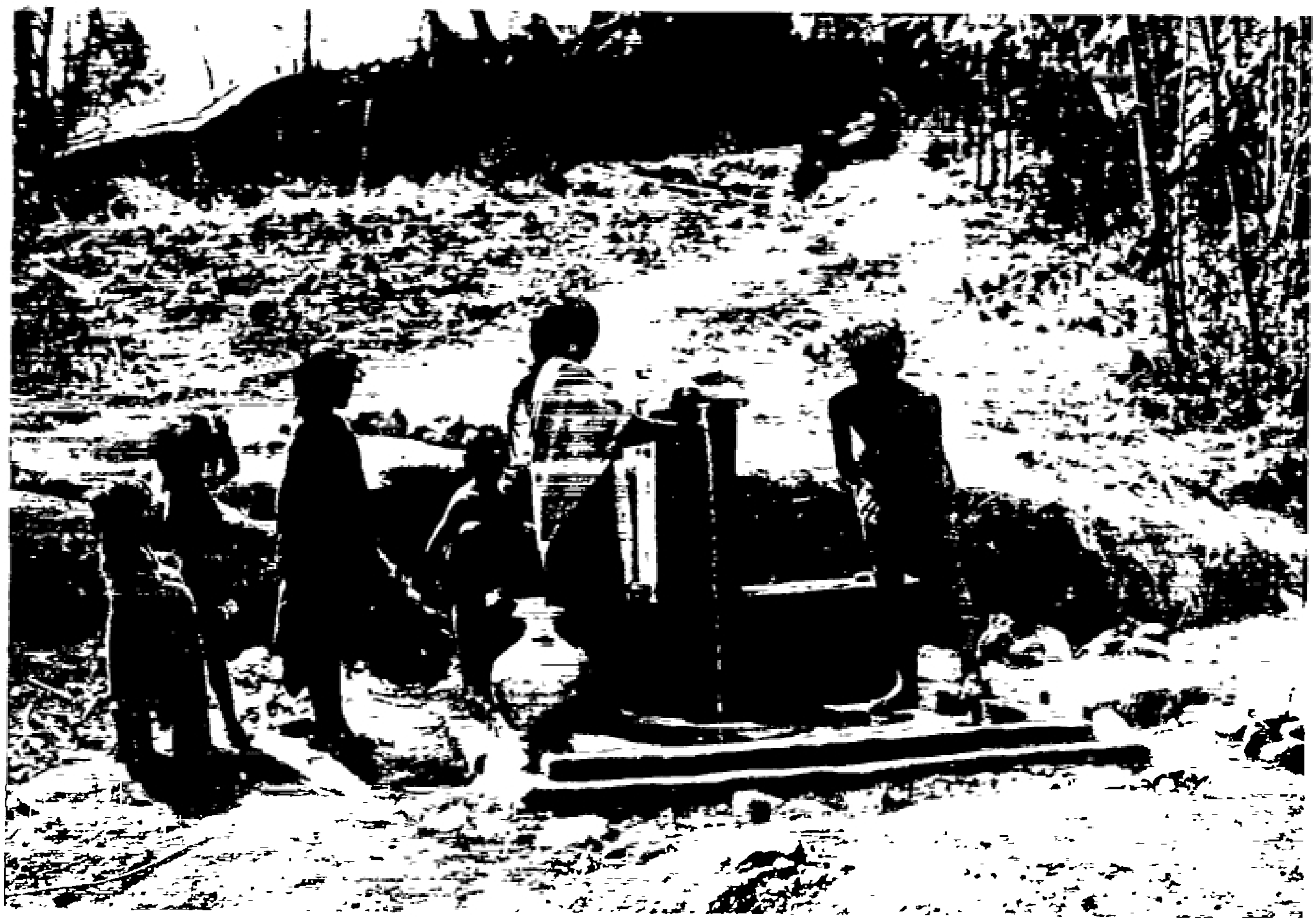
UNIVERSITY of LIVERPOOL

DEPARTMENT OF CIVIL ENGINEERING

SUSTAINABLE AND LEAST COST GRAVITY WATER SUPPLY

FOR

A RURAL COMMUNITY IN THE HILLS



SUSTAINABLE AND LEAST COST GRAVITY WATER SUPPLY

FOR

A RURAL COMMUNITY IN THE HILLS.

Drona Raj Ghimire

Dissertation

submitted in partial fulfilment of requirements

for

the degree of Master of Science (Eng)

in

Environmental Civil Engineering.

The University of Liverpool

Department of Civil Engineering

October 1991.

To

the millions of rural people

who are deprived of reasonable

water supplies.

Acknowledgement

I am thankful to my supervisor Professor A B Templeman at the Department of Civil Engineering, the University of Liverpool, for his constant guidance in preparation of this dissertation; his comments and suggestions in each chapter were invaluable basis for improvements and corrections.

I am indebted to my friends Mr Ajaya M Dixit, Mr Mukund P Neupane and my brother Mr Ganesh K Ghimire, who in a short notice managed to send me some useful information from Nepal.

I would like to acknowledge and appreciate the help of my friend Mr Ganesh B Thapa in typing and making sketches.

I am grateful to all those who directly or indirectly helped, encouraged, and supported me to complete the present study.

Summary

Providing water to the rural communities is one of the high priority goals in the rural development policy in the Third World. However, making the rural water supply schemes least cost and long lasting is proving to be a difficult challenge, despite the fact that the schemes are small, and technically simple.

This dissertation primarily describes the Sustainability and the Optimization of the rural water supply schemes in the Third World with special reference to Nepal.

Advantages of providing an adequate quantity of safe and wholesome water are critically discussed in the socio-economic and cultural context of the rural areas. The sustainability of the rural water supply scheme is dependent on various aspects such as: technological choice; consideration to the local skill and manpower; problems associated with repair, operation and maintenance; socio-economic situations; cultural aspects and traditional values; and even the political realities. As almost every aspects of rural life come together at these rural level schemes, these schemes are most likely to be sustainable only if they become a part of a comprehensive rural development programme.

The optimization of one single scheme which is small and cost low may not be an attractive area. However, it becomes important when there are hundreds of communities in need of such schemes and the resources available are limited. The dissertation describes a general approach of water supply system optimization. The Linear Programme (LP) models for the optimization of the looped and the branched pipe network are presented. A LP model for the optimization of the gravity water supply in the hilly terrain is developed, application of which is demonstrated by solving a hypothetical example problem.

NOTATIONS:

A = cross-sectional area of a pipe.

C = Hazen-William constant

C_m = Cost of pipe 'm'

C_t = Cost of a tank

C_T = Total cost

D = Diameter

D_m = Diameter of pipe 'm'

d_p = commercially available diameter.

d_{max} = Maximum permissible diameter

d_{min} = Minimum " "

E_{ij} = Elevation difference between points i and j

f = Friction factor

g = Acceleration due to gravity

H_i = Pressure at node i

H_{sm} = Hydrostatic pressure anywhere in the pipeline.

H_{min} = Minimum permissible pressure at a node.

h = Pressure head

h_L = Head loss in a pipe

h_{pt} = Maximum permissible hydrostatic pressure for pipe type 't'

K = Head loss coefficient

K_s = Surface roughness

L_m = Length of pipe 'm'

l_p = Length of pipe segment of diameter d_p

N = Number of tanks

n = Manning's roughness coefficient.

P = Hydrostatic pressure

P_w = Wetted perimeter

Q = Rate of flow, Q_m = rate of flow at pipe m

R = Hydraulic radius

Re = Reynold's number

S = Slope of hydraulic (or energy) grade line.

V = velocity of flow

V_m = Velocity in a pipe 'm'

V_{max} = Maximum permissible velocity.

V_{min} = Minimum permissible velocity.

z = Potential head

γ = Unit weight of water

γ_t = Cost coefficient for the pipe type 't'

ρ = Density of water

μ = Viscosity of water

ν = Kinematic viscosity of water

Content

CHAPTER 1

1. Introduction

1.1 SCOPE OF THE STUDY:

This dissertation covers two aspects of the rural water supply schemes in the third world; the first is the Sustainability, and the second is the Optimization. Though many aspects of the discussions presented here are likely to be common to many other developing countries also, the situation of Nepal was predominant in the mind of the author while preparing this thesis.

A brief introduction of Nepal, and the characteristics and definition of the rural communities in the hills are presented in this chapter.

Chapter 2 starts with a general review of the advantages of potable water. Realization of these benefits are critically discussed considering the social, economic and other aspects of the rural life. The second half of the chapter deals with sustainability aspect of the rural water supply schemes. The role of Appropriate technology, Community Participation, Institutional development etc. and aspect of Operation & Maintenance for the long term viability of the schemes are described. A brief critical overview of the present approach and policy of rural development in Nepal is presented.

The second part of the thesis covers the optimization aspects. First a brief review of the basic theory of hydraulics as used in

the analysis and the design of pipe flows are described in Chapter 3. The conventional methods of pipe network design are summarised. Then Chapter 4 describes the model-based optimum design of water supply networks. A general strategy for the water supply system optimization is discussed and Linear Programme (LP) models for the optimization of the Looped as well as Branched network are presented. Considering the hilly terrain of Nepal a LP model is developed for the optimization of the branched gravity water supply network when the various points in it have high elevation differences. The use of the developed model is demonstrated through the solution of a hypothetical example in Chapter 5.

1.2 NEPAL AND RURAL COMMUNITY IN THE HILLS:

Nepal

Nepal is a small mountainous country sandwiched between two Asian giants, China and India ; with a latitude of 26° 22′ N to 30° 27′ N and longitude of 80° 4′ E to 88° 12′ E. The total area of the country is 147,181 sq.km. of which about two-thirds is occupied by hills and mountains. In 1990 the estimated total population was 18,916,304 while the population growth rate was estimated to be 2.56% during 1989-1990.

The ecological distribution of population in 1981 showed that 56.4 % lived in mountains and hills, and 43.6 % lived in the plain area (Terai). About 91 % of the population was estimated to

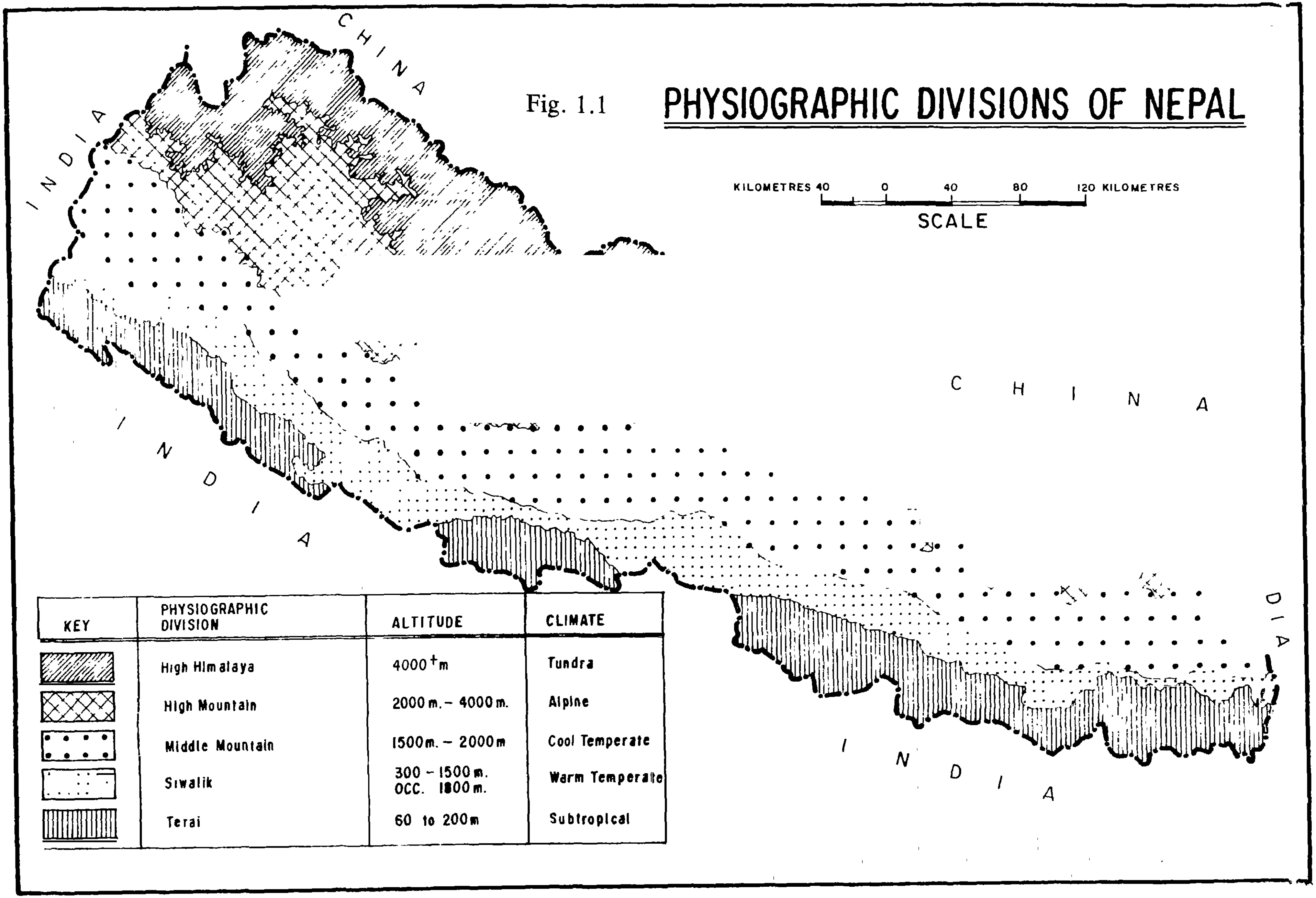


Fig. 1.1 PHYSIOGRAPHIC DIVISIONS OF NEPAL

live in rural areas under severe poverty and in an underdeveloped state.

The country has widely varying topographical and climatological features (Fig 2.1). The ground level rises from about 70.0 m to the highest mountain of the world (8848 m) in just about 200 km width. This variation in elevation causes climatic variation which varies from sub-tropical in the plain area (Terai) to Arctic in the Himalayan Range.

In the hills and mountains, ground water exploration and abstraction is not practical. However, there are numerous ever flowing streams, snow fed rivers and springs through-out the country. In the Siwalik Range, small springs are found on the Northern slopes, but Southern slopes are usually dry and do not retain enough water to produce sources that yield year round[2]. The hydrogeological, geographical and topographical conditions; and nature of settlements,unavailability of power etc. make gravity water supply from surface sources the only practical and feasible option in the mountains.

Characteristics and
Definition of a Rural Community:

Often urban and rural communities are differentiated on the basis of their population size. However, there are important

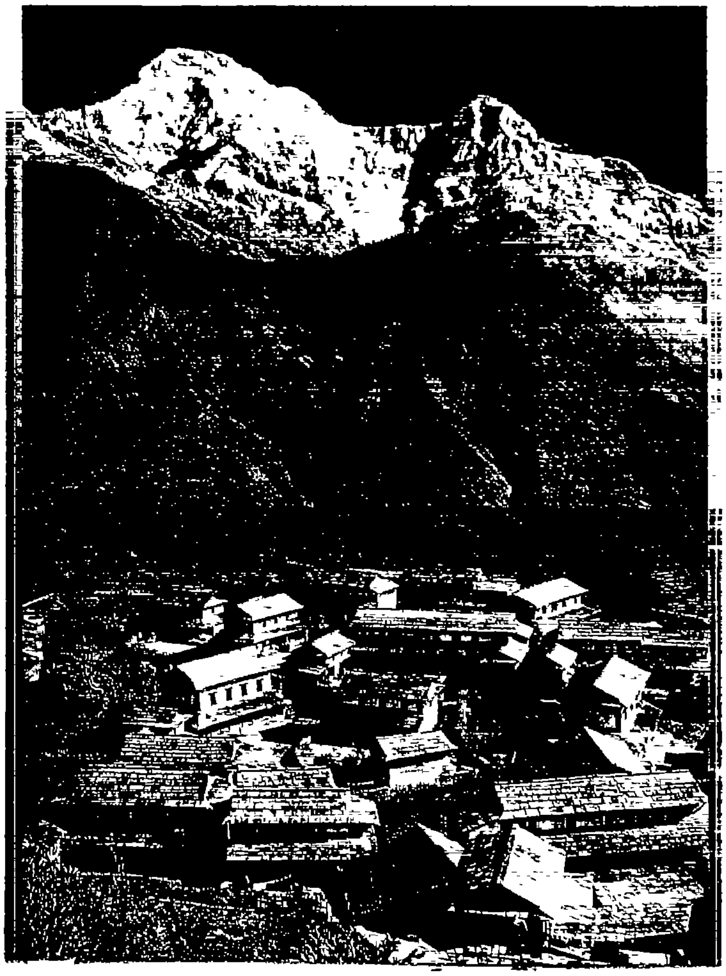


Fig. 1.2a Typical settlement in the high altitude.

(Ghandruk village, west Nepal)

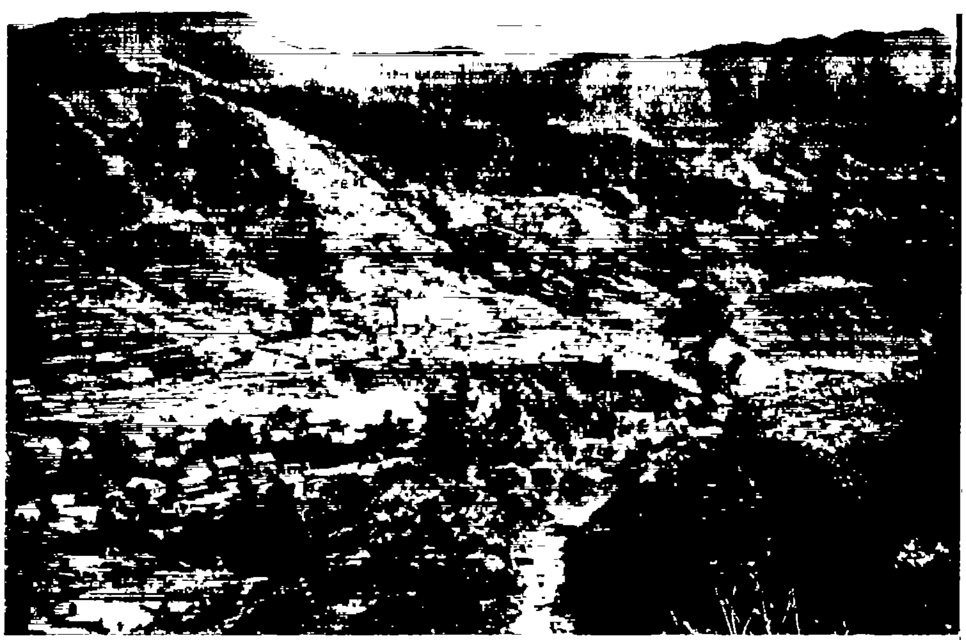


Fig. 1.2b Typical settlement in the mid-hills.

(Palung village, central Nepal)

differences in the characteristics of urban and rural communities,other than population size. The most important among those is whether the predominant activity is agriculture or not. The urban communities are more fully integrated into national market economy and the majority of the population are not engaged in agriculture, where as the rural communities are more isolated, have less commercial exchange with the outside. Their activities are predominately agricultural, and they have relatively few relationship with regard to their daily livelihood which extend outside the community. For a few things like cloth, kerosine oil, salt etc they have direct links with the national market economy, for other things, by and large, the communities are traditionally self-reliant.

In rural areas almost all community members are known to one another and contacts are on a personal basis. All members of the community have access to land for cultivation, and underemployment is largely a matter of having little or nothing to do on the land at certain times of year. However at planting and harvesting periods, agriculture requires all available labour. In slack period of agricultural involvement, voluntary work could be organised , where as in urban areas such voluntary unpaid labour is usually not available.

A rural community is less aware of its political other rights compared to better- educated urban population. In the mountains

and hills, where modern means of transportation & communication are not available, relationships among community members are influenced by the ease of physical communication. A real community is generally the geographically separate settlement, defined usually by topography.

A community in the full social sense is not just a locality where people live but is defined by a dense network of social relationships between its members and a strong sense of belonging. An administrative zone where usually more than one such settlement is included may not be taken as a community. A community , therefore, can be defined as a social entity, organised in some fashion however loose and informal, and with a sense of identity, not just the habitation of a locality[3].

The sense of community tends to be very strong in relatively isolated places, and weak in urban neighbourhoods. The smaller the village ,the more likely it is to be homogeneous with everyone on a similar social level. In such situations cooperation may usually be easier to organise.

The population settlement in Nepal is distinguished by ethnical group or by caste. The places of settlements of various ethnical groups roughly follow the physiographical and hence climatic division of the country. The communities in the higher altitude are more or less homogeneous in ethnical, economic and caste sense; and they are composed of small nucleated villages. The communities that live in the lower altitudes are more heterogeneous, and divided by ethnical groups and / or by caste;

and houses are scattered. A heterogeneous community can actually be seen as many separate 'real' communities (or sub-communities) depending on their sense of common identity, interest,ethnical group, and caste.

In a rural community economic and political power are traditionally controlled by one particular group of people. The situation is more so in heterogeneous communities than in homogeneous one . Any development attempt from an external agency, might change, reduce or destroy such power and influence of these people. Clearly there is a potential for tension between external agencies involved in such developmental activities and those who find their power and influence in their communities under challenge.

CHAPTER 2

2. Advantages of Water supply and Aspects of Sustainability.

2.1 INTRODUCTION:

This chapter starts with a general discussion on the potential benefits of a good water supply system; emphasis is given on the health benefits. These advantages are critically reviewed in the rural context of a developing country like Nepal. The second part of the chapter deals with the sustainability aspects of the small rural water supply schemes in which discussion on aspects like; Appropriate technology, Community Participation, Operation and Maintenance, Institutional Development,etc are presented.

2.2 BENEFITS OF POTABLE WATER:

2.2.1 General benefits:

Water is essential for life. All human communities must have some kind of water source to sustain life.Need for water varies depending on the climatic condition and lifestyle, a person needs water each day for drinking and other domestic uses e.g. cooking, bathing, laundry, cleaning etc. Concern regarding water to a community arises when available water is insufficient, or it is dirty, or the source is far away as is the case in the rural areas of the third world. Over half of the population in the developing world do not have easy access to safe drinking water. An easy and reliable access to a safe and wholesome water

provides many direct and indirect (or intangibles) benefits to a community, especially the one which is underprivileged and underdeveloped. The direct benefits are related to health, saving of time and money, and the increase in productivity. The indirect benefits include secondary and tertiary effects on the whole aspects of people's way of life, and consequent socio-economic changes, effects on traditionally attitudes, values, and beliefs. Table 2.1 gives summary of aims and potential benefits of water supply improvements[4].

However, in the third world, often no goals (other than to have water flowing out of the end of pipe), and benefits are clearly aimed at when a decision to provide water supply for a rural community is taken. The realization of the expected benefits are much more complex than it seems at first glance, and are influenced by complex interrelation between various socio-economic issues. The complementary inputs necessary for the achievements of the various aims and benefits set out in Table 2.1 are summarised in Table 2.2[4] .

The health benefit is the strongest and most frequent argument put forward in favour of community water supplies. Therefore, a discussion on health aspect of water supply follows in section 2.2.2

Table 2.1 Aims & Potential Benefits of Water Supply Improvements

Immediate Aims	Stage I Benefits	Stage II Benefits	Stage III Benefits
Improved Water quality quantity Availability Reliability	Save time Save energy Improved health	Labour release Crop innovation Crop improvement Animal husbandry innovation Animal husbandry improvements	Higher cash income Increased and more reliable subsistence Improved health Increased leisure

[Source: Feachem, R G : 1983]

Table 2.2 Complementary inputs necessary for the achievement of the various aims and benefits set out in table 2.1

Aims or Benefits (Table 2.1)	Complementary inputs or prerequisite conditions
Immediate aim	Active community participation and support. Competent design. Adequate facilities for operation and maintenance. Appropriate technology utilised.
Stage I benefits	New supply used in preference to old. New supply closer to dwelling than old. Water use pattern changed to take advantage of improved quality, availability and reliability. Hygiene changed to utilise improved supply. Other environmental health measure taken. Supply must not create new health hazards (e.g mosquito breeding site).
Stage II benefits	Good advice and extension services must be provided by the government personnel concerned with agriculture, animal husbandry, cooperatives, marketing, education, credit etc.
Stage III benefits	Water supply development must be just a single component of an integrated rural development programme which has the active support of the local community.

[Source: Feachem, R G: 1983]

2.2.2 Health aspect of a water supply scheme:

About 80% of the sicknesses and diseases in the world comes from unsafe water[5]. In the developed nations increase in the life expectancy of general public in the last century is attributed as much to the provision of an adequate quantity of safe and wholesome water for domestic purpose as to the development in medical science.

There are many infectious diseases related to the microbiological quality of water, while others are related to the quantity of available water and / or to sanitation. There are many health problems associated with the chemical quality of the drinking water e.g. Tooth decay. Therefore provision of an adequate quantity of safe (free from pathogens) drinking water is important in control of the water-related infectious diseases, where as provision of wholesome (of good chemical quality) water is helpful in controlling non-infectious water related health problems. Table 2.3 gives a summary of infectious diseases related to deficiencies in water and / or sanitation[5].

Many disease causing organism which (e.g. an insect, snail, or other cold-blooded organism, or which undergo development in the soil) infect and / or spread water related diseases, need fairly warm temperatures if they are to complete their stages of life

Fig. 2.1 Rural women fetching drinking water

(East Nepal Tarai)

spent outside man. The tropical climate is very suitable for the organism to complete its life-cycle. On the other hand most of the people from the developing nations cannot afford a conventionally good water supply. Because of these two reasons: warm climate and poverty, diseases related to water and / or sanitation are widespread in the tropical developing world.

Water-borne diseases[6,7,8] are transmitted when water that is contaminated with pathogens (the disease causing organism) is consumed by a person. The pathogen reaches the water from infected human or animal faecal material. It must be noted that all water -borne diseases can also be transmitted by any route which permits faecal material to pass into the mouth (i.e. any other faecal-oral route e.g. Cholera may be transmitted by contaminated food). Therefore, provision of safe drinking water is important but not sufficient in controlling water-borne diseases. In the absence of general health care on the part of consumers, water supply alone is unlikely to produce the expected health benefits.

Insufficient quantity of water within easy access will usually lead to poor personal hygiene and domestic cleanliness. Such a condition favours the spread of many infectious diseases known as water wash-diseases[6,7,8]. These water-wash diseases include infections of the intestinal track such as diarrhoea & other diseases transmitted by faecal-oral routes (e.g.typhoid,

bacillary dysentery & other water-borne diseases), infections of the body surface such as skin sepsis, scabies, fungal infections

Table 2.3 Diseases related to deficiencies in water supply and/ or sanitation.

Group	Diseases
* Diseases transmitted by water (water-borne diseases). Water acts only as a passive vehicle for the infecting agent. All of these diseases depend also on poor sanitation.	Cholera,Typhoid,Bacillary dysentery, Infectious hepatitis, Leptospirosis, Giardiasis, Gastro enteritis
* Disease due to lack of water (water-washed diseases). Lack of adequate quantity of water and poor personal hygiene create conditions favourable for their spread. The intestinal infections in this group also depend on lack of proper human waste disposal.	Scabies, Skin sepsis and ulcers Yaws, Leprosy, Lice & typhus, Trachoma, Conjunctivitis, Bacillary dysentery, Amoebic dysentery, Salmonellosis, Enterovirus diarrhoea, Paratyphoid fever, Ascariasis, Trichuriasis, Whipworm (Enterobius), Hookworm (Ankylostoma).
* Disease caused by infecting agents spread by contact with or ingestion of water. (Water-based diseases). An intestinal part of the life cycle of the infecting agent takes place in a aquatic animal. Some are also affected by waste disposal.	Schistosomiasis (urinary rental), Dracunculosis (guinea worm), Bilhariosis, Philariosis, Oncholersosis, Treadworm.
* Diseases transmitted by insects which live close to water. (Water related vectors) Infections are spread by mosquitos, flies, insects that breed in water or bite near it. These are especially active and aggressive near stagnant open water. Unaffected by disposal.	By mosquito (Yellow fever, Dengue, Haemorrhagic fever, West-nile and Rift valley fever,Arbovirus Encephalitides, Bancroftian Filariasis, Malaria, Diarrhoea@) and by flies (Onchocerciasis@ by Simulium fly, and Sleeping sickness@ by Tsetse fly).
* Diseases caused by infecting agents. Mostly contracted by eating uncooked fish and other food. (Faecal-disposal diseases)	Through fish: Clonorchiasis and Diphyllobothriasis Through plant: Fasciolopsiasis Through Cray fish: Paragnimiasi

@ Unusual for domestic water to affect these much.

[Source: Hofkes, E H at el: 1983]

of the skin and eye infections e.g. trachoma; and infections carried by the insects parasitic on the body surface such as mites cause scabies & promote asthma, louse-borne epidemic typhus, relapsing fever. People's awareness regarding importance of personal hygiene is as important as providing adequate quantity of water in controlling water-washed diseases.

All water-based disease[6,7,8] are due to infection by parasitic worms which must spend a part of its life cycle on an intermediate aquatic host before it infect man. For example Schistosomiasis, in which water polluted by schistosomiasis patients excreta may contain aquatic snail in which the Schistosome larvae develop until infective Cercariae are shed into the water and reinfect man through his skin. Another example is Guinea worm, the larvae of which escape from man through skin lesions and develop in small aquatic Crustacea, and man is reinfected by drinking water containing these Crustacea.

Some water-related diseases[6,7,8] are spread by insects which either breed in water or bite near water. Malaria, Yellow Fever, Dengue and Onchocerciasis (river blindness) etc are transmitted by insects which breed in stagnant water, pools and sometime even in domestic water containers. Sleeping sickness (Trypanosomiasis) is transmitted by the riverine Tsetse fly which bites near water. The Yellow Fever and Dengue are transmitted by Aedes mosquito

which chiefly breed in temporary water containers, pots and jars, used to store water in the household where the supply is intermittent or has to be carried.

The appropriate preventive strategies[4,7] for control of the infectious diseases related to the water are presented in Table 2.4 .

Table: 2.4 __Water related disease transmission mechanism and the appropriate preventive strategies__.

Transmission mechanism	Preventative strategies
Water-borne	Improve water quality (e.g. microbiological sterility to control Typhoid,Cholera and microbiological improvement to control infective hepatitis). Prevent causal use of other unimproved water sources
Water-washed	Improve water quantity (e.g.for controlling Scabies, Trachoma, Bacillary dysentery etc. provide greater volume of water) Improve hygiene.
Water-based	Decrease need for water contact Control snail populations Improve quality (e.g. to control schistosomiasis, protect the users and to control Guinea worm protect source)
Water-related insect vector	Improve surface water management Destroy breeding sites of insects Decrease need to visit breeding sites. (e.g. to control sleeping sickness use piped water from source ,& to control Yellow fever use piped to site of use)
Infections primarily of defective	Improve sanitary faecal disposal (e.g. control of Hookworm)

[Source:Feachem, R G ; Brandley, D J : 1983]

The health problems are also related to the chemical composition

as opposed to microbiological composition , of drinking water[6]. The taste, colour, odour, and even the feel of water is affected by the chemicals contained within it. Health problems may arise either if there is an absence of necessary constituents or if there is an excess of harmful chemicals. These problems may be avoided simply by adding the chemicals which are deficient or removing those which are harmful. Examples of health problems due to deficiency of essential chemicals are:

a) A deficiency of fluoride in water and diet can cause poor growth of bones and teeth in the young.

b) A deficiency of Iodine in water & diet can cause a wide-spread goitre.

c) Women with goitre, who are also poorly nourished and repeatedly sick, are prone to bear children with damage to the brain and central nervous system.This is known as endemic Cretinism and likely to occur among isolated rural communities in mountainous areas where iodine is deficient in soil , water and staple foodstuffs.

d) Population using hard water (containing the carbonates and sulphates of calcium and magnesium) have a lower incidence of cardiovascular disease.

Chemicals, if present in excess of certain limits, may create health problems. They are organic or inorganic in nature. Some organic compound, or groups of compounds, are known to be either toxic, or carcinogenic (cancer-producing) or to produce odours or

taste , sometimes after reacting with the chlorine used for disinfection. Most of the toxic organic chemicals are pesticides (including herbicides, fungicides, insecticides, and molluscicides) , the bulk of which are applied in agriculture. Thus most human intake of pesticides is in food, not in water.The other organic chemicals in which particular attention are paid to their presence in drinking water are Polynuclear Aromatic Hydrocarbons (PAHs), and Trihalomethanes (THMs).

The presence of harmful inorganic chemicals is considered more dangerous than the presence of organic chemicals. A number of metallic ions cause metabolic disturbances in man, or cause a variety of other toxic effects. The common toxic inorganic chemical are Arsenic, Barium, Beryllium, Boron, Cadmium, Cobalt,Lead, Mercury, Molybdenum, Selenium, Tin, Vianium, Vanadium etc. The limit of their quantities in drinking water is defined by WHO or by the concerned authority in the country.

In developing countries, the presence of chemicals such as salt (mainly chlorides and sulphates) in ground water may make the water unpalatable, and so lead people to use surface water which is more likely to be bacteriologically polluted.

The concentration of over 2 mg/l of fluoride has been associated with mottling of tooth enamel, and concentration of over 4 mg/l, consumed for many years, may cause stiffness and pain in the joints and skeletal deformities, particularly in hot climate where people drink more water, where concentration tend to be

increased by evaporation of stored water and where peoples diets may also be rich in fluoride or nutritionally deficient.

Nitrate concentrations of over 45 mg/l in drinking water can cause a serious blood condition in infants known as methaemoglobinaemia (infantile cyanosis), particularly if their diet is not rich in vitamin C. The high concentration of nitrate may also cause gastric cancer.

2.2.3 Advantages of water supply to a rural community: Illusion or reality ?

The links between water quality and diseases have been discussed in the preceding sections. It is worth noting that about 80 % of diseases in the world are water related. The developing nations suffer 96 % of all infant mortality (less than five years of age) and the majority of these are connected with inadequate water supply[9]. In Nepal, poor water supply and sanitation are among the leading causes of diseases such as Diarrhoea, Dysentery, Hepatitis, Gastro-enteritis and parasitic infections. Children are the main victims: the diarrhoeal episodes per child per year was estimated at 6.2 in 1988. The death ratio for children aged one to five is 35 per 1000 and 46 % of these are associated with diarrhoea. In 1988, Nepal has the highest under-five mortality rate in Asia with the exception of Afghanistan and the lowest life expectancy for women[2]. It is obvious that the provision of a reliable and clean water supply is an essential element in

Fig. 2.2 Traditionally used water source.

Note the hygiene condition.

(Jumla, Nepal)

improving the quality of life for the rural population in the developing world. In addition to the direct health benefits, it is anticipated that improved access to water would greatly ease the burden for women who carry water over long distances. It would also release labour for greater input into agriculture & cottage industries, or time saved could be used for child care , voluntary works or leisure. Therefore the women and the children are the one who benefit the most from an improved water supply system.

It is too simplistic or even illusory to think that a rural community will naturally enjoy all the benefits so forcefully argued in favour of safe & adequate water within easy access. Though the water supply is an essential element, the actual realization of these goals depends equally on the socio-cultural values & beliefs, hygiene awareness, economic level, and attitude of the consumers. These issues are much more difficult and complex to deal with than the technical design and construction of a water supply scheme, especially when the population is uneducated, backward, and at an under-developed state.

The potential health benefits of a clean water supply frequently fail to be realised because infectious and parasitic diseases continue to be transmitted by routes which remain unaffected[3], for example:

+ Old or polluted sources of water may continue to be used for drinking purposes for reasons of preference or convenience. Presence of some chemicals may lead to an unpleasant taste or appearance and this may cause people to abandon a source of good microbiological quality in favour of a source of poor microbiological quality.

+ The water from improved supplies may be contaminated between point of

delivery and the point of ingestion - in carrying vessel, storage vessels, drinking vessels, and handling.

+ Water , though made more accessible, may not be optimally used in personal and domestic hygiene. The hygiene habits may have presumably remained unchanged. A particular focus may need to be given to hand washing.
  It has been reported that quantity of water used in a rural community increases up to 5-13 litre per day when the water is available within 1.6 km. Thereafter usage does not significantly increase until a tap is provided within each house when per capita consumption rise to 30-100 litres per day[9].

+ Waste disposal methods and environmental sanitation may not be improved. There might not be a change in the habit of open and random defecation. Woman who carefully prepares a meal for her family, may send her child to defecate just beyond the doorstep.

In the absence of an integrated effort to improve the overall quality and general standard of rural life, a water supply scheme in isolation may prove to be undesirable or disadvantageous . For example, women may be replaced by children as water-carriers, a new water supply scheme may interfere with the social life of women by depriving them from opportunities for socialising and communicating with other women, if proper drainage facilities were not provided it may increase the possibility of water-based and/or water related vector diseases etc.

Example of harmful effects of water supply may be worth noting at this point. It is reported that in Burkina Faso, the irrigation of 1200 ha. of rice fields in the plain of Loumana created conditions which induced an outbreak of river blindness (Onchocerciasis) resulting in desertation of the settlement and the dilapidation of the system five years later (1957-1962) after 15 % of women and 20 % of men became blind[10].
It has been reported that in the Ryukyu Island water tap was provided along with a towel on which children wipe their hands. This was found responsible in spreading Trachoma among the children[15].

To achieve the anticipated goals, therefore, a rural water supply programme should be developed as an integral part of the

comprehensive rural development programme. That means it should be closely integrated or actively co-ordinated with health education, sanitation, hygiene awareness etc.[11,12,13] in the immediate sense, and with the standard of rural life in broader sense. It should be noted that the effects of a successful programme do not confine themselves to the project locality, they spread spontaneously into other areas through induced diffusion.

2.3 SUSTAINABILITY OF WATER SUPPLY SCHEME:

Sustainability refers to the maintenance or enhancement of resource productivity on a long-term basis. If put in the simplest way a sustainable development has to satisfy two main criteria: the first is that the development should at least not degrade & exhaust , if it does not enhance, the natural resource being harnessed or the other natural resources which are inter-linked with the one being utilised and on which peoples' long-term livelihood is based. The second is that the type of development pursued (e.g. technology used) should be compatible with the socio-economic conditions and readily acceptable to the people so that it becomes a natural part of their way of life, sustain in the society, and improves with the socio-economic development. Sustainability is, therefore, closely linked with the long-term self-reliance. The self-reliance (and hence sustainability) in one sphere such as water supply cannot be

expected without a comprehensive framework of local socio-economic development to make self-reliance possible in many spheres.

Often it is the human health and economic aspect of the project that gets the most consideration in a new water supply scheme. When a technician or an administrator responsible for a water supply scheme works for a rural community he/she does not usually have any clear objective in mind other than to have 'safe' water flowing out of the end of a pipe. Thus the role of the socio-economic situation in the long-term viability of the scheme is often neglected. Such a narrow engineering approach may work in areas with well-developed supporting infrastructures, where people are economically better-off, generally educated and are more hygiene conscious. In contrast, if a rural water supply scheme is to yield the expected benefits and be sustainable, then it must include, besides engineering, consideration of many issues of socio-economic, cultural institutional, administrative and also political nature. In fact it should be seen as an integral part of the overall development programme for a rural community. Therefore making a community water supply sustainable requires interrelated evaluation of the technical, social, and organizational dimensions right from the inception of the project.

It is therefore worthwhile to discuss the sustainability of a rural water supply scheme in terms of appropriate technology, community participation, operation and maintenance, institutional

development, and other parameters which together affect the long-term viability of the scheme.

2.3.1 Appropriate Technology:

A very careful evaluation and analysis of the prevailing situation is needed in order to have a scheme appropriate to rural circumstances (e.g. economic level, socio-cultural aspects, local skills and manpower etc.) and for it to be long-lasting and acceptable to the rural population. The selection and use of appropriate technology is therefore vital for the long-term success of a water supply scheme. A technology is appropriate if it fits with local circumstances; specifically it must be[6,9]

a) appropriate in terms of cost in order that it is affordable.
b) appropriate in performance so that it does the job required.
c) simple so that it can be operated and maintained with local skill.
d) acceptable to the community from socio-cultural view point.

The rural water supply schemes are usually financed by outside agencies, either governmental or non-governmental. Therefore , from a villager's point of view it is not the total cost that matters, it is that part of the cost which the villagers have to pay themselves, which plays a vital role in the long-term acceptance & sustainability of the scheme. For example, the Bamboo tube well[9] got wide acceptance in Bihar, India because people were able to pay the cost of the tube-well; whereas in the hills of Nepal PVC pipes are widely used because they are sponsored and the maintenance cost, which is the responsibility

of villagers, is minimal.

In a rural community, technical knowledge and skills available are at a low level. This technical ability of the local, rural inhabitant should be kept in mind when designing a water supply system and the system should be simple so that the tasks are not too difficult. The simpler the system and the more closely it takes socio-economic & cultural realities into account, the more likely it is that it will be widely accepted.

The components of a water supply system in the rural areas are subjected to harsh operating condition and therefore it is preferable, if possible, to have robust and self-contained system components that will not require regular skilled maintenance e.g. use of ferro-cement storage tanks.

Another important aspect is the use of indigenous techniques and materials, and maximum use of in-country industries and services. The use of indigenous techniques have advantages even when they appear less efficient and more costly; they use local labour and materials which make repair and maintenance easier for local craftsmen, and as such the demand for skilled manpower and spare parts will be less.

As the whole range of social, economic, cultural as well as technical aspects are involved, aims and objectives of rural water supply have to be clearly defined taking these aspects into account. Obviously, therefore, service levels desired in a rural water supply schemes are significantly different than those for urban water supply. The Intermediate Technological Development

Group (ITDG) has suggested the goals and objectives (table 2.3) for water supply improvement in rural areas of developing nations. Table 2.4 illustrates the criteria of appropriateness, suggested by ITDG, against which a water supply project could be judged.

Table 2.3 Goals and objective of water supply improvements in rural areas of developing countries.

Immediate objectives	Further goals stage I[1]	Further goals stage II[2]	Further goals stage III[3]
Functional: to improve the quality, quantity availability & reliability of the supply. **Others:** to carry out this improvement in a manner which a) secures the support of users. b) conserve scarce resources (eg capital). c) avoid adverse environmental consequences (eg lowering water table, encouraging mosquitoes).	**Health:** to reduce incidence of water-borne & water-based diseases. **Energy/Time (Economic):** to save time & energy expended in carrying water. **Social:** to arouse interest in the further health & economic benefits which may arise from the water supply **Economic:** to provide more water for livestock and garden irrigation (water may be used for this even if it is intended solely for domestic water supply).	**Health:** to reduce incidence of water-washed infections (input required: improved hygiene health education improved sanitation) **Social/Technical:** to ensure good long-term maintenance of water supply and sanitation facilities (inputs required: training, clear allocation of responsibility, build-up of local maintenance organization). **Economic:** to use energy/time savings and increased water available to achieve better agricultural output (input required: extension work, fertilizer, supply etc.)	to achieve the the greater well-being of the people through: a) social change greater self-reliance in the community, better organization, better deal for the poor, women etc. b) improved standard of living, health, nutrition, income, leisure.

1. These follow as consequences when the immediate objective have been met.
2. These follows from previous stages if complementary inputs provided.
3. These are consequences of reaching the previous goals which follow if there are also inputs on many other fronts.

[Source: ITDG : 1978]

Table 2.4 Criteria of appropriateness against which a water supply project should be judged.

Criteria derived from immediate objectives	Criteria derived from stage I goals	Criteria derived from stage II goals
1. Criteria of technical appropriateness:		
Functional appropriateness (fitness for purpose)	Health & sanitary appropriateness (water-borne diseases data & water quality)	Health & sanitary appropriateness (water-washed diseases data and water quality and availability)
Environmental appropriateness, fitness for hydrological conditions, avoidance of environmental damage.		
2. Criteria of social appropriateness:		
Community appropriateness (felt needs and stated preferences in community, scale in relation to community size & organization.	Consumer appropriateness (changes in water carrying and in water use patterns)	Maintenances appropriateness (organization, administration, village/government responsibilities, spare parts supply training, record-keeping.
Work appropriateness (organization of labour force)	Educational appropriateness (degree of interest created in health, hygiene & other development)	
3. Criteria of economic appropriateness:		
Resource utilization appropriateness (capital & labour intensity, import bill, fuel consumption, scale economies.)		Production appropriateness (amount of time/ energy saving and volume of water available for productive purpose.

[Source: ITDG : 1978]

The WHO water quality standard for small-scale community is criticised as inappropriate[14], especially the criteria which states that water should contain less than one Escherichia or 10.0 total coliform per 100.0 ml. If implemented , this standard

would condemn most rural water supplies in the developing world. Simple design criteria can be applied to improve the quality of existing supplies rural areas. These criteria can be upgraded as the socio-economic standard of rural life enhances.

2.3.2 Community Participation:

The highest potential for sustainability is achieved when the community is involved in all phases of the project, starting from the planning stage. If the scheme is to continue to operate satisfactorily, villagers have to recognise the need for improved service, be able and willing to maintain the system, and also be able and willing to pay for the maintenance. The importance and need for community participation in local level development is well recognised by those involved in this field. But achieving it is a complicated task which involves local level social and political issues[8], for example factionalism, conflicts generated by local political interests, traditional power structure and influence, and many more.

The realization by the people that a scheme is necessary (i.e. felt-need) and their willingness to help themselves (i.e.self-help) are two basic elements of community participation and long-term sustainability. Community participation is too often understood in the narrow sense of obtaining voluntary labour or cash contribution, to an already selected and designed scheme, and is understood as a means of saving the government resources through community contribution.

Fig. 2.3 A common tap built under a community water supply project. (East Nepal)

The villagers may provide voluntary labour in implementing such a scheme, but it can hardly be called community participation and hardly ensures sustainability unless the community is involved in decision-making[3]. Thus when an outside agency remains in total control of the process and merely calls upon the beneficiaries to give their labours directly, one can not speak of real community participation though there is an element of self-help labour.

Every community (in fact every individual family) has its own sense of priorities as to what needs to be done and as to acceptable ways of doing it. A community may show little response to a project if it has higher priorities. A community may reject a contribution towards a water supply scheme not because they are against the scheme but because they think they can use their resources to better advantage in building a road or a school or an irrigation scheme or a local bridge etc.[15]. The outsider should not assume that they know what the villagers want, but undertake a persistent and patient exercise of interacting with them again and again. It is necessary to identify and put the peoples' priorities first. The greatest need appears to be for an open approach to be adopted to dialogue with the community to identify the community's needs and priorities. Intensive consultation and interaction with the community members is probably the best way of identifying people's priorities and needs. If an external agency intends to contribute in local level development, therefore, the agency should go to the community with an open

mind, not with predetermined schemes and programmes, but with determination to help the community in their own priorities and felt-need[16]. Such a process of social consultations and interactions provides opportunities for the community and outside agency to assess[16]

+ the community's current preferences
+ the community's current ability to meet their perceived needs
+ the capability of the community to adapt to new facilities
+ the likelihood of maintaining the system by the community in the face of changing preferences over time.

A great deal of patience, commitment and cautious effort is needed to avoid any source of bias or to avoid misreading the people's intentions while assessing the community's needs and prospects of community involvement. In assessing the felt-need it is usual practice to compare one village to another in terms of their receptive character and willingness to share the cost. If this remains the sole criteria for extending a contribution from outside agency toward a scheme, then there is a danger that more backward communities are likely to be left even farther behind, since those which are already better-off have more capability to pay, likely to be more receptive, and organise the things better. A real felt-need is more important than mere monetary or labour contribution from the community. Therefore, a positive discrimination in favour of the backward may be necessary, and subsidies, for example food-for-work, may some time be necessary or desirable. But such subsidies may diminish the relevance or sustainability of a programme[16]. People who are paid in food or cash are prepared to undertake work in which they have neither

interest nor faith, and this might mislead the outsiders into thinking that people want what they really do not.

2.3.3 Operation and Maintenance:

Only a properly functioning scheme can discharge the intended benefits. The study of 300 rural water supply schemes in Nepal and Bolivia revealed that about 50 % of the newly constructed projects had operating problems within days and 6 months of completion[17]. There are many reasons, why the rural water supply schemes in the third world are failing at a fast rate, malfunctioning or are being rejected by the people. Many of these situations could be avoided by use of appropriate technology and community participation. The shabby state of the schemes are related to the aspects of inappropriate design, local skill and manpower, lack of funds for regular maintenance, lack of a sense of responsibility or responsible institution to look after the system, negligence of socio-cultural aspects in the design process of the system etc. The diversity of causes for the break down, or malfunctioning, or rejection of a scheme may be illustrated by some typical situations such as;

+ Damage can result from unfamiliarity of the users with the facilities, e.g. taps, unless an explanation of their operation is given.

+ If the scheme fails to meet the needs of parts of the population,there is the danger of deliberate damage by those excluded.

+ Inappropriate design, e.g. low flow which might lead to queuing and sometimes causing people to prefer polluted sources or to damage the facilities in the effort to get water.

+ Use inappropriate technology, e.g. if the scheme involves treatment of water with chemical, like chloride, it is most likely that people will abandon treatment (chlorination) when the chemical stock runs out. Similarly when slow sand filter is used, a by-pass is often considered an easy way out if the slow sand filter become clogged.

+ Unless it is very clear who is responsible to undertake preventive maintenance, to organise a work party to re-bury an exposed pipe, or to take initiative in carrying out a repair, it is very likely that these tasks will remain undone, at least until the supply breaks completely.

+ The system used may be too complicated for the local skill and technical knowledge to carry out minor repair & maintenance works, and thus the minor repair problems may become the major one.

+ Lack of a local level institution which collects funds for regular minor maintenance and can contact external agency for help in need of major repair.

+ Inappropriate selection of water source may lead to the rejection of the scheme, for example when the water at a source contains some chemical (e.g. chlorides, sulphates) which cause an unpleasant taste or appearance, this may cause people to abandon water from these sources, and use other source which is more likely to be bacteriologically polluted.

Experiences have shown that village participation in construction (through contributing cash and labour) has been successful in many cases, but village participation in operation and maintenance has often been minimal. The system will quite often become sustainable through secure and exclusive family or household rights to resources, for example private taps, exclusively for family are better cared for than common taps.

A good O & M programme takes into account human resources development, appropriate technology, willingness and capability of recipient of the water supply system to financially support the system as well as logistics and the availability of spare parts & supply. O & M can not be considered as simply an

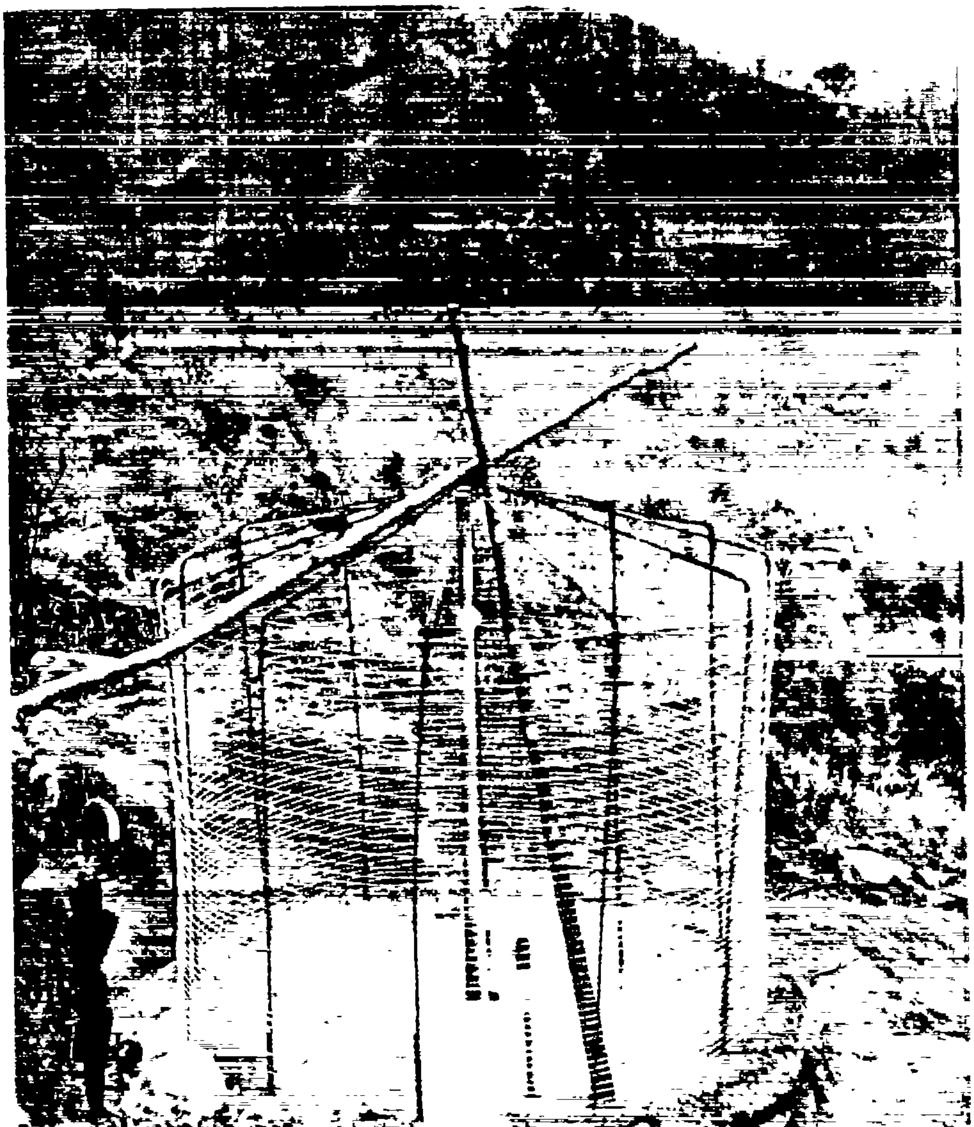


Fig. 2.4 A ferrocement tank under construction, East Nepal.

engineering exercise or a funding question. AT the planning and technical design phase , the maintenance requirements should be considered in a clear and comprehensive manner.

2.3.4 Institutional Development:

It is essential to have an organization responsible at the local level to keep the system functioning. In the cities water supply systems are looked after by specialised organizations, the consumers are not responsible for operation and maintenance. The urban population seldom have an alternative water source, and are mostly aware of the general health benefits of water supply services. Therefore in urban areas inhabitants have usually accepted the principle or at least are familiar with the requirement that they must pay for the water supply.

To keep the system functioning, one of the major issues that has to be settled from the beginning is: What are the expenses incurred for running the system and how will they be met? In the cities water supply agencies are responsible for financial management, the collection of water charges, and operation and maintenance of the system. Therefore in the cities there is no confusion, whatsoever, regarding the responsibility of ensuring safe delivery of water to the consumers.

Too often the situation of villages are different. Typically the rural communities lack formal organization, of the type available in the cities, which look after the essential facilities.

Administratively and financially it is too big a burden to the government to look after all the small scale village water supply schemes. The problem becomes more severe for the government to manage these systems when the villages are in remote places where transportation and communication are very difficult, eg. in the hills of Nepal. Therefore the projects have virtually no chance of long-term success if the villages are not responsible for managing them. Therefore the completed projects are handed over to the community and it is hoped that they will be taken care of by the community. The villagers are asked to maintain and operate their systems themselves. But individual members of a community are likely to value the project less important in relation to their own properties. In absence of an institution with clear responsibility to look after, maintain and run the community project at local level, it is likely that the minor maintenance problems will become major ones and the system will break down or stop functioning properly. Managing regular expense accrued in operation and maintenance of a rural water supply project is an important aspect of sustainability. The rural people frequently have a choice between alternative water sources, and they have developed their own criteria to chose between them. In rural areas water is usually seen as a free commodity like air. Health consideration plays a minor role, and the principle of paying for water is usually not widely accepted.

The importance of institution development at local level to look after the system is obvious from the above discussion. In

addition to the management of the water supply system, the local organization may be useful in co-ordinating other development activities at local level, e.g. health education, sanitation programme, child care etc. so that full advantage of water supply scheme could be achieved. In fact an organised community carries out activities rather than merely contributing to it.

The skills available at local level may not be sufficient to take full responsibility of managing the system. In such situations the responsibility should be transferred gradually. Training of the community members for repair, maintenance and operation of the system is essential. Conducting such training and extending other types of support the community might need to manage the system may be organised if an effective local level institution exists.

The external water agency, usually the government agency at regional or district level, is supposed to provide support to the community in major repair and maintenance works as and when needed. But in many cases the village schemes fails due to bureaucratic delays in response to the request for support from the community. Therefore bureaucratic reform of the existing governmental or non-governmental agencies to obtain an improved relation between local and external organization, to establish an effective line of communication, and cooperation between them is as important as establishing community level institution.

2.3.5 Broader Perspective:

It has been shown in the preceding discussion that, though important, technical design and construction of a rural water supply scheme does not necessarily yield the expected benefits to the community. To achieve any degree of success effective integration of water supply scheme at least with sanitation and a hygiene awareness programme is vital. Any attempt to deal with water supply schemes in isolation is most likely going to be futile. Full advantage of a water supply scheme can not be realised unless such a scheme becomes a part of a comprehensive integrated rural development programme (IRDP) designed to enhance quality and standard of rural life. An integrated rural development programme if designed, implemented and managed effectively, may be expected to increase local organizational capacity and level of consciousness, to make local people self-reliant, and to encourage better distribution of wealth, and hence increase the probability of success and sustainability of the overall programme.

Obviously a comprehensive IRDP has to take every aspect of rural life into account, not just a water supply scheme. At local level it has to consider social, cultural, traditional, and economic aspects of villagers. At a broader perspective , the IRDPs become a part of national development policy, and therefore become in part a question of national politics: whether the government represents or is genuinely committed to the interest of the poor

majority of the population? A very strong and real commitment for the rural development is necessary at the national political level. Therefore in national context IRDP is concerned directly with democracy, employment, income distribution, devolution of powers etc.

Nepal's experiences in this regard are not encouraging. In the past every one talked about rural development, it has become a fashion to some extent. The rhetoric of rural development is generally approved, but the real commitment of the government can be judged from the low budget allocation in this sector. The technical ministries and the departments have generally shown little interest and enthusiasm for small scale local development. There may be many reasons for that, such as the policy pursued by the government, budgetary aspects, political interference and control,complicated administrative / bureaucratic procedures etc. Consequently government department staffs are usually found not to have closeness and empathy to the villages. The government organizations were considered as a socially desirable mass employer. As a consequence all most all departments are now overstaffed and underpaid.

There is a need of big shake up at national political level. In fact complete reorientation of national development policies, programmes and strategies, and administrative reforms are pre-requisite to the success of rural development programmes

including water supply schemes. In other words complete rethinking about the type and way of development is necessary. Considering the recent (1990) change in the political system, this may be the right time to bring about such changes.

Conceptually it is advisable to keep the government organization small, effective, and manageable. At national and regional level, government role should be to formulate broad policy for development, and to take responsibility of providing national and regional infrastructures, e.g. establishing transportation network. The policy of involving non-government organizations (NGOs) actively at local level development in partnership with local people is expected to relieve pressure and burden from the government. The NGOs, of which there are many, are generally found to be better in understanding and representing the point of view of rural people. They are free from the administrative and bureaucratic hassle of the government organization, and their staffs are generally found to be more sensitive to, and determined in working with and for poor. A NGO working in a limited geographical area may be in a very good position to build up knowledge of the needs and wishes of various sections of the population, and to implement a development policy which is based on local decision. Of course to achieve better result in such a situation, the NGOs should have complete freedom to work within the policy guide lines, without interference from the government department and / or politicians. It is likely that efforts which

improve the position of weaker sections of population may be opposed and frustrated by local power structures, because they might see this as a challenge to their traditional influence. The government department, local politician, social worker, and the NGOs should remain in close contact and in good communication to overcome such difficulties. Thus government department's role at the local level may be to monitor, without interference, the performance of the NGOs ensuring that the project undertaken are of the type which benefits the poor, to encourage the coordination among various parties involved and to help resolve any conflict of views.

CHAPTER 3

3. Review of Pipe Flow Hydraulics:

3.1 INTRODUCTION:

The object of this chapter is to review the basic theory of hydraulics as applied to the analysis of pipe flow problems. Effort is made to make the chapter as concise as possible and at the same time comprehensive by bringing relevant information together.Proceeding from these basic theories of hydraulics and information gathered in the first part, the second part of the chapter illustrates the conventional use of the theory to analyze the pipe flow and design of pipelines. The basic hydraulics and the illustration of the conventional method of design together form the background on which the next chapter proceeds in which the optimal approach of pipe networks design will be presented.

3.2 COMPONENTS OF WATER SUPPLY SYSTEM:

A typical water supply system consists of an intake structure, facilities for the storage of water, facilities to transport and distribute the water , facilities to improve and/or alter the quality of water etc (fig 3.1)

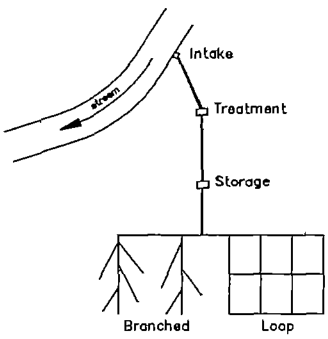


Fig. 3.1

Facilities used to transport water from the source and distribute it to the users consist usually of a system of pipelines. Layout of the system of pipes depends on the source/demand points and the topography of the area. In a relatively flat area many alternative layouts of pipelines are possible but in a hilly region the layout is usually unique and is controlled by topographical condition.

3.3 HYDRAULICS OF PIPE FLOW:

3.3.1 Steady and Unsteady Flow: When the characteristics of flow, such as flow rate, velocity, or pressure remains constant over a finite period of time, the flow is called steady otherwise flow is unsteady.

Uniform and Non-uniform Flow: When the cross -sectional area, depth of flow and hence the mean velocity remain constant from section to section, the flow is called uniform otherwise non-uniform.

3.3.2 Basic Equations:

The basic equations used to analyze the flow in pipes are the continuity equation and Bernoulli's (or Energy) equation.

Continuity Equation:

The equation of continuity expresses the principle of conservation of mass[20]. The flow in pipes is considered incompressible and the continuity equation can be written as:

$$Q = A_1V_1 = A_2V_2 = \text{Constant} \qquad (3.1)$$

Where Q is the flow rate in m^3/s

A is the cross-sectional area of flow in m^2

V is the mean velocity in m/s

Bernoulli's (or Energy Equation):

Flowing water consists of the following types of energies[20,21] :

i. Kinetic energy of movement, known as velocity head, which is usually expressed as $V^2/2g$.

where V = mean velocity (m/s)

and g = acceleration due to gravity.

ii. Pressure Energy (or Elastic energy) which is known as pressure head. This is given by $\frac{p}{\gamma}$ where :

p = hydrostatic pressure of a column of water.

γ = unit weight of water

The energy form of $\frac{p}{\gamma}$ follows from a consideration of a

piston operating in a cylinder[22]. If p is a sustained pressure and v is the swept volume, the pressure energy per unit weight (work done per unit weight) is evidently $\frac{p}{\gamma}\frac{v}{v} = \frac{p}{\gamma}$.

The hydrostatic pressure $p = \gamma h$ where:
h = height of free column of water
Therefore, $\frac{p}{\gamma} = h$ and pressure head is given by depth of

water under free flow condition.

iii. Potential Energy : Which is known as potential head and is given by the height (z) of the position of a point in flowing fluid above some given (or assumed) datum.

iv. Thermal or Internal Energy : Which is known as head loss of head gain. Energy may be transferred to or from the fluid by thermal or mechanical process[20] for example by pumps and losses. Losses are the fraction of energy which has been transferred into forms non-recoverable for the hydraulic system such as energy dissipated as heat or noise.

Bernoulli's equation can be obtained by use of the principle of energy conservation between any two points if the following assumptions are made.

(i) There is no head loss by friction or by other reasons and there is no head gain (energy input) from a pump etc.

(ii) Flow is steady and within a stream-line.

The principle of conservation of energy, applied along a stream-line, says that the sum of potential head(z), pressure head($\frac{p}{\gamma}$) and velocity head ($\frac{v^2}{2g}$) at any points in the stream line remains the same provided no energy is added to (eg.pump) or lost (eg.by friction) from the system. Therefore, between points 1, 2 and 3 (fig3.2):

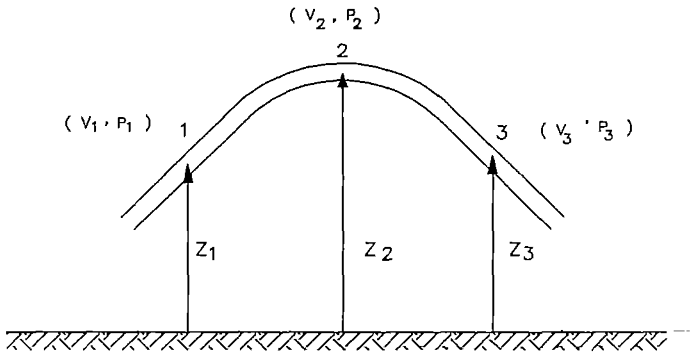


Illustration of Bernoulli's Equation

fig 3.2

$$
\begin{aligned}
z_1 + \frac{p_1}{\gamma} + \frac{v_1^2}{2g} &= z_2 + \frac{p_2}{\gamma} + \frac{v_2^2}{2g} \\
&= z_3 + \frac{p_3}{\gamma} + \frac{v_3^2}{2g} \\
&= constant
\end{aligned}
\qquad (3.2)
$$

Equation (3.2) is known as Bernoulli's equation.

Real pipe flow and use of Bernoulli's Equation:

Discussion on assumption (i)

In practical cases energy is lost by friction known as friction loss, and in expansion/ contraction of pipes, in pipe bends, in pipe valves and fittings etc. which are all known as minor losses. If all of these losses are taken into account then we will get a modified form of Bernoulli's equation:

$$z_1 + \frac{p_1}{\gamma} + \frac{v_1^2}{2g} = z_2 + \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + h_L \qquad (3.3)$$

where h_L = head (energy) loss due to friction etc. while flowing from point 1 to point 2 (fig 3.2). If the energy input (such as that by a pump) is considered, then:

$$z_1 + \frac{p_1}{\gamma} + \frac{v_1^2}{2g} + h_p = z_2 + \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + h_L \qquad (3.4)$$

where h_p = Energy (head) input by a pump.

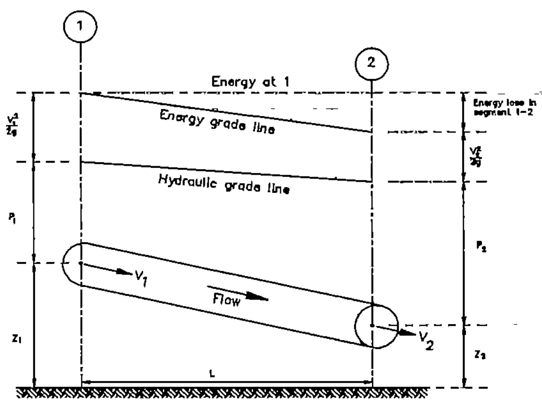
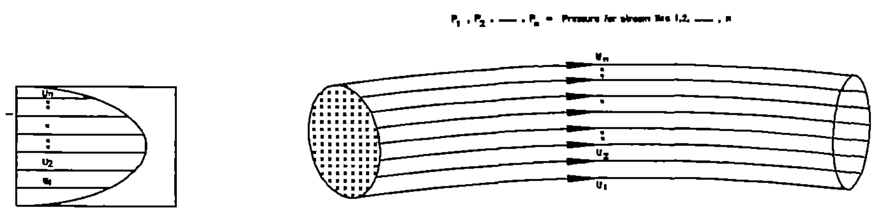


Fig. 3.3 Flow in Pipes and Various Heads

Discussion on Assumption (ii)

As discussed earlier, Bernoulli's equation applies to any stream-line flow which is at a steady state. However, the flow within a real pipe is not uniform[2]. The velocity of water at the boundary is zero and is maximum at the centre. The flow can be visualised as an infinite number of stream-lines.



(a) Velocity Profile within a pipe

(b) Infinite stream lines

fig.3.4

Each stream line has its own velocity head, pressure head and potential head. Bernoulli's equations in the form of equation (3.2) to (3.4) are not strictly true because they use mean velocity, v to compute velocity head and pressure head is compared to the centre line of the pipe. Thatis

$$\frac{v^2}{2g} = \sum \left(\frac{u_1^2}{2g} + \frac{u_2^2}{2g} + \ldots\ldots\ldots + \frac{u_n^2}{2g} \right)$$

$$\frac{p}{\gamma} = \sum \left(\frac{p_1}{\gamma} + \frac{p_2}{\gamma} + \ldots\ldots\ldots + \frac{p_n}{\gamma} \right) \quad \text{and}$$

$$z = \sum \left(\frac{z_1 + z_2 + z_3 + \ldots\ldots\ldots + z_n}{n} \right)$$

The ratio of $v^2/2g$ to ($u^2/2g$) is about 0.98, and use of mean velocity introduces very small errors which can be ignored for almost all practical cases. By similar reasoning errors introduced by taking $\frac{p}{\gamma}$ at the centre line can be ignored and

$z = \frac{\sum z_n}{n}$ introduces no error so long as the pipe is flowing full[21].

3.3.3 Laminar and Turbulent Flow

A flow is called laminar (or viscous) if all the fluid particles proceed along parallel paths and there is no traverse component of velocity. That is, fluid may be considered to be flowing in discrete layers with no mixing taking place[22,23].

In turbulent flow, motion of individual fluid particle are not well defined as in laminar flow. The individual particles are subject to fluctuating traverse velocities, with low velocities and haphazard interchanges of positions. The motion of a fluid particle within a turbulent flow is complex and irregular. A turbulent flow is associated with a relatively high velocity.

A flow in which some degree of unsteadiness becomes apparent may comprise a short 'burst' of turbulence embedded in a laminar flow.

The type of flow can be predicted by a dimensionless parameter

known as Reynolds number(Re), which is defined as the ratio of inertia force to viscous force or mathematically,

$$Re = \frac{VD}{\nu}$$

Where V = Velocity

D = Characteristic length,m

(diameter in case of pipe flow)

ν = Kinematic viscosity of fluid

$$\{ = \frac{density}{viscosity}\}$$

Types of flows in commercial pipes can be predicted on the basis of Re as follows[23];

Laminar flow	:	$Re < 2000$
Transitional flow	:	$2000 < Re < 4000$
Turbulent flow	:	$Re > 4000$

In pipelines and open channel hydraulics, the velocities are nearly always sufficiently high to ensure turbulent flow, although a thin laminar layer persists in proximity to a solid boundary[23]. Due to its irregular fluctuating nature, turbulent flow has defied rigorous mathematical treatment, and for the solution of practical problems it is necessary to rely largely on empirical or semi-empirical relationships.

3.3.4 Head Loss, Pipe Flow Equations and Friction Factor

Bernoullis' equation of the form

$$z_1 + \frac{p_1}{\gamma} + \frac{v_1^2}{2g} = z_2 + \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + h_l$$

connects flow at any section of pipe line with the flow at any other section of the same line. The difficulty in using this relation is determination of the head loss term (h_L) accurately. The total head loss in a pipe line basically consists for two types:

(a) Head loss due to friction between the fluid and the boundary i.e. frictional head loss.

(b) Minor losses: due to abrupt changes in pipe size i.e.contraction or expansion, entrance and exit losses, bends, valves, and fittings of all types. In long pipe lines these minor losses can often be neglected but they may be quite important in short pipes. Minor losses in turbulent flow are approximately proportional to the square of the velocities and are usually expressed as a function of velocity heads in the following form.

$$h_{Lm} = k\,\frac{V^2}{2g} \qquad (3.5)$$

where k is the coefficient value given in table 3.1 below.

Frictional Headloss(h_L):

For laminar flow the frictional headloss in the pipe is

(a) Enlargements

[Values of K_L in $h_{Lm} = \frac{(V_1 - V_2)^2}{2g}$]

Θ^*	$D_2 / D_1 = 3$	$D_2 / D_1 = 1.5$
10	0.17	0.17
20	0.40	0.40
45	0.86	1.06
60	1.02	1.21
90	1.06	1.14
120	1.04	1.07
180	1.00	1.00

* The angle Θ is the angle in degrees between the sides of the tapering section.

(b) Abrupt contractions

[Values of K_L in $h_{Lm} = K_L \frac{V_2^2}{2g}$]

D_2 / D_1	K_L
0.0	0.5
0.4	0.4
0.6	0.3
0 8	0.1
1.0	0.0

(c) Pipe Entrance From Reservoir

Bellmouth $h_L = 0.04 \frac{V^2}{2g}$

Square Edge $h_L = 0.5 \frac{V^2}{2g}$

(d) Bends

(Values of K_l in $h_{lm} = K_L \frac{V^2}{2g}$, the head loss in excess of that in a straight pipe of equal length)

Radius of bend	Deflection Angle of Bend		
Pipe diameter	90°	45°	22.5°
1	0 50	0.37	0.25
2	0 30	0.22	0 15
4	0.25	0.19	0.12
6	0.15	0.11	0.08
8	0.15	0.11	0.08

(e) Valves and Fittings

(Values of K_L in $h_{Lm} = K_L \frac{V^2}{2g}$)

Globe valve (wide open)	10
Swing check valve (wide open)	2 5
Gate valve (wide open)	0 2
Gate valve (half open)	5.6
Return Bend	2.2
Standard tee	1 8
Standard 90 degree elbow	0.9

Table 3.1

proportional to the velocity and can be calculated by Hagen-Poiseuille's equation which reads:

$$h_{Lf} = \frac{32\mu LV}{\rho g d^2} \tag{3.6}$$

Where: μ = Viscosity

L = Length of pipe within where headloss h_{Lf} occurs

V = Velocity of flow

ρ = Density of fluid

g = Acceleration due to gravity

D = Diameter of pipe

Pipe Flow Equations:

For almost all practical pipe flow cases, velocity is sufficiently high to make the flow turbulent. For turbulent flow frictional headloss is proportional to the square of the velocity $h_{LF} \propto V^2$.

One of the main interests of a pipeline design engineer in the process of design is to compute headloss in the pipeline system accurately. Two tapes of formulae are available for the purpose of headloss computation[21].

(a) Dimensionally correct: mathematically derived and based on experimental results.

(b) Empirical.

Dimensionally Correct Formulae:

Darcy-Weisbach Equation:

The general equation for calculating headloss due to friction in pipe for a turbulent flow is Darcy-Weisbach equation which reads:

$$h_{Lf} = f\frac{Lv^2}{2dg} \tag{3.7}$$

Where f = friction factor which is dimensionless and may be

used with any system of units[23].

For non-circular pipes diameter may be replaced by hydraulic radius[20]:

$$R = \frac{A}{P_w}$$ Where ; R = Hydraulic radius, m

A = Cross-sectional area of flow, m^2

P_w = Wetted perimeter, m

The Friction Factor (f)

Originally it was presumed that the friction factor was constant, but it has subsequently been found that the coefficient of friction (friction factor) depends on the Reynolds number and the relative roughness. The relative roughness (K_s/D) is defined as the ratio of the effective roughnes (K_s) to the pipe diameter(D).

Nikuradse's Experimental Results:

In the 30s, Nikuradse produced a set of experimental results of f and Re for a range of relative roughness (K_s/D) of 1/30 to 1/1014. He plotted his results as log f against log Re for each value of K_s/D as shown in fig. 3.5.

The plot shows that there are five regions of flow which are:

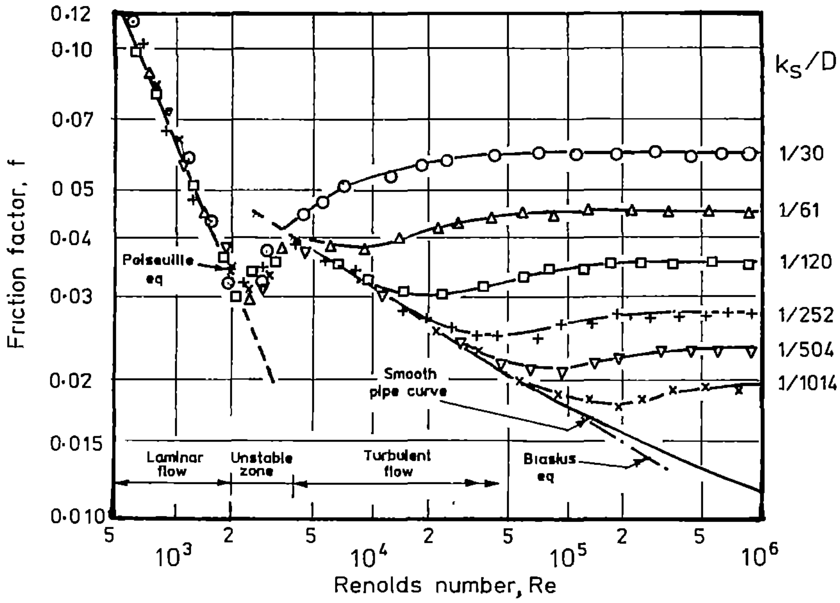


Figure 3.5. Nikuradse's f-Re curves for artificially roughened pipes.
(plotted points shown diagrammatically only)

(i) **Laminar Flow:** The region in which the relative roughness has no influence on the friction factor. Equating Hagen-Poiseuille,s equation to Darcy-Weisbach's equation we get,

$$f = \frac{64}{Re} \qquad (3.8)$$

Here, the Darcy-Weisbach equation may also be used for laminar flow provided that 'f' is equated by equation (3.8)

(ii) Transition From Laminar to Turbulent Flow:
An unstable region between Re = 2000 to 4000. Pipe flow normally lies outside this region.(iii) Turbulent Flow: Three distinctive zones were found;

(a) Smooth Turbulence: Zone of hydraulic smooth pipes, where f depends on the Re only - friction factor in this region can be calculated by Karman-Prandtl's equation which reads as:

$$\frac{1}{\sqrt{f}} = 2\log\frac{Re\sqrt{f}}{2.51} \qquad (3.9)$$

(b) Rough Turbulence: The zone of hydraulic rough pipes where 'f' can be obtained by another formula proposed by Karman -Prandtle which reads as :

$$\frac{1}{\sqrt{f}} = 2\log\frac{3.71D}{K_s} \qquad (3.10)$$

(c) Transitional Turbulence: Zone where 'f' depends on both Re and relative roughness(K_s/D). The friction factor in this region

can be computed by Colebrook-White's formula which reads as:

$$\frac{1}{\sqrt{f}} = 2\log\left[\frac{2.51}{Re\sqrt{f}} + \frac{k_s}{3.71D}\right] \qquad (3.11)$$

Flow in sewers and water supply pipes usually falls in this region of turbulence flow[1]. Substituting S for the slope of energy grade line, $S = h_L/L$ and combining Darcy-Weisbach's equation (3.7) and Colebrook-White formula (5.11) and rearranging, we get:

$$v = -2\log\left(\frac{2.51\nu}{D\sqrt{2gDs}} + \frac{k}{3.71D}\right)\sqrt{2gDs} \qquad (3.12)$$

as $Q = AV$

or, $$Q = \frac{\pi D^2}{4}\left[-2\log\left(\frac{2.51\nu}{D\sqrt{2gDs}} + \frac{k}{3.71D}\right)\sqrt{2gDs}\right] \qquad (3.13)$$

In practical design any two of these variables (Q,D,and S) are known. In the case of a new pipe line design required discharge (Q), and the available head, are known and the requisite diameter must be found. This will involve trial and error procedure. The use of charts (or tables), e.g. the charts produced by Hydraulic Research, make the work easier. In the case of the existing pipeline, the diameter and available head are known and hence discharge may be found directly from (3.13). In the case of the analysis of a pipe network, the required discharges and pipe diameters are known and the headloss must be computed, which may be done by explicit formula for 'f' or by the use of charts.

The first attempt to make engineering calculations easier was made by Colebrook-White's formula (3.11) for commercial pipes. The diagram known as the Moody diagram is shown in fig 3.6. The use of the Moody diagram involves trial and error process. For example, if D, Ks and available head (H) and the length of pipe is known, then solution by use of the Moody diagram involves the following steps:

(i) Calculate Ks/d

(ii) Guess a value for V

(iii) Calculate Re

(iv) Estimate 'f' using the Moody diagram

(v) Calculate h_{Lf} by Darcy-Weisbach's equation

(vi) Compare h_{Lf} with available head (H)

(vii) If H is not equal to h_{Lf}, then repeat from step(ii).

Where as if Q instead of D were known and D is to be determined then D has to be guessed, and V, Re, f etc. to be computed, as above.

Moody has also presented an explicit formula for 'f'

$$f = 0.0055\,[1+(\frac{200K_S}{D}+\frac{10^6}{Re})^{\frac{1}{3}}] \qquad (3.14)$$

which gives f correct to $\pm$ 5% for $4x10^3 < Re < 1x10^7$
and for Ks/D < 0.01

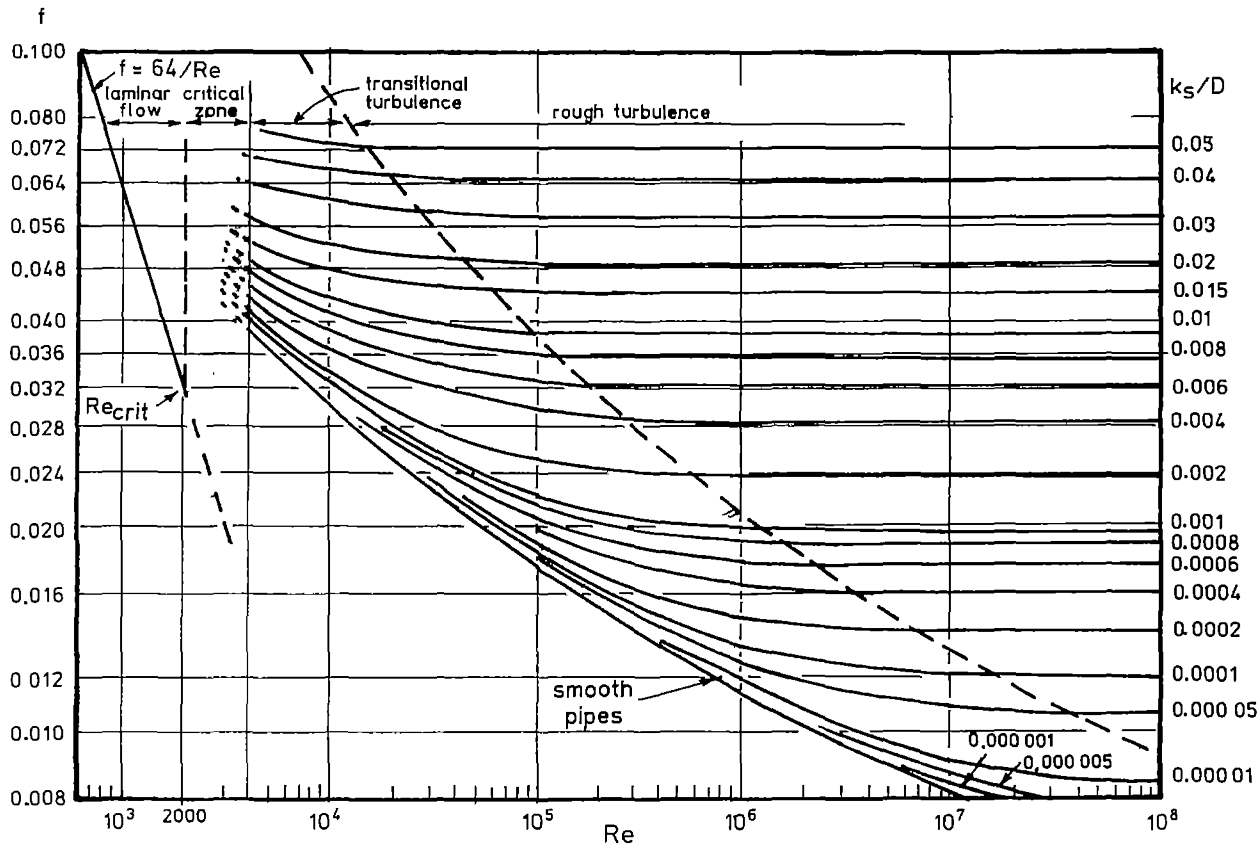


Fig 3.6. The Moody diagram

Barr has presented another explicit formula for 'f':

$$\frac{1}{\sqrt{f}} = -2\log\left(\frac{K_s}{3.7D} + \frac{5.1286}{Re^{0.89}}\right) \qquad (3.15)$$

for Re > 10^5 this provides solution for f to an accuracy of ± 1%.

Another explicit formula was proposed by Jain A K and Swamee P K[24]:

$$\frac{1}{\sqrt{f}} = 1.14 - 2\log\left(\frac{Ks}{D} + \frac{21.25}{Re^{0.9}}\right) \qquad (3.16)$$

The K_s value in the above equations (3.10 to 3.16) represents surface roughness of pipe material. Two factors should be borne in mind when selecting a roughness value:

(a) The surface roughness is likely to increase during operating life due to corrosion encrustation and deposition of solids.

(b) The roughness of the entire pipe system is higher than the roughness of a single pipe, due to joints, fittings and various other discontinuities which disrupt the flow patterns.

Recommended values for K_s taking those factors into account are given in table 3.2.

Table: 3.2

Recommended Values of Ks for Water Supply pipes (taken from ref 20)

	Ks in mm
Straight main	
Steel or cast iron pipe with bitumen or cement coating	0.1
Prestressed concrete or asbestos cement pipes ...	0.1
Mainly straight main	
Same material as above. Additionally steel and cast iron pipes without coating if no sedimentation occurs	0.4
Design of new networks	
The value includes the impact of dense looping ..	1.0

Empirical Equations:

The use of the Colebrook-White transition formula with the aid of charts or tables is not so difficult in the case of single pipelines. However for pipes in series or parallel or for the more general case of pipe network, it rapidly becomes impossible to use for hand calculations. For this reason simple empirical formulae are still in common use[23].

Hazen-Williams Formula:

The formula is reasonably accurate over a range of pipe sizes and flows (and have Re) which are widely experienced in water networks. The formula can be expressed as:

$$v = 0.355CD^{0.63}\left(\frac{h_L}{L}\right)^{0.54} \tag{3.17}$$

or rearranging

$$h_L = \frac{6.78L}{D^{1.165}}\left(\frac{V}{C}\right)^{1.85} \tag{3.18}$$

Where C = a coefficient, the value of which varies from 70 to 150 depending on pipe diameter, material and age. In calculation C is usually assumed constant. In reality, C should change with Re, and caution should be exercised in its use[23]. The values of C which may be taken are as set out in fig 3.7.

Flow rate by Hazen-William formula becomes

$$Q = \frac{\pi D^2}{4}\left[0.355CD^{0.63}\left(\frac{h_L}{L}\right)^{0.54}\right] \tag{3.19}$$
$$i.e. \quad Q = 0.278CD^{2.63}S^{0.54}$$

Where S = h_L/L , slope of energy grade line.

The formula is sufficiently accurate for pipe sizes of 150 mm upward, and for values of C not substantially below 100. It is more accurate for larger diameter of pipes and for flows of the order of 1 m/s[21].

Manning's Formula:

The formula is expressed as

$$V = \frac{1}{n}R^{\frac{2}{3}}S^{\frac{1}{2}} \tag{3.20}$$

Where R = Hydraulic Radius (A/P_w); A = cross sectional area

P_w = wetted perimeter

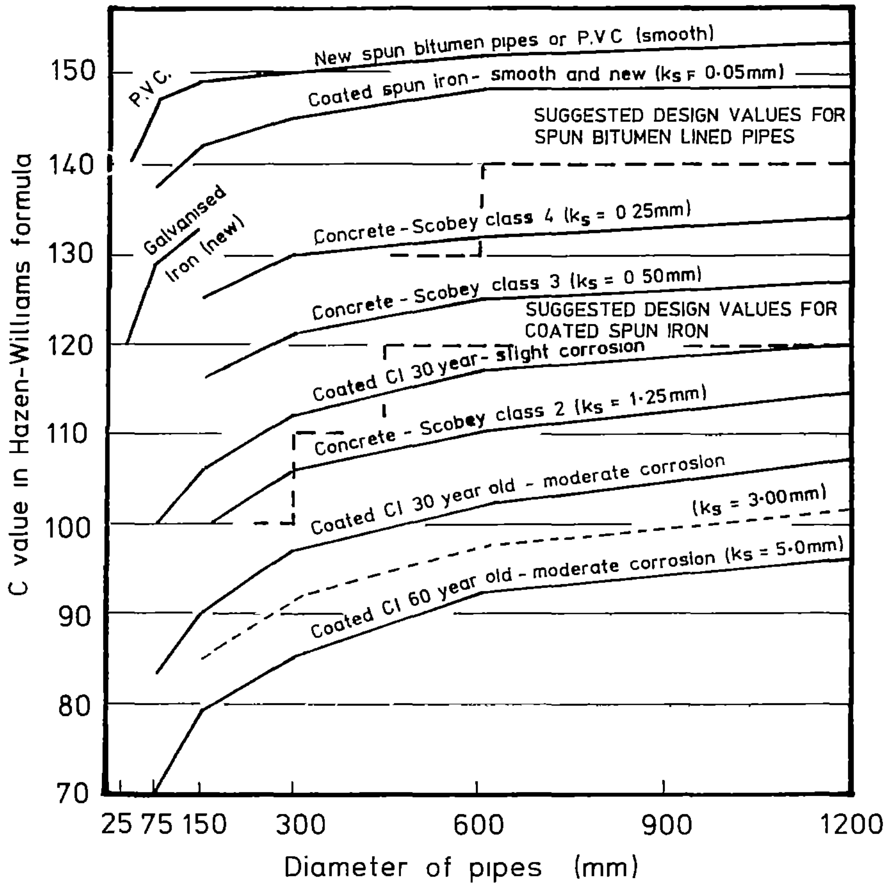


Fig. 3·7 C values in the Hazen-Williams formula and k_s values in the Colebrook-White formula according to P Lamont (WWE, January 1969)

NOTES:-

1) Suggested design values are for a non-aggressive and non-sliming water

2) The curves apply to a 1m/s flow rate For 2m/s reduce C values by 5% below 100, 3% below 130 and 1% below 140 For 0·5m/s increase C values by the same amounts

3) Scobey classes are: class 4 - first class interior finish with all joint irregularities removed, class 3 - good interior finish with joints filled and concrete made on steel forms, class 2 - imperfect interior finish, and as tunnel linings; class 1 - old concrete pipes with mortar not wiped from joints (k_s = 5·00ms)

4) For asbestos cement pipes use values for spun bitumen lined pipes

[Source : Twort et al, 1985]

$S = h_L/L$ slope of hydraulic energy gradeline.

h = roughness coefficient

Equation (3.20) can be written as

$$h_L = n^2 \frac{L}{R^{\frac{4}{3}}} V^2$$

or $h_L = KV^2$ (3.21)

Where $K = n^2 \frac{L}{R^{\frac{4}{3}}}$

Values of n for Mannings equation are given in table 3.3 Manning's formula has the advantage that $h_L \propto V^2$, as roughness coefficient is constant for a given pipe type. Since headloss caused by fittings is also expressed as KV^2, the Manning's formula is conveniently used for pipelines involving many fittings whose effect is appreciable.

The formula is more accurate than Hazen-Williams formula for estimating high flows or flows in old rough-surfaced pipes where C value in the Hazen-Williams formula is well below 100[21].

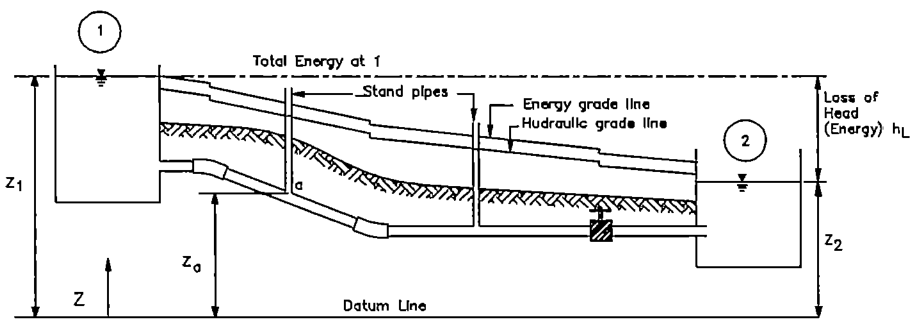
Table : 3.3

Recommended Values of n to be used with the Manning's Equation[1]

Surface	Best	Good	Fair	Bad
Uncoated cast iron pipe	0.012	0.013	0.014	0.015
Coated cast iron pipe	0.011	0.012*	0.013*	-
Vitrified sewer pipe	0.010-0.011	0.013*	0.015	0.017
Common clay drainage tile	0.011	0.012*	0.014*	0.017
Concrete pipe	0.012	0.013	0.015*	0.016
Concrete lined channels	0.012	0.014*	0.016*	0.018
Canals with rough stony beds, weeds on earth bunks	0.025	0.030	0.035*	0.040
Canals with earth bottom, rubble sides	0.028	0.030*	0.033*	0.035
Natural stream channels: very weedy reaches	0.075	0.100	0.125	0.150

* Values commonly used in designing.

3.3.5 Energy and Hydraulic Grade Line:



Hydraulic and Energy Grade Line in a Pipe Line

fig. 3.8

Application of the Bernoulli's equation between (1) and (2) reads:

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_l$$

Where, h_L = total head loss which is summation of frictional head loss (h_{Lf}) and minor losses (h_{Lm}).

If the water is flowing continuously in a pipeline and a standpipe is installed at a point along the pipeline such that water can rise up the standpipe freely, then the term ($\frac{P}{\gamma} + Z$) in the Bernoulli's equation gives the height to which the water rises freely in the standpipe. If all such points are joined together along a pipeline then the line joining the points is called the hydraulic grade line. The line joining the points representing total energy ($\frac{P}{\gamma} + \frac{V^2}{2g} + Z$) along the pipeline gives the energy grade line. Both of these lines have a slope of S (where S = h_{Lf} / L) and minor losses are shown by a step in the fig 3.8.

The location minimum and maximum pressure may be found by finding minimum and maximum heights between the pipe and hydraulic gradient. If the hydraulic gradient is below the pipe, then there is sub atmospheric pressure at that point. This condition is to be avoided since cavitation may occur (if $\frac{P}{\gamma} <$ - 7.o m), and if there are any leaks in the pipeline, matter will be sucked into the pipe, possibly causing pollution of the water supply[23].

3.3.6 Pipes in series; in parallel; and Equivalent pipes:

When a single pipeline consists of different sizes, these pipes are said to be in series (fig. 3.9a).

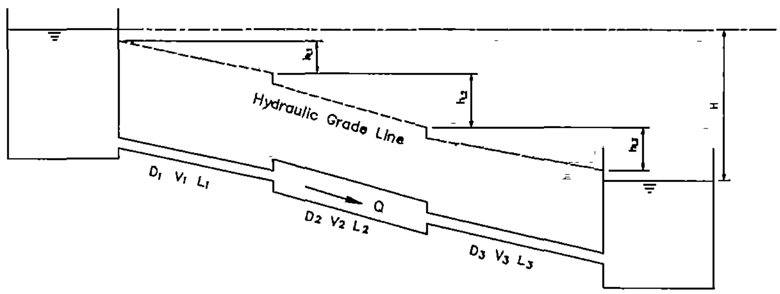


Fig. 3.9 a.

When pipes are in series, frictional head loss (neglecting minor losses) can be written as:

$$h_L = \sum \textit{frictional head losses}$$
$$= f_1 \frac{L_1 V_1^2}{2gD_1} + f_2 \frac{L_2 V_2^2}{2gD_2} + f_3 \frac{L_3 V_3^2}{2gD_3}$$

or $$h_L = \frac{\cancel{16Q}}{2g\pi^2} \left[\frac{f_1 L_1}{D_1^5} + \frac{\cancel{f_2 L_2}}{D_2^5} + \frac{f_3 L_3}{D_3^5} \right] \qquad (3.22)$$

The pipe system can be replaced for analyses purposes by a single equivalent pipe of uniform section with the same head loss. If D_e, L_e, and f_e are equivalent diameter, length and friction factor respectively ,then

$$h_L = \frac{16Q}{2g\pi^2} \left(\frac{f_e L_e}{D_e^5} \right) \qquad (3.23)$$

Equating (3.22) and (3.23)

$$L_e = \frac{D_e^5}{f_e}\left[\frac{f_1 L_1}{D_1^5} + \frac{f_2 L_2}{D_2^5} + \frac{f_3 L_3}{D_3^5}\right] \tag{3.24}$$

When the pipes are in parallel (fig 3.9b)

$$h_L = \frac{f_1 L_1 V_1^2}{2gD_1} = \frac{f_2 L_2 V_2^2}{2gD_2}$$

i.e.,
$$V_1 = \left(\frac{2gD_1 h_L}{f_1 L_1}\right)^{\frac{1}{2}}$$
$$V_2 = \left(\frac{2gD_2 h_L}{f_2 L_2}\right)^{\frac{1}{2}}$$

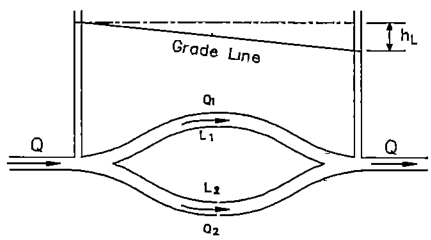


Fig. 3.9b

By continuity equation:

$$Q_1 = \frac{\pi D_1^2}{4} V_1$$
$$= \frac{\pi D_1^2}{4}\left[\frac{2gD_1 h_L}{f_1 L_1}\right]^{\frac{1}{2}}$$

i.e.,
$$Q_1 = \frac{\pi}{4}\sqrt{2g}\left(\frac{D_1^5}{f_1 L_1}\right)^{\frac{1}{2}} h_L^{\frac{1}{2}}$$
$$Q_2 = \frac{\pi}{4}\sqrt{2g}\left(\frac{D_2^5}{f_2 L_2}\right)^{\frac{1}{2}} h_L^{\frac{1}{2}}$$

Now $Q = Q_1 + Q_2$

or $$Q = \frac{\pi}{4}\sqrt{2g}\, h_L^{\frac{1}{2}}\left[\left(\frac{D_1^5}{f_1 L_1}\right)^{\frac{1}{2}} + \left(\frac{D_2^5}{f_2 L_2}\right)^{\frac{1}{2}}\right]$$

As before to find equivalent pipe length ;

$$\left(\frac{D_e^5}{f_e L_e}\right)^{\frac{1}{2}} = \left(\frac{D_1^5}{f_1 L_1}\right)^{\frac{1}{2}} + \left(\frac{D_2^5}{f_2 L_2}\right)^{\frac{1}{2}}$$

or $$\left(\frac{D_e^5}{f_e L_e}\right)^{\frac{1}{2}} = \sum_{i=1}^{n} \left(\frac{D_i^5}{f_i L_i}\right)^{\frac{1}{2}}$$

i.e. $$L_e = \frac{D_e^5}{f_e \left[\sum_{i=1}^{n} \left(\frac{D_i^5}{f_i L_i}\right)^{\frac{1}{2}} \right]^2} \tag{3.25}$$

3.4 CONVENTIONAL DESIGN OF PIPE NETWORK:

3.4.1 Fundamental laws:

Three basic conditions govern the analysis and design of any pipe network;

1. At any node the algebraic sum of flows must be equal to zero i.e. the sum of flows out of the node is equal to sum of flows into the node.

2. The algebraic sum of the hydraulic losses (pressure drops) around any closed loop must be zero.

3. A relationship between the flow rate 'Q' and head loss h_L in any element is maintained. The relationship may be in the form of equation (3.7), (3.13), (3.19), (3.20) etc.

The basic idea of analysis is to obtain the value of 'Q' and 'h_L' for every element of network while the three condition are satisfied simultaneously. Therefore it is basically a solution of several non-linear simultaneous equations. As the number of elements increases, direct solution of such equations becomes complicated and laborious.

3.4.2 Convenient Form of Flow Equation:

All the flow equations discussed previously can be expressed in the following form.

$$h_{Lf} = K\,Q^{m} \qquad (3.26)$$

The formula and appropriate value of K & m are as follows.

Formula	h_{Lf}	Remarks
Hazen-William: $V = 0.335\,C\,D^{0.63}\,S^{0.54}$	$\frac{10.67\,LQ^{1.85}}{C^{1.85}\,D^{4.87}}$	m =1.85 $K = \frac{10.67\,L}{C^{1.85}\,D^{4.87}}$
Darcy-Weisbach : $h_{Lf} = \frac{f\,L\,V^2}{2\,g\,D}$	$\frac{8\,f\,L\,Q^2}{\pi^2\,g\,D^5}$	m = 2 $K = \frac{8\,f\,L}{\pi^2\,g\,D^5}$
Mannings : $V = \frac{1}{n}\,R^{2/3}\,S^{1/2}$	$\frac{10.31\,n^2\,L\,Q^2}{D^{5.33}}$	m = 2 $K = \frac{10.31\,n^2\,L\,Q^2}{D^{5.33}}$

When the standard f-Re diagram is consulted in order to assess values of 'f' likely to pertain in the various components of the system , it often leads to simplification if a single exponential expression is derived which is a good approximation to the resistance law throughout the range of condition which are applicable e.g. the diameter is known , expected range of velocities & thus flow rate (Q_{max} , Q_{min}) are known and corresponding 'f' is estimated. Then equivalent m & K can be evaluated as follows;

$$h_{Lf} = \frac{fLQ^2}{\pi^2 g\, D^5} = K\,Q^m \quad \text{so that} \quad \left(\frac{Q_{max}}{Q_{min}}\right)^m = \frac{f_2}{f_1}\left(\frac{Q_{max}}{Q_{min}}\right)^2$$

$\frac{f_2}{f_1}$ = Known , $\frac{Q_{max}}{Q_{min}}$ = Known . Therefore equivalent 'm' can be determined and similarly the equivalent 'K' can be determined.

3.4.3 Design of a pipeline:

Referring to the figure 3.10, known quantities are :

Flow rate (Q).

Total head between source and demand point (H).

Minimum head required at demand point (h_{min}).

Pipe material and hence (K_s).

Pipeline length (L)

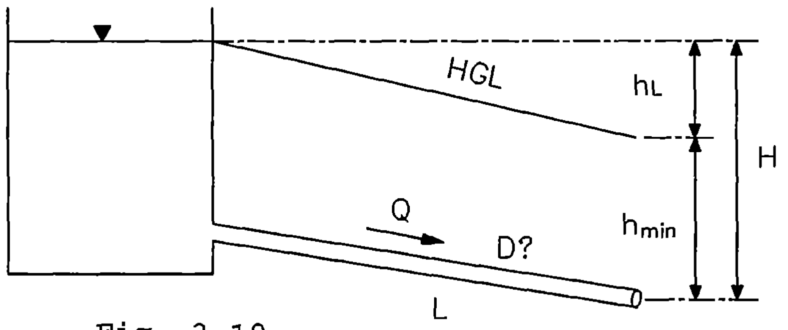


Fig. 3.10

Design steps:

- Compute total head that can be lost (h_L) = $H - h_{min}$
- Compute slope of hydraulic grade line (S) = $\frac{h_L}{L}$

To use equation (3.13):

- Guess D
- Find Q, if calculated Q equal to required Q the diameter is okay, otherwise guess new D. Check if the flow lies on the transitional turbulence , if not then use appropriate formula. Check for total head loss and available head .

To use Moody's diagram:

- Guess D
- Compute $\frac{K_s}{D}$
- Compute V by $V = \frac{4Q}{\pi D^2}$
- Calculate Reynold's number by $Re = \frac{VD}{\nu}$

- Estimate 'f' from Moody's diagram.

- Calculate frictional head loss by $h_{Lf} = \frac{fLV^2}{2gD}$ and

  minor head loss by $h_{Lm} = K\frac{V^2}{2g}$ and

  find $h_L = h_{Lf} + h_{Lm}$

- Compare h_L with available h_L .

- If these are not equal, repeat the process.

The use of charts:

Charts such as prepared by Hydraulic Research gives direct solution.

3.4.4 Design of Pipes in Series:

Referring to fig. 3.11, known quantities are: Flow rate (Q); Pipe material and roughness (K_s); Total static head available (H); Minimum head (h_{min}) required at the end of pipeline and thus total permissible head loss(h_L), and Total lengths of pipes (L).

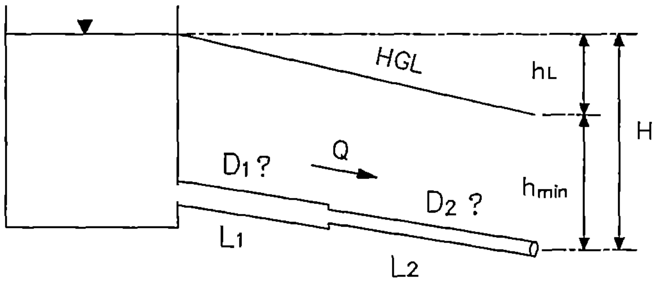


fig. 3.11

Solution strategy :

- Guess D_1 , D_2 , L_1 , L_2 *etc.*

  ($L_1 + L_2$ should be equal to the total length L)

- Compute $\dfrac{K_s}{D_i}$, for all D_i .

Use of Moodys' Diagramme

- Compute mean velocities $V_i = \dfrac{4Q}{\pi D_i^2}$ in all pipes.

- Calculate $\sum h_{Lf} = \sum f_i \dfrac{L_i V_i^2}{2gD_i}$ and

$$\sum h_{Lm} = \sum K_m \frac{V_m^2}{2g}$$

  [All flow formulae can be expressed in the form

  of $\begin{aligned} h_{Lf} &= f_i \frac{L_i V_i^2}{2gD_i} \\ &= K \frac{L_i Q_i^2}{D_i^5}, \quad where\ K = \frac{8f_i}{\pi^2 g} \end{aligned}$]

- Find $h_L = \sum h_{Lf} + \sum h_{Lm}$

- Compare computed h_L to permitted h_L .

- If these two are not equal the process is

to be repeated.

[Note: With increase in number of pipes, the solution becomes tedious. *The use of charts and nomographs facilitates the computation.*]

3.4.5 Design of Pipes in a Branched Network:

Known quantities are:

Flow rates (Q_i), Length of pipes(L_i), Pipe materials and thus roughness(K_s), Total elevation difference (H_i) between nodes, and minimum head required ($h_{\min}$) at demand nodes.

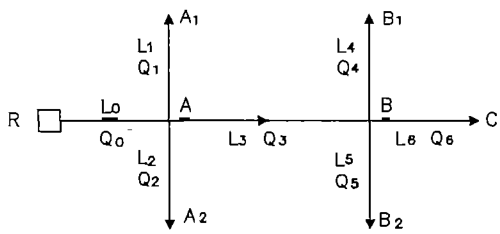


fig. 3.12

Conditions to be satisfied are:

- Required flow demand should be supplied by each branch.
- Minimum head condition should be satisfied at each demand node.
- Hydraulic grade line should not fall below pipe elevation at any point.

Design Strategy:

Design each branch (or portion) individually, for example in fig. 3.12 design branch RA, AA1, AA2, AB, BB1, BB2, and BC individually.

Design of Pipe RA

- Let available piezometric head at R = elevation of R
- Compute required piezometric head at A

  = elevation of A + Required minimum pressure at A

- Compute maximum allowable head loss in RA(h_L)

  = Elevation of R - Minimum required piezometric head at A

(Note : If A is not a supply node and if head lost in portion RA does not affect downstream head requirements, all the available head in RA i.e. elevation difference between R and A may be allowed to be lost).

- Guess D, compute $\frac{K_s}{D}$ and design the pipe as discussed before in design of a pipeline.

Design of branch A-A1

Compute as follows;

- Available piezometric head at A

  = Required piezometric head at A (computed as above).

- Minimum required piezometric head at A1

  = Elevation of A1 + Minimum required pressure at A1.

- Maximum allowable head loss in A-A1 (h_L)

  = Available piezometric head at A

  - Minimum required piezometric head at A1.

Now this branch can be designed as in the first case. Similarly all the branches may be designed. If there are any higher ground level point along the pipeline, the position of hydraulic grade line at that point should be checked. The hydraulic grade line should never fall below the pipe elevation.

Obviously use of the design charts facilitates hand computation.

[Note: Single pipe diameter between two consecutive nodes was assumed in the case discussed, which is a usual case in conventional practice. The provision of a single size pipe diameter between nodes is neither a hydraulic necessity nor a economical option. It is practised because of simplicity in computation. That means if hydraulic conditions are satisfied, a pipe between two consecutive nodes could consists of several segments of different diameter (fig. 3.13) in which case computation becomes more complicated. Design of a pipeline between nodes in such situation is similar to the design of pipe in series discussed before.]

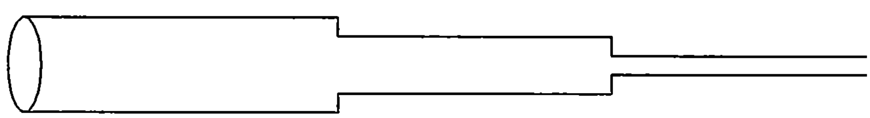


Fig. 3.13

3.4.6 Design of pipes in a looped network:

Analyses and design problem of a loop network is conventionally solved by method of successive approximation. The Hardy-Cross method is one of such popular method. The Hardy-Cross method requires that the flow in each pipe be assumed so that the principle of continuity is satisfied at each junction. A correction to the assumed flow is computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

The correction in a loop is given by;

$$\Delta Q = -\frac{\sum (KQ^{m})}{\sum \left| K\, m\, Q^{m-1} \right|} \qquad (3.27)$$

Where ΔQ = Correction to the assumed flow.

K , m = Same as in equation (3.26).

Q = Assumed flow in pipe.

The known quantities in a loop network design are ;

Total flow rate to satisfy demand at each node (Q_1 , Q_2 , Q_3 , Q_4) in fig 3.14 and hence total supply flow rate (Q_o), length ,pipe material, elevation of all nodes and critical points (e.g. higher elevation points).

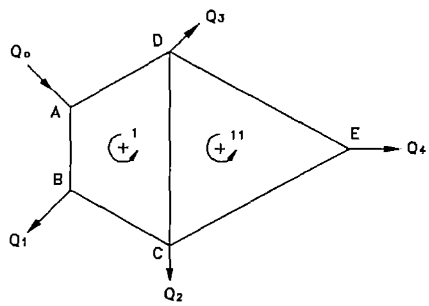


fig. 3.14

Design strategy:

- Guess diameters.
- Guess flows satisfying conditions of nodal equilibrium i.e. at any node algebraic sum of flows must be equal to zero.
- Compute K for each pipe, for example K_{AB} , K_{BC} , K_{CD} etc.,

  depending on the equation chosen.

  ('f' may be read from Moody's diagramme if the Darcy-Weisbach equation is used. The Hazen-William equation is simpler to use as the 'C' in it is assumed to be constant which is strictly speaking not true but gives fairly reliable results).

Pipe	AB	BC	CD	DE	CE
Diameter					
Length					
K					

- Compute sum of head losses ($\sum KQ^m$) around a loop of the network keeping track of signs. If the direction of movement (clockwise or anti-clockwise) around the loop is opposite to the direction of flow in the pipe this h_L is negative.

  The summation $\sum (KQ^m)$ around a loop must be zero when the solution is reached. If it is not zero then it is the numerator of flow correction relation in the Hardy-Cross method.

- Compute denominator $\sum | m\,K\,Q^{m-1}|$.

- Compute flow correction by $\Delta Q = -\dfrac{\sum (KQ^m)}{\sum |m\,K\,Q^{m-1}|}$.

- Similarly compute ΔQ for all loops.
- Compute corrected flows in each pipe. In a pipe common to two or more loops apply correction from all loops keeping track of signs.

- Repeat the process until ΔQ is zero or very small (insignificant).

The tabular form is the most convenient for repeated hand computation. A tabular form is suggested in table 3.4.
The Hardy-Cross method applies only one iterative correction to each loop before proceeding to the next loop. After applying one iterative correction to all loops the process is repeated until acceptable convergence is achieved. The minimum required head at all nodes and the position of hydraulic grade line at critical points should be checked.

If the initially guessed diameter does not give a satisfactory result then a new set of diameters should be guessed and the whole process has to be repeated. The computation is repetitive, lengthy and becomes tedious as the network size increases. It is further complicated if different pipe diameters are tried to provide between two nodes. Obviously the repetitive nature of computation can efficiently be handled by a simple computer programme.

The conventional design approach presented in this chapter considers only hydraulic aspects, not the economic aspects of the network. The next chapter will attempt to incorporate cost aspect of the network in the design approach.

Table 3. A Suggested form of tabular computation

Loop number	Pipe-line	Dia.	First trial			Second trial		
			Q	KQ^m	$\lvert mKQ^{m-1}\rvert$	Q	KQ^m	$\lvert mKQ^{m-1}\rvert$
I	AB BC			$\sum KQ^m$	$\sum \lvert mKQ^{m-1}\rvert$			
			$\Delta Q = -\dfrac{\sum KQ^m}{\lvert mKQ^{m-1}\rvert}$					
II			Note: Q may be +ve or -ve. and the Q in second trial is corrected from first one. First trial, second trial,.. etc. as in loop I.					

CHAPTER 4

4. Optimization of Distribution System:

4.1 INTRODUCTION:

This chapter develops the background information presented in previous chapters especially regarding Pipe Flow hydraulics and Conventional Design approach to the water supply networks. It has been shown in the discussion on the Conventional Design approach that the basic theory of pipe flow is conventionally applied to satisfy the hydraulic conditions of flow rates and head losses in the process of design without due consideration to the cost aspects of the distribution networks.

In this chapter model-based design of distribution system and a general design philosophy/strategy for optimal water supply distribution networks design is discussed. The general mathematical models for optimal looped and branched network are presented. The approach is further extended to deal with the branched network for gravity water supply system in a hilly terrain. A linear programming (LP) model for the optimal design of a branched network is then developed on the basis of mathematical models proposed by many researchers in the last decade. It is hoped that the model is directly applicable to practical situation in a mountainous country like Nepal.

4.2 OPTIMAL WATER SUPPLY SYSTEM DESIGN: PHILOSOPHY/STRATEGY

A typical water supply scheme (fig. 4.1) consists of many components such as intake structures, pipelines, treatment facilities, storage facilities, distribution networks etc.

The true cost of the system includes the construction cost of all of these components and operation and maintenance costs of the system during its design life. Therefore a true cost-optimal system is the one which incurs the minimum combined (e.g. construction + operation + maintenance) cost in its span of life.

The cost of constructing a water supply system is the combined cost of the individual components present in the system. Therefore the optimal construction cost results from a particular combination of components in relation to their size, location and interrelation with other components. The subjects can perhaps be better explained by a general discussion which follows.

Optimal Location and Size of Tanks:

A) Sedimentation Tank:

Let us suppose the source of water is a stream which has a high concentration of suspended solids. The removal of solids is necessary to make it suitable for domestic use. The suspended solids are removed often by a process of desiltation through a sedimentation tank. Water is allowed to stay still or to flow at

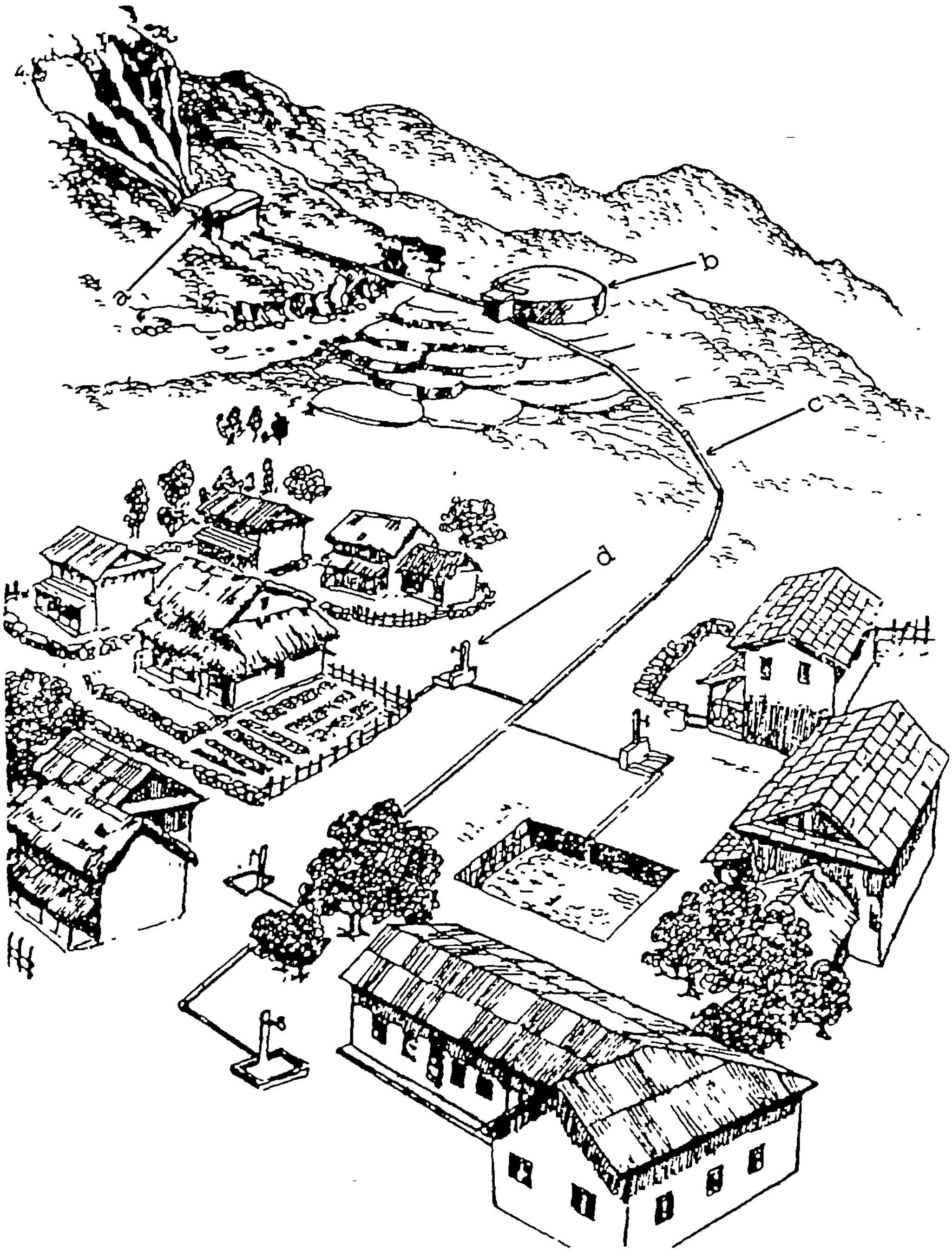


Fig. 4.1 <u>A typical gravity water supply scheme in the hills of Nepal</u>. a. source b. storage tank c. pipeline d. common tap.

[Source : IRC International Water and Sanitation Centre, 1988]

a very low velocity in the tank so that the particles settle at the bottom by gravity. Particles thus accumulated at the bottom of the tank are usually flushed by using the water in the tank. The accumulated sediment may be flushed continuously or at regular intervals. In both cases a certain amount of water is used to flush the sediment. This implies that the transmission pipe between the intake and the sedimentation tank should be capable of carrying a flow higher than the flow downstream of the tank i.e. $Q_o > Q_1$ and thus D_o is larger than that required for passing flow for actual consumer demand. Typical cost of transmission pipe is given by :

$$C_m = \gamma_t L_m D_m^n \qquad (4.1)$$

Where C_m = cost of pipe of internal diameter D_m and length L_m

γ_t = cost coefficient (constant)

n= power of D_m

Equation (4.1) suggests that for a known diameter, cost increases with the length of pipe. Obviously the shorter the length of the pipe the less the cost.

Siting a sedimentation tank as close to the intake as possible therefore reduces the length of the large diameter transmission pipe and hence results in a reduction of total cost.

Another possibility for cost reduction related to the sedimentation tank is it's size. Knowing the density and effective size of the particle needed to be removed, the time to settle the particle at the bottom may be determined. As the discharge and flow velocity are known the size of the tank may

easily be found. The dimensions of the tank depend on the size of the particle to be removed by sedimentation. The smaller the size of particle to be removed, the larger the size of the tank and the higher the cost.

B) Storage Tank:

The cost of the system is also influenced by the location of the storage tank. The diameter of the transmission pipe upstream of the storage tank is larger compared to the size of pipes in the distribution system downstream of the tank. The pipeline between the sedimentation tank and the storage tank is used to transmit water from the first to the second tank; water is usually not provided to the consumer in this portion and therefore larger size pipes may be used. On the other hand, the same water is distributed through a network of smaller sized pipes downstream of the tank. Transmitting a certain amount of water through larger sized pipes costs less compared to the transmission of the same amount of water through smaller sized pipes under same head loss condition. The case is demonstrated by an example below: (fig 4.2)

Let the flow that is to be carried between A and B be 2q

Typical discharge-head loss relation is:

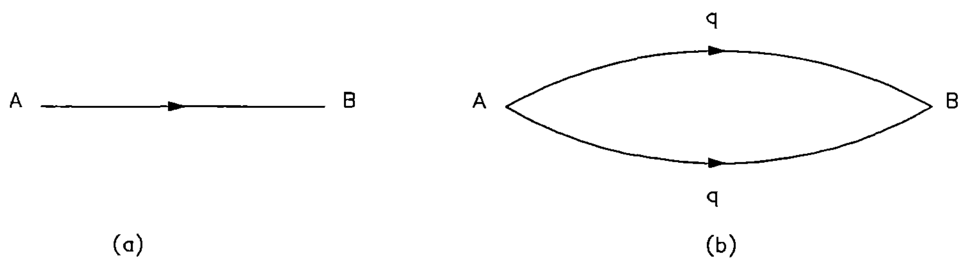
$$h_L = K\ Q_m^2\ / D_m^5 \qquad (4.2)$$

Typical value of n in equation (4.1) is 5/4. Therefore equation

(4.1) becomes:

$$C_m = \gamma_t L_m D_m^{5/4} \tag{4.3}$$

Where K = constant, Q_m = discharge, h_m = head loss and the rest are the same as in equation (4.1)



Single large size Vs smaller multiple size pipes between point A and B.

Fig. 4.2

a) Case I: Single Large Pipe

If total flow = 2q , Head available = h, then application of equation (4.2) gives

$$h_L = K \ (2q)^2 / D_m^5$$

Rearranging,

$$D = 4^{1/5} \ (\ K \ q^2/h_L \ )^{1/5} \tag{4.4}$$

Substitution of equation (4.4) in (4.3) result,

$$C_m = 1.41\ \gamma_t\ L_m\ (K\ Q_m^2 /\ h_L)^{1/4} \quad (4.5)$$

b) Case II: Two smaller diameter pipes:

If two pipes, each of which carries a discharge q, are provided then application of equation (4.2) as before gives the diameter of a pipe as:

$$D = (K\ q^2/h_L\ )^{1/5}$$

and the cost of one pipe by equation (4.3) is

$$C_m = \gamma_t\ L_m (K\ q^2/h_L)^{1/4}$$

and therefore the cost of two pipes is

$$C_m = 2\ \gamma_t\ L_m (K\ q^2/h_L)^{1/4} \quad (4.6)$$

The costs given by equations (4.5) and (4.6) are the costs to transmit the same quantity of water between the same points by a single large sized pipe and two smaller sized pipe. A comparison of these two costs shows that the second option is 29.5% costlier than the first one.

In light of the above example it can easily be inferred that an increase in the number of smaller sized pipes, as in case of the distribution network downstream of the storage tank, will cause the cost to increase. This general conclusion implies an optimization strategy which may be stated as :

Locate the storage tank as close to the demand centre as possible to obtain the minimum cost.

As in the case of the sedimentation tank the cost of the storage tank itself depends on its size. The size is determined by the volume of water intended to be stored, in other words how long the water stored in the storage tank can supply the demand. The decision of this time period is associated with the risk and is another optimization possibility.

Optimal Distribution Network:

Though the total cost of establishment of a water supply system is the summation of the cost of all the elements, the largest proportion of the money is taken up by the pipeline network. The cost of the pipe network is the function of the lengths and the diameters of the pipes used. Many separate attempts are made for the layout optimization to obtain the minimum total length, and for the pipe size optimization to obtain the minimum diameters. However, as in the layout, the lengths and the diameters are hydraulically interrelated. The layout which gives the minimum length, does not necessarily gives the minimum diameters, and vice versa.

Therefore, a true optimal network is obtained only if both layout and diameter are optimised simultaneously.

The demand points and corresponding demands are known in a network design problem.If the cost of a distribution network is assumed to be influenced by pipe length only (not by diameter) and the topography does not affect the network layout, then a minimum spanning tree technique[28] is probably the simplest method of finding the cheapest way of connecting the pipes in the network. In other words, a minimum spanning tree gives the optimal layout under the assumption stated above only.

More general methods of obtaining optimal layout of a water distribution network were suggested by Bhave & Lam[29], Rowell & Barnes[30].

For a given layout, the optimization problem is to find the diameters or sets of diameters of pipes and corresponding lengths, so that total cost of the network is minimum and all the hydraulic conditions are satisfied. In the last two decades or so, various researchers e.g., Karmeli et al[31], Robinson & Austin[32], Alperovits & Shamir[33], Su et al[34], Fujiwara & Silva[35] etc., have proposed the use of mathematical programming techniques in identifying optimal solutions to a given layout of a distribution network.

A method of two linked linear programming formulations have been proposed by Goulter & Morgan[36] for simultaneous optimization of layout as well as pipe sizes.

4.3 SOLUTION TECHNIQUE FOR OPTIMIZATION PROBLEMS:

An attempt to represent a real world situation in the form of a mathematical model may result in a model without any constraint, or with simple equality constraints, or inequality constraints; and linear or non-linear variable functions. Many optimization techniques are used to solve the formulated mathematical models[20,28] depending on the nature of the constraints and the types of variable functions. These optimization techniques are as follows:

A) Classical Optimization:

i. Model without constraints: To optimise such a model differential calculus may be used efficiently.

ii. Model with equality constraints: The technique of Lagrange multipliers may be used to transform the model into an equivalent model without constraints and optimum solution may be obtained as in (i).

B) Optimization of a Linearly Constrained Model:

When a mathematical model consists of a linear objective function and a set of linear constraints then the model may be solved by the simplex method of Linear programming.

C) Optimization of Non-linearly Constrained Models:

i) Model with convex non-linear functions: By the property of convexity they contain no local optima. The solution techniques that may be used are:

Linearization: The non-linear functions are transformed into linear functions, which gives sufficiently accurate approximation of the real model.

Quadratic Programming: A model which has a quadratic objective function and linear constraints may be solved by the technique of quadratic programming.

Gradient Method : It is a stepwise procedure. The solution of the model improves in the direction of the gradient of the objective function.

ii) Models with non-convex non-linear functions:

If a model consists of non-convex functions ,it may contain local optima.There is no criterion by which the existence of local optima could be excluded, and therefore, a solution technique such as gradient methods may end in a local optimum.Such models may be solved by:

Explicit enumeration: Comparison of all finite number of feasible solution. A Decision tree may be used for enumeration.

Implicit or Partial enumeration: The enumeration process may be structured so that only a fraction of all feasible solution has to be examined.

Heuristic procedures: This does not guarantee the optimal solution, but improve an initial solution by any rational approach.

D) Other Optimization Techniques:

Dynamic Programming(DP): Irrespective of linear or non-linear functions, if a problem can be expressed as a series of sequential events, it may be solved by dynamic programming .

Network Analysis: Network analysis techniques eg: Network flow, shortest path problem, minimum spanning tree etc., may be used to find optimal solution of many problems.

4.4 FORM OF FLOW EQUATION IN DESIGN MODEL:

In formulating a mathematical model for analysis, design, or optimization of a pipe network, the relationships between diameter (D_m), head loss (h_L) and flow rate (Q_m) are decisive. In a model-based design approach, the flow equation is often rearranged so that the diameter is expressed as a function of flow rate and rate of head loss (i.e. slope of hydraulic or energy grade line). For example, the Hazen-William equation may be expressed as:

$$D_m = Q_m^{0.38} / (0.615\ C^{0.38}\ S^{0.21}) \tag{4.7}$$

Manning's equation as:

$$D_m = (n\ Q_m / 0.312)^{3/8}\ S^{-3/16} \qquad (4.8)$$

Flow equation (3.13), which resulted from combination of the Darcy-Weisbach equation and the Colebrook-White equation, cannot be explicitly solved for the diameter. In practice, the equation is solved by an iterative procedure, commonly by Newton's method. The Darcy-Weisbach equation may be rearranged in the form of equations (4.7) or (4.8) by substituting $\frac{4Q}{\pi D^2}$ for the velocity.

Darcy-Weishbach equation is:

$$h_L = f\ (L_m/D_m)\ (V^2/2g) = \frac{8\ f\ L_m}{\pi^2 g}\frac{Q_m^2}{D_m^5} \qquad (4.9)$$

Which may be written as;

$$D_m^5 = \frac{8f}{\pi^2 g}\frac{Q_m^2\ L}{h_L}$$

or $$D_m = \frac{K_1 Q_m^{2/5}}{S^{1/5}} \qquad (4.10)$$

Where $K_1 = \frac{8f}{\pi^2 g}$

S = Slope of hydraulic (or energy) grade line

h_L = head loss in length L

f = friction factor (In true sense f is a variable, the value of which depends on the Reynolds Number and / or boundary roughness. Equations 3.8 to 3.11 may be used when appropriate, to calculate the value of f.

Computation becomes tedious if f is considered variable. In practice, therefore, f is considered constant and K_1 becomes constant)

Therefore equation (4.9) can actually be written as;

$$D_m = K_1\ Q_m^{2/5}/S^{1/5} \tag{4.10}$$

Observation of the equations (4.7), (4.8), (4.9 or 4.10) reveal that all of these flow equations are of the same form which may be written as;

$$D_m = K\ Q_m^{m1}/S^{m2} \tag{4.11}$$

Where K = Constant, depending upon the pipe roughness and the flow equation being used .

m1,m2 = Exponent of Q and S respectively, the value of which depends on the flow equation being used.

D_m, Q_m, S has the usual meaning as used previously.

4.5 DESIGN CRITERIA, MODEL FORMULATION AND CONSTRAINTS:

Design Criteria:

A pipe network has to satisfy many design criteria specified in the code of practice. These criteria are mainly related to velocity and pressure. The arrangement of a distribution network and the pipes used in it, must satisfy these criteria, such that required quantities of the flows are available at the demands points.

The criteria related to velocity are the maximum and minimum

flow velocities in pipes. The maximum allowable velocity, which depends on the pipes material, is limited to a certain value to avoid excessive scouring of the pipe wall. The minimum permissible velocity is specified to avoid deposition of the sediments on the pipe and therefore depends on the size of sediments in the water.

The criteria related to pressure are the minimum service pressure at the service points, and the maximum permissible pressure anywhere in the pipelines. The maximum allowable pressure depends on the pipe material and the wall thickness. The minimum pressure requirements are different for the ordinary use and for the fire fighting. In the urban areas where pipes are connected to individual houses, the minimum pressure requirement depends on the height of the buildings. In the rural areas of developing countries, where a common tap is provided for many houses, the minimum pressure criteria is defined so that the container used by the villagers to fetch the water is filled easily and efficiently.

4.6 MODEL FORMULATION:

Governing Laws and Constitutive Equation:

The flow of water in a pipeline network is determined by two fundamental physical laws referred to as Kirchhoff's first and second law[37]. One is the law of continuity which states that all flows leaving a node must equal flows entering the node. The

second law states that the total algebraic sum of head losses around any closed loop must be zero.

The law of continuity can be written as;

$$\sum QI_i - \sum QO_i = 0 \quad \text{for all nodes } i = 1,\ldots,I \qquad (4.12)$$

where, QI_i = Incoming flows at node i

(Q_a for node A and Q_{bc} , Q_{ac} for node C in fig 4.3)

QO_i = Outgoing flows at node i

(Q_{ab}, Q_{bc} for node A and Q_{cd} for node C in fig 4.3

By equation (4.12) at node A ; $Q_a - Q_{ab} - Q_{ac} = 0$

and at node C; $Q_{ac} + Q_{bc} - Q_{cd} = 0$)

Kirchhoff's second law can be written as ;

$$h_{Lm} = 0 \quad \text{for all loops } j = 1,\ldots,J \qquad (4.13)$$

(In fig 4.3 for loop I head losses are h_{Lac}, $-h_{Lcb}$, $-h_{Lba}$

and by equation (4.13) $h_{Lac} - h_{Lcb} - h_{Lba} = 0$)

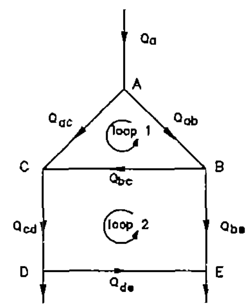


Fig. 4.3

The equations (4.12) and (4.13) are linked together by the flow equation in the form of equation (3.18, 3.19, 3.21 etc). In those

equations if V is replaced by $\frac{4Q}{\pi D^2}$ and K is used to denote the constant part, then the equation takes slightly different form of equation (4.11). The equation may be written as:

$$h_{Lm}=\frac{KL_m Q_m^{e1}}{D_m^{e2}} \tag{4.14}$$

where h_{Lm} = head loss in pipe m

Q_m = flow in pipe m

L_m = length of pipe m

D_m = diameter of pipe m

K = constant which depends on the flow equation.

e1 & e2 = exponent of Q^m and D^m respectively; the value of which depends on the head loss equation used.

The values of e1,e2 & K for Darcy-Weishbach, Hazen-William, and Mannings equations are given below:

Darcy-Weishbach	$K = 8f/ \pi^2 g$,	e1 = 2,	e2 = 5
Hazen-William	$K = 10.6/ C^{1.85}$,	e1 = 1.85,	e2 = 4.865
Manning's	$K = n^2/0.0973$,	e1 =2,	e2 = 16/3

(where f is friction factor, C is Hazen-William constant, n is Manning's roughness coefficient)

The Colebrook-White equation cannot be explicitly expressed in this form.

4.7 OPTIMIZATION MODEL:

Loop Network:

The equations (4.12), (4.13) and (4.14) represent a water distribution network in mathematical form. In design problems all flows external to the network eg: Q_1, Q_2, Q_3, Q_4 in fig 4.4 are known (i.e., external supply and demand at nodes are known). The network design problem is then to find the diameters(D_m) of pipes, such that equations (4.12 to 4.14) are satisfied and the minimum service pressure are maintained at all nodes without exceeding the maximum permissible pressure at any point in the system.

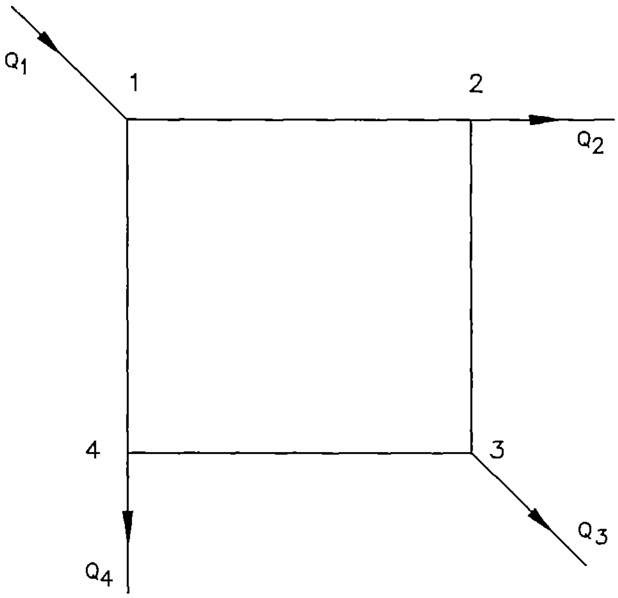


fig. 4.4

Difficulty in determining the appropriate pipe diameter (D) in a loop network arises from the fact that:

i) Both the diameters and the flow rates in the pipes are unknown eg: the flow rates between nodes 1-2, 2-3, 1-4 etc. and the diameters of pipes 1-2, 2-3, 1-4 etc. are unknown; and

ii) the equation (4.14) is non-linear.

The commonly used solution method is to guess a set of diameters. With these diameters as known values, the flow rates are guessed such that equation (4.12) is satisfied. Then the head loss is computed by the equation (4.14). Finally it is checked as to whether equation (4.13) is satisfied or not. If it is not satisfied, then a correction to the initially assumed flow is applied by the Hardy-Cross method, or the Flow method, or the Head method[39]. By applying a correction successively, an acceptable result can be obtained. When the equations (4.12 to 4.14) are satisfied, it is necessary to check if the conditions of minimum and maximum pressure are satisfied; if not, another set of diameters are to be guessed and the whole steps are to be repeated. The process is repetitive and a computer programme makes the job easier.

The aim of optimal pipe network design is to minimise the cost. To achieve this objective, a cost function should be included in the mathematical representation of the pipe network. The cost function can be typically expressed as;

$C_m = \gamma_t \, L_m \, D_m^{5/4}$; where, C_m is cost of pipe m

and the total cost is;

$$C_T=\sum \gamma_t L_m D^{\frac{5}{4}} \tag{4.15}$$

The objective function of the optimization problem may be written as;

$$Minimise \quad C_T=\sum \gamma_t L_m D_m^{\frac{5}{4}} \quad \text{for all m} \tag{4.16}$$

The conditions to be satisfied are;

Equation (4.12)

Equation (4.13)

pressure constraints

$H_i > H_{min}$ at all nodes i = 1, ..., I

$H_{sm} < H_{max}$ at any point in the pipeline

Where, H_i = Pressure at node i

H_{sm} = Hydrostatic pressure anywhere in pipe m

H_{min} = Minimum permissible pressure at a node respectively.

h_{pt} = Maximum permissible hydrostatic pressure for pipe Type 't'

V_m = Velocity at pipe m

V_{min}, V_{max} = Minimum & maximum permissible velocity respectively.

The general network optimization model, therefore , becomes;

$$Minimise \quad C_T=\sum \gamma L_m D_m^{\frac{5}{4}}$$

Subject to:

$\sum QI_i - \sum QO_i = 0$ for all nodes i= 1, ... I

$\sum h_{Lm} = 0$ for all loops j = 1, ..., J

$H_i \geq H_{\min}$ at all nodes i= 1,......, I

$H_{sm} \leq h_{pt}$ any where in the pipeline

$V_m \geq V_{\min}$ at any pipe m

$V_m \leq V_{\max}$ at any pipe m

In these equation head loss (h_{Lm}) is calculated by equation (4.14) which is $h_{Lm} = \dfrac{K\, L_m\, Q_m^{e1}}{D_m^{e2}}$.

The non-linear nature of the objective function and the constraint equations, coupled with the unknown pipe flows (Q_m) and the pipe diameters (D_m), makes the solution of the model difficult. The linear programming (LP) technique can be used to solve the model by making some simplifying assumptions. A method suggested by Alperovits and Shamir[33] assumes;

i) Known initial set of flows (Q_m) such that equation (4.12) is satisfied.

ii) Length L_m consists of several segments of known diameters.

The initial set of flows may be obtained by the Hardy-Cross method, the Flow method, or the Head method and the initial

chosen flows may be modified by the Quindry, Brill, Lebman method[39,40]. The complete solution is obtained by iterative process.

Optimization by the non-linear programming[41] and the dynamic programming[42,43] principles are proposed by various researchers. Various other methods based on different techniques are available.

4.8 OPTIMIZATION MODEL FOR BRANCHED NETWORK:

The most important difference in the analysis, the design of a Loop Network, and a Branched Network system, is that the flow in pipes is uniquely known in the latter. In a branched network system, there is no alternate route of supply to satisfy the nodal demand. Therefore, knowing the nodal demand for a known layout, the discharges in all the pipes can be uniquely computed such that equation (4.12) is satisfied. The analysis and the design problem of a branched network is to use the head loss equation, for example equation (4.14) , to find the diameter such that the velocity and the pressure requirements are satisfied. In the head loss equation $h_{Lm} = \dfrac{K\, L_m\, Q_m^{e1}}{D_m^{e2}}$, D_m is the only unknown. The allowable (h_{La}) can be computed by knowing the elevation difference between the points and the required minimum service head at the supply points (or at the critical points such as

higher elevation points using the condition that hydraulic grade line should not fall below pipe elevation). Equating h_{La} to h_{Lm} only unknown D_m can be computed. The mean flow velocity in the pipe can then be easily computed by the continuity equation $V_m = \frac{4\ Q_m}{\pi\ D_m^2}$. Finally this velocity is compered with the velocity criteria to see if the maximum and the minimum velocity conditions are satisfied or not. The maximum static pressure condition is also checked, if any of these criteria is violated, the process is repeated for another D_m such that $h_{Lm} \leq h_{La}$.

Therefore, design steps are:

Compute D_m by $D_m = [\frac{KL_m Q_m^{e1}}{h_{Lm}}]^{\frac{1}{e2}}$ (4.21)

such that $h_{Lm} \leq h_{La}$ (4.22)

and $V_m \leq V_{max}$

$V_m \geq V_{min}$

$H_{Sm} \leq h_{pt}$

A trial value of h_{Lm} can be taken. A few trials, usually yields the D_m which satisfy these conditions. Thus the analysis and the design of a branched network is simpler as compared to a looped network.

To obtain a true optimal solution, however, the cost function has to be included in the mathematical representation of the network. Again, the total pipe cost, in a network can be typically

expressed by equation (4.15), which is: $C_T = \sum \gamma_t \, L_m \, D^{5/4}$

The optimal design problem is to find the diameters to minimise the total pipe cost, while meeting the pressure and the velocity conditions. The problem can be formulated as:

Minimise $\quad C_T = \sum \gamma_t \, L_m \, D_m^{5/4} \qquad \sum \gamma L_m D_m^{5/4}$

Subject to:

$$h_{Lm} \le h_{La}$$

$$H_{sm} \le h_{pt}$$

$$V_m \le V_{\max}$$

$$V_m \ge V_{\min}$$

Various researchers have proposed a number of optimization models and, the solution techniques for a branched water supply distribution network. Liang[43] has solved the single dead end problem through dynamic programming. Wantanatada[44] formulated the design problem as a non-linear programming problem, and solved it by variable metric method. Karmeli et al[31], Robinson & Austin[32] and Alperovits & Shamir[33], formulated the optimization model as a linear programming problem. Deb[45] extended the method suggested by Cowan[46] to obtain an analytical solution considering pipes in a series, for optimal pipe sizes. Appleyard[47], proposed Lagrange multiplier method which has been further developed by Chipunkar & Khanna[48] for calculator computation purpose. Fujiwara & Dey[49]

proposed a two stage solution method . In the first stage pipe sizes are obtained by the Lagrange multiplier method; and in the second stage, the method uses LP model, where the solution obtained in the first stage, is used to select a restricted candidates list, for input to the model. The application of this method is limited to branched networks, located on a flat terrain with a single source node, and where the required minimum head for each end node is equal.

The linear programming (LP) technique, similar to the one suggested by Alperovits & Shamir, truly yields the global optimal solution for branching distribution networks. A constant flow part of the network, may be assumed to consist of one or more pipes of known diameters (ie commercially available diameters). The equations (4.14) & (4.15) suggest that the head loss and the cost of the pipes of known diameter, (D_m) and with the known flows, (Q_m) are linear function of its length (L_m). Therefore, the constant flow part can be assumed to consist of pipe segments, of known sizes (d_p) but unknown lengths(l_p), eg in fig 4.5. Thus l_p becomes decision variable of the LP model.

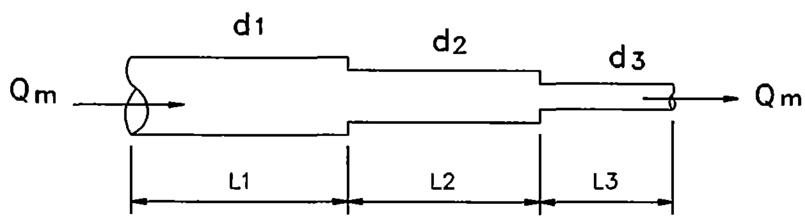


fig. 4.5

With these assumptions the optimization model would become:

Minimise $\sum_{m=1}^{M} \gamma_t \sum l_p \, d_p^{5/4}$ (4.23)

Subject to :

$\sum l_p = L_m$ (4.24)

$E_{ij} - \sum h_{Lm} \geq H_{\min}$ (4.25)

$E_{ij} \leq h_{pt}$ (4.18)

$V_m \geq V_{\min}$ (4.19)

$V_m \leq V_{\max}$ (4.20)

$l_p \geq 0$ (4.26)

Where E_{ij} = elevation difference between the points i & j

By using the whole range of commercially available diameters as an input to the model, the number of decision variables (l_p) will be unnecessarily increased. The range of d_p is actually restricted to a few, by the velocity criteria. By using continuity equation, the maximum, and the minimum velocity requirements, one may obtain;

$$d_{\max} = \sqrt{\frac{4\,Q}{\pi\,V_{\min}}} \quad (4.22a)$$

and $$d_{\min} = \sqrt{\frac{4\,Q}{\pi\,V_{\max}}} \quad (4.22b)$$

where $d_{\max}$ = Maximum permissible pipe diameter

$d_{\min}$ = Minimum permissible pipe diameter

Thus, in a constant flow part, only those commercially available diameters are necessary to consider which fall within the range of d_{max} and d_{min} .

This model is directly applicable to obtain the optimal solution of a branching pipe network, in a relatively plain terrain, where complete gravity flow exist, only one type of pipe (pipe material and/ or wall thickness) is used, and no pressure release devices (eg pressure release valves or pressure release tanks) are used.

4.9 Model For a Branched Network in Hills:

4.9.1 INTRODUCTION:

In the hilly topography, the high elevation difference along the pipeline introduces some new conditions which were not considered in the LP models discussed in the previous sections. In this section, a discussion on the nature of these new conditions and the conventional way of dealing with them will be presented. Then a LP model similar to the previous models will be developed for the optimization of a branched gravity water supply network in the hills. The next chapter will show the use of the developed model through the solution of a hypothetical example.

4.9.2 PROBLEM STATEMENT:

The topography of a hilly area is characterised by high elevation differences. Large elevation difference along a pipeline implies high static pressure (H_{sm}) at some points. The pipe used to supply water can only withstand a certain level of pressure, which depends on the pipe material and the wall thickness. Some means should be introduced to release the pressure, when the elevation difference between two points along a pipeline exceeds the pressure that the pipes in the pipeline can withstand. In hilly areas of Nepal, Break Pressure Tanks (BPTs) are commonly used to release the additional pressure. Introduction of the BPTs

in the network creates new constraints in the distribution network optimization model, discussed in the previous section. PVC pipes are commonly used in rural water supply schemes in developing countries like Nepal. The pipes come in various wall thicknesses, which can withstand different amounts of internal pressure. Depending on the internal pressure created by the elevation difference, pipes of the same internal diameter, but of a different wall thickness, may be used along a pipeline branch. Thicker wall pipes are costlier than the ones with thinner walls. This difference in cost further modifies the optimization model. The cost coefficient γ in equation (4.15) remains constant only for one pipe type (material and wall thickness). As soon as the pipe wall thickness and /or the pipe material differs, a different γ , which reflects the cost of that type of pipe, should be used.

The layout of a network in a hilly terrain is usually controlled by the topographical (e.g., ridge/valley) conditions, other natural conditions (e.g.: landslide prone areas, suitable stream crossing, difficult cliff area etc.), and nature of settlement. Therefore, there is little or practically no possibility of layout optimization. An optimization model for a hilly terrain therefore, should include diameters (or lengths), BPTs, and pipe type (thick or thin wall) as basic variables.

A complete gravity water supply system is usually used for small rural communities in the mountains of Nepal. A pumping system is

particularly out of the question as no power is available in the rural areas. On the other hand the use of water source located at a higher elevation than the settlements favour a gravity system. At a branching node, a Distribution Box (DB) is used to facilitate the water distribution in a different branch. These DBs are tanks which, in addition to facilitating a pipe branching, effectively serve the purpose of pressure release tanks.

Therefore, a network may be optimised by optimising each branch between such nodes individually.

4.9.3 OPTIMIZATION STRATEGY:

The commonly used components of a gravity water supply system in the rural hills are intake, sedimentation tank, storage tank and pipelines.

It is obvious from the discussion presented at section (4.2) that siting a sedimentation tank as close to the intake as possible; and a storage tank as close to the demand centre as possible, results in a reduction in the total system cost. In light of the discussion presented in the preceding section, the network optimization model of the form given by the equations (4.23) to (4.26) has to be modified to incorporate BPTs and the pipes wall thickness. Introduction of the BPTs has two effects; first it releases the internal pressure created by elevation difference, and the second is that as a consequence of pressure release, it is possible to use a thinner walled pipe which is cheaper. Thus, it results in a reduction in total pipe cost. However, the tanks

have their own cost. Therefore, the optimum network is only possible from the optimum combination of tanks and pipes.

As the distribution boxes are used in all of the branching nodes whose positions are known from the layout, these boxes also serves the purpose of a pressure release tank. Each branch of the network may be optimised independently.

The cost of a branch is related to three things which are:

A) Positions and number of BPTs

B) Lengths and diameters of pipes

C) Pipe wall thickness(or pipe type)

All three are interrelated and contribute to the cost of a branch. The optimal policy for a given number of BPTs is the combination of positions of the BPTs, lengths & diameters of pipes, and pipe wall thicknesses, which gives minimum cost. Obtaining the optimal policy for a given number of BPTs involves evaluating the cost of the branch for all possible positions of the tanks, lengths & diameters of pipes, and pipe wall thicknesses. The global optimal policy for a branch can be obtained by comparing the optimal policies for all possible number of BPTs. Therefore, computation for obtaining a global optimal policy for a branch becomes repetitive. The following process may be used to obtain a global optimal solution for a branch .

First obtain an optimal policy when the absolute minimum number(N_{min}) of BPTs are provided. Then increase the number of BPTs by one and obtain an optimal policy for the present

number of BPTs. Increase the number of BPTs in steps of one and find optimal policy for each case; terminate the process when the number of BPTs becomes the absolute maximum (N_{max}). Each policy is valid optima for the number of tanks provided. Then select the one policy from these optimal policies, which has the minimum cost.

The global optimal policy includes positions and numbers of the tank(s), lengths and diameters of various types of pipes. Therefore, the global optimal policy

$$= \text{Minimum of} \quad \begin{array}{l} \text{Optimum policy when number of BPT} = N_{min} \\ \text{Optimum policy when number of BPT} = N_{min} + 1 \\ \cdots\cdots\cdots\cdots\cdots\cdots \\ \text{Optimum policy when number of BPT} = N_{min} + N \\ \cdots\cdots\cdots\cdots\cdots\cdots \\ \text{Optimal policy when number of BPT} = N_{max} \end{array}$$

4.9.4 DEFINITION OF TERMS :

Before formulating and examining the optimization model for the hilly conditions, the following terms will be defined:

Branch : A branch is the part of a network that has constant flow. eg S-T1-J1, T1-T2-J2, J2-E1, J1-J4 etc.

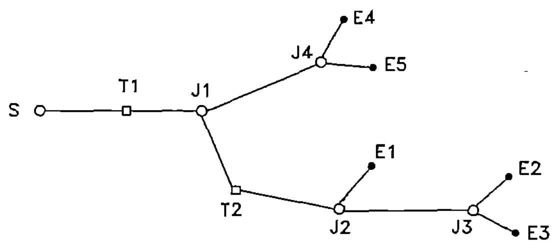
Link : A link is considered to be a continuous part (without break by BPT) of a branch . eg S-T1, T1-J1, J1-J4 etc.

Part : A part is defined as the portion of a link which has only one type of pipe (i.e., the same wall thickness and same material).

Node : A node is a point where two or more link meets, or a link starts, or a link ends eg S, T1, J1, E1 etc. The source (S), the BPTs (T), the branching point (J), and the end points (E), are all nodes. The source (S), junction(J), and service points (E), are fixed node where as the position and number of the BPTs nodes are unknown initially. The determination of the total number and positions of tank nodes are a part of the optimization process.

Path : A path is any continuous sequence of branches in a network.eg S-T1-J1-J4-E5 , S-T1-J1-T2-E1 , T1-J1-T2-J2-J3-E3 , T1-J1-T2 etc.

A path may start from a source(S), or from a tank (T) where pressure is atmospheric, and may ends at a service points(E), or at a tank.



Definition diagram

Fig 4.6

4.9.5 ASSUMPTIONS AND KNOWN QUANTITIES:

Assumptions:

1. The layout of the pipe network is known. This is a reasonable assumption as a layout in the hills is usually controlled by the topography and the nature of the settlement.
2. Relationship between the head loss, the flow, and the pipe diameter is given by the equation (4.14)

$$h_{Lm} = \frac{KL_m Q_m^{e1}}{D_m^{e2}}$$

3. The pipe cost is given by the equation of the type (4.15)

$C_m = \sum \gamma_t \, L_m \, D_m^{5/4}$ C_t is pipe cost.

4. The cost of the break pressure tanks is given by the equation

$$C_t = N\, c_t \qquad (4.28)$$

where C_t = Total cost of tanks.

c_t = Cost of one tank.

N = Number of tanks.

[The BPTs used in a small community water supply system are of a standard size and therefore, the cost is the same for every tank in a particular system. That is, the total cost of the tanks is only given by the product of the number of tanks, and the cost of a tank.]

By assumption (3) & (4), the total cost of a network system is given by :

$$C_T = N\, c_t + \sum \gamma_t \, L_m \, D_m^{5/4} \qquad (4.29)$$

Where C_T = Total network cost.

5. The pressure at each branching node is atmospheric. The use of a distribution box at a branching node makes this assumption valid.
6. The ground has a uniform slope along the pipeline in a branch.

The known Quantities:

Quantities known from feasibility survey are:

1. The flow in each branch (Q_m) is uniquely Known .
2. The ground elevation of the points along all paths. (i.e., the ground profile and the elevation difference between the points are known).
3. The cost of a tank.
4. The cost per unit length of all types (thin & thick wall) and the diameters of the pipes.
5. Length of each branch (L_m)

The quantities known from criteria laid down in the code of practice are:

1. The minimum pressure requirements at service nodes.
2. The minimum and the maximum allowable velocities.
3. The maximum allowable pressure for each type of pipe.

4.9.6 MODEL FOR A BRANCH:

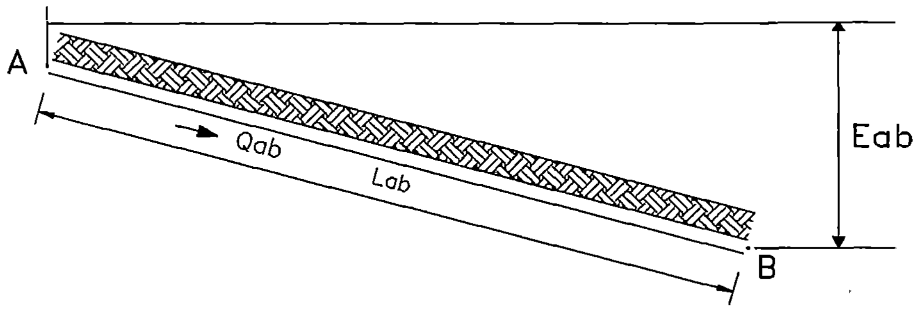
<u>A Branch with a uniform slope:</u>

Considering a branch, between points A and B, Fig 4.7 in which ;

Q_{ab} = Flow in branch A-B

E_{ab} = Elevation difference between points A & B

L_{ab} = Total length between points A & B



A uniform slope branch

Fig 4.7

For the purpose of simplicity in discussion and subsequent model formulation it is assumed here, that pipes are available in two wall thicknesses. Theoretically, there can be more than two wall thicknesses or pipe materials.

The following symbols are used in the discussion.

h_{p1} = Maximum pressure that type 1 (thin wall type) pipe can withstand.

h_{p2} = Maximum pressure that type 2 (thick wall type) pipe can withstand.

Obviously $h_{p2} > h_{p1}$ and type 2 is costlier than type 1.

	γ_t =	Cost	coefficient	of pipe	type	t.
ie	γ_1 =	"	"	"	"	1
	γ_2 =	"	"	"	"	2

Note : γ_t varies with wall thickness and/ or pipe material.

Two situations are possible in a branch:

A) Branch without a BPT

B) Branch with the BPTs

Branch without a BPT may exist in two conditions, which are;

i) when $E_{ab} \leq h_{p1}$

ii) when $h_{p2} \geq E_{ab} > h_{p1}$

Branch with BPTs exist when $E_{ab} \geq h_{p2}$

Each possibility will now be studied in detail.

Model Formulation for a branch:

1. A BRANCH WITHOUT BPT WHEN $E_{ab} \leq h_{p1}$:

In such a situation, a thinner wall pipe (type 1 & cheaper one) can be provided in the whole length of the branch AB. That means introduction of a pressure release tank does not result in a reduction of the pipe cost, instead, the additional cost of the tanks increases the total cost. Therefore, it is not necessary to

consider BPT and pipe type 2 in the optimization model.

The objective function would be:

$$\text{Minimise} \quad C_{ab} = \sum \gamma_t \, L_{ab} \, D_{ab}^{5/4}$$

$$\text{or Minimise} \quad \sum \gamma_1 L_{ab} D_{ab}^{5/4}$$

If the length L_{ab} is assumed to consists of many pipe segments of unknown lengths l_p and known diameters d_p . The d_p are the commercially available diameters within the range of d_{max} and d_{min} . Let there are n number of diameters available within this range. Then the objective function become:

$$\text{Minimise} \quad C_p = \gamma_1 \sum_{p=1}^{n} (l_p \, d_p^{5/4}) \tag{4.30}$$

and the constraints are:

$$\sum_{p=1}^{n} l_p = L_{ab} \tag{4.31}$$

If the minimum pressure requirement at B is $H_{min,b}$ and the summation of head losses in A-B is $\sum h_{Lab}$ then the pressure condition is:

$$E_{ab} - \sum h_{Lab} \geq H_{min,b}$$

$$\text{or} \quad E_{ab} - KQ_{ab}^{e1} \sum (l_p / d_p^{e2}) \geq H_{min,b} \tag{4.32}$$

$$\text{and all} \quad l_p \geq 0 \tag{4.33}$$

Therefore, the complete LP model would be:

$$\text{Minimise} \quad \gamma_1 \sum_{p=1}^{n} (L_p \, d_p^{5/4})$$

Subject to:

$$\sum l_p = L_{ab}$$

$$E_{ab} - K \, Q_{ab}^{e1} \sum_{p=1}^{n} (l_p / d_p^{e2}) \geq H_{min,b}$$

$$l_p \geq 0$$

2. A BRANCH WITHOUT BPT WHEN $h_{p2} > E_{ab} > h_{p1}$:

When the situation $h_{p2} > E_{ab} > h_{p1}$ exists, two options are available, either;

I). Provide the Type 1 pipe up to the point where internal pressure in the pipe equals h_{p1} and provide the pipe Type 2 in the remaining portion.

or;

II). Provide a BPT (or BPTs) and use pipes Type 1 and 2 as appropriate.

MODEL FOR A BRANCH WITHOUT BPT

Defining following symbols;

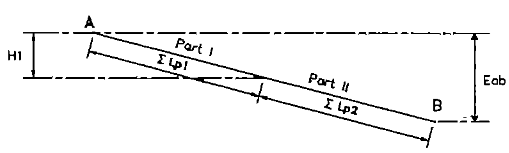
$$S_{ab} = \frac{E_{ab}}{L_{ab}} = \text{ slope of the line A-B (uniform)}$$

$\sum l_{p1}$ = Total length of the pipe Type 1

$\sum l_{p2}$ = Total length of the pipe Type 2

The cost of the pipes is given by :

$$C = \gamma_1 \sum (l_{p1}\, d_{p1}^{5/4}) + \gamma_2 \sum (l_{p2}\, d_{p2}^{5/4}) \qquad \text{for all p} \qquad (4.34)$$



Using two types of pipes

Fig 4.8

Up to the hydrostatic pressure of h_{p1} , the cheaper i.e. Type 1 pipe could be used. If the Type 1 pipe is used up to an elevation difference of H_1 from A, then the condition of the internal pressure in the Type 1 pipe should not exceed h_{p1} and can be expressed as ;

$$H_1 \leq h_{p1}$$

or $$S_{ab} \sum l_{p1} \leq h_{p1} \qquad (4.35)$$

If $\sum h_{Lp1}$ and $\sum h_{Lp2}$ are the head losses at the part 1 and the part 2 respectively, the minimum pressure requirement at point B can be stated as;

$$E_{ab} - \sum h_{Lp1} - \sum h_{lp2} \geq H_{\min,b}$$

or $$E_{ab} - KQ_{ab}^{e1} \sum (l_{p1}/d_{p1}^{e2}) - KQ_{ab}^{e1} \sum (l_{p2}/d_{p2}^{e2}) \geq H_{\min,b} \qquad (4.36)$$

Total length should be equal to L_{ab} .

ie $$\sum l_{p1} + \sum l_{p2} = L_{ab} \qquad (4.37)$$

Therefore, complete LP formulation would be:

Minimise $$C_p = \gamma_1 \sum (l_{p1} d_{p1}^{5/4}) + \gamma_2 \sum (l_{p2} d_{p2}^{5/4}) \qquad (4.38)$$

Subject to:

$$S_{ab} \sum l_{p1} \leq h_{p1} \qquad (4.35)$$

$$E_{ab} - K\, Q_{ab}^{e1} \sum (l_{p1}/d_{p1}^{e2}) - K\, Q_{ab}^{e1} \sum (l_{p2}/d_{p2}^{e2}) \geq H_{\min,b} \qquad (4.36)$$

$$\sum l_{p1} + \sum l_{p2} = L_{ab} \qquad (4.37)$$

and $$l_{p1}\ ,\ l_{p2} \geq 0 \qquad (4.39)$$

3. A BRANCH WITH BPT:

The BPT, one or more as the case may be, must be provided when the elevation difference exceeds the maximum permissible pressure for the Type 2 pipe ($E_{ab} > h_{p2}$). When $E_{ab} = h_{p2}$, the provision of a BPT is not a strict hydraulic requirement, since the Type 2 pipe can withstand the maximum pressure exerted in the system. In such a case a decision whether to provide a tank (or tanks) or not has to be taken entirely on the cost basis. But when $E_{ab} > h_{p2}$, a certain minimum number of tanks (or some other pressure releasing devices e.g. valves) have to be provided. The minimum number of tanks between the two points A & B, may be obtained by the integer part of $\frac{E_{ab}}{h_{p2}}$. Similarly the maximum number of the BPTs can be obtained by the integer part of $\frac{E_{ab}}{h_{p1}}$.

MODEL FOR A BRANCH WITH BPT:

Situation I : When $E_{ab} = h_{p2}$

The optimization problem is to obtain the position of the tank(s); in addition to the lengths and diameters of the pipe Type 1 & 2, such that the total cost is minimum.

Using the following symbols:

T1 = Position of tank 1

E_l = Elevation difference between the starting and the ending points of a link 'l'

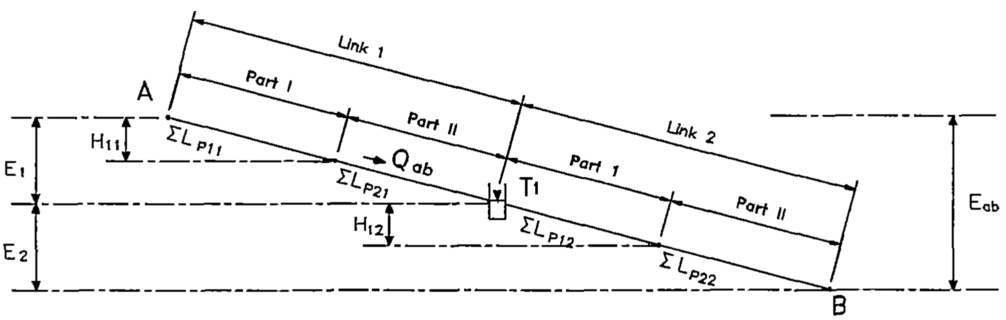
ie E_1 = Elevation difference between the starting and the ending points of link 1.

l_{ptl} = Length of pipe 'p' of type 't' in link 'l'

h_{Lptl} = Head loss in pipe 'p' of type 't' in link 'l'

H_{tl} = Maximum head subjected to pipe type 't' in link 'l'.

In the Figure 4.9, Link 1 = A-T1, and Link 2 = T1-B. Each link consists of two parts. Part-I of the pipe type 1 and part-II of the pipe type 2.



Two types of pipes and a BPT

Fig. 4.9

The cost of the system is given by :

C_T = Cost of tank (C_t) + Cost of part I in link 1
+ Cost of part II in link 1
+ Cost of part I in link 1
+ Cost of part II in link 2

or
$$C_T = C_t + \gamma_1 \sum (l_{p11}\, d_{p1}^{5/4}) + \gamma_2 \sum (l_{p21}\, d_{p2}^{5/4}) + \gamma_1 \sum (l_{p12}\, d_{p1}^{5/4}) + \gamma_2 \sum (l_{p22}\, d_{p2}^{5/4}) \qquad (4.40)$$

The conditions to be satisfied are:

<u>In link 1</u>:

* The maximum permissible pressure at part I should not exceed h_{p1} .

ie $H_{11} \le h_{p1}$

or
$$S_{ab} \sum l_{p11} \le h_{p1} \qquad (4.41)$$

* The total head loss in the link 1 ($\sum h_{lpt1}$) should be equal to the available head (E_1).

ie $E_1 - \sum h_{Lpt1} = 0$

or
$$S_{ab} \left(\sum l_{p11} + \sum l_{p21}\right) - K\, Q_{ab}^{e1} \sum (l_{p11}/d_{p1}^{e2}) - K\, Q_{ab}^{e1} \sum (l_{p21}/d_{p2}^{e2}) = 0 \qquad (4.42)$$

Similarly <u>in link 2</u>:

$H_{21} \le h_{p1}$

or $$S_{ab} \sum l_{pt2} \leq h_{p1} \tag{4.43}$$

* The minimum head requirement at B;

$$E_2 - \sum h_{Lpt2} \geq H_{\min,b}$$

or $$E_2 - \sum h_{Lp12} - \sum h_{Lp22} \geq H_{\min,b}$$

or $$S_{ab} \left(\sum l_{p12} + \sum l_{p22} \right) - K Q_{ab}^{e1} \sum (l_{p12}/d_{p1}^{e2}) - K Q_{ab}^{e1} \sum (l_{p22}/d_{p2}^{e2}) \geq H_{\min,b} \tag{4.44}$$

* The total lengths of the pipes should be equal to the length between points A & B (or L_{ab}).

i.e. $$\sum l_{p11} + \sum l_{p21} + \sum l_{p12} + \sum l_{p22} = L_{ab} \tag{4.45}$$

And
* All the decision variable should be positive.

i.e. $$l_{ptl} \geq 0 \tag{4.46}$$

The complete LP formulation would be:

Minimise equation (4.40)

Subject to :

Equation (4.41)

Equation (4.42)

Equation (4.43)

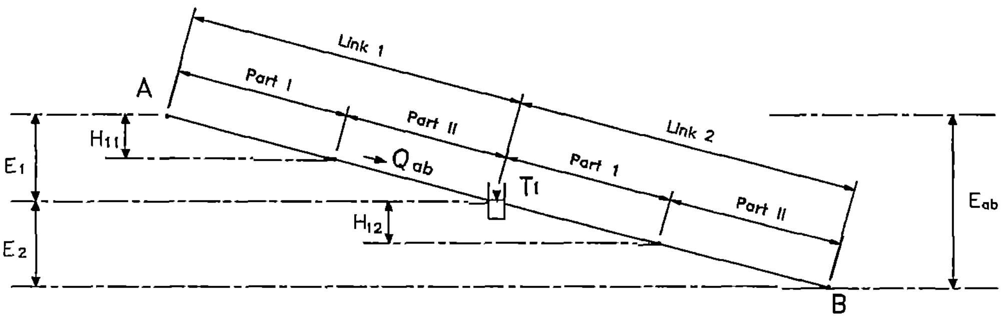
Equation (4.44)

Equation (4.45)

and Equation (4.46)

Situation II: When $E_{ab} > h_{p2}$

When the condition $E_{ab} > h_{p2}$ exists then some more constraints, in addition to those presented in preceding model, have to be introduced. The new constraints are related to the maximum permissible internal pressure in the Type 2 pipes. In part II of both the links, internal pressure should not exceed h_{p2} .



Two types of pipes and a BPT

Fig. 4.10 Branch A-B with a BPT when $E_{ab} > h_{p2}$

i.e. additional constraint for link 1 is:

$$E_1 \le h_{p2}$$

$$\text{or} \quad S_{ab} \left(\sum l_{p11} + \sum l_{p21}\right) \le h_{p2} \qquad (4.47)$$

and for link 2:

$$E_2 \le h_{p2}$$

$$\text{or} \quad S_{ab} \left(\sum l_{p12} + \sum l_{p22}\right) \le h_{p2} \qquad (4.48)$$

Therefore, the complete LP formulation would be :

Minimise equation (4.40)

Subject to:

Equation (4.40)

....

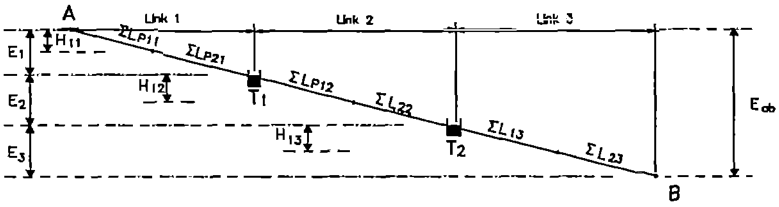
....

Equation (4.48)

MODEL FOR MORE THAN ONE BPT SITUATION:

First let us consider two tanks, T1 and T2 in an uniform slope branch A-B. Then link 1 = A-T1, link 2 = T1-T2, and link 3 = T2-B. The number of links in a branch is equal to the number of BPT in the branch plus one.

As before each link may be assumed to consist of two parts; PartI consisting the pipe Type 1 and part II consisting the pipe Type 2 only. The optimization model formulation is exactly similar to the previous case. The only difference is the number of BPT, therefore, the number of links has increased; which in turn increases the number of decision variables (l_{ptl}) and the constraints to be satisfied.



Branch AB with more than one BPT

Fig. 4.11

The LP model for two BPT situations (see fig.4.11) may be formulated as follow.

Minimise the total cost,which is given by:

$$\begin{aligned} C_T = 2\ C_t &+ \gamma_1 \sum l_{p11}\ d_{p1}^{5/4} \\ &+ \gamma_2 \sum l_{p21}\ d_{p2}^{5/4} \\ &+ \gamma_1 \sum l_{p12}\ d_{p1}^{5/4} \\ &+ \gamma_2 \sum l_{p22}\ d_{p2}^{5/4} \\ &+ \gamma_1 \sum l_{p12}\ d_{p1}^{5/4} \\ &+ \gamma_2 \sum l_{p23}\ d_{p3}^{5/4} \end{aligned} \qquad (4.50)$$

Subject to :

<u>In Link 1 :</u>

$$H_{11} \leq h_{p1}$$

or $\quad S_{ab} \sum l_{p11} \leq h_{p1} \qquad (4.50)$

and $\quad E_1 \leq h_{p2}$

or $$S_{ab}\ (\sum l_{p11} + \sum l_{p21})\ \le h_{p2} \qquad (4.51)$$

and $$E_1 - \sum h_{Lt1} = 0$$

or $$E_1 - \sum h_{L11} - \sum h_{L21} = 0$$

or

$$S_{ab}\ (\ \sum l_{p11} + \sum l_{p21}\ )\ -K\ Q_{ab}^{e1} \sum (l_{p11}/d_{p1}^{e2}) \\ -K\ Q_{ab}^{e1} \sum (l_{p21}/d_{p2}^{e2}) = 0 \qquad (4.52)$$

<u>In Link 2:</u>

$$H_{12} \le h_{p1}$$

i.e. $$S_{ab} \sum l_{p12} \le h_{p1} \qquad (4.53)$$

$$E_2 \le h_{p2}$$

i.e. $$S_{ab}(\sum l_{p12} + \sum l_{p22})\ \le h_{p2} \qquad (4.54)$$

and, $$E_2 - \sum h_{Lt2} = 0$$

i.e. $$S_{ab}(\sum l_{p12} + \sum l_{p22})\ -\ K\ Q_{ab}^{e1} \sum (l_{p12}/d_{p1}^{e2}) \\ -\ K\ Q_{ab}^{e1} \sum (l_{p22}/d_{p2}^{e2}) = 0 \qquad (4.55)$$

<u>In Link 3:</u>

$$H_{13} \le h_{p1}$$

i.e. $$S_{ab} \sum l_{p13} \le h_{p1} \qquad (4.56)$$

$$E_3 \le h_{p2}$$

and i.e. $$S_{ab}\ (\sum l_{p13} + \sum l_{p23})\ \le h_{p2} \qquad (4.57)$$

and $$E_3 - \sum h_{Lt3} \ge h_{\min,b}$$

i.e. $$S_{ab}\ (\sum l_{p13} + \sum l_{p23})\ -\ K\ Q_{ab}^{e1} \sum (l_{p13}/d_{p1}^{e2}) \\ -\ K\ Q_{ab}^{e1} \sum (l_{p23}/d_{p2}^{e2}) \ge H_{\min,b} \qquad (4.58)$$

The total length constraint:

$$\sum l_{p11} + \sum l_{p21} + \sum l_{p12} + \sum l_{p22} + \sum l_{p13} + \sum l_{23} = L_{ab} \qquad (4.59)$$

and finally,

the decision variable $l_{ptl} \ge 0$ (4.60)

As the number of links (or BPT) increases, the number of

decision variables l_{ptl} increases. It has been shown in the preceding model formulation that each link has at least three constraints to be satisfied. The velocity criteria may be taken into account by considering the range of d_p only within d_{min} and d_{max} . This will also reduce the number of decision variables. The number of the decision variables (n) in a part is equal to the number of commercially available diameters within d_{min} and d_{max} .

Therefore, the number of the variables in a link

= Number of the variables in part I

+ Number of the variables in part II

= 2 n

Total number of the variables in a branch

= number of the links(N_l) multiplied by 2n

= 2 n N_l

and the number of BPT = N_l - 1

The LP model for a branch consisting of N_l number of links (i.e. N_l - 1 tanks) in a uniform ground slope S can, therefore, be written as :

Minimise $(N_l-1)\ C_t + \sum_{l=1}^{N_l} \sum_{t=1}^{t} [\gamma_t \sum_{p=1}^{n} (l_{ptl}\ d_{pt}^{5/4})]$ (4.61)

Subject to :
For all links;

$$H_{1l} \le h_{p1} \tag{4.62}$$

$$S \sum l_{ptl} \le h_{p1} \tag{4.63}$$

For <u>all nodes</u> except services nodes;

$$S \sum l_{ptl} - K\, Q_m^{e1} \sum (l_{ptl}/d_{pt}^{e2}) = 0 \tag{4.64}$$

For <u>all service nodes;</u>

$$S \sum l_{ptl} - K\, Q_m^{e1} \sum (l_{ptl}/d_{pt}^{e2}) \ge H_{\min} \tag{4.65}$$

The total length constraint;

$$\sum l_{ptl} = L_m \tag{4.66}$$

and non-negative of the decision variables;

$$l_{ptl} \ge 0 \tag{4.67}$$

For a given number of the BPTs, the solution of the model gives the tanks positions; lengths and diameters of each type of pipes to be used; so that the minimum cost is incurred.

Therefore to obtain a policy for the absolute minimum cost, the model has to be solved many times. First the model may be solved to obtain an optimum policy when the minimum number of BPTs are provided. The model may be solved repeatedly for the whole range of possible BPTs, and the optimum policy for each case may be obtained. The truly optimum policy for a branch may be obtained by comparing the individual optimum policies.

4.9.7 ADDITIONAL CONSTRAINTS WHEN GROUND LEVEL RISES ALONG A PIPELINE.

When the ground elevation rises along a pipeline some more constraints have to be introduced in the model presented in the preceding discussion.

The additional constraints are related to the pressure at the lowest and the highest points in the pipeline e.g. point B and point C in fig 4.12.

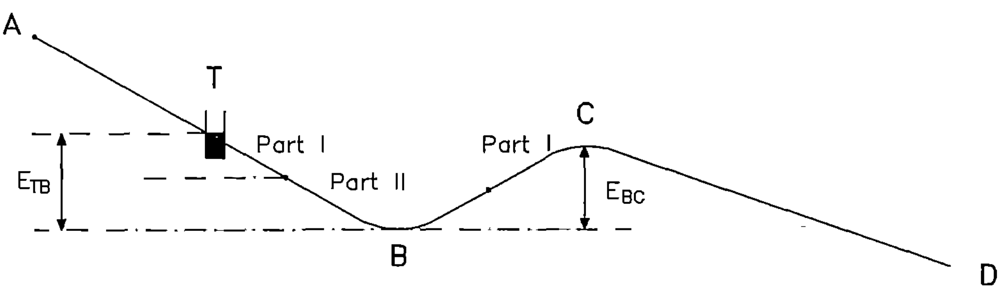


Fig. 4.12 Rising ground slope

The BPT should be located so that at point B, internal pressure created due to static head must not exceed the maximum allowable pressure; and at point C, hydraulic grade line should not fall below the ground elevation (i.e. head available at C must be more than or equal to atmospheric).

Considering portion TBC in fig 4.12, both TB and BC portion of the pipeline may be assumed of consisting of partI and partII of the Type 1 and Type 2 pipes respectively.

Defining $l_{pt\,(ab)}$ = length of pipe 'p' of type 't' in portion 'AB'

$d_{pt\,(ab)}$ = diameter of pipe 'p' of type 't' in portion 'AB'

$h_{Lp\,(ab)}$ = head loss in pipe 'p' in portion 'AB'

The pressure condition at point B may expressed as;

$$E_{tb} - \sum h_{Lp(tb)} \leq h_{p2}$$

$$\text{i.e.}\quad S_{ab}\ (\sum l_{p1\,(tb)} + \sum l_{p2\,(tb)} - KQ^{e1} \sum (l_{p1\,(tb)}/d^{e2}_{p1\,(tb)}) - KQ^{e1} \sum (l_{p2\,(tb)}/d^{e2}_{p2\,(tb)}) \qquad (4.68)$$

and the head condition at C may be expressed as;

$$E_{tb} - \sum h_{Lp\,(tb)} - \sum h_{Lp\,(bc)} - E_{bc} \geq 0$$

$$\begin{aligned}\text{i.e.}\quad S_{ab} \sum l_{p1\,(tb)} + \sum l_{p2\,(tb)} &- KQ^{e1} \sum (l_{p1\,(tb)}/d^{e2}_{p1\,(tb)}) - KQ^{e1} \sum (l_{p2\,(tb)}/d^{e2}_{p2\,(tb)}) \\ &- KQ^{e1} \sum (l_{p1\,(bc)}/d^{e2}_{p1\,(bc)}) - KQ^{e1} \sum (l_{p2\,(bc)}/d^{e2}_{p2\,(bc)}) \\ &- S_{bc}\ (\sum l_{p1\,(bc)} + \sum l_{p2\,(bc)} \geq 0 \qquad (4.69)\end{aligned}$$

By incorporating these constraints into the LP model already presented in the preceding discussion, the situation of higher ground elevation along a pipeline can be tackled.

The next chapter will describe the use of these models in designing some example networks.

CHAPTER 5

5. Application of the Optimization Model:

5.1 INTRODUCTION:

The purpose of this chapter is to illustrate the application of the optimization model developed in the previous chapter. The model will be used to solve a hypothetical example case of a branched network in a hilly terrain. The solution of the model will give the least cost policy and determine the diameters of the commercially available pipes to be used, and the type of pipe materials and lengths to be provided. If break pressure tank(s) should be provided it then also tells the optimum location(s) of the tanks.

The limitation of the model is: firstly, it assumes a uniform ground slope along a single branch, and secondly, for particular number of BPTs, each branch has to be solved separately to obtain the optimum location(s) of the BPT(s) and the corresponding optimum cost of the branch.

All the numerical data such as flows, commercially available pipe sizes, cost coefficients, strength of pipes to resist static pressure etc. are hypothetical. Therefore, these input data and the consequent results should not be considered to represent an existing real world situation. However, an actual technical report on the design of a gravity water supply in the hills of Nepal has been consulted while assuming these data. The sole

purpose of this exercise is to demonstrate the use of the optimization model.

5.2 SUMMARY OF DESIGN CRITERIA AND CONSIDERATION:

The relevant design criteria adopted in the design of a community water supply in the hills of Nepal are summarised below. These criteria are recommended in the Design Guidelines for Rural Water Supply (1990) published by the **Department of Water Supply & Sewerage, Regional Directorate, Western Region, Nepal.** Wherever necessary these criteria will be used in the solution of the hypothetical example problem.

Summary of the Design Criteria:

Minimum flow velocity:

At river intake, if no sedimentation is provided the minimum flow velocity shall be;

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

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

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

Maximum flow velocity:

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

Static Pressure:

Gravity Pipelines

- for PE pipes pressure class 10 kg/cm^2 not more then 100.0m
- for GI pipes pressure class conforming to BS 1387 medium grade not more than 160.0 m

Residual Pressure:

Tapstand	ideal	5 to 10 m
	acceptable	10 to 15 m
BPTs and Storage tanks		10 to 20 m

5.3 ASSUMPTIONS:

The following assumptions are made in the application of the model to solve the example problem.

1. Two types of pipes are available;

   Pipe type 1 : that can withstand a static pressure of up to (h_{p1}) 30.0 m
   Pipe type 2 : that can withstand a static pressure of up to (h_{p2}) 60.0 m

2. The cost of a break pressure tank = NRs. 2000.00

   (NRs.= Nepali Rupee; £ 1.0 = NRs. 70.0 approximately)

3. The cost co-efficient γ_t to be used to calculate pipe cost in the equation (4.3); $C_T = \gamma \; L \; D^{5/4}$ are;

   γ_1 = 0.45, for pipe type 1, and

   γ_2 = 0.65, for pipe type 2.

   These coefficients give cost(C_T) in NRs., when length(L) is in metre and diameter in millimetre.

4. The frictional headloss is given by equation (4.14);

$$h_L = K\frac{L\,Q^{e1}}{D^{e2}}$$

For Hazen-William equation;

$$K=\frac{10.6}{C^{e1}}\ , \quad e1 = 1.85\ , \quad e2 = 4.865$$

From Fig. (3.7), taking C= 145 as an average value.

$K= 10.6 \times 10^{-4}$ and therefore,

$$h_L=10.0 \times 10^{-4}\ Q^{1.85}\frac{L}{D^{4.865}}$$

5. The commercially available diameters are;

10 mm, 15 mm, 20 mm, 25 mm,30 mm, 40 mm, 50 mm, 60 mm,75 mm, 90 mm, 110 mm, 125 mm, 140 mm, 160 mm, 180 mm, 200 mm, 225 mm, 250 mm, 280 mm, 315 mm, 355 mm, 400 mm, and 450 mm.

5.4 HYPOTHETICAL EXAMPLE:

5.4.1 The Basic Data:

Let us consider a branched network as shown in Fig. 5.1, with the data as shown in Table 5.1.

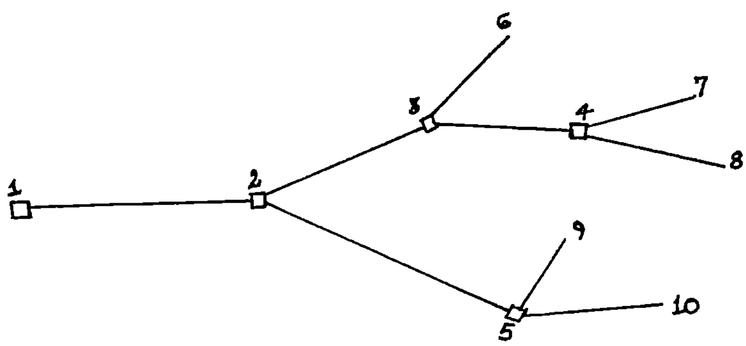


Fig. 5.1 A hypothetical branched network.

Table 5.1 Data for the network shown in Fig. 5.1

Branch	Length (L) m	Flow (Q) x 10^{-3} m^3/s	Elevation difference between starting & end point (E) m	Average slope (S)
1-2	500.0	1.80	75.0	0.150
2-3	275.0	1.10	58.0	0.211
3-4	180.0	0.70	46.0	0.211
3-6	78.0	0.40	23.0	0.295
4-7	80.0	0.40	32.0	0.400
4-8	112.0	0.30	41.0	0.366
2-5	325.0	0.70	63.0	0.194
5-9	62.0	0.40	25.0	0.403
5-10	90.0	0.30	44.0	0.489

5.4.2 Preparation of the Data for Input:

The diameters of pipes for input to the model may be limited to a few by making use of the maximum and the minimum flow velocities. If the maximum flow velocity (V_{max})= 2.5 m/s, and the minimum flow velocity (V_{min})= 0.40 m/s, then the maximum and minimum permissible diameters may be computed respectively by;

$$D_{max} = \sqrt{\frac{4\ Q}{\pi\ V_{min}}}$$

$$D_{min} = \sqrt{\frac{4\ Q}{\pi\ V_{max}}}$$

Knowing the maximum and the minimum permissible pipe sizes, the range of the pipe diameters used as input into the model may be selected from the commercially available diameters.

For each branch of the example network the permissible maximum and minimum diameters are calculated by the above relations, and tabulated in Table 5.2 . The table also shows the commercially available pipe diameters within the maximum and minimum permissible limits.

Table 5.2 The Maximum and Minimum permissible diameters and Commercially available diameters.

Branch	Q x 10^{-3} m^3/s	D_{max} mm	D_{min} mm	Commercially available diameters mm
1-2	1.80	75.7	30.30	75, 60, 50, 40 (total 4 sizes)
2-3	1.10	59.2	23.7	50, 40, 30, 25 (total 4 sizes)
3-4	0.70	47.2	18.9	40, 30, 25, 20 (total 4 sizes)
3-6	0.40	35.7	14.3	30, 25, 20, 15 (total 4 sizes)
4-7	0.40	35.7	14.3	30, 25, 20, 15 (total 4 sizes)
4-8	0.30	30.9	12.4	30, 25, 20, 15 (total 4 sizes)
2-5	0.70	47.2	18.9	as in branch 3-4
5-9	0.40			as in branch 3-6
5-10	0.30			as in branch 4-8

5.4.3 Application:

Branch 1-2

From Table 5.1, Elevation difference between 1-2 (E_{12}) = 75.0 m

Therefore, minimum number of BPT = Integer part of$\{ \frac{E_{12}}{h_{p2}} \}$

= " " { 75.0/60.0}

= 1

Similarly, maximum number of BPTS= Integer part of $\{ \frac{E_{12}}{h_{p1}} \}$

= " " {75.0/ 30.0}

= 2

Slope (S) = 0.150, length(L) = 500.0 m,

Flow (Q) = 1.8×10^{-3} m^3/s

From Table 5.2, commercially available pipe sizes in mm are:

75, 60, 50, 40, (4 sizes).

There are two possibilities regarding the provision of the BPT;

1. Either provide a BPT and find out the optimum policy for this case or,
2. Provide two BPTs and find out the optimum policy for this case.

To find the global optima both cases have to be solved and their optimal policies have to be compared, the minimum of the two will give the global optimum solution for this branch. Now the model will be used to solve both the cases.

Case: I Branch 1-2 with a BPT

The objective function is:

Minimise $C_T = N\,C_t + \sum_{l=1}^{N_l} \sum_{t=1}^{t} [\gamma_t \sum_{p=1}^{n} (l_{ptl}\, d_{pt}^{5/4})]$

where, N = Number of BPT = 1

N_l= Number of links = N+1 = 2

p = number of parts = pipe sizes = 8

t = type of pipes = 2

C_t= cost of a BPT = NRs. 2000.0

γ_t = cost co-efficient, $\gamma_1 = 0.45$, $\gamma_2 = 0.65$

The objective function becomes;

$$\begin{aligned}\text{Minimise } C_T = {} & 2000.0 \\ & + 0.45\ (l_{111}\ 75^{1.25} + l_{211}\ 60^{1.25} + l_{311}\ 50^{1.25} + l_{411}\ 40^{1.25}) \\ & + 0.65\ (l_{121}\ 75^{1.25} + \ldots\ldots\ldots\ldots + l_{421}\ 40^{1.25}) \\ & + 0.45\ (l_{112}\ 75^{1.25} + \ldots\ldots\ldots\ldots + l_{412}\ 40^{1.25}) \\ & + 0.65\ (l_{122}\ 75^{1.25} + \ldots\ldots\ldots\ldots + l_{422}\ 40^{1.25})\end{aligned}$$

$$\begin{aligned} = {} & 2000.0 \\ & + 99.3\ l_{111} + 75.1\ l_{211} + 59.8\ l_{311} + 45.3\ l_{411} \\ & + 143.5\ l_{121} + 108.5\ l_{221} + 86.4\ l_{321} + 65.4\ l_{421} \\ & + 99.3\ l_{112} + \ldots\ldots\ldots\ldots\ldots\ldots + 45.3\ l_{412} \\ & + 143.5\ l_{122} + \ldots\ldots\ldots\ldots\ldots\ldots + 65.4\ l_{422}\end{aligned}$$

Number of Decision Variables = (4x2)x2 = 16

Number of conditions to be satisfied = 7, which are as follows:

Constraints:

1. The static pressure subjected to pipe type 1 in link 1 ≤ 30.0 m

Or $0.150\ (l_{111} + l_{211} + l_{311} + l_{411}) \le 30.0$

2. The static pressure subjected to pipe type 2 in link 1 $\leq$ 60.0 m

Or $0.150\ (\ l_{111} + \ldots\ldots + l_{411} + l_{121} + \ldots\ldots\ldots + l_{421}\ ) \leq 60.0$

3. The static pressure subjected to pipe type 1 in link 2 $\leq$ 30.0 m

Or $0.150\ (\ l_{112} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + l_{412}\ ) \leq 30.0$

4. The static pressure subjected to pipe type 2 in link 2 $\leq$ 60.0 m

or $0.150\ (\ l_{112} + \ldots\ldots + l_{412} + l_{122} + \ldots\ldots\ldots + l_{422}\ ) \leq 60.0$

5. The residual head at BPT $\geq$ 10.0 m

$$0.150\ (\ l_{111} + \ldots + l_{411} + l_{121} + \ldots\ldots\ldots + l_{421})$$
$$- 10.6\text{x}10^{-4}\text{x}(1.8\text{x}10^{-3})^{1.85}\text{x}\{l_{111}\ /(.075)^{4.865} + l_{211}\ /(.06)^{4.865} + \ldots\ldots + l_{411}/(.04)^{4.865}$$
$$+ l_{121}\ /(.075)^{4.685} + \ldots\ldots\ldots\ldots + l_{421}/(..04)^{4.865}\} \geq 10.0$$

Or $0.147\ l_{111} + 0.142\ l_{211} + 0.131\ l^{311} + 0.094\ l_{411}$
$+ 0.147\ l_{121} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + 0.094\ l_{421} \geq 10.0$

6. The residual pressure at node 2 $\geq$ 10.0 m

Or $0.147\ l_{112} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + 0.094\ l_{412}$
$+ 0.147\ l_{122} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + 0.094\ l_{422} \geq 10.0$

7. Total length of the pipes = 500.0 m

Or $l_{111} + \ldots\ldots\ldots\ldots + l_{411} + l_{121} + \ldots\ldots\ldots + l_{421}$
$+ l_{112} + \ldots\ldots\ldots\ldots + l_{412} + l_{122} + \ldots\ldots\ldots + l_{422} = 500.0$

This is a Linear Programming (LP) problem, which may be solved by the Simplex Method. The computer programme EZLP, available at the University of Liverpool, Department of Civil Engineering, which solves LP problem by the simplex method is used to solve the above problem. The input data for the EZLP programme and the computer output of the solution are included in the Appendix. The result of the solution is summarised in Table 5.3.

Table 5.3 Result of LP model solution for case I branch 1-2

Link	Pipe type	Length (m) Size (mm) 40	Total length (m)	Remarks
1	1 2	200 100	200.0 100.0	The BPT is located at the end of link 1
	The BPT is located at 300 m from node 1			
2	1 2	200 -	200.0 -	
			500.0 m	

The optimum cost for case I = NRs. 26,660.00

<u>Case II: Branch 1-2 with two BPTs</u>

The formulation of the LP model and its solution technique are exactly the same as for case I. In this case there are two BPTs which cost 2 x 2000.0 or NRs 4,000. There are three links: therefore, the number of variable = (4 x2) x 3 = 24. Each link should satisfy the static pressure conditions, which means pipe Type 1 and pipe Type 2 in each link should not be subjected to pressure more than h_{p1} and h_{p2} respectively. So there are 3x2 = 6 static pressure conditions to be satisfied. There are three residual pressure conditions to be satisfied: one for each BPTs and one for service node 2. Lastly there is a length condition to be satisfied. Thus the total number of constraints are 10 in this case.

Data for the EZLP computer programme are prepared exactly in the same manner as in the first case, and the result of the LP solution is summarised in Table 5.4

Table 5.4 Result of LP solution for branch 1-2 case II

Lin	Pipe type	Length (m) size (mm) 40	Total length (m)	Remark
1	1 2	193.6 -	193.6 -	BPT-1 is located at the end of link 1
	BPT-1: Located at 193.6 m from node 1			
2	1 2	200.0 -	200.0 -	BPT-2 is located at the end of link 2
	BPT-2: Located at 200 m from BPT-1			
3	1 2	106.4 -	106.4 -	
			500.0	

The optimum cost for case II = NRs. 26,650.00

Now the global optimum policy for the branch 1-2

= Minimum of ; (a) case I : with a BPT = NRs. 26,660.00

(b) case II: with two BPT = NRs. 26,650.00

It is obvious from the cost comparison between case I and case II that the global optimum policy is to provide two BPTs.The types, sizes, and corresponding lengths of the pipes that are to be used are suggested in Table 5.4 The interpretation of the Table is as follows:

From node 1, for the first 193.6 m which is link 1, provide pipe Type 1 of 40 mm diameter. At the end of the link 1 provide the first BPT. In link 2, which starts at BPT-1, provide pipe Type 1 of 40 mm diameter for 200.0 m

Provide the second BPT at the end of link 2. In the third link provide pipe Type 1 of 40 mm diameter for the remaining 106.6 m.

Branch 2-3:

The options available in this branch are:

(a) Branch without BPT, and

(b) Branch with a BPT.

The optimum solution for each of the options is obtained by the same method. The comparison of the optimum solutions for these possibilities suggest that the option without a BPT is cheaper, and then it is the global optimum solution for branch 2-3. Table 5.5 represents the global optimum policy.

Table 5.5 Global optimum policy for branch 2-3

Lin	Pipe type	Length (m) size (mm) 30	25	Total length (m)	Remark
1	1	100.2	42.0	142.2	
	2	-	132.8	132.8	
				275.0 m	

The optimum cost = NRs. 9,045.53

Other Branches:

All the remaining branches are solved by the same method. The options available for each of the remaining branches are listed in Table 5.6. The global optimum policy for each of these branches are summarised in Table 5.7

Table 5.6 Various options for the remaining branches.

Branch	Options available	Global optimum policy	Cost NRs.
2-5	(a) with a BPT (b) with two BPTs	option (a)	9,934.90
3-4	(a) without BPT (b) with a BPT	option (a)	4,455.19
3-6	(a) without BPT	option (a)	1,387.33
4-7	(a) without BPT (b) with a BPT	option (a)	1,292.72
4-8	(a) without BPT (b) with a BPT	option(a)	1,666.79
5-9	(a) without BPT	option(a)	1,017.07
5-10	(a) without BPT (b) with a BPT	option (a)	1,366.04

Total cost for these branch = 21,120.04

Total cost of the pipe network and the BPTs

= cost of branch 1-2

+ cost of branch 2-3

+ cost of the remaining branch

= 26,650.00 + 9,045.53 + 21,120.04

= NRs. 56,815.57

Table 5.7 Policies of the global optimum option for he remaining branches.

Branch	Link	Pipe type	Length (m) sizes (mm) 25	20	15	Total length (m)	Remarks
2-5	1	1	134.9	19.7	-	154.6	a BPT
		2	-	15.7	-	15.7	
	2	1	127.4	27.3	-	154.7	
		2	-	-	-	-	
						325.0	
3-4	1	1	81.5	36.2	-	117.7	
		2	-	62.3	-	62.3	no BPT
						180.0	
3-6	1	1	-	61.4	16.6	78.0	no BPT
		2	-	-	-	-	
						78.0	
4-7	1	1	-	35.0	40.0	75.0	
		2	-	-	5.0	5.0	no BPT
						80.0	
4-8	1	1	-	-	82.0	82.0	
		2	-	-	30.0	30.0	no BPT
						112.0	
5-9	1	1	-	38.8	28.2	62.0	
		2	-	-	-	-	no BPT
						62.0	
5-10	1	1	-	-	61.3	61.3	
		2	-	-	28.7	28.7	no BPT
						90.0	

CHAPTER 6

6. Conclusions and Recommendations

Sustanability:

The advantages of providing an adequate quantity of safe and wholesome water are beyond doubt. However, considering the rural circumstances of the Third World, the realization of the benefits are unlikely unless the water supply schemes are effectively integrated at least with sanitation and hygiene awareness programmes. If full advantage of the water supply schemes are to be derived, and if these schemes are to sustainable , it is essential to see and understand the rural situations in broader perspective of overall development. A water supply scheme as a component of a comprehensive integrated rural development (CIRD) package which covers all aspects of the rural life, is most likely to be both successful and sustainable.

It is an interesting area of a study to develop a CIRD package for a small rural community. A visit to the community with an open mind without any decided plan is deemed necessary to develop such a package. The consultations and the interactions with as wider sections of the community as possible will help establish the villager's needs, and priorities. In fact, an important element of a CIRD package developed by an open mind approach will be the documentation of the villager's needs, wishes, and preferences as perceived by themselves, not imposed from outside.

In addition to the documentation of the villager's view points, due consideration to the socio-economic situations, cultural and traditional values, local skills and organizational infrastructure etc. may be given and an appropriate strategy of the community development may be suggested in the CIRD package.

Optimization:

As the rural water supply schemes are small and simple, the cost of a single such project is low. This is one of the reasons why the designers of these projects are not attracted to the optimal design approach. However, in a developing country hundreds of the rural communities may be in need of these schemes. The total cost of these schemes is substantial, and considering the limitations of the resources, the optimization of the schemes become important.

Various possibilities for the optimization of a water supply scheme exist such as optimization of storage tank, treatment facilities, pipe network, etc. In a water supply system, the cost of the pipes usually constitutes the biggest proportion of the total system cost. The general concepts and strategy for water supply system optimization presented in the dissertation can, if applied, substantially reduce the total cost. The Linear Programming (LP) model presented here can be used for the

optimal design of the branched networks for the gravity water supply schemes.

For the solution of the LP models a help of computer is usually needed. A PC can easily solve the LP models for the branched networks. The PCs are now available at the most of the regional water supply offices in Nepal.

The LP model developed in the dissertation assumes a uniform slope in a branch. In reality, this may not be the case. Therefore, development of a model to tackle the multiple ground slopes will be an useful contribution. There is a need to find out a simplified method of optimization for use in places where PCs are out of reach.

6. Conclusions and Recommendations

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Appendix : A

APPENDIX - A

<u>Input data for the EZLP computer programme</u>.

<u>Branch 1-2</u> <u>Case I: With a BPT</u>

```
16      7    ( Number of  decision variables, and constraints)

-1      30.0 ( Constraint number 1)
0.15    0.15    0.15    0.15 ( coefficients of the decision
0.0     0.0     0.0     0.0    variables)
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0

-1      60.0 ( constraint number 2)
0.15    0.15    0.15    0.15 ( coefficients )
0.15    0.15    0.15    0.15
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0

-1      30.0 ( constraint number 3 )
0.0     0.0     0.0     0.0 ( coefficients )
0.0     0.0     0.0     0.0
0.15    0.15    0.15    0.15
0.0     0.0     0.0     0.0
```

-1	60.0 (constraint number 4)		
0.0	0.0	0.0	0.0 (coefficients)
0.0	0.0	0.0	0.0
0.15	0.15	0.15	0.15
0.15	0.15	0.15	0.15

1	10.0 (constraint number 5)		
0.147	0.142	0.131	0.094 (coefficients)
0.147	0.142	0.131	0.094
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

1	10.0 (Constraint number 6)		
0.0	0.0	0.0	0.0 (coefficients)
0.0	0.0	0.0	0.0
0.147	0.142	0.131	0.094
0.147	0.142	0.131	0.094

0	500.0 (constraint number 7)		
1.0	1.0	1.0	1.0 (coefficients)
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0

```
1       1 ( objective function type )
2000.0    ( Cost of a BPT )
99.3    75.1    59.8    45.3 (coefficients)
143.5   108.5   86.4    65.4
99.3    75.1    59.8    45.3
143.5   108.5   86.4    65.4
```

Branch 1-2 <u>case II: with two BPT</u>

```
24      10 ( Number of decision variables, and the constraints)

-1     30.0 ( constraint number 1 )
0.15    0.15    0.15    0.15 ( coefficients )
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0
0.0     0.0     0.0     0.0
```

-1	60.0		
0.15	0.15	0.15	0.15
0.15	0.15	0.15	0.15
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

-1	30.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.15	0.15	0.15	0.15
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

-1	60.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.15	0.15	0.15	0.15
0.15	0.15	0.15	0.15
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

-1	30.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.15	0.15	0.15	0.15
0.0	0.0	0.0	0.0

-1	60.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.15	0.15	0.15	0.15
0.15	0.15	0.15	0.15

1	10.0		
0.147	0.142	0.131	0.094
0.147	0.142	0.131	0.094
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

1	10.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.147	0.142	0.131	0.094
0.147	0.142	0.131	0.094
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0

1	10.0		
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0
0.147	0.142	0.131	0.094
0.147	0.142	0.131	0.094

0	500.0 (constraint number 10)		
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0
1.0	1.0	1.0	1.0

```
1        1 ( type of objective function )
4000.0     ( cost of two BPT )
99.3     75.1     59.8     45.3 ( coefficients )
143.5    108.5    86.4     65.4
99.3     75.1     59.8     45.3
143.5    108.5    86.4     65.4
99.3     75.1     59.8     45.3
143.5    108.5    86.4     65.4
```

Appendix : B

Appendix- B

Output of the compute solution

Branch 1-2 case I: with a BPT

```
            **** PROGRAM EZLP ****

  SOLUTION OF LINEAR PROGRAMMING PROBLEMS

      A.B.TEMPLEMAN          JULY,1985
            (MODIFIED : NOV89)



THE NUMBER OF VARIABLES IS  16

THE NUMBER OF CONSTRAINTS ( EXCLUDING NON-NEGATIVITY
REQUIREMENTS ) IS   7
```

THE INPUT CONSTRAINT INFORMATION IS:-

CONSTRAINT NO. 1 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.300000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 2 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.600000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 3 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.300000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 4 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.600000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00

CONSTRAINT NO. 5 IS GREATER THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.100000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 6 IS GREATER THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.100000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01

CONSTRAINT NO. 7 IS STRICTLY EQUAL TO A RIGHT-HAND SIDE OF 0.500000E+03

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01

THE OBJECTIVE FUNCTION IS TO BE MINIMIZED AND ITS COEFFICIENTS OF VARIABLES ARE, IN THE INPUT ORDER:-

0.993000E+02	0.751000E+02	0.598000E+02	0.453000E+02
0.143500E+03	0.108500E+03	0.864000E+02	0.654000E+02
0.993000E+02	0.751000E+02	0.598000E+02	0.453000E+02
0.143500E+03	0.108500E+03	0.864000E+02	0.654000E+02

THE OBJECTIVE FUNCTION ALSO CONTAINS A CONSTANT TERM 0.200000E+04 WHICH WILL BE INCLUDED IN ALL CALCULATIONS

PHASE ONE COMPLETED

SOLUTION POINT REACHED

OPTIMUM VALUES OF THE 16 INPUT VARIABLES ARE:-

```
X(1) = 0.000000E+00  X(2) = 0.000000E+00  X(3) = 0.000000E+00    Link 1
     ( size: 75 mm )      ( size: 60 mm)       ( size: 50 mm)    Pipe
X(4) =0.200000E+03                                               Type 1
     ( size: 40 mm )

X(5) = 0.000000E+00  X(6) = 0.000000E+00  X(7) = 0.000000E+00     Pipe

X(8) = 0.100000E+03                                               Type 2
                     Location of BPT.

(9) = 0.000000E+00  X(10) = 0.000000E+00 X(11) = 0.000000E+00    Similarly

X(12) = 0.200000E+03                                             for Link 2.

X(13) = 0.000000E+00 X(14) = 0.000000E+00  X(15) = 0.000000E+00

X(16) =0.000000E+00
```

THE OPTIMUM VALUE OF THE OBJECTIVE FUNCTION IS 0.266600E+05

(Cost in NRs.)

THE ROW NUMBERS OF THE ACTIVE CONSTRAINTS ARE:-

1 3 7

**** END OF SOLUTION ****

SOLUTION TIME = 5.330 SECONDS

Branch 1-2 case II: with two BPT

**** PROGRAM EZLP ****

SOLUTION OF LINEAR PROGRAMMING PROBLEMS

A.B.TEMPLEMAN JULY,1985

(MODIFIED : NOV89)

THE NUMBER OF VARIABLES IS 24

THE NUMBER OF CONSTRAINTS (EXCLUDING NON-NEGATIVITY REQUIREMENTS) IS 10

THE INPUT CONSTRAINT INFORMATION IS:-

CONSTRAINT NO. 1 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.300000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 2 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.600000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 3 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.300000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 4 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.600000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 5 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.300000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 6 IS LESS THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.600000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00
0.150000E+00	0.150000E+00	0.150000E+00	0.150000E+00

CONSTRAINT NO. 7 IS GREATER THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.100000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 8 IS GREATER THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.100000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00

CONSTRAINT NO. 9 IS GREATER THAN OR EQUAL TO A RIGHT-HAND SIDE OF 0.100000E+02

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.000000E+00	0.000000E+00	0.000000E+00	0.000000E+00
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01
0.147000E+00	0.142000E+00	0.131000E+00	0.940000E-01

CONSTRAINT NO. 10 IS STRICTLY EQUAL TO A RIGHT-HAND SIDE OF 0.500000E+03

THE COEFFICIENTS OF THE LEFT-HAND SIDE FUNCTION ARE, IN THE INPUT ORDER:-

0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01
0.100000E+01	0.100000E+01	0.100000E+01	0.100000E+01

THE OBJECTIVE FUNCTION IS TO BE MINIMIZED AND ITS COEFFICIENTS OF VARIABLES ARE, IN THE INPUT ORDER:-

0.993000E+02	0.751000E+02	0.598000E+02	0.453000E+02
0.143500E+03	0.108500E+03	0.864000E+02	0.654000E+02
0.993000E+02	0.751000E+02	0.598000E+02	0.453000E+02
0.143500E+03	0.108500E+03	0.864000E+02	0.654000E+02
0.993000E+02	0.751000E+02	0.598000E+02	0.453000E+02
0.143500E+03	0.108500E+03	0.864000E+02	0.654000E+02

THE OBJECTIVE FUNCTION ALSO CONTAINS A CONSTANT TERM 0.400000E+04 WHICH WILL BE INCLUDED IN ALL CALCULATIONS

PHASE ONE COMPLETED

SOLUTION POINT REACHED

OPTIMUM VALUES OF THE 24 INPUT VARIABLES ARE:-

```
X(1)  = 0.000000E+00  X(2) = 0.000000E+00   X(3) =  0.000000E+00    Link 1
       ( size: 75 mm )      ( size: 60 mm )        ( size: 50 mm)     Pipe
X(4) =  0.193617E+03                                                  Type 1
        ( size: 40 mm )

X(5)  = 0.000000E+00  X(6) = 0.000000E+00   X(7) =  0.000000E+00     Pipe

X(8) =  0.000000E+00                                                  Type 1

              Location of BPT-1                                       Link 2

X(9)  = 0.000000E+00  X(10) = 0.000000E+00  X(11) = 0.000000E+00      Pipe

X(12) =  0.200000E+03                                                 Type 1

X(13) = 0.000000E+00  X(14) = 0.000000E+00  X(15) = 0.000000E+00      Pipe

X(16) =  0.000000E+00                                                 Type 2

              Location of BPT -2                                      Link 3

X(17) = 0.000000E+00  X(18) = 0.000000E+00  X(19) = 0.000000E+00      Pipe

X(20) =  0.106383E+03                                                 Type 1

X(21) = 0.000000E+00  X(22) = 0.000000E+00  X(23) = 0.000000E+00      Pipe

X(24) =  0.000000E+00                                                 Type 2
```

THE OPTIMUM VALUE OF THE OBJECTIVE FUNCTION IS 0.266500E+05

(The cost in NRs.)

THE ROW NUMBERS OF THE ACTIVE CONSTRAINTS ARE:-

3 9 10

**** END OF SOLUTION ****

SOLUTION TIME = 9.289 SECONDS