Design and Optimization of Irrigation Distribution Networks

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FAO Irrigation and Drainage: Paper 44

Published by: Food and Agriculture Organization of the United Nations
Publications Division
Via delle Terme di Caracalla
00100 Rome
Italy

Available from: UNIPUB
4611-F Assembly Drive
Lanham MD 20706-4391
U.S.A.

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FAO IRRIGATION AND DRAINAGE PAPER

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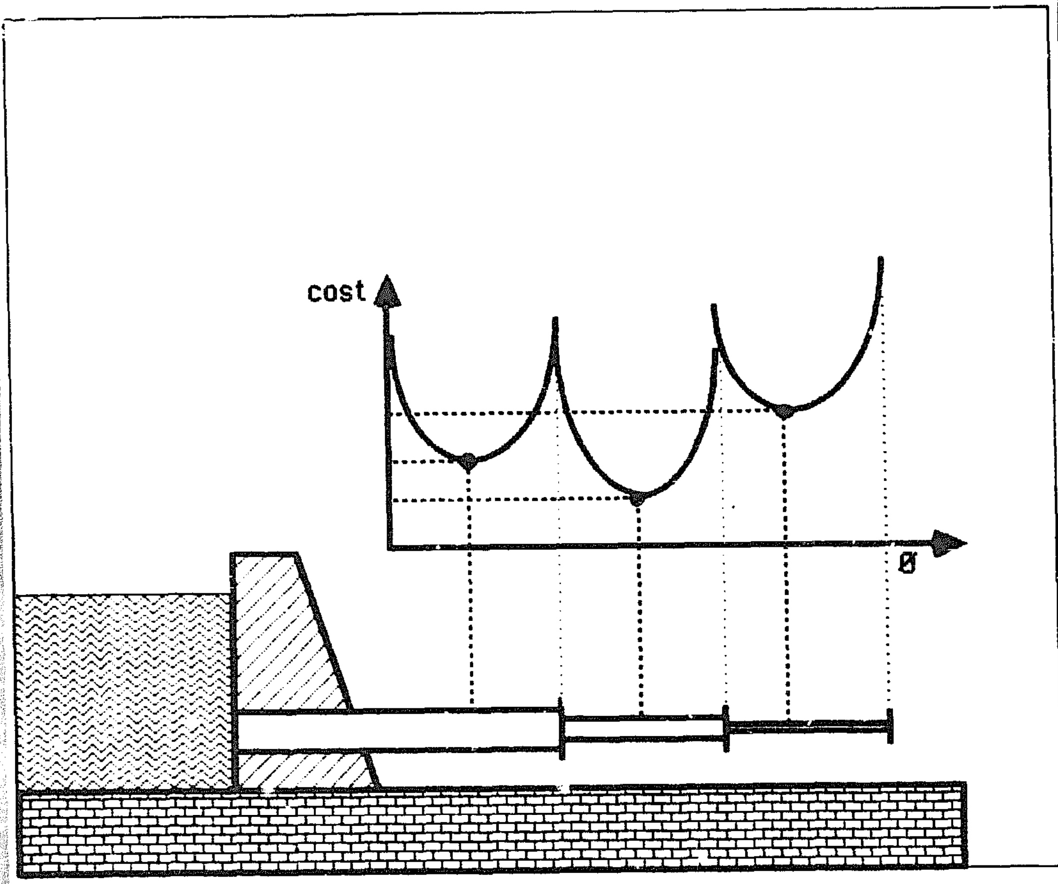

FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS

Design and optimization of irrigation distribution networks

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Rome, 1988

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M-56
ISBN 92-5-102666-1

PREFACE

There is a great deal of information and many schools of thought regarding technical analysis for the optimization of irrigation water conveyance networks.

The Food and Agriculture Organization of the United Nations has been working with leading experts in design techniques for optimization in order to review the approaches and to condense them into a publication for practical application to irrigation.

Techniques and automated procedures developed over the past decades may be modified and refined and new ones may be elaborated. Consequently this publication does not aim to be either a comprehensive or definitive treatise on all aspects of design alternatives. It does, however, seek to provide a recognized procedure for practical application and to give guidance to experts and others concerned with the design of water conveyance systems, who would need to have a degree in engineering with some years of experience in irrigation and its application in the field.

The first three Chapters deal with layout and design criteria for irrigation schemes, and with hydraulic and economic principles, while in Chapters 4 and 5 the methods and mathematical tools for the optimization of pipe and channel systems are developed. Chapter 6 is limited to special considerations on systems protection.

The Food and Agriculture Organization would like to thank all those individuals and organizations who have assisted in the preparation of this edition through advice and contributions.

Particular appreciation is expressed to Mr. X. Pitsaer of Belgium for the computer programming work and to the Senior Editor, Mr. J.C. Henry from France.

Comments and suggestions for improvement of this publication are welcome and should be forwarded to:

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43. Water lifting devices, 1986 (E*)
44. Design and optimization of irrigation distribution networks, 1988 (E* F***)

Availability: April 1988

Ar	—	Arabic	* Available
C	—	Chinese	** Out of print
E	—	English	*** In preparation
F	—	French	
S	—	Spanish	

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M-56
ISBN 92-5-102666-1

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1.1 INTRODUCTION

A well planned, designed and constructed distribution network for irrigation purposes should deliver water in the right quantities, and rate, with the right pressure and at the right time without causing management and operational problems to the water authority or to the consumers. To this end, the distribution system has to incorporate all necessary structural and operational aspects such that the above requirements and any constraints imposed at the source or in other parts of the system are satisfied.

Distribution systems vary greatly in size and complexity, from spreading of flood water over adjacent areas to the conveyance and distribution of surface water or groundwater to areas of intensive agricultural development.

1.2 DEFINITIONS OF NETWORK COMPONENTS

Irrigation distribution networks may include some or all of the following components:

- headworks (intake or diversion structures);
- feeder and main conveyor;
- regulation and protective structures;
- main secondary and tertiary canals;
- bridges, desilting basins, measuring and control equipment and structures;
- pumping stations;
- balancing, storage and pump regulating reservoirs;
- wells;
- any other related equipment or structure required to ensure that the system serves its purpose.

For a better understanding, definitions of the most important components are given below.

- Headworks (intake or diversion structures): Such structures are built at the source of the water supply (river, canal or reservoir) and include intake structures, diversion weirs or dams or any other work designed to withdraw water from the source.

- Feeder: This is a conduit or a channel which takes off from a headwork and feeds a balancing reservoir or a main conveyor directly. This conduit passes through the dead area.

- Main conveyor: This connects either the source of water directly (intake structure or diversion structure) or the feeder conduit with the distribution system. This conduit may be a gravity main or pumping main. Direct supply of water to the farms from this conduit is generally not recommended.

- Distribution system: This is the part of the irrigation system which connects the main conveyor or the feeder or the intake structure to the farm outlets. It is made up of main, secondaries and tertiaries and all ancillary structures and equipment.

However, it must be mentioned that in modern irrigation systems there may be no clear distinction between mains, secondaries and tertiaries, this depending on the size of the network and the mode of water distribution, etc.

- Main: This connects the main conveyor or the feeder or sometimes the source itself to the secondaries or tertiaries. Direct water supply to the farms is generally avoided in this section.

- Secondary: This connects the main with the tertiary and again the direct supply of irrigation water to the farms from this section is not recommended.

- Tertiary: This is connected to a secondary or main and delivers water to the farms.

- Protective structures and equipment: These are structures or equipment whose purpose is to avoid overspilling of water from open canals and overpressures in closed conduits. In canals examples of such structures are emergency spillways or automatic siphons whereas in pipes these are air valves, non return valves, pressure relief valves, etc.

- Irrigation hydrant: This is a composite valve which consists of a pressure reducing valve, a flow limiting device, an isolation valve and a water meter and it is installed at the head of the tertiary conduit or at the farm gate. It may carry up to four independent outlets.

- Farm outlet: This is an outlet on the distribution system serving one farm. The farm outlet is equipped with an isolation valve and often with a water meter.

- Balancing reservoir: This is used for storing excess supply during periods of low demand, the stored water being released during periods of high demand. It may also permit a reduction of the design capacity of the feeder or main conveyor.

- Night storage reservoir: This is a special type of balancing reservoir designed to store the flow of the feeder during the night for release during the day together with the incoming flow. By this method the feeder conduit capacity is reduced, its cost minimized, and night irrigation avoided. Night storage reservoirs usually have a storage capacity equal to eight hours of continuous supply.

- Regulating tank: This is an elevated or ground level reservoir constructed primarily to ensure adequate pressure in all parts of the distribution system and to provide the means of regulating the operation of the pumps. The storage capacity of the reservoir is a function of the type of regulation provided.

- Pressure relief tank: This is a small reservoir, usually concrete and situated at ground level whose object is to reduce the pressue along a long conduit with large differences of elevation between the two ends, thus reducing the pressure in the lower section of the pipeline. These tanks are also installed at dam outlets to maintain a constant pressure downstream. For automatic operation pressure relief tanks are usually equipped with float valves and their size are a function of the pipeline capacities, the float valve operational chacteristics and the pipeline surge limitation.

1.3 DEFINITIONS OF NETWORK PARAMETERS

Although there are no strict and uniform rules to subdivide irrigation schemes and techniques vary from country to country, one way of doing this is given below as an example.

An irrigation scheme can be subdivided into regions, the regions into zones, the zones into sub-zones, the sub-zones into sectors and the sectors into blocks. The blocks are then made up of individual plots or farms. The following definitions may be given of the different components of an irrigation scheme.

- Irrigation Scheme Perimeter: This is the extreme boundary line enclosing the project area. Within this perimeter there may be lakes, rivers, villages, etc.

- Irrigation Region Perimeter: This is the land within the irrigation scheme perimeter which is supplied by a single conveyor. The conveyor may be fed by one or more water supply sources.

- Irrigation Zone Perimeter: This is the line defining the area enclosed in a zone.

- Irrigation Sub-zone Perimeter: This is the line defining the area enclosed in a sub-zone. Two or more sub-zones make up a zone, whereas sub-zones are divided into sectors.

- Irrigation Sector Perimeter: This is the line defining the area enclosed in a sector. Two or more sectors form a sub-zone whereas the sector is divided into blocks.

- Irrigation Block Perimeter: This is the line defining the area enclosed in a block. Two or more blocks make up a sector. A block consists of one or more farms or plots.

According to their suitability or location the areas included in an irrigation project are defined as follows:

- Dead area or external area: A non irrigable area outside the irrigable scheme perimeter and lying between the supply sources and the perimeter of the irrigation scheme.

- Gross scheme area: The total area within the extreme limit of the scheme perimeter.

- Gross irrigable area: The gross area included in the gross scheme area less such areas as are excluded from the project because they are unsuitable for irrigation, either on account of the nature of the soil, or because the ground is too high to be irrigated by gravity of pumping.

- Gross commanded area: The portion of the gross irrigable area which can be commanded by gravity irrigation. In some countries gross commanded area includes land irrigated by pumping. In this Paper unless otherwise stated the gross irrigable area is the area commanded by gravity.

- Gross lift area: The portion of the gross irrigable area which can be irrigated only by lifting the water with pumps or other devices. Gross lift area plus gross commanded area equals gross irrigable area.

- Net irrigated area: The gross irrigable area less the area not available for irrigation due to off-farm operations such as land lost for villages, roads, land terracing, land drainage, land fencing, on-farm storage, on-farm roads, etc.

1.4 IRRIGATION NETWORK CLASSIFICATION

Distribution systems that deliver water to the farms can be of different types depending on the nature of the source of water, the type of main conveyor and the distribution adopted, and whether the system is supplied by gravity or by pumping. One type of classification is given below.

1.4.1 Classification According to Source of Supply

The supply of water to a distribution network may originate from surface run-off, groundwater or from a wastewater recycling system or a combination of two or more types of these sources.

- Surface water scheme: A system supplied with natural clean water from a surface supply, such as a storage reservoir, a river or a lake.

- Groundwater system: The source of water is an underlying or nearby aquifer.

- Conjunctive use system: This is a system that uses both surface water and groundwater supply sources.

- Recycling system: This is system that uses treated wastewater effluents.

- Mixed system: This is a system that uses surface water or groundwater mixed with treated wastewater effluents.

1.4.2 Classification According to Type of Conveyance System

- Open canal systems: In these systems the main conveyor closely approximates to a contour line and is given a very small gradient whereas the secondary follows the main slope of the ground. The tertiary also follows the contour lines but with a very small gradient.

  In order to ensure full control of the water supply, the main and secondary must be provided with regulating gates at the intersection points and at the points of tertiary offtakes.

  Other ancillary structures such as culverts, siphons, bridges, facilitate agricultural activities in the area.

- Pressurized systems: These systems consist of pressurized conduits. They are classified according to the layout and interconnection of the conduits:

  - Branching networks: This system is used in areas where considerable elevation differences are found and where economy is first sought. It is best suited to systems which are to be operated on rotation from the tertiary and when the farm rate is fixed. Continuous distribution may also be considered. Branching networks are sometimes used in modern irrigation systems for the supply of irrigation water on demand, but in this case the outflow at the tertiary must always be controlled so that pressures are kept above the minimum necessary.

- Looped networks: These are mostly used for domestic water supply but may also be adopted in irrigation networks. They are best suited for on demand distribution and where the topography allows for equipressure loops.

  Looped networks offer greater security of supply since the flow to a certain point on the conduit is possible from two directions and in case of a breakdown a reduced supply may be secured from the other end of the loop.

- Mixed type networks: Mixed systems can be of the branching type for the main and secondary consisting of canals or pipes while the tertiary systems can be looped.

  Any combination of canals and conduits can form a network, the choice depending mainly on the topography. In such systems, however, the canals are always placed upstream of the pipes.

1.4.3 Classification According to Pressure Availability

Irrigation distribution systems may deliver water to farm outlets under enough pressure for the operation of a pressurized on-farm irrigation system or at a low pressure to apply the water by furrow or flooding. According to the pressure the irrigation networks can be classified into pressurized and non-pressurized systems:

- Pressurized distribution systems: Those systems deliver water to the farm outlet at a pressure suitable for on-farm application with closed conduits.

  These systems are closed type networks and are equipped with all necessary accessories such as flow limiting devices, pressure regulating devices, water meter, etc. to maintain the pressure between established minimum and maximum levels.

- Non-pressure distribution systems: Are those systems which deliver water to the farm gate at such a pressure that irrigation can be carried out only by surface methods. For irrigation by a pressurized on-farm irrigation system boosting is required.

1.4.4 Classification According to Origin of Pressure

The pressure in a pressurized distribution network may result either from differences of elevation between the source and the supply point or be created by mechanical means. Depending on the origin of the pressure the systems are classified as gravity systems, pumped systems and mixed systems as follows:

- Gravity systems: These systems are characterized by the fact that the supply source lies higher than the irrigated area, i.e. all the irrigated area is commanded by the distribution system. These systems may be pressurized or non-pressurized systems or mixed.

- Pumping systems: Pumps or boosters ensure the flow of water from the source to the farm outlet.

  All groundwater schemes (non-artesian) are of this type as well as many surface and conjunctive use systems where the irrigated area lies higher than the source or where extra head is required.

- Mixed gravity and pumped systems: These are systems where one or more regions are irrigated by pumping.

1.4.5 Classification According to Mode of Distribution

Irrigation water supply entails the artificial control of flow from the source to the farm. It is achieved through the interaction of physical facilities of the system (hardware) and operational management activities (software). However the distribution system must allow for the delivery of a quantity of water to each farm outlet such that the irrigation requirements are satisfied.

In general there are three basic patterns of water distribution to the farms:

- Continuous supply systems: In these systems the supply of irrigation water to the farms or plots is continuous. This type of supply yields the minimum possible design capacity but requires storage at the farm. The rate of water supply to each farm is proportional to its area and the specific discharge required for the crop.

- Supply on rotation: As the term implies, the water required by each individual farm, block or zone is supplied by rotation. There are many variations of water distribution by rotation.

  - Rotation by fixed turn: The water is delivered to the individual farm unit during a fixed period according to the size of the field or the water-right of the field, so that the turn of each holder occurs on a fixed day at a fixed hour for a fixed time.

    This system is applicable where the water is owned by proportions or has to be distributed to the farms according to some definite pattern because of ab-antiquario rights or other arrangemens between the irrigators. It has the advantage that every irrigator knows exactly when and for how long he can use the water. On the other hand, the system has the disadvantage of being inflexible and ill-suited to variations of cropping patterns with different water requirements or frequencies of application.

    The design capacity of the main conveyor is more or less the same as in the continuous supply system but the tertiaries are designed to carry higher rates (modules).

  - Rotation on follow-on system: Water distribution on the rotation on follow-on system (called programmed rotation) is based on the condition that the area to be irrigated is subdivided into blocks, each block assigned a fixed rate of water supply (module) depending on the specific discharge and its surface area. The flow to each block is continuous at the fixed rate and the different farms within the block are supplied with the whole of the flow on rotation, at a predetermined time for a fixed period. Both the timing and the amount of water (period of delivery under constant supply) to be delivered depend on the surface area and the cropping pattern of the farm.

The operation of such a system obviously requires very good programming but the work of irrigation programming may be reduced tremendously when the rotation of supply is based on the tertiary. In such cases the flow in the main, secondary and tertiary conduits is continuous but the water supply to each farm is obtained by rotation of the flow in the tertiary.

This system of water supply is very flexible requiring the minimum irrigation programming by the farmers. However the layout of the system has to be such that the blocks are independant of each other. Furthermore the tertiary pipe design capacity is always larger than for normal rotation.

- Supply on free demand: Water distribution on free demand allows the irrigator a quantity of water which exceeds the amount he is entitled to when the distribution system is on rotation without **a priori** fixing the time and duration of delivery. This system of supply is suitable for areas with small holdings and for highly heterogeneous crops and soils for which the system by rotation would not be practical.

  Inherent with this system is a certain over-capacity as compared to a network operating on rotation.

- Supply on modified demand: This system supplies water to the block on demand and the farmers have to agree to a flexible irrigation programme as on the rotation method. In this case the design of the system is carried out in the same way as in the free demand method but the blocks are treated as individual farms. This system is more economical than the free demand and is best suited for very small farm holdings.

2. BASIC HYDRAULICS AND ECONOMICS

A. HYDRAULICS

2.1 INTRODUCTION

This chapter describes the basic hydraulics, the hydraulic equations and the elements of engineering economics used in the design of irrigation networks.

2.2 CHARACTERISTICS OF WATER

Irrigation water, for all practical purposes, can be considered to have a specific gravity equal to unity since in the range 0 to 30°C the departure from unity is only four per ten thousand.

Depending on the nature of the dissolved minerals and their concentration, water can be corrosive or tend to form scale, a matter discussed at some length in Chapter 6. The quality of water for agriculture is dealt with in detail in FAO (1985).

The temperature of water in an irrigation network will vary according to its source and the ambient conditions. Apart from plant physiological aspects, variations in temperature are important as they determine the value of the kinematic viscosity of the water as shown in Table 1.

Table 1 KINEMATIC VISCOSITY OF WATER (ν)

Temp (°C)	($m^2/s \times 10^{-6}$)	Temp (°C)	($m^2/s \times 10^{-6}$)
0	1.787	25	0.893
5	1.519	30	0.801
10	1.307	35	0.724
15	1.140	40	0.658
20	1.004	45	0.602

For head loss calculations in pipe networks the following equations can be used to determine kinematic viscosity:

from 0°C to 20°C $\log \nu = 14.106 - 8.152 \log T$ (1a)

from 20°C to 50°C $\log \nu = 9.053 - 6.104 \log T$ (1b)

where: ν = the kinematic viscosity (m^2/s)
T = absolute temperature (t°C + 273)

2.3 FLOW OF WATER IN PIPES

2.3.1 State of flow

Flow of water in pipes can be **laminar** or **turbulent** depending upon the value of the Reynolds number (Re). This number is a dimensionless quantity defined as:

$$Re = VD/\nu \tag{2}$$

where: V = mean velocity of flow (m/s)
D = inside diameter of pipe (m)
ν = kinematic viscosity of the fluid (m^2/s)

Observations show that the flow in a pipe is laminar for values of the Reynolds number less than 2000, transitional from laminar to turbulent for values lying between 2000 and 4000 and fully turbulent for values much greater than 4000. In general the flow in irrigation networks occurs in the transitional zone where the state of flow is ill-defined. When flow in a pipe is laminar the maximum velocity is equal to twice the mean velocity, whereas for turbulent flow the maximum velocity is equal to about 1.25 times the mean velocity, due to interchange of momentum between the centre and the periphery.

Flow is said to be **steady** when the discharge is constant with respect to time. If the flow varies with time, as during the emptying of a tank or as the position of a valve is adjusted, flow is said to be **unsteady**. If the discharge varies along the length of a pipe, as is the case in a perforated pipe, then the state of flow is said to be **spatially variable.** If the cross-sectional area of a conduit remains constant, the flow is said to be **uniform** and when the cross-sectional area changes, as in a venturi or orifice, the flow is said to be **non-uniform** or, more often, **varied**.

2.3.2 Flow Relationships

The three basic concepts of fluid mechanics are **continuity, momentum** and **energy**. For steady incompressible non-rotational flow in a pipe the **continuity equation** can be written

$$Q = A_1V_1 = A_2V_2 = A_nV_n \tag{3}$$

where: Q = discharge (m^3/s)
A = cross-sectional area (m^2)
V = mean velocity (m/s)

If the flow is spatially variable then the continuity equation can be written

$$A_1V_1 \pm q\Delta x = A_2V_2 \tag{4}$$

where: q = inflow or outflow per unit length of pipe (m^3/s)
Δx = length of pipe between sections 1 and 2 (m)

For steady flow in a pipe the momentum concept states that the change in momentum flux between two sections is equal to the sum of the forces on the fluid which cause the change. The **momentum equation**, which can be used to determine forces on pipe bends and nozzles for example, is written

$$\Delta F_x = \rho Q \Delta V_x \tag{5}$$

where: F = force (N)
ρ = fluid mass density (kg/m^3)
x = refers to the direction 'x'

The basic principle most often used in hydraulics is the law of conservation of energy. The energy equation is derived by equating the work done on an element of fluid by gravitational and pressure forces to the change in energy. The resulting **energy equation**, known as the Bernoulli equation, can be written (for water)

$$V_1^2/2g + p_1 + z_1 = V_2^2/2g + p_2 + z_2 + h_f \qquad (6)$$

where: $V^2/2g$ = velocity head (m)
p = pressure (m = metres of water column)
g = acceleration due to gravity (m/s^2)
z = elevation above arbitrary datum (m)
h_f = head loss due to friction or turbulence between sections 1 and 2 (m)

In equation (6) the sum of pressure and elevation above the datum is known as the hydraulic head and the sum of hydraulic head and velocity head is the total head. In collective networks for irrigation, the velocity head is generally small compared to other heads and may often be neglected.

The variation of hydraulic and total head along a pipe system is illustrated in Figure 1. Friction loss and local losses are discussed in sections 2.3.3 and 2.3.4 respectively.

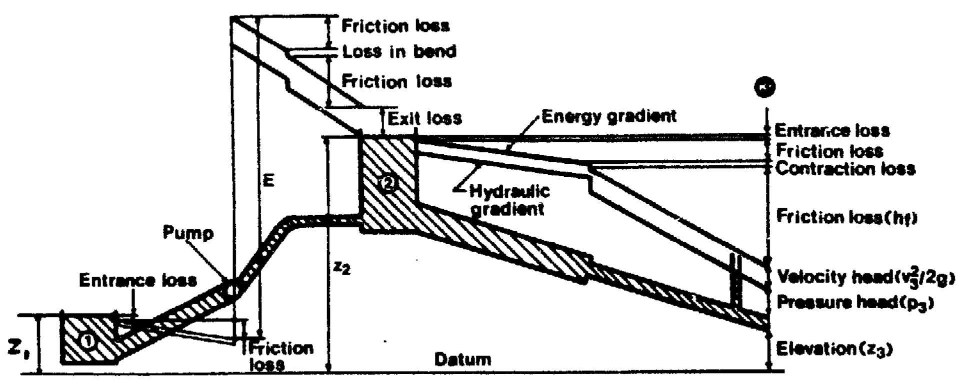


Figure 1 Head loss in a pipe system

Neglecting the very small approach velocities in the two reservoirs, the energy equations for the system can be written:

- From reservoir 1 to reservoir 2

$$z_1 + E = z_2 + h_f + h_m$$

where: E = energy input by pump
h_f = pipe friction loss from 1 to 2
h_m = minor loss in entrance, bend and exit

(The pressure terms are zero since both tanks are at atmospheric pressure.)

- From reservoir 2 to pipe section 3

$$z_2 = z_3 + p_3 + V_3^2/2g + h_f + h_m$$

where: V_3 = velocity at section 3
p_3 = pressure head
h_f = pipe friction from 2 to 3
h_m = minor loss at entrance and at change of pipe diameters

The velocity at any point is determined by applying the continuity equation (Eq. 3).

2.3.3 Head Loss due to Pipe Friction

Ever since liquids have been conveyed in pipes it has been realized that loss of energy or head occurs along the length of the pipe as evidenced by fall of pressure along a horizontal pipeline. Since Chezy first developed in 1775 an equation to express friction slope in terms of velocity many formulae have been proposed. Amongst these the following are still commonly used:

Darcy-Weisbach : $$V = (8gRs_f/\lambda)^{\frac{1}{2}} = (2gDs_f/\lambda)^{\frac{1}{2}} \qquad (7)$$

Manning : $$V = R^{\frac{2}{3}}\, s_f^{\frac{1}{2}}/n \qquad (8)$$

Hazen-Williams : $$V = C_H R^{0.63}\, s_f^{0.54} \qquad (9)$$

where: V = mean velocity (m/s)
R = hydraulic radius = D/4 for full flow pipes (m)
D = pipe diameter (m)
s_f = energy gradient
n, C_H = dimensional friction coefficients
λ = friction factor which is dimensionless

Although simple to use these equations offer the difficult choice of the correct friction coefficient. Manning and Hazen-Williams originally developed their empirical equations from relatively narrow ranges of flow conditions in which the friction coefficients depended only upon the nature of the surface of the pipe. In practice these equations tend to be used beyond the range of the original experiments and it has been shown experimentally that in all three of the above equations the coefficients vary according to both the pipe diameter and velocity of flow as well as the roughness of the pipe wall.

On the basis of experimental data for flow in commercial pipes, combined with the theory of turbulent flow, Colebrook-White (1939) wrote the well-known semi-empirical equation:

$$1/\lambda^{\frac{1}{2}} = -2 \log \left[\frac{k}{3.7D} + \frac{2.51}{Re\,\lambda^{\frac{1}{2}}} \right] \qquad (10)$$

where: λ = friction factor or relative roughness of the Darcy-Weisbach equation = k/D
k = a factor known as roughness height (m)
Re = Reynolds number
D = pipe diameter (m)

An explicit function relating head-loss (s_f) to the friction factor (λ) can be obtained by combining the Colebrook and Darcy equations:

$$1/\lambda^{\frac{1}{2}} = -2 \log \left[\frac{k}{3.7D} + \frac{2.51\nu}{D(2gDs_f)^{\frac{1}{2}}} \right] \qquad (11a)$$

Experiments performed by Lamont (1954) and Ackers (1958) show that this equation defines the flow relationship for a wide range of pipes (or channels if D is replaced by 4R, where R = hydraulic radius) including all the data which served as a basis for the Manning and Hazen-Williams equations, and equation (11a) becomes:

$$1/\lambda^{\frac{1}{2}} = -2 \log \left[\frac{k}{14.8R} + \frac{2.51\nu}{R(128gRs_f)^{\frac{1}{2}}} \right] \qquad (11b)$$

For ease of application of the Colebrook-White function, Barr (1975) developed the following explicit equations which enable a pipe to be designed given any three of the four variables:

- to find V, given k, D and s_f

$$V = -(8gDs_f)^{\frac{1}{2}} \log \left[\frac{k}{3.7D} + \frac{5.02\nu}{D(8gDs_f)^{\frac{1}{2}}} \right] \qquad (12)$$

- to find s_f, given k, D and V

$$s_f = V^2 \Big/ 8gD \left[\log \left\{ \frac{k}{3.7D} + 5.13 \left(\frac{\nu}{VD} \right)^{0.89} \right\} \right]^2 \qquad (13)$$

- to find D, given k, V and S_f

$$D = V^2 \Big/ 8gs_f \left[\log \left\{ \frac{1.558}{(V^2/2gs_f k)^{0.8}} + \frac{15.045}{(V^3/2gs_f \nu)^{0.73}} \right\} \right]^2 \qquad (14)$$

where:
s_f = friction slope
D = pipe diameter (m) - flowing full
V = velocity (m/s)
g = acceleration due to gravity (m/s^2)
ν = kinematic viscosity (m^2/s)
k = roughness height (m)

The Colebrook-White equation expressed in this form is readily solved with hand-held calculators. It is fundamentally sound and covers the whole range of turbulent flows whilst allowing for temperature and should be regarded as the most reliable equation that can be used for the hydraulic design of pipes (HRS 1977). It is not, however, applicable to corrugated culverts or channels with sediment deposit for which the Manning equation will give the correct solution. Representative values of the roughness height (k) are listed in Table 2.

Table 2 ROUGHNESS HEIGHT (k) FOR USE WITH THE COLEBROOK-WHITE EQUATION
(Values of k quoted here in mm must be entered in metres in the equations)

Pipe material	k (mm)	Pipe material	k (mm)
Butt welded steel		**Cast iron**	
Severe tuberculation and incrustation	2.4 to 6.1	New	0.1 to 1.0
General tuberculation	0.95 to 2.4	Bitumen lined	0.03 to 0.2
Heavy brush coated enamels and tars	0.37 to 0.95	Cement lined	0.03 to 0.2
Light rust	0.15 to 0.37	Galvanized	0.06 to 0.2
Hot asphalt dipped	0.06 to 0.15	**Plastic**	0.01 to 0.1
New smooth centrifuged enamels	0.01 to 0.06	**Brass, copper, lead (new)**	smooth
Concrete		**Aluminium**	0.015 to 0.06
New centrifuged	0.03	**Asbestos-cement** (new)	0.03 to 0.1
New smooth moulded	0.2 to 0.5	Manufacturers should be consulted	
New rough moulded	1.0 to 2.0		

The roughness height for a given pipe may vary with time on account of sedimentation, mineral deposits, organic growths, tuberculation and corrosion.

- Sedimentation can generally be avoided by excluding sand and silt and by ensuring that operational velocities are sufficiently high to flush the system.
- Organic growth and slime deposits occur when the water originates from streamflow or reservoirs. Chemical treatments are available to overcome this problem when serious.
- Mineral deposits may occur due to hard water under unstable conditions.
- Active corrosion may be overcome by selecting pipes lined with cement mortar, coal-tar enamel, bitumen, etc. Plastic pipes are usually considered to retain their original flow characteristics.

EXAMPLE 1 - HEAD LOSSES IN PIPES

Calculate the head loss in a 1500 m long cement lined pipe of 300 mm diameter conveying 30 l/s of water at 30°C if the pipe roughness height is 0.1 mm.

From Table 1 the kinematic viscosity (ν) is 0.801×10^{-6} m²/s.

The flow velocity $V = Q/A = 4\,Q/\pi D^2 = 0.424$ m/s.

Solving equation (13):

$$Ls_f = 5.66 \times 10^{-4} \times 1500 = 0.85 \text{ m}$$

(If the water had been at 5°C the head loss would have been 0.93 m.)

2.3.4 Minor Losses

Head loss in pipe networks due to abrupt changes in flow geometry resulting from bends, valves and fittings of all types are generally known as minor losses. Often negligible in long networks, minor losses can become large in short pipe assemblies and can exceed the pipe friction loss.

Minor losses can be expressed in terms of either velocity head (kinetic energy) or equivalent length of straight pipe:

$$h_m = K_m V^2/2g \tag{15}$$

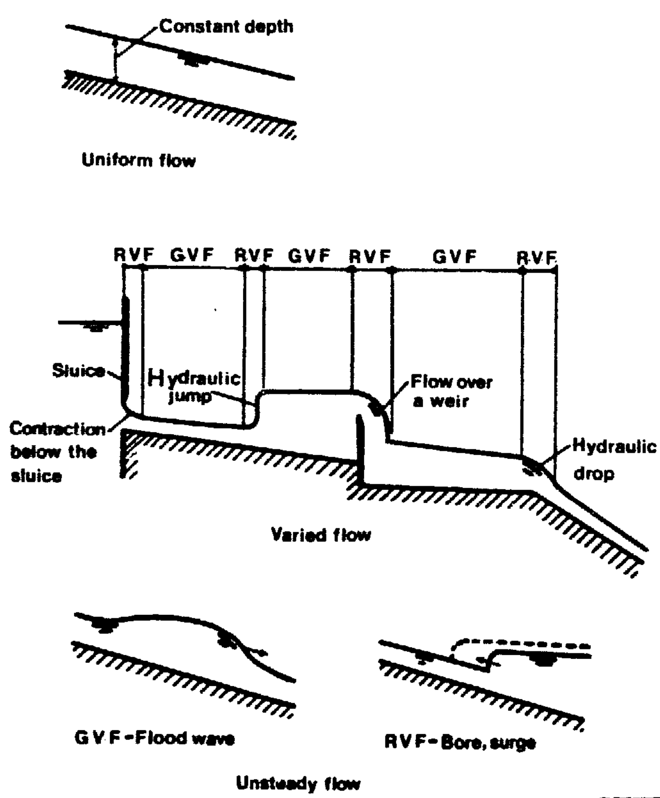
with: h_m = local or minor head loss (m)
K_m = local friction loss coefficient
V = velocity (m/s)
g = acceleration due to gravity (m/s²)

Values of local friction loss coefficients (K_m) are listed in Table 3.

2.4 FLOW OF WATER IN OPEN CHANNELS

2.4.1 Types of Flow

The concepts of flow in pipes can be extended to open channels which are in fact conduits with a free surface. Flow in open channels can be classified as follows (Fig. 2):



1. Steady flow
   a. Uniform flow
   b. Varied flow
      - gradually varied flow
      - rapidly varied flow
2. Unsteady flow
   a. Unsteady uniform flow
   b. Unsteady varied flow
      - gradually varied unsteady flow
      - rapidly varied unsteady flow

Fig. 2 Various types of open-channel flow
GVF = gradually varied flow
RVF = rapidly varied flow
Adapted from Chow (1959)

Table 3 MINOR LOSS COEFFICIENTS ($h_m = K_m V^2/2g$)

		K_m
1.	Tank to pipe	
	pipe projecting into tank	0.80 to 1.0
	pipe flush with wall	0.40 to 0.5
	small radius at orifice	0.1 (radius < 4D)
	large radius at orifice	0.05 (radius > 4D)
2.	Pipe to tank, all cases	1.0
3.	Bends: r = radius, D = diameter (US National Bureau Standards)	
	90° r/D = 1	0.35
	2	0.18
	5	0.20
	10	0.32
	15	0.38
	20	0.42
	60° bend 0.83 K_m, 45° bend 0.7 K_m, 22.5° bend 0.42 K_m (USBR)	
4.	Valves (full opening)	
	- angle	2.5 to 5
	- globe	10
	- swing check (reflux)	0.6 to 2.5
	- gate : 300 mm pipe	0.07
	: 50 mm pipe	0.16
	- needle	0.5
	- butterfly	0.3
5.	Tee, through side outlet	1.8
	through side inlet	1.3
	straight through	0.1
	Elbow, short	0.9
	medium	0.8
	long	0.6
	45°	0.4
	return	2.2
	square	1.27
	Manufacturers' catalogues should be consulted.	

6. Expansion $h_m = K_m V_1^2/2g$

sudden $K_m = (1 - D_1^2/D_2^2)$

gradual K_m from table below, θ = total cone angle

$\theta°$	D_1^2/D_2^2 = 0	0.2	0.4	0.6	0.8	1.0
7.5°	.13	.08	.05	.02	0	0
15°	.32	.24	.15	.08	.02	0
30°	.78	.45	.27	.13	.03	0

Contraction, sudden $h_m = K_m V_2^2/2g$

D_1^2/D_2^2	0	0.2	0.4	0.6	0.8	1.0
K_m	0.5	0.37	0.25	0.15	0.07	0

Source: Stephenson (1981)

V_1 to V_2 and D_1 to D_2 in the flow direction.

2.4.2 State of Flow

The flow of water in open channels is always turbulent in practice, laminar flow occurring only where thin sheets of water flow over the ground or in certain model test channels.

The transition from laminar to turbulent flow occurs when the viscous forces become weak relative to the inertial forces; it is characterized by Reynolds numbers in the range 500 to 2000. For free surface flow the Reynolds number (Re) takes the form:

$$Re = VR/\nu \qquad (2a)$$

where the hydraulic radius (R) is taken as the characteristic length corresponding to the diameter (D) in the case of closed conduits under pressure (Equation 2). The hydraulic radius is defined in Section 2.4.3.

The effect of gravity upon the state of flow depends upon the ratio of inertial forces to gravity forces which is known as the Froude number (Fr):

$$Fr = \frac{V}{(gD)^{\frac{1}{2}}} \qquad (16)$$

where: V = mean velocity (m/s)
g = acceleration of gravity (m/s^2)
D = hydraulic depth (m) = A/T
A = cross-sectional area (m^2)
T = width of the free surface (m)

The Froude number, like the Reynolds number, is dimensionless. When it is equal to unity

$$V = (gD)^{\frac{1}{2}} \qquad (17)$$

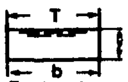
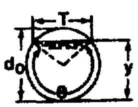
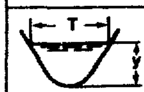
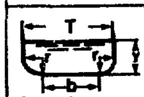
the flow is said to be in the **critical** state. When Fr is less than unity, the flow is said to be **subcritical**; the gravitational force is dominant and the flow is described as streaming or tranquil. When Fr is greater than unity the flow is **supercritical**, the inertial forces are dominant and the flow is described as torrential or shooting.

2.4.3 Channel Geometry

When built with unvarying cross-section and constant bottom slope a channel is said to be prismatic. Normally irrigation canals are prismatic channels with cross-sections of well-defined geometric shape.

The most common cross-section for canals is the trapezoid whose side slopes ensure stability when unlined. Rectangular sections are also common in cross-over, drop or spillway structures or for canals excavated in stable material. Small ditches are frequently triangular and the round-bottom triangle is a form excavated by power shovels. Semi-circular sections are often used for prefabricated distributaries.

The geometric elements of seven distinct channel sections are illustrated in Figure 3 together with their definition in terms of the geometry of the section and the depth of flow. These geometric elements are very important on account of their repeated use in hydraulic computations. Only two need to be defined here, the others are self-explanatory:

Section	Area A	Wetted perimeter P	Hydraulic radius R	Top width T	Hydraulic depth D
 Rectangle	by	$b + 2y$	$\frac{by}{b + 2y}$	b	y
 Trapezoid	$(b + zy)y$	$b + 2y \sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y \sqrt{1 + z^2}}$	$b + 2zy$	$\frac{(b + zy)y}{b + 2zy}$
 Triangle	zy^2	$2y \sqrt{1 + z^2}$	$\frac{zy}{2 \sqrt{1 + z^2}}$	$2zy$	$y/2$
 Circle	$(\theta - \sin\theta)d_o^2 /8$	$\theta d_o/2$	$\left(1 - \frac{\sin\theta}{\theta}\right) d_o/4$	$(\sin \frac{1}{2}\theta)d_o$ or $2\sqrt{y(d_o - y)}$	$\left(\frac{\theta - \sin\theta}{\sin\frac{1}{2}\theta}\right) d_o/8$
 Parabola	$2Ty/3$	$T + \frac{8y^2}{3T}$ *	$\frac{2T^2 y}{3T^2 + 8y^2}$ *	$\frac{3}{2}\frac{A}{y}$	$2y/3$
 Round-cornered rectangle (y>r)	$\left(\frac{\pi}{2} - 2\right) r^2 + (b+2r)y$	$(\pi-2)r + b + 2y$	$\frac{(\pi/2-2)r^2 + (b+2r)y}{(\pi-2)r + b + 2y}$	$b + 2r$	$\frac{(\pi/2-2)r^2}{b + 2r} + y$
 Round-bottom triangle	$\frac{T^2}{4z} - \frac{r^2}{z}(1-z\cot^{-1} z)$	$\frac{T}{z}\sqrt{1+z^2} - \frac{2r}{z}(1-z\cot^{-1} z)$	$\frac{A}{P}$	$2[z(y-r) + r\sqrt{1+z^2}]$	$\frac{A}{T}$

* Satisfactory approximation for the interval $0 < x \leqslant 1$, where $x = 4y/T$.
When $x > 1$, use the exact expression $P = (T/2)[\sqrt{1 + x^2} + 1/x \ln (x + \sqrt{1 + x^2})]$

Source: Chow (1959)

Figure 3 Geometric elements of channel sections

- Hydraulic radius: R = A/P
- Hydraulic depth : D = A/T

where: A is the cross-sectional area (m^2)
P is the wetted perimeter (m)
T is the top width or width of the channel at the free surface (m)

These symbols together with those used in Figure 3 will be retained throughout the present publication.

2.4.4 Velocity Distribution and Measurement of Discharge

Due to wall friction and to the presence of a free surface, the velocities in a channel are not uniformly distributed in the channel cross-section. The maximum velocity usually occurs below the free surface at 0.05 to 0.25 of the depth, this depth increasing as the distance from the bank decreases.

In prismatic channels the vertical velocity distribution is such that the approximate mean velocity occurs at 0.6 of the depth on any vertical. A more accurate determination of the mean velocity on a vertical is obtained by taking half the sum of the velocities measured at 0.2 and 0.8 of the depth. These properties form the basis of the stream-gauging procedure of the US Geological Survey: the average of the mean velocities in any two adjacent verticals multiplied by the area between the verticals is equal to the discharge through the vertical element of cross-section. The total discharge is obtained by summating the discharges through the vertical elements. The mean velocity is, therefore, equal to the total discharge divided by the whole area (Q = V/A).

Because of the nonuniform velocity distribution in an open channel section, the true velocity head should be written $\alpha V^2/2g$ where V is the mean velocity and α is an **energy coefficient** (Coriolis coefficient) whose value increases from 1.03 to 1.36 for straight prismatic channels as the width decreases. Similarly, the momentum in open channels is affected by nonuniform velocity distribution and should be written $\beta wQV/g$ where β is the **momentum coefficient** (Boussinesq coefficient) whose value, for straight prismatic channels varies from 1.01 to 1.12 and w is the unit weight of water. In straight channels these two velocity distribution coefficients are usually assumed to be equal to unity since their effect is small compared to the other uncertainties involved in the flow relationships. They should however be taken into account when dealing with irregular channels such as natural streams or in the vicinity of obstacles.

2.4.5 Flow Relationships

i. Continuity

The continuity equation for free-surface steady flow is the same as for flow in pipes with

$$Q = V_1 A_1 = V_2 A_2 = V_n A_n \tag{3}$$

and if the main canal divides into branches conveying discharges Q_2, Q_3 .... Q_n then

$$Q_1 = Q_2 + Q_3 + \ldots . Q_n$$

with Q_1 = discharge of main canal.

ii. **Energy**

For practical purposes the velocity heads for all points in a channel section can be considered to be equal and the total energy (H) at the channel section is written:

$$H = z + d \cos \theta + \alpha V^2/2g \qquad (18)$$

where:
- z = elevation above a datum (m)
- d = depth perpendicular to the bottom (m)
- θ = slope angle of the channel bottom
- α = energy coefficient
- V = mean velocity (m/s)
- g = acceleration due to gravity (m/s²)

For small slopes the total energy in the section is:

$$H = z + y + V^2/2g \qquad (19)$$

Considering gradually varied flow in a steep channel (Fig. 4) and applying the principle of conservation of energy to the flow in cross sections 1 and 2

$$z_1 + d_1 \cos \theta + \alpha_1 V_1^2/2g = z_2 + d_2 \cos \theta + \alpha_2 V_2^2/2g + h_f \qquad (20)$$

where: h_f = loss of energy.

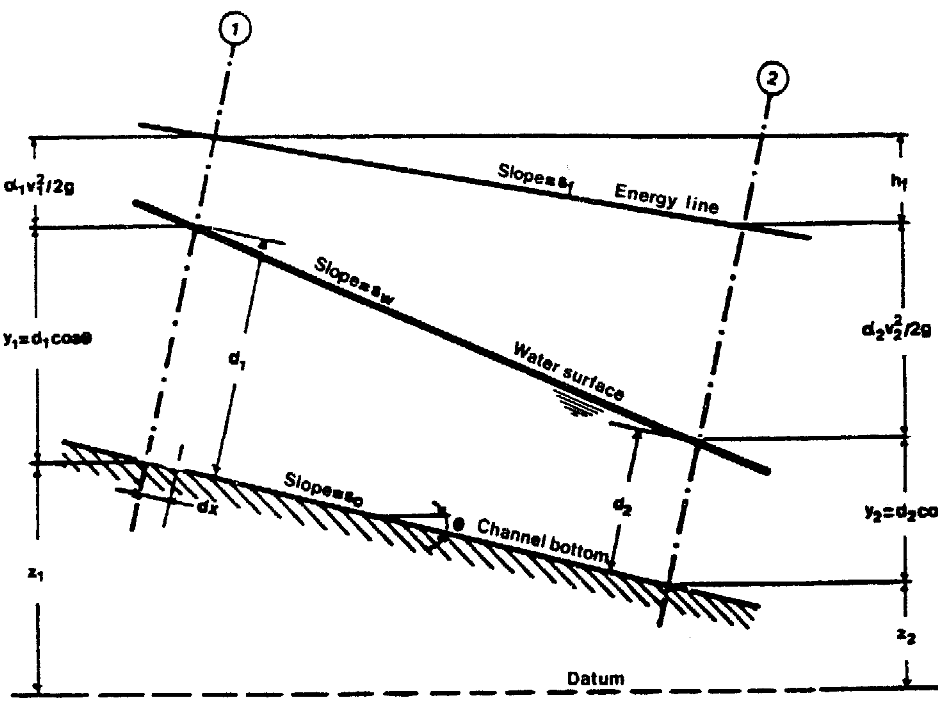


Figure 4 Gradually varied flow in a steep channel

If the channel slope is small then $y = d = d \cos\theta$ and

$$z_1 + y_1 + \alpha_1 V_1^2/2g = z_2 + y_2 + \alpha_2 V_2^2/2g + h_f \qquad (21)$$

which, if $\alpha_1 = \alpha_2$ and $h_f = 0$ then

$$z_1 + y_1 + V_1^2/2g = z_2 + y_2 + V_2^2/2g = \text{constant} \qquad (22)$$

which is the **Bernoulli energy equation.**

Referring to Figure 4 it will be noted that the energy line represents the total head of flow. Its slope is termed the **energy gradient,** denoted s_f. The water surface slope is s_w and the bottom slope $s_o = \sin\theta$. For varied flow each slope has a different value whereas for uniform flow

$$s_f = s_w = s_o = \sin\theta$$

EXAMPLE 2 - USE OF CONTINUITY AND ENERGY EQUATIONS

A rectangular horizontal channel 10 m wide is equipped with a sluice gate and the water levels upstream and downstream of the gate are 5 m and 2 m respectively. Calculate the discharge passing under the gate and define the state of flow in the channel.

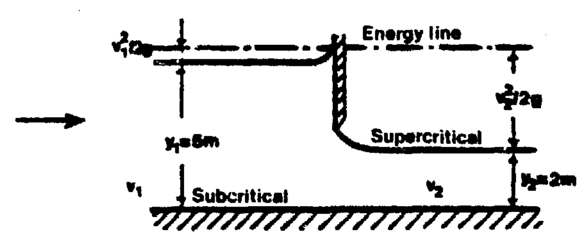


Figure 5 Flow profile

The continuity equation (3) states that

$$50\,V_1 = 20\,V_2$$

and from the energy equation (22)

$$5 + V_1^2/2g = 2 + V_2^2/2g$$

whence $V_2 = 5V_1/2$ and by substitution $V_1 = 3.348$ m/s.

Since $Q = VA$, the discharge $Q = 167$ m³/s.

Upstream of the gate $Fr = V_1/(gy_1)^{\frac{1}{2}} = 0.48$

The flow approaching the gate is subcritical.

Immediately downstream of the gate $Fr = V_2/(gy_2)^{\frac{1}{2}} = 1.89$

Therefore the flow is supercritical. In actual fact a sluice gate always creates subcritical flow upstream and supercritical flow downstream.

iii. <u>Specific energy and alternate depths: critical flow</u>

In the above example, the downstream flow depth was indicated immediately downstream of the gate. This is because two depths of flow may be possible, denoted alternate depths, as will be demonstrated by introducing the fundamental concept of specific energy (E), or energy referred to the channel bed as datum (z = 0)

$$E = y + V^2/2g \tag{23}$$

If q is the discharge per unit width of rectangular channel then q = Q/b = Vy and substituting in equation (23)

$$E = y + q^2/2gy^2 \tag{24}$$

Figure 6 represents a graphical solution of equation (24). It can be seen that given E and q there are two values of y which satisfy the equation (in fact there are three but the third is negative and unreal). When two depths of flow are possible for a given E and q, they are referred to as alternate depths. Each represents a different regime: the upper corresponds to subcritical flow, the lower to supercritical flow. The crest of the E-y curve corresponds to the point of minimum energy; it coincides with the occurrence of critical flow.

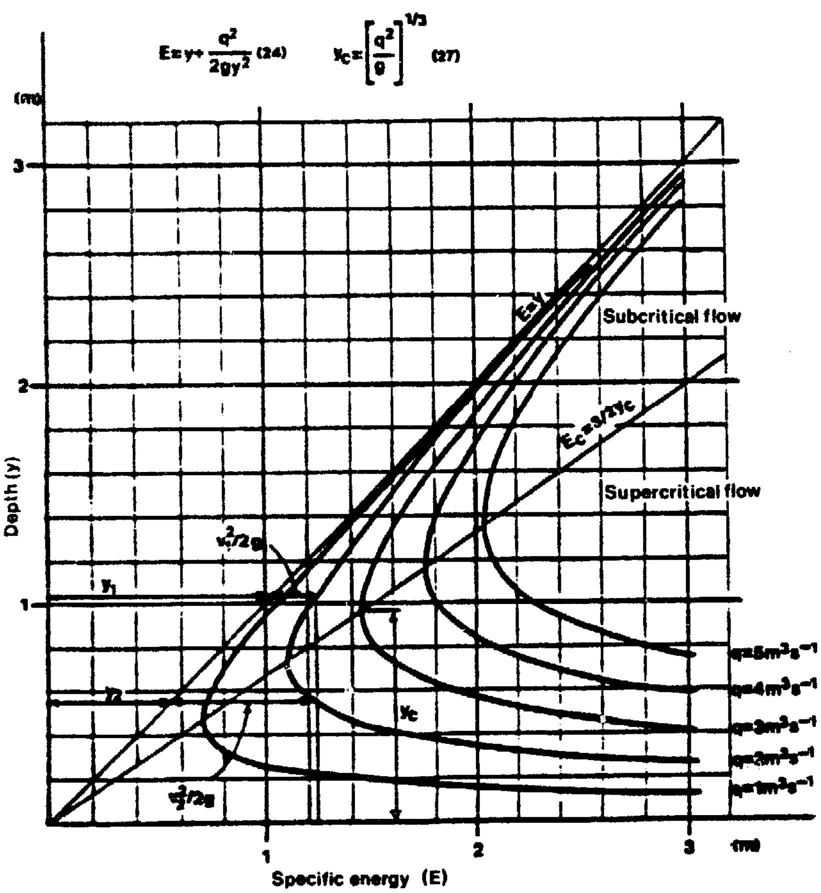


Figure 6 Specific energy curve in a rectangular channel

Differentiating Eq. (24) with respect to y and equating to zero

$$q^2 = gy^3 \qquad (25)$$

hence
$$V_c^2 = gy_c \qquad (26)$$

where the subscript c indicates critical flow conditions.

Rewriting Eq. (25)

$$y_c = (q^2/g)^{\frac{1}{3}} \qquad (27)$$

and from Eq. (26)

$$V_c^2/2g = \tfrac{1}{2}y_c$$

which produces

$$E_c = y_c + V_c^2/2g = \tfrac{3}{2}y_c$$

and
$$y_c = \tfrac{2}{3}E_c \qquad (28)$$

For all channels the criterion for critical flow is

$$V^2/2g = D/2 \quad (29) \qquad \text{or} \qquad Q^2/g = A^3/T \quad (30)$$

Table 4 CRITICAL FLOW FORMULAE

Element	Channel section: General	Rectangular	Trapezoidal	
E_c	$y_c + D/2$	$\frac{3}{2}y_c$	$\dfrac{(3b + 5zy_c)y_c}{(2b + 4zy\ )}$	(31)
depth	$D_c = V_c^2/g$	$y_c = V_c^2/g$	$y_c = \dfrac{V_c^2}{g} - \dfrac{b}{2z} + \left[\dfrac{b^2}{4z^2} + \dfrac{V_c^4}{g^2}\right]^{\frac{1}{2}}$	(32)
	$= Q_c^2/A^2g$	$= (q_c^2/g)^{\frac{1}{3}}$		(32a)
		$= \frac{2}{3}E_c$		(32b)
		$= [(Q/b)^2/g]^{\frac{1}{3}}$		(32c)
V_c	$(gD_c)^{\frac{1}{2}}$	$(gy_c)^{\frac{1}{2}}$	$\left[\left(\dfrac{b + zy_c}{b + 2zy_c}\right)gy_c\right]^{\frac{1}{2}}$	(33)
		$(gq_c)^{\frac{1}{3}}$		(33a)
		$(gQ_c/b)^{\frac{1}{3}}$		(33b)
Q_c	$A(gD)^{\frac{1}{2}}$	$y_c^{\frac{3}{2}} \quad bg^{\frac{1}{2}}$	$y_c^{\frac{3}{2}}\left[\dfrac{g(b + zy_c)^3}{b_c + 2zy}\right]^{\frac{1}{2}}$	(34)
		$\left(\frac{2}{3}E_c\right)^{\frac{3}{2}} \quad bg^{\frac{1}{2}}$		(34a)

EXAMPLE 3 - USE OF SPECIFIC ENERGY CURVE

A. EFFECT OF STEP IN CHANNEL BED

A rectangular channel 10 m wide conveys a flow of 10 m^3/s with a depth of 1 m. Determine the effect upon the water level of a short step 30 cm high placed on the channel bed (Fig. 7). Assume that there is no friction loss. Calculate the height of a step to achieve critical flow.

The specific energy curve is drawn for $q = Q/b = 1\ m^2/s$. At the original depth the specific energy (E_1) is:

$$E_1 = y_1 + V_1^2/2g = 1 + 0.05 = 1.05\ m$$

Raising the bed by 0.30 m, q remains constant and:

$$E_2 = 1.05 - 0.30 = 0.75\ m$$

and from the specific energy curve

$$\text{for } E_2 = 0.75\ m \qquad y_2 = 0.62\ m$$

which corresponds to a depth of 0.92 m above the original bed level. The effect of the step is to drop the water surface profile by 8 cm. Since the flow over the step does not attain the critical depth, the water surface rises to its original level beyond the step.

From Eq (27)

$y_c = (q^2/g)^{\frac{1}{3}} = 0.47\ m$ and $E_c = \frac{3}{2} y_c = 0.70\ m$

if Δz_c is the height of the "critical step",

then $E_c = 1.05 - \Delta z_c = 0.70\ m$ hence $\Delta z_c = 0.35\ m$

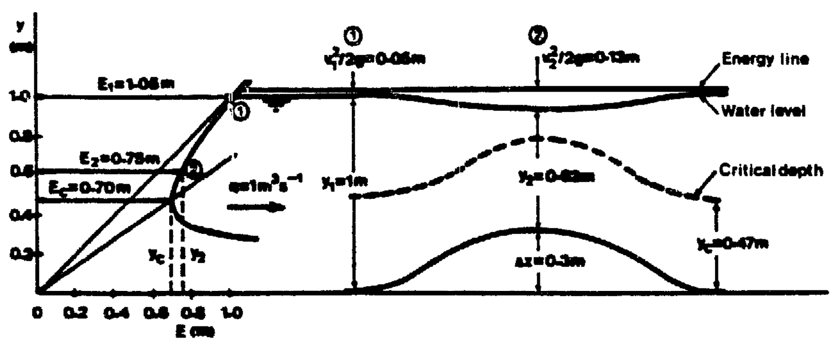


Figure 7 Use of specific energy curve to determine water surface profile resulting from step in channel bed

B. EFFECT OF CONTRACTION IN CHANNEL WIDTH

In the channel of Ex 3A the step is replaced by a smooth contraction in width from 10 m to 8 m. Determine the effect upon the water level and calculate the width to achieve critical flow.

In this case $q_2 = 10/8 = 1.25\ m^3/s$

and $E_1 = E_2 = y^2 + V_2^2/2g = y_2 + q_2^2/2gy_2 = 1.05\ m$

Solving graphically $y_2 = 0.96$ m hence the effect of the contraction is to drop the water level by 4 cm only. The critical width can be determined from Eq (27) and (31) assuming a constant energy level

$$y_c = (q_c^2/g) = \tfrac{2}{3}E = 0.70\ m$$

which yields $q = 1.83\ m^3s$

and since $q = Q/b$ $\quad b = 5.46\ m$

It was demonstrated in the two cases examined in Example 3 that channel constrictions can bring about a change from subcritical to critical flow. Any further reduction of channel width or depth beyond that which achieves critical flow will result in a backing up of the water level upstream with either a reduction of the discharge per unit width (q) or an increase of the specific energy (E_1) depending upon the upstream conditions. In other words, when the obstruction becomes larger than that which achieves critical flow, the flow can no longer be maintained with the available specific energy.

The state of flow downstream of the critical flow constriction (gate, step, convergent) will obviously depend upon hydraulic conditions further down the channel. If due to channel friction or the presence of another obstacle, the flow cannot remain supercritical, the change of depth will take place more or less abruptly through what is known as a hydraulic jump. This change involves both the energy equation and the momentum equation since it occurs with turbulence and energy loss.

iv. Momentum

According to Newton's second law of motion, the change of momentum per unit of time in the body of water in a flowing channel is equal to the resultant of all the external forces that are acting on the body. This principle, applied to flow between cross-sections 1 and 2 is expressed by the **momentum equation** written

$$\frac{Qw}{g}(\beta_2 V_2 - \beta_1 V_1) = P_1 - P_2 + W \sin\theta - F_f \qquad (35)$$

where: P_1 and P_2 = resultants of pressures acting on the two sections

W = weight of water enclosed between the sections

F_f = total external friction force acting along the surface of contact between the water and the channel

w = unit weight of water

In a short reach of a rectangular channel of small slope and length L

$$P_1 = \tfrac{1}{2} w\, b\, y_1^2$$

$$P_2 = \tfrac{1}{2} w\, b\, y_2^2$$

and if $F_f = w\, h_f'\, b\, \bar{y}$

where h_f' = friction head

$\bar{y} = (y_1 + y_2)/2$

also $Q = \tfrac{1}{2}(V_1 + V_2) b\bar{y}$

and since $W = w\, b\, \bar{y} L$

and $\sin\theta = (z_1 - z_2)/L$

by substituting in Eq. (35)

$$z_1 + y_1 + \beta_1 V_1^2/2g = z_2 + y_2 + \beta_2 V_2^2/2g + h_f' \qquad (36)$$

This equation differs from the energy equation (Eq. 21) only in that the momentum coefficient (β) replaces the energy coefficient (α) and that h_f' measures the losses due to external forces whereas h_f measures the losses due to internal forces. It is important to distinguish between energy, a scalar quantity, from momentum, a vector quantity, when solving problems in hydraulics.

v. Specific Force and sequent depths: critical flow

In a short horizontal prismatic channel the external force of friction and the weight effect of water can be ignored and Eq. (35) can be written:

$$\frac{Qw}{g}(V_2 - V_1) = P_1 - P_2$$

substituting $w\bar{y}$ for P and Q/A for V where $\bar{y}$ = distance between water surface and centre of water area (A)

$$Q^2/gA_1 + \bar{y}_1 A_1 = Q^2/gA_2 + \bar{y}_2 A_2 \qquad (37)$$

or

$$F = \frac{Q^2}{gA} + \bar{y}A \qquad (38)$$

where F is the **specific force.**

Plotting depth against specific force for a given channel section and discharge the specific force curve, analogous to the specific energy curve, can be drawn. It can be shown by differentiating F with respect to y and equating to zero that the minimum value of the specific force occurs at the critical depth.

At the critical depth:

$$F_c = \tfrac{3}{2} y_c^2 \qquad (39)$$

and since

$$E_c = \tfrac{3}{2} y_c \qquad (28)$$

the relation between critical specific energy and critical specific force in a rectangular channel follows

$$F_c = E_c\, y_c \qquad (40)$$

As with the specific energy curve, two possible values of depth are forthcoming from the specific force curve except at the critical depth. These are known as the **initial** and **sequent depths** or **conjugate depths.**

2.4.6 The Hydraulic Jump

The change from supercritical flow produced by an upstream control (gate, chute, etc.) to subcritical flow produced by a downstream control (gate, sill, etc.) can take place abruptly in the form of a hydraulic jump. This change is accompanied by turbulence and a large internal energy loss as illustrated in Figure 8.

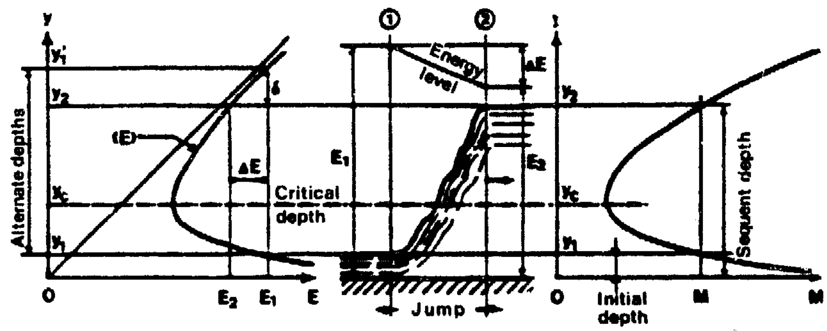


Figure 8 Loss of total energy in hydraulic jump
Adapted from Crausse (1951)

Since this internal energy loss cannot be evaluated in the energy equation, the momentum principle is used to analyse the problem. As the external forces of friction and the weight effect of water in a hydraulic jump on a horizontal channel are negligible, the specific forces before and after the jump are considered to be equal.

$$Q^2/gA_1 + \bar{y}_1 A_1 = Q^2/gA_2 + \bar{y}_2 A_2 \qquad (37)$$

since for a rectangular channel

$$Q = V_1 A_1 = V_2 A_2, \quad A_1 = by_1 \text{ and } A_2 = by_2$$

$$\bar{y}_1 = y_1/2 \text{ and } \bar{y}_2 = y_2/2$$

and

$$Fr = V/(gy)^{\frac{1}{2}} \qquad (16)$$

$$(Fr = \text{Froude number})$$

it follows that

$$Fr_1^2 = \tfrac{1}{2} y_2/y_1 (y_2/y_1 + 1) \qquad (41)$$

this quadratic is solved by

$$y_2/y_1 = \tfrac{1}{2} [(1 + 8 Fr_1^2)^{\frac{1}{2}} - 1] \qquad (42)$$

or if the downstream conditions are known

$$y_1/y_2 = \tfrac{1}{2} [(1 + 8 Fr_2^2)^{\frac{1}{2}} - 1] \qquad (43)$$

The energy loss in the jump can be written (rectangular channels):

$$\Delta E = (y_2 - y_1)^3/4y_2 y_1 \qquad (44)$$

$$\Delta E = \frac{y_2 y_1}{2(y_1 + y_2)} \left[\frac{y_2 - y_1}{y_c}\right]^3 \qquad (45)$$

i. Energy loss in hydraulic jump in non-rectangular channel sections

There are no simple equivalents of Eq. (41 to 43) for non-rectangular channels but the loss of energy in a hydraulic jump can be determined graphically by plotting both the specific energy and specific force curves (Fig. 8). On either side of the jump the specific forces are equal and are seen to correspond to the specific energy values E_1 and E_2. The energy loss (Δz) in the jump, shown by the drop of the energy line is equal to E_1-E_2. When plotting the force curve for a non-rectangular section, the moment of area about the water surface must be determined by numerical or graphical integration. For a trapezoidal section the term ($\bar{y}A$) in Eq. (37) becomes:

$$\bar{y}A = (2zy + 3b)y^2/6 \qquad (46)$$

ii. Occurrence of hydraulic jump subject to downstream conditions

Whether a hydraulic jump can exist or not depends not only on the occurrence of supercritical flow upstream but also upon whether the downstream control will permit the jump to form.

In Figure 9 for instance, flow under gate A is supercritical and changes to subcritical due to the setting of gate B. In this case no jump can form because the depth y_2 is equal to or greater than the sequent depth of the depth y_o. The jump is drowned and equations (41) to (43) do not hold in view of the contact of the roller with gate A.

Below gate B a jump can form since the depth y_2 resulting from the setting of gate C is less than the sequent height of the depth y_o. In this case the jump forms and is located in a position determined by the imposed sequent depth (y_2) and the resulting initial depth (y_1) to satisfy Eq. (43).

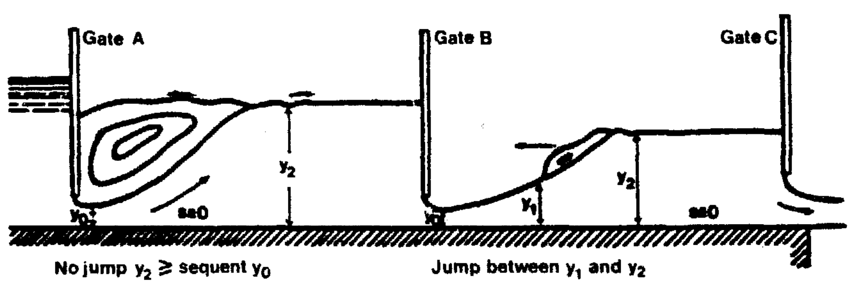


Figure 9 Dependency of hydraulic jump upon downstream conditions

iii. Position of hydraulic jump

In theory a hydraulic jump occurs in a horizontal rectangular channel when the initial and sequent depths and approaching Froude number satisfy Eq. (42) or (43) but in practice its location also depends on the length of the jump. For a complete discussion of the hydraulic jump and its application as an energy dissipator, the reader is referred to Crausse (1951), Chow (1959), Henderson (1966).

iv. Length of hydraulic jump

Unlike its height, the length of a jump cannot be determined analytically and reliance must be placed upon experimental values. In horizontal rectangular channels the length of an ordinary jump is often asumed to be vary from 5 to 7 times its height. In non-rectangular channels the jump tends to be longer. The length of jumps in terms of sequent depth and Froude number as determined from data and recommendations of the US Bureau of Reclamation (USBR 1955) are reproduced in Fig. 10. (See also Stilling basins - Chapter 6).

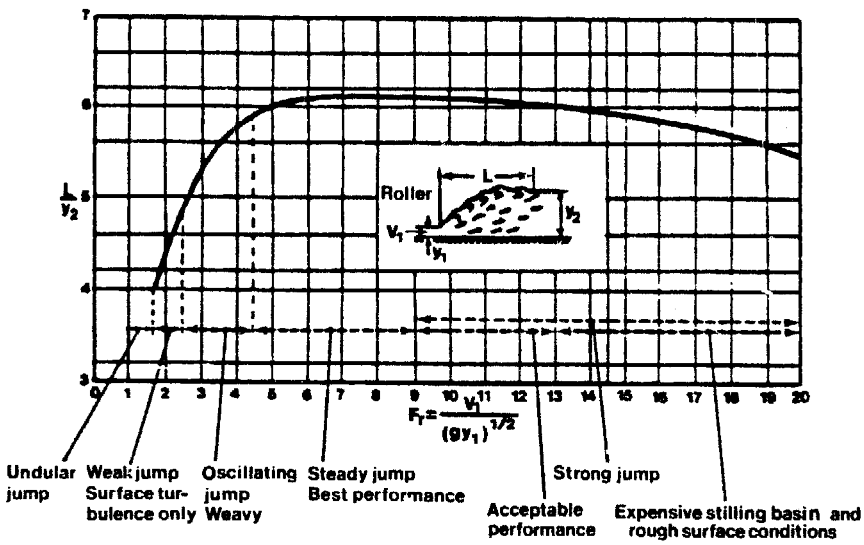


Figure 10 Length in terms of sequent depth y_2 of jumps in horizontal channels
Source: USBR in Chow (1959)

v. Surge

The surge or hydraulic bore is an example of rapidly varied flow. Equations to determine surge heights and velocities are presented in Section 6.2.3.

2.4.7 Flow Resistance due to Friction

i. Uniform flow

Flow in a channel is said to be uniform when the depth, water area, velocity and discharge at every section of the channel reach are constant and the channel bottom, water level and energy level are all parallel. When these conditions hold the gravity forces which cause the water to move down the sloping channel are exactly balanced by the friction forces of the channel bottom and walls.

a. **Velocity of flow**

As early as 1775, the French engineer Chézy proposed an expres-

EXAMPLE 4 - NORMAL AND CRITICAL DEPTH IN UNIFORM FLOW

A trapezoidal channel has the following characteristics

$b = 4$ m $\quad z = 1 \quad n = 0.015 \quad s = 0.001$

Determine the normal depth and critical depth for uniform flow with a discharge of 25 m³s and show that for the given slope and roughness values the flow in the channel is subcritical at all discharges.

Since $\qquad V = R^{\frac{2}{3}} s^{\frac{1}{2}}/n \qquad (48)$

hence $\qquad Q = AR^{\frac{2}{3}} s^{\frac{1}{2}}/n$

from Fig. 3 we have for a trapezoidal section

$$R = \frac{(b + zy)y}{b + 2y(1 + z^2)^{\frac{1}{2}}} \quad \text{and} \quad A = (b + zy)y$$

substituting $b = 4$ and $z = 1$

$$R = \frac{(4 + y)y}{4 + 2^{\frac{3}{2}}\ y} \quad \text{and} \quad A = (4 + y)y$$

The normal depth (y_0) is the value of y which by trial and error is found to yield values of (A) and (R) which satisfy the Manning formula. This value is

$$y_0 = 1.835 \text{ m for } Q = 25 \text{ m}^3/\text{s}$$

When several different discharges are likely to be examined it is expedient to plot the normal stage discharge relation $Q(y_0)$ as shown in Fig. 11 together with the variation of the cross-sectional area (A), the hydraulic radius (R) and the normal velocity (V_o).

The criterion for critical flow is:

$$V_c^2/2g = D/2 \qquad (29)$$

$$\text{now } D = A/T = \frac{(4 + y)y}{4 + 2y}$$

$$\text{and } V = Q/A = \frac{Q}{(4 + y)y}$$

substituting in Eq (29)

$$Q^2(4 + 2y_c) = [(4 + y)y_c]^3 g$$

Solving by trial and error for Q = 25 m³/s the critical depth (y_c) is found to be 1.40 m. Since $y_c < y_0$ the flow is subcritical at the normal depth. Values of the relation $Q(y_c)$ are shown on Fig. 11 and it can be seen that flow in the channel remains subcritical throughout the range of stage. The $Q(y_c)$ curve is obtained by plotting the critical depth (y_c) as a function of

$$Q_c = (gA^3/T)^{\frac{1}{2}} = g\left[\frac{[(4 + y)y]^3}{(4 + 2y)}\right]^{\frac{1}{2}} \qquad (30)$$

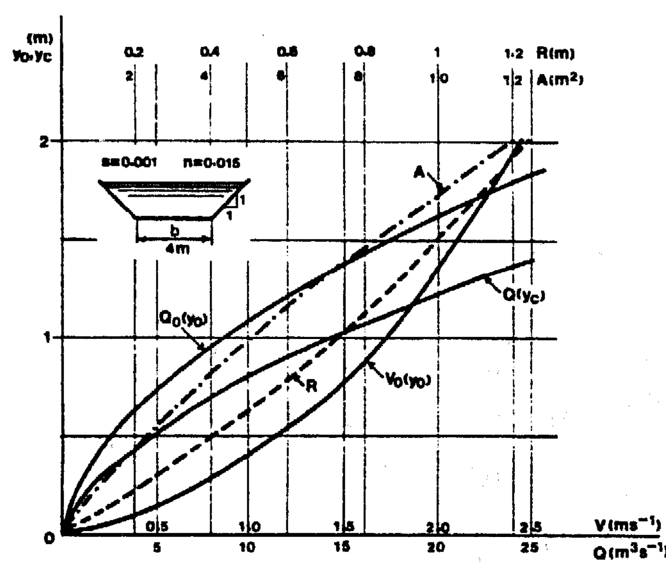


Figure 11 Stage - discharge, velocity and critical depth curves in a trapezoidal channel

sion which related the square of the mean velocity to the slope and the hydraulic radius of the channel and which is usually written:

$$V = C(Rs)^{\frac{1}{2}} \qquad (47)$$

where C is a factor of flow resistance. Although this equation is still employed (with expressions for C developed notably by Ganguillet and Kutter in 1869 and by Bazin in 1897) it is common practice to adopt the well-known formula presented by the Irish engineer Manning in 1889:

$$V = R^{\frac{2}{3}} \; s^{\frac{1}{2}}/n \qquad (48)$$

where n is a roughness coefficient generally known as Manning's n.

This formula (sometimes known as the Strickler formula with n in the numerator) is used throughout the world and was recommended for use by the Executive Committee at the Third World Power Conference in 1936.

As can be seen from the formula it is of the utmost importance to select the correct value of the roughness coefficient. This requires sound engineering judgement, particularly when dealing with natural streams or unlined channels excavated in rough heterogeneous materials. In the absence of experience the reader is referred to Chow (1959) where values of n are tabulated for a very wide range of conditions together with a series of photographs illustrating these conditions. Roughness coefficients are also listed here in Chapter 5.

The uniform flow formula is of considerable importance since it determines the minimum cross-sectional area required to pass a given discarge for a given lining material and a given slope. Its use is illustrated by examples 4 and 5.

The relationship between slope, depth, channel cross section and discharge for uniform flow is illustrated in Fig. 12 where:

- limiting discharge curves indicate the variation of discharge with channel slope;

- critical slope curves indicate the transition from subcritical to critical flow.

It may be seen that in the case of the rectangular channel the critical slope curve has a limit at $s_c = 0.00348$. This is the limit slope or smallest possible critical slope. Limit slopes also exist for trapezoidal channels when $z < 1$. It should be noted that when the critical slope curve has a limit there is a range of slopes for which a decrease in the discharge will result in a change of regime from subcritical to critical and vice versa.

The critical slope curve is obtained by solving Eq. (17) for Q = VA with assumed values of y, substituting y and Q (= VA) in the Manning formula (Eq. 48) and solving for s_c.

When solving uniform flow problems it is of great importance to select the correct value of the roughness coefficient (n) since the velocity and hence the discharge are inversely proportional to n and the slope is directly proportional to n^2.

EXAMPLE 5 - NORMAL AND CRITICAL SLOPES IN UNIFORM FLOW

1 - Determine the slope required to convey 25 m^3/s with a normal depth of 1.5 m and check the state of flow in:

a. a trapezoidal channel with $n = 0.015$ $b = 4$ m $z = 1$
b. a rectangular channel with $n = 0.015$ $b = 4$ m

2 - For both channels determine critical slope and discharge for the normal depth of 1.5 m.

1. The slope to convey a given discharge is found by rearranging the Manning formula (Eq 48)

$$s = n^2Q^2/AR^{\frac{4}{3}} \tag{48a}$$

a. For the trapezoidal channel with $y = 1.5$, $b = 4$ and $z = 1$

$$R = \frac{(4 + 1.5)1.5}{4 + (2)^{\frac{3}{2}}\ 1.5} \quad \text{and} \quad A = (4 + 1.5)1.5$$

hence $s_o = 0.00206$ (Point A in Fig. 12)

From the previous example it is known that the critical depth for this cross-section is 1.4 m and therefore the flow is subcritical for a depth of 1.5 m.

b. For the rectangular channel with $y = 1.5$ and $b = 4$

$$R = \frac{by}{(b + 2y)} \quad \text{and} \quad A = by$$

hence $s_o = 0.00480$ (Point B in Fig. 12)

With $Q = 25$ m^3/s and $A = 6$ m^2 and since $V = Q/A$, $D = y$

then
$$F_r = V/(gD)^{\frac{1}{2}} \tag{16}$$
$$= Q/A(gy)^{\frac{1}{2}} = 1.086$$

Since the Froude number is greater than unity, the flow in the channel is supercritical.

2. The channel slope and discharge can be adjusted so that a critical uniform flow occurs at a given normal depth. The equations used are:

$$V_c = (gD)^{\frac{1}{2}} \quad (17) \qquad \text{and} \qquad V_c = R^{\frac{2}{3}}\ s_{co}^{\frac{1}{2}}/n \quad (48)$$

a. For the trapezoidal channel

$$V_c = \left[g\,\frac{(b + zy)y}{b + 2zy}\right]^{\frac{1}{2}} = 3.40 \text{ m/s} \tag{17a}$$

$$= \left[\frac{(b + zy)y}{b + 2y(1 + z^2)^{\frac{1}{2}}}\right]^{\frac{2}{3}} s_{co}^{\frac{1}{2}}/n \tag{48}$$

$$= Q/A = Q/(b + zy)y$$

Substituting $y_{co} = 1.5$ m, $b = 4$ m, $z = 1$ and $n = 0.015$, it follows that the slope required to achieve critical flow at the given normal depth of 1.5 m is $s_{co} = 0.00260$ and that corresponding discharge is $Q = 28.05$ m^3/s (Point C in Fig. 12).

b. For the rectangular channel

$$V_c = (gy)^{\frac{1}{2}} = 3.84 \text{ m/s} \tag{17}$$

$$= \left[\frac{by}{b + 2y}\right]^{\frac{2}{3}} s_{co}^{\frac{1}{2}}/n \tag{48}$$

$$= Q/A = Q/by$$

hence $s_{co} = 0.00407$ and $Q = 23.02$ m^3/s (Point D in Fig. 12).

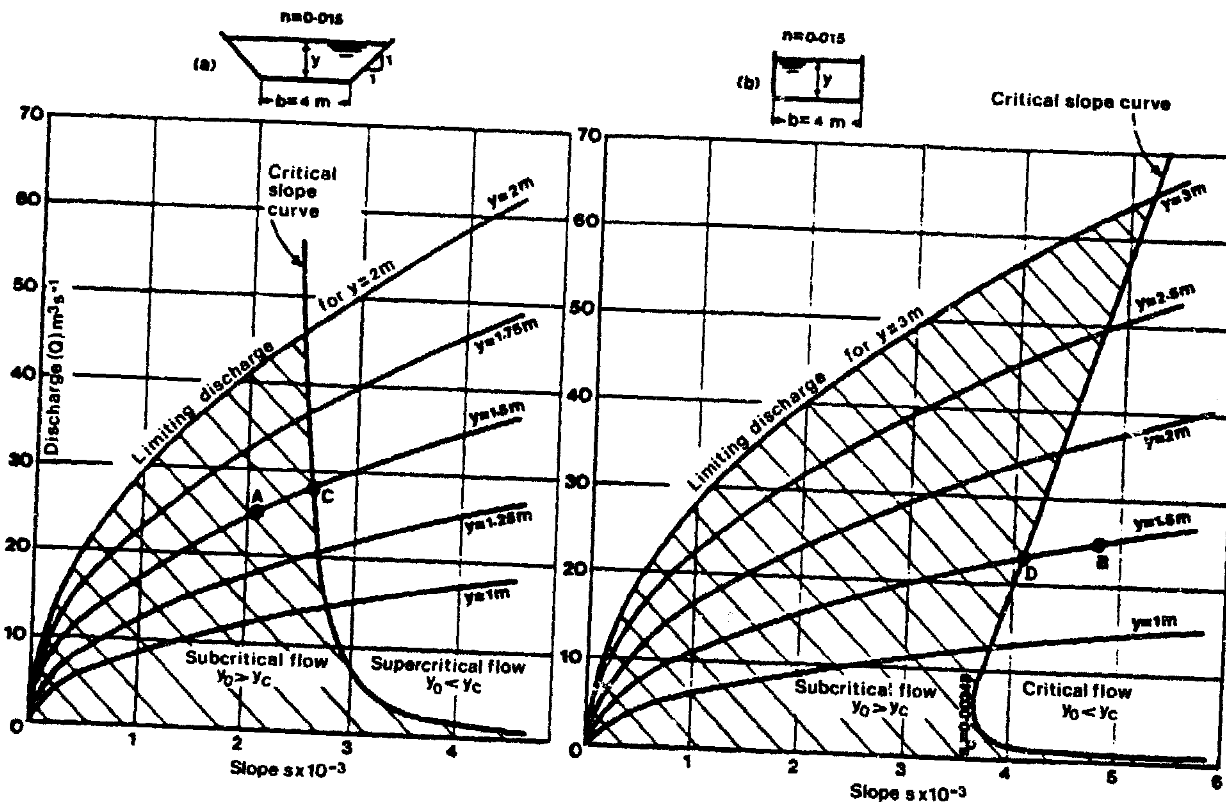


Figure 12 Limiting discharge and critical slope

b. Most efficient channel section

Examination of the Manning formula shows that from the purely hydraulic standpoint, the most efficient channel cross-section is the one with the smallest wetted perimeter.

For open channels, the section with the least wetted perimeter is the semicircle and for rectangular channels that in which the width is equal to twice the depth.

In the case of trapezoidal sections, the most efficient section is the half hexagon, which is the trapezoid most closely approximating the semicircle.

Practical considerations (construction, lining and maintenance) usually dictate the choice of trapezoidal channels having rather flat side slopes. In this case the most efficient sections have proportions such that semicircles can be inscribed in them. The performances of channels of various shapes are compared in Fig. 13.

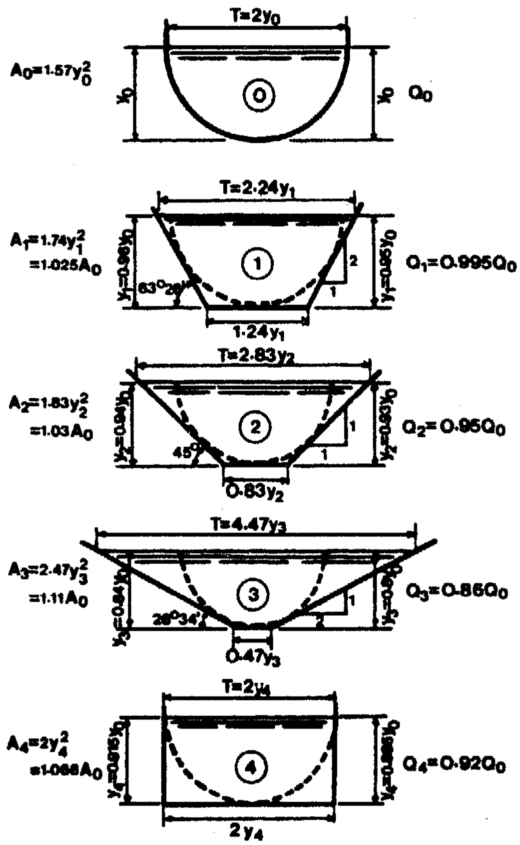


Figure 13 Comparison of trapezoidal sections for maximum flow
Left : areas for constant discharge
Right : discharge with constant area
(The slope and rugosity remain constant)
Source: Crausse (1951)

ii. Nonuniform flow

a. **The gradually varied flow equation**

When designing canals it is often necessary to determine the hydraulic conditions in a reach in which the depth of steady flow gradually varies along the length of the channel as a result of the presence of some form of control upstream or downstream. Such controls may be any structure, change of channel slope, roughness or cross-section which brings about a gradual change of uniform flow depth. Since the change is gradual, channel friction cannot be neglected as for rapidly varied flow and the problem to be resolved is the determination of the shape of the resulting water surface profile.

Referring once more to Fig. 4 where x is measured along the bottom of the channel, differentiating Eq. (18) we have:

$$\frac{dH}{dx} = \frac{dz}{dx} + \cos\theta \frac{dd}{dx} + \alpha \frac{d}{dx} \left(\frac{V^2}{2g}\right) \tag{49}$$

By convention the slope is the sine of the slope angle and is said to be positive when it descends in the flow direction and negative when it ascends. Hence the friction or energy slope $s_f = -dH/dx$ and $s_o = \sin\theta = -dz/dx$ which when substituted in Eq. (49) gives

$$\frac{dd}{dx} = \frac{s_o - s_f}{\cos\theta + \alpha d(V^2/2g)/dd} \tag{50}$$

This differential equation is known as the gradually varied flow equation. Since in most problems involving gradually varied flow the slope is small

$$\cos\theta = 1, \quad d = y \quad \text{and } dd/dx = dy/dx$$

hence

$$\frac{dy}{dx} = \frac{s_o - s_f}{1 + \alpha d(V^2/2g)/dy} \tag{51}$$

Assuming that the head loss at a section of gradually varied flow is the same as for uniform flow having the velocity and hydraulic radius of the section then the friction or energy slope s_f is given by rewriting the uniform flow equation (Manning)

$$s_f = n^2V^2/R^{\frac{4}{3}} \tag{48a}$$

b. **Classification of flow profiles**

Flow profiles are universally classified (after Bakhmeteff) in five classes according to the slope of the channel bed and three zones depending on the position of the profile with respect to the critical and normal depth lines:

Class	channel slope	
M	mild	$y_o > y_c$
C	critical	$y_o = y_c$
S	steep	$y_o < y_c$
H	horizontal	$y_o = \infty$
A	adverse	$y_o > y_c$

Zone	location of profile
1	above y_o
2	between y_o and y_c
3	below y_c

Since the uniform flow depth (y_o) in horizontal or adverse slope channels is infinite, the above classification gives rise to thirteen different flow profiles: M1, M2, M3; C1, C2, C3; S1, S2, S3; H2, H3; and A2, A3. In each case the flow is gradually varied with the exception of C2 which corresponds to uniform critical flow and is in fact a straight line.

These profiles are illustrated in Fig. 14 together with examples of situations which frequently arise.

c. Control sections

A control section is a channel cross-section at which, for a discharge or a range of discharge, there is a fixed relation between depth of flow (stage) and discharge. This condition is most commonly met at sections where critical depth occurs, such as the upstream end of a steep channel or the downstream end of a channel of mild slope.

Channel constrictions, weirs and sluices are also control sections as long as they are not drowned out by the downstream level when the discharge increases due to the presence of a further control downstream.

It is important to realize that since critical depth varies with discharge, a section may not be a control section throughout the range of operating conditions.

Computations to determine the shape of gradually varied flow profiles are generally initiated at the control point since its position is well defined. The computation then proceeds in the upstream or downstream direction depending on the state of flow in the channel: upstream direction for downstream control (subcritical flow); downstream direction for upstream control (supercritical flow).

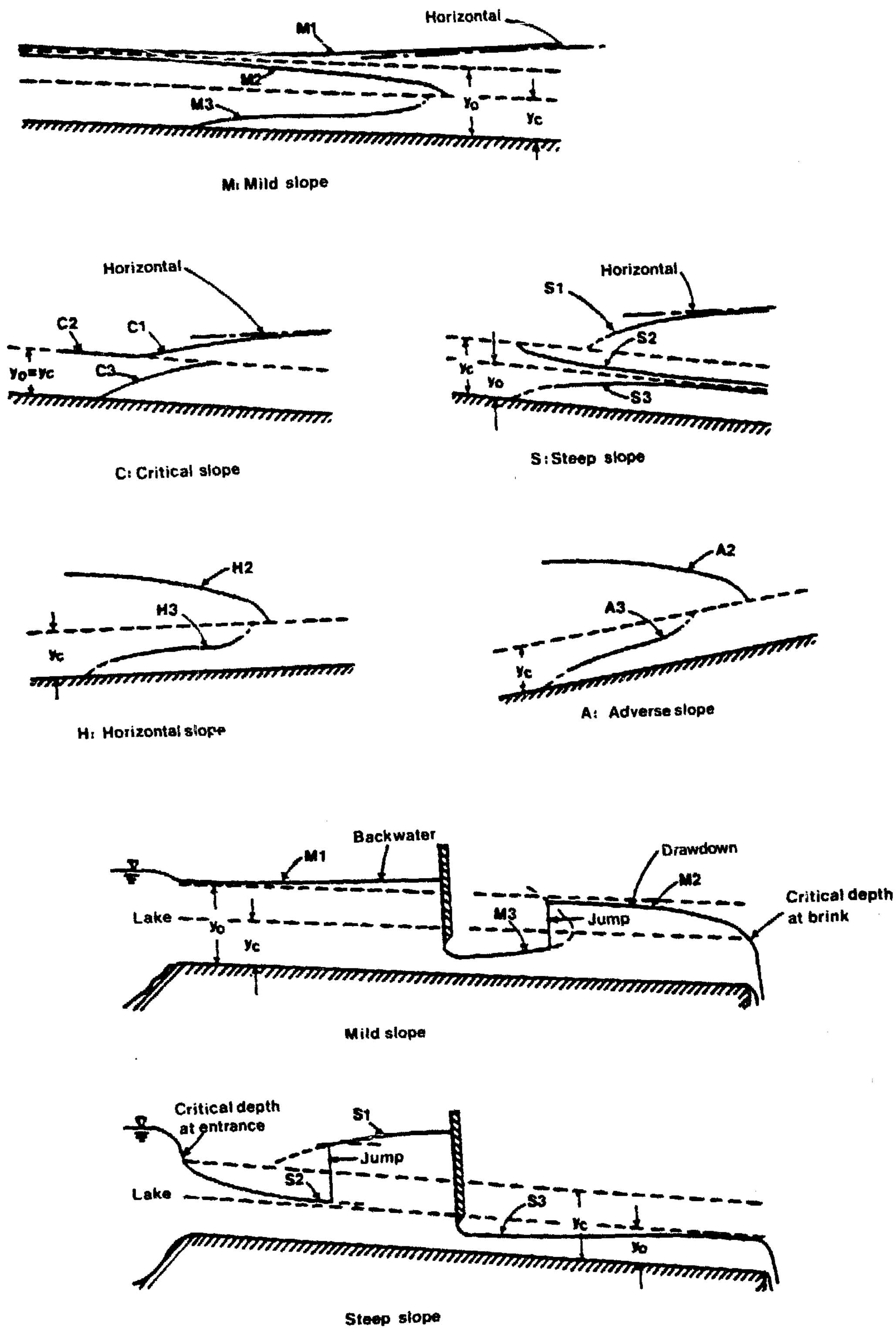


Figure 14 Classified flow profiles

EXAMPLE 6 - COMPUTATION OF GRADUALLY VARIED FLOW PROFILE

A trapezoidal channel conveying 6 m^3/s flows into a lake whose water level is 1.5 m above the channel bottom. Determine the shape of the water surface profile as it approaches the lake. $n = 0.014$, $s_o = 0.002$, $b = 3$ m and $z = 2$.

a. Uniform flow depth (y_o)

$$V = Q/A = R^{\frac{2}{3}} s^{\frac{1}{2}}/n \qquad (48)$$

$$\text{or} \quad AR^{\frac{2}{3}} = nQ/s^{\frac{1}{2}} = 1.878 \qquad (48b)$$

the normal depth (y_o) is the value of y which satisfies

$$y(3 + 2y)\left[\frac{(3 + 2y)y}{3 + 2y\,5^{\frac{1}{2}}}\right]^{\frac{2}{3}} = 1.878$$

by trial and error it is found that $y_o = 0.686$ m

b. State of flow

At the critical depth (y_c)

$$A^3/T = Q^2/g = 3.6697 \qquad (30)$$

the critical depth (y_c) is the value of y which satisfies

$$\frac{[(3 + 2y)y]^3}{3 + 4y} = 3.6697$$

by trial and error it is found that $y_c = 0.639$ m. Uniform flow in the channel is therefore subcritical and controlled by the lake level with a resulting backwater curve in the channel.

c. Determination of the profile by the direct step method

The procedure is as shown in Table 5 where arbitrary values of the depth (y) are selected which gradually decrease from $y = 1.5$ m (lake level above channel bottom) to $y_o = 0.686$ (uniform flow depth). Referring to Table 5 it will be recalled that

Col. 4	E	= specific energy = $y + V^2/2g$ (23)
Col. 5	ΔE	= change of E with respect to previous step
Col. 6	s_f	= friction slope = $n^2V^2/R^{\frac{4}{3}}$ (48a) [Manning]
Col. 7	$\bar{s}_f$	= average slope between two successive steps
Col. 8	$s_o - \bar{s}_f$	= channel bottom slope $- \bar{s}_f$
Col. 9	Δx	= $\Delta E/(s_o - \bar{s}_f)$
Col. 10	x	= distance from origin (channel end) $= \sum \Delta x$

The resulting profile is shown in Fig. 14. The decrease in the selected size of the depth increments takes into account the rapid growth of Δx as the backwater curve tends towards the uniform depth.

Table 5 COMPUTATION OF FLOW PROFILE BY DIRECT STEP METHOD (EX 6)

1 y (m)	2 $R^{\frac{4}{3}}$	3 V (m/s)	4 E (m)	5 ΔE (m)	6 s_f 10^{-4}	7 $\bar{s}_f$ 10^{-4}	8 $s_o-\bar{s}_f$ 10^{-3}	9 Δx (m)	10 $\sum\Delta x$ (m)
1.5	0.904	0.667	1.5227		0.965				
1.4	0.839	0.739	1.4278	0.0949	1.276	1.120	1.888	50	50
1.3	0.775	0.824	1.3346	0.0932	1.717	1.497	1.850	50	100
1.2	0.711	0.926	1.2437	0.0909	2.364	2.041	1.796	51	151
1.1	0.648	1.049	1.1561	0.0876	3.328	2.846	1.715	51	202
1.0	0.585	1.200	1.0733	0.0828	4.825	4.076	1.592	52	254
0.9	0.523	1.388	0.9983	0.0750	7.220	6.022	1.398	54	308
0.85	0.491	1.504	0.9653	0.0330	9.030	8.125	1.187	28	336
0.80	0.461	1.630	0.9355	0.0298	11.296	10.163	0.984	30	366
0.75	0.430	1.778	0.9111	0.0244	14.406	12.284	0.771	32	398
0.725	0.414	1.860	0.9013	0.0098	16.379	15.392	0.461	21	419
0.70	0.399	1.948	0.8934	0.0079	18.641	17.510	0.249	39	458
0.69	0.393	1.985	0.8909	0.0025	19.651	19.146	0.085	29	487
0.686	0.391	2.001	0.8900	0.0009	20.062	19.856	0.001	63	550

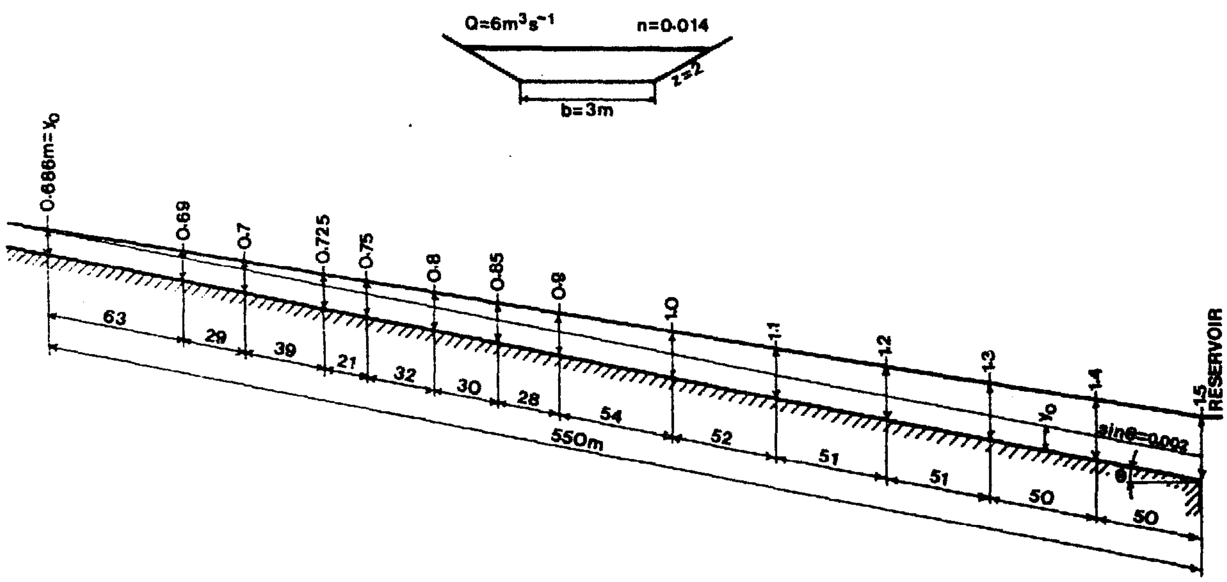


Figure 15 Backwater curve in a trapezoidal channel (Ex 6)

B. ECONOMICS

2.5 PHASES OF A PROJECT APPRAISAL STUDY

The selection of irrigation infrastructures and most suitable ancillary equipment involves that both technical and economic factors be taken into account during the investment studies. The approach can best be illustrated by the following example.

Reconnaissance surveys have identified an area of 6000 ha on the bank of a large river in the African Sahel as suitable for irrigated agriculture. If the farmers are provided with a regular supply of water in sufficient quantity, agricultural production can be considerably increased and diversified. At present the area supports 2000 families each having an average yearly income of US$ 400. Flood recession agriculture (millet, sorghum, maize) yields about 500 kg/ha of cereals and 300 kg/ha of cowpeas. Okra, onions and sweet potatoes are also grown as well as paddy, more rarely,, and on small areas.

By irrigating this area and introducing improved farm practices it would be possible to grow 10 to 12 t/ha of paddy by double cropping. Other cropping patterns are also possible (paddy + tomatoes, maize + tomatoes, etc.) with tomatc yields in excess of 40 t/ha. Five rotations in two years have been experimented by the local agricultural research centre.

The benefits that can be expected to result from the increased agricultural output of the area are the following:

- qualitative and quantitative improvement of the diet of the local inhabitants;
- substantial increase of the income of the farmers;
- production of surpluses which can be marketed in the cities thus reducing rice imports and saving foreign exchange;
- creation of employment in other sectors such as agro-industry and transport (provision of a rice mill, of a tomato paste plant, etc.).

These various aspects were briefly analysed during the identification stages (site, project) as were the various possible water supply networks and three possible alternatives were finally retained each with its sub-variants:

- 6000 ha in one unit with a large pumping station, a main canal along the river bank with distribution canals.
- Six small units, each supplied by a small pumping station through a system of canals.
- Layout of a pipe network for pressure distribution of the water.

An initial estimate of overall costs of the operation amounted to US$ 60 million (1983). After consultation, seven funding agencies accepted to provide the necessary funds through low interest loans. After calling for international tenders, a consulting firm was selected to undertake a feasibility study covering in detail all the aspects examined by the identification study.

The feasibility study was carried out in three phases:

First Phase: This covers all the basic studies (physical and socio-economic) which are not interdependent. The purpose of the first phase is to define the initial conditions or starting point and to quantify the target or point of arrival in terms of agricultural output. The government's policy with respect to appropriation or distribution of land and water is taken into account as are the relocation of people and the balance of the regions.

At the end of the first phase the average size of the farms has been decided upon with possibly one or two variants. This enables the number of water outlets which have to be provided to be defined as well as the discharge of each.

Information which will be needed later is also collected during the first phase. The analyses of farm models and cropping patterns are used in the third phase for the preparation of farm budgets and accounts to determine the profitability of the project.

Second Phase: This phase is entirely devoted to the technical and economic studies of the proposed development, both items being closely interdependent throughout. The final objectives of this phase are to determine the total investment and running costs of the project and to identify the type of water distribution system which will yield the lowest cost investment and operation at the farm level.

In the example described above, the project involves three distinct fields of activity:

- building of dykes and other basic infrastructures,
- excavation of the canal system,
- on-farm development work.

For each of the alternatives that were retained, the above fields of activity are not in fact independent of each other and the final or "best" layout should not be selected until each solution has been fully explored. In practice experience enables the procedure to be shortened and, as this phase proceeds, the number of alternative solutions is reduced.

Third Phase: This phase is devoted to the evaluation of the profitability of the solution that has been selected. The calculation is based on the farms taken as a whole and the result depends upon the choice of crops, farm practices, type of scheme as well as many other factors, some of which cannot be quantified (adaptability of the farmers to new techniques, percentage of failures, etc.).

The preceding schematic description of the contents of a feasibility study is intended to throw some light not only on the multiplicity of themes which are involved but also upon the close relation between them.

The diagram reproduced in Figure 16 illustrates the progression of a project study. The numbers within the frames represent the numbers given to each item under study (list of tasks). Economic studies are shown in double frames. The tasks within the circle correspond roughly to the second phase as defined previously. The design of such a diagram implies the resolution of three problems:

- choice of method of selection of the solution;
- division into elemental tasks (sub-studies) and their inter-relationship;
- evaluation of the time and means required to complete these tasks.

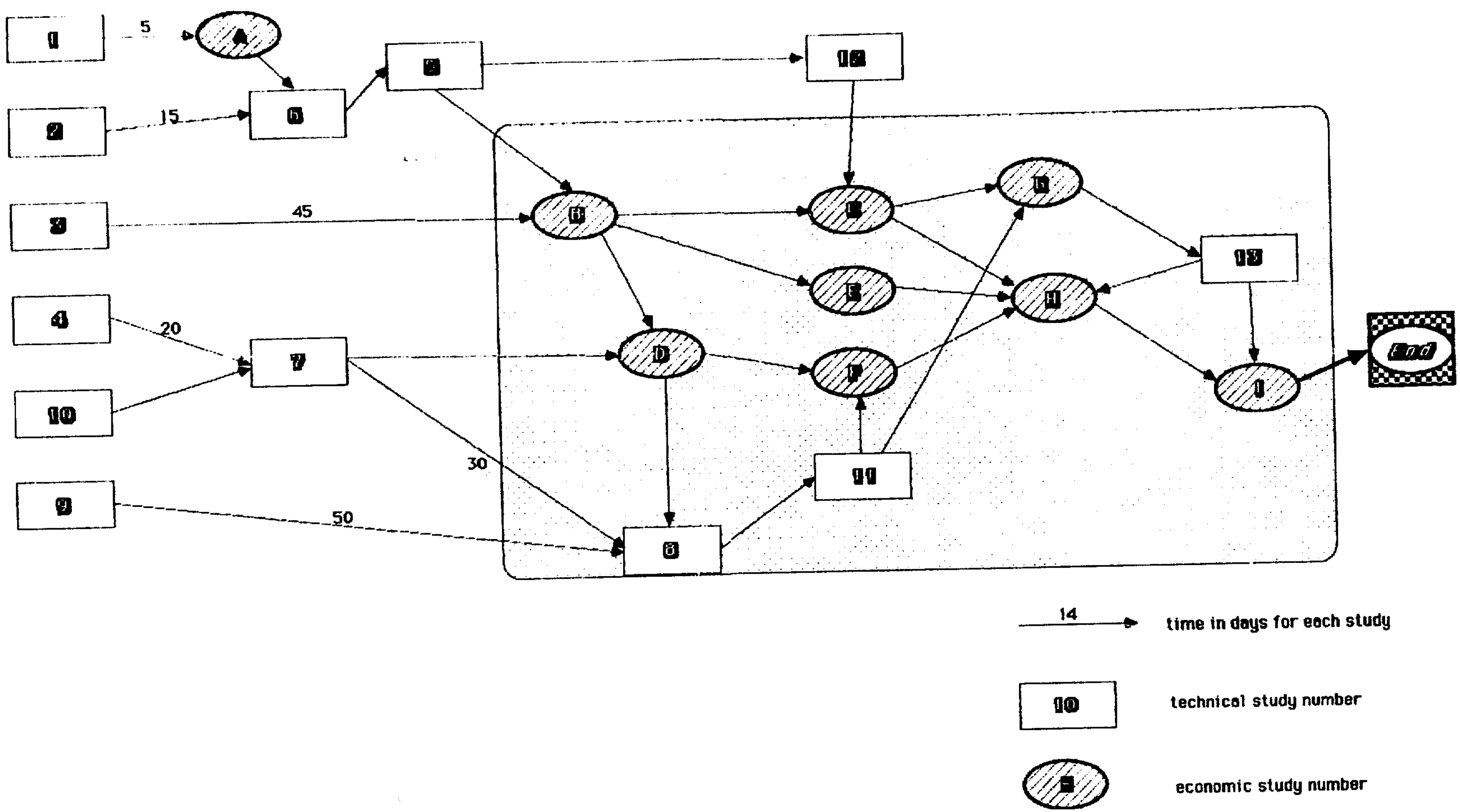


Figure 16

PERT chart

2.6 GENERAL METHODS: MULTI-CRITERIA ANALYSES

It is clear that an irrigation project will have positive and negative effects upon both the local and the national economy.

Multi-criteria analyses consist of:

- selection of the main fields of activity affected by the project;
- identification of a descriptive criterion for each field;
- weighting of each criterion according to the importance it has upon decision making (choice of solution);
- adoption for each criterion of a rank for each of the solutions which have been examined.

This results in the attribution of an overall mark which enables all the solutions to be ranked. Thus the example used previously could be analysed as follows:

- Percent increase of cereal production. (1)
- Percent increase of market garden production. (2)
- Improvement of competence of farmers (in terms, for example, of farmers who can read and write). (3)
- Numbers of jobs created by activities induced by the project. (4)
- Saving of foreign exchange by the replacement of imports with local products. (5)
- Percent increase of farmer incomes. (6)
- Project internal rate of return. (7)
- Annual repayment of loans. (8)

The above criteria can be weighted in the following way: the criteria are arranged in decreasing order of importance, increasing the margin between two criterion as required (e.g. criterion 7 may be ranked 1 and criterion 1 placed in rank 3, rank 2 remaining vacant). Finally, for each criterion, each solution is awarded a mark of merit (between 0 and 10 and 0 and 5 etc.).

This is a very simple method of weighting and measure of criteria and many more elaborate methods have been used. Nevertheless this type of analysis is rarely employed in agricultural development studies.

Studies of agricultural projects generally follow procedures laid down by the international organizations and the sequence of economic analyses is described in the following section.

2.7 ECONOMIC AND FINANCIAL ANALYSIS OF AGRICULTURAL PROJECTS

The analysis involves three items:

- Calculation of costs (investment, replacement, running costs).
- Financial analysis of project expenditure and income.
- Economic analysis or study of the profitability of the project.

2.7.1 Cost Calculation

The cost of goods or services is an economic variable which defines the value of the goods or services at a given moment in a given place and within a given economic environment.

It is of the form $\sum_i p_i q_i$ where p_i is the unit price of component i of the goods or the service multiplied by the quantity q_i of the same component.

In the following example, an irrigated area in the Senegal River valley is to be provided with pumps manufactured in the United Kingdom. It is therefore necessary to determine the cost of the pumps delivered on site on the basis of the following:

- Price f.o.b. Liverpool (£/unit)
- Cost of insurance and freight (US$/ton)
- Unloading and warehousing Dakar (CFA/tonne)
- Port and customs dues Dakar (CFA and percentage of CFA value of imported items)
- Handling and transport costs from Dakar to project site (CFA/tonne and per tonne-km)

After applying suitable exchange rates to convert US dollars and £ Sterling to local currency the total cost is obtained by addition $\sum_i p_i q_i$.

The price obtained is a function of place and time. Inflation both in the UK and in Senegal will raise this price whereas technical modifications introduced by the manufacturers may lower it. The price is also subject to changes in currency exchange rates. Lastly, detaxation policies on the part of the exporting country as well as taxation or subsidies by the importing country will also have an impact upon the final price.

By convention, basic prices used in project studies are market prices at the time of the study. They are therefore dated. They are termed financial or market prices and are used without adjustment for all calculations within the limits of the financial analysis.

When undertaking cost calculations, the first task is to draw up the complete table of detailed and commented investment and of their cost. An example of such a table is shown in Table 6.

Item V (Miscellaneous) is expressed as a percentage of the preceding costs and the choice of percentage depends upon the degree of uncertainty involved in the estimation of costs. There is no general rule and a value of 10 percent is frequently accepted. In practice this item increases the flexibility during the construction phase and may help to avoid financial problems. During periods of high inflation it may be revised upwards especially if the time required for study and construction is long.

On the basis of Table 6 and technical studies the tables of annual running costs and renewal of equipment can be drawn up.

In these tables, costs are once more referred to the date at which the study was undertaken. In certain cases costs at current prices, based upon an estimate of future inflation are calculated. This is only possible when rates of inflation are relatively low and when future inflation can be substantially justified.

The table of annual running costs includes:

- manpower (salaries, social security etc.);
- maintenance of infrastructures and buildings;
- depreciation costs (these will not be taken into account later in the economic analysis);
- miscellaneous financial costs.

The cost of equipment entered in the appropriate columns in the tale of renewals is the actual cost of the equipment at the time of completion of the study unless future inflation can be estimated correctly in which case current prices are listed.

Table 6 INVESTMENT COSTS (US$, JUNE 1984)

I.	Studies	
	Topography and soil surveys	60 000
	Engineering	12 000
	Total studies	**72 000**
II.	Infrastructures	
	Earthworks	5 524 000
	Civil engineering structures	1 467 000
	Buildings	609 000
	On-farm improvements	128 000
	Total infrastructures	**7 728 000**
III.	Irrigation Equipment	
	Electro-mechanical equipment of pumping stations	495 000
	Electric lines	251 000
	On-farm equipment (pipes, siphons)	65 000
	Total irrigation equipment	**811 000**
IV.	Agricultural machinery and others	
	Power shovels (canal maintenance)	15 000
	Tractors, ploughs, threshing machines	267 000
	Other equipment (trucks ...)	103 000
	Total agricultural machinery	**385 000**
V.	Miscellaneous items (10 percent)	**900 000**
VI.	Interest during construction	**345 000**
	GRAND TOTAL	**10 241 000**

2.7.2 Financial Analysis

This task consists of the study of future cash flows or, at least, of those which can be estimated since some will remain unknown.

The procedure closely resembles that which is followed in standard accountancy. It is normally applied to the project as a whole and to its constituent economic units where applicable. Thus, for example, in the case of a 2000 ha irrigated area which is split up into 100 ha units of 50 farms each, the financial analyses are undertaken at the level of:

- the farmer;
- the cooperative (grouping 50 farmers in each unit);

- the authority responsible for the management of collective irrigation works (network, pumping stations);
- the complete irrigation area.

The financial analysis is made at constant prices (prices at the time of the study) at current price and consists firstly in calculating year by year the receipts of each economic unit (farmers, cooperatives, etc.) and secondly, with reference to the table of running costs, in preparing the operating account and the profit and loss account.

The financial analysis should facilitate:

- the evaluation of the financial validity of the proposed investment;
- the analysis of the financial structure of the economic units under consideration;
- the determination of the adequacy of the financing plan;
- the formulation of suggestions to enhance the financial validity of the project.

The financial analysis is based upon components which reflect the economic environment (in the broad sense of the term) of the moment. The economic and socio-political characteristics have therefore a direct impact upon the financial viability of the project (primary policy, government assistance, employment situation, etc.). The financial analysis should clearly bring out their impact and proposals should be made to modify these policies where necesary in order to improve the financial viability of the project.

2.7.3 Economic Analysis

It has been shown that the financial situation of an economic entity depends largely upon the conditions prevailing at the time of the study and it is quite possible that the project is penalized by its environment. For this reason other criteria can justifiably be employed by which to judge the project.

The aim of the economic analysis is to eliminate these influences and to appraise the project without taking into account purely conjunctural influences.

The project analysis is therefore repeated, no allowance being made this time for:

- tax policy (tax free calculation);
- governmental aid (no subsidies);
- pricing policy (shadow prices are calculated, see below);
- exchange controls (economic exchange rates are used which more realistically reflect the true value of the currency);
- monetary policies and other policies which have a bearing on price levels. Since the subject is sensitive the difficulty is avoided by calculating in constant prices.

The final objective of the economic analysis is to establish the rate of return and to determine whether it is sensitive to technical constraints (fall of yields for example) or economic factors (price variations).

All calculations made for the financial analysis are repeated for the economic analysis and the results listed in Table 7. All prices used are tax and subsidy free and expressed in constant prices. Before taking up the subject of shadow prices it should be noted that provision is not made for depreciation under the heading "running costs" in Table 7. Investments are attributed in whole to the initial years and are not ventilated over the years of life of the equipment.

Table 7 SUMMARY OF RECEIPT AND EXPENDITURE

Years	Receipts (I)	Expenditures (II)				Result (I - II)
		Investments	Running and maintenance	Renewal	Total expenditure	
1	-	365.2			365.2	-365.2
2	-	270.2			270.2	-270.2
3	-	305.7	29.8		336.5	-336.5
4	44	58.1	81.1		139.2	- 95.2
5	99.9	12.5	111.9		124.4	- 24.5
6	165.2		116.5		116.5	48.7
7	196.6		123	4.2	127.2	69.4
8	233.8		128.9		128.9	105.9
9	248.2		128.1		128.1	120.1
10	252		128.1	1.9	130	122
11	255.3		128.1	44.6	172.7	82.6
12	255.3		128	2.7	130.7	124.6
13	255.3	129			129	126.3
14	255.3		128.2		128.2	127.1
15	255.3	128	27.9	155.9	99.4	
16	255.3		128		128	127.3
17	255.3		128	116.7	244.7	10.6
18	255.3		128	36.1	164.1	91.2
19	255.3		128	5.3	133.2	122
20	255.3		128	4.0	132	123.3
21	255.3		128		128	127.3
22	255.3		128		128	127.3
23	255.3		128	10.1	138.1	117.2
24	255.3		128		128	127.3
25	255.3		128	37.9	165.9	89.4
26	255.3		128		128	127.3
27	255.3		128	26.2	154.2	101.1
28	255.3		128		128	127.3
29	255.3		128		128	127.3
30	255.3		128		128	127.3

2.7.4 Shadow Prices

Many market prices (in the broad sense of the term) at the time of the study do not correspond to the true economic situation. Local currency exchange rates for instance may be maintained artificially with a resulting bias of the local value of imported products. In the same way wages may not reflect labour availability. Also the government may have chosen a tax policy designed to protect the internal market from international competition whilst subsidizing certain foodstuffs as inputs.

Giving due weight to these and other considerations which may not be included above, the economist will review the prices which were used for the financial analysis.

An example of the calculation of a shadow or import substitution price is shown in Table 8.

Table 8 RICE-GROWING PROJECT: ESTIMATING THE IMPORT SUBSTITUTION PRICE OF RICE

		1981-1984	1985-1989	1990 +
Price fob Bangkok (US$/t) [1]		469.00	533.00	551.00
- 30% (compensation for quality) [2]		328.00	373.00	386.00
Freight and insurance (US$/t)		55.00	55.00	55.00
Price CIF - Port of arrival (US$/t)		383.00	428.00	441.00
Price CIF (MU/t) $1 = MU 270		103 410	115.560	119 070
+ import contingency cost [3]	2 470	-	2 470	2 470
+ reception costs:				
. unloading and handling	1 668	-	1 668	1 668
. warehousing	2 068	-	2 311	2 381
+ port dues	735	-	735	735
+ transit agent	4 332	11 273	4 332	4 332
+ loading and transport (rail)	3 012			
+ wholesale warehousing	800			
+ conveyance	871			
+ wholesale margin	1 000			
+ retail margin	1 500	7 183	7 183	7 183
Import substitution price of rice at capital city (MU/t)		121 866	134 259	137 839
Official Price in June 1981:		**75 000 MU/t**		
- transport from capital to rice mill at project site and unloading (rail) (MU/t)	1 270			
- processing costs	5 000	MU/t	MU/t	MU/t
- depreciation of sacking plant	855	7 125	7 125	7 125
Price ex-rice mill		114 741	127 134	130 714
Equivalent paddy price (67%)		76 877	85 180	87 180
+ value of by-products		1 600	1 600	1 600
		78 477	86 780	89 1788
- loss by dessication in warehouse		1 507	1 670	1 717
- depreciation paddy sacking plant		855	855	855
- margin of state-owned companies		4 000	4 000	4 000
- transport to rice mill (average distance)		4 515	4 515	4 515
- cost of paddy collection		2 000	2 000	2 000
Economic price of paddy		65 600	73 740	76 091
Official price in June 1981:		**47 000 MU/t**		

MU = Monetary unit of country

[1] World Bank projection for 1981 in 1980 US dollars

[2] Notably for a broken rice rate in the reference product (5%) which is less than that of ordinary rice

[3] 2.205% ad valorem

Warehousing at port:
1 - 15 days - free
16 - 30 days - 7% CIF value
31 - 45 days - 10% CIF value
46 - 60 days - 20% CIF value
60 - days - 40% CIF value

Loading on railcar/unloading 400 MU/t

By-products: (B.R. 10 kg at 20 MU = 200 MU;
bran 70 kg at 20 MU = 1 400 MU)

2.7.5 Final Stage of Economic Analysis: Calculation of the Internal Rate of Return

After all prices and costs have been recalculated according to the procedure described above, new tables of investments, running costs and operating account (without depreciation costs) are drawn up and cash flows are summarized as in Table 7. The last column of this table indicates for each year the balance in constant prices (profit or loss). This set of figures shows that the expenditure in the first years (investments) results in profits during the later years. The problem is how to compare the two flows and how to evaluate the investment prospect.

Whenever money is deposited (savings bank, stocks and shares, etc.) it is customary to estimate the rate of return (percentage, interest rate, dividends) which it will produce each year. In the present case a similar parameter is determined which is known as the internal rate of return and which is linked to the notion of discount.

2.7.6 Discount

It is common knowledge that a unit of money available today does not represent the same value as it did the year before and that it will have yet another value next year. In the first place this is due to monetary erosion or loss of purchasing power on account of inflation (on the contrary, negative inflation enhances its value).

Moreover, if this unit of money is available today instead of next year it can bring in returns, by investing it for example, or it may be used to purchase goods (rather than saving up and waiting to dispose of the necessary sum for the purchase of a car is it preferable to borrow the money and buy the car right away?).

It is common practice to distinguish the two aspects discussed above. Thus, for example, calculations can be based on the real interest rate by subtracting the rate of inflation from the nominal interest rate (interest rate charged for any loan).

Discount calculation is a process which is used to calculate or estimate (when forecasts are available) at a given moment the value of a sum of money" s" available "i" years (or other time period) later or earlier. For example, one can calculate the value on 1 January 1980 of a sum of 100 units available on 1 January 1985:

From 1 January 1980 to 1 January 1985 five complete years have passed. Assuming that the purchasing power of the sum is to be calculated and that inflation was as follows:

1981: +8%; 1982: +10%; 1983: +12%; 1984: +9%; 1985: +7%

then the value on 1 January 1980 of the 100 units of 1 January 1985 was:

100/(1.08)(1.1)(1.12)(1.09)(1.07) = 64.4 units

Conversely the value on 1 January 1985 of the 100 units at 1 January 1980 is:

100.(1.08)(1.1)(1.12)(1.09)(1.07) = 155.2 units

Let a_i be the actual discount factor for year i, then

$$S_o = S_n \prod_{i=1}^{i=n} \frac{1}{(1+a_i)}$$

and $$S_n = S_o \prod_{i=1}^{i=n} (1+a_i)$$

If the discount rate is constant throughout the period, that is if

$$a_1 = a_2 = .. = a_n$$

then $S_o = \dfrac{S_n}{(1+a)^n}$ and $S_n = S_o (1 + a)^n$

where S_o is in the first case the discounted value at year zero of S_n available in year n. In the second case S_n is the actualized value (projected in year n) of S_o available in year zero.

2.7.7 Calculation of the Internal Rate of Return

Coming back to Table 7 which shows a chronological series of profits and losses in constant prices which will be given the symbol B_i, then B_1 to B_5 are negative (investments) and B_6 to B_{30} are positive. All flows are referred to the same date of each year (31 December for an operating account and since during the initial years the investments include interest during construction also calculated on the last day of each fiscal year).

The problem to be solved is to find the discount factor which results in the sum of flows being equal to zero when discounted to year zero. Stated otherwise, which value of a satisfies the relation:

$$\sum_{i=1}^{n} \frac{B_i}{(1+a)^i} = 0 \qquad (52)$$

where $B_i/(1 + a)^i$ is the discounted value of B_i at rate a.

What is the significance of rate a? It is not a measure of inflation since all flows have been calculated in constant prices (at the time of study).

It is in actual fact a measure of the return of the investment. Since in Table 7 the first five terms of the series are negative, the first part of Eq. (52) will also be negative:

$$\sum_{i=1}^{5} \frac{B_i}{(1+a)^i} < 0$$

and since the remaining 25 terms are positive, then:

$$\sum_{i=6}^{30} \frac{B_i}{(1+a)^i} > 0$$

Eq. (52) is therefore satisfied if

$$\sum_{i=1}^{5} \frac{B_i}{(1+a)^i} = \sum_{i=6}^{30} \frac{B_i}{(1+a)^i} \tag{53}$$

which means that the net present worth of expenditure (investment costs) is equal to the net present worth of income (operating profit generated by the investment).

The result would have been the same if any other year had been selected for discounting purposes since raising (1 + a) to the same power on both sides of Eq. (53) does not affect its equality.

The above calculation was conducted with the series B_i of "profits" expressed in constant prices and a discount factor a_1 such that

$$\sum_{i} \frac{B_i}{(1+a_1)^i} = 0 \tag{54}$$

which is the internal rate of return of the project.

Considering now the series D_i of profits in current prices, obtained by multiplying each B_i by $(1 + j)^i$ where j is the mean rate of annual inflation and which will be assumed constant throughout the period.

Resuming the calculation of discount rate, a value of a_2 is to be found such that

$$\sum_{i=1}^{n} \frac{D_i}{(1+a_2)^i} = 0 \tag{55}$$

By substituting $D_i = B_i (1 + j)^i$ it follows that

$$\sum_{i=1}^{n} \frac{B_i(1+j)^i}{(1+a_2)^i} = 0 \tag{56}$$

Comparing Eq. (56) and Eq. (54) it may be seen that

$$\frac{(1+j)^i}{(1+a_2)^i} = \frac{1}{(1+a_1)^i}$$

hence

$$1 + a_1 = \frac{1+a_2}{1+j} \tag{57}$$

and $\quad a_2 = a_1 + a_1 j + j$

If the inflation rate is low ($j < 0.1$ for example) the second order term ($a_1 j$) can be omitted for a first approximation, hence $a_2 = a_1 + j$.

a_1 corresponds to constant prices or observed prices when the economy under consideration is inflation free. a_1 is a measure of the profitability of the investment in constant prices, a_2 represents its profitability with inflation. a_1 is known as the internal rate of return.

In practice a_1 is obtained by iteration. Starting with an average value of a_1 the value of

$$\sum_{i=1}^{n} \frac{B_i}{(1+a)^i}$$

is calculated. If the result is positive a second attempt is made with a large value of a until a result is obtained which is close to zero and negative. The calculation is refined by linear interpolation of the last two values having opposite signs. If the first trial yields a negative value, a_1 is decreased until a positive value close to zero is obtained after which the calculation is completed by interpolation.

Thus, referring to Table 7 with a = 6.1%

$$\sum \frac{B_i}{(1+a)^i} = 15.6$$

the value of the expression drops to + 11 for a = 6.2% and becomes negative with a = 6.3%. The internal rate of return is therefore close to 6.2% which is rounded off to 6% since decimal values are rarely considered.

In certain publications one may find as series to be discounted P(a), the series

$$B_o + \frac{B_1}{(1+a)} + \frac{B_2}{(1+a)^2} + \dots + \frac{B_n}{(1+a)^n} \qquad (58)$$

The notation has the same mathematical signification as the one used earlier (52) because

$$\frac{1}{(1+a)^o} = 1 \quad \text{and therefore} \quad B_o = \frac{B_o}{(1+a)^o}$$

The series (58) may be written $\sum_{i=o}^{n} \frac{B_i}{(1+a)^i}$

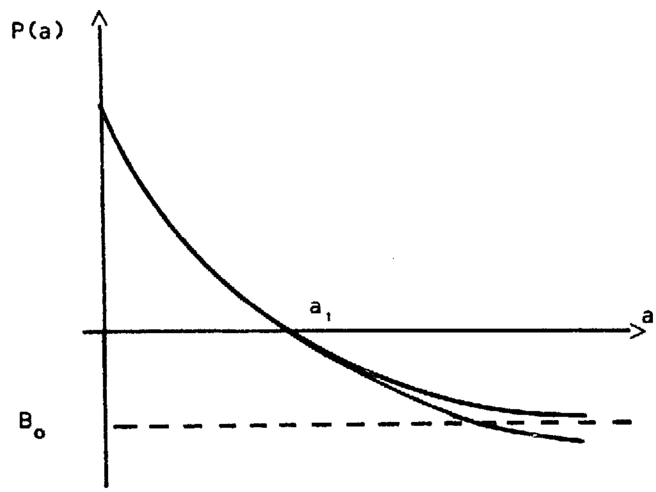
All finite solution for (52) is a solution for (58) as well.

Under its form (58) it can be shown that:

- if for a=o, $\sum_{i=o}^{n} \frac{B_i}{(1+a)^i}$ is negative, then the solution (internal rate of return) al, so that P(al) = $\sum_{i=o}^{n} \frac{B_i}{(1+a_1)^i} = 0$

is negative.

- if for a=o, P(a) is positive, then a_1 with $P(a_1)$ = o is the only solution and P(a) is a curve of the form:



3. NETWORK LAYOUT AND DESIGN DISCHARGE

3.1 INTRODUCTION

The structures that are commonly found on all irrigation schemes of some size can be placed in three categories:

- Conveyance structures which carry the water from the headworks to the irrigation area. These structures normally transport large discharges over long distances. They are linear, such as canals flumes or tunnels.
- Distribution structures which carry the water from the conveyance structure to the farm turnouts within the irrigation area. These structures are generally of the branching type.
- On-farm equipment which conveys the water from the turnouts or hydrants to the plants.

The present chapter deals specifically with the distribution structures, their layout and the discharges for which they are designed.

During the initial stage of project formulation it is essential to develop the physical design of the system simultaneously with the basic choices concerning technical management. The general organizatin of a network must be related to the mode of water distribution whether it be by rotation, on demand or continuous. Recognition at the design stage of the constraints associated with each type of water distribution contributes to the acceptance by the network operator and the farmers of the new tool placed at their disposal. As an illustration, it is clear that a pressure network designed to allocate water on a rotation basis cannot be used efficiently if the planner has not organized the layout so as to individualize the distribution branches serving each irrigation block within which the water duty is rotated (see 3.2).

For the sake of clarity, the subject-matter of this chapter will be treated in two distinct sections:

- Structure and layout of the distribution network including the principal alternatives: open-channel systems and underground pressure-conduit systems.
- Determination of the design discharge for each mode of water supply: rotation, on-demand and continuous.

3.2 STRUCTURE AND LAYOUT OF OPEN CHANNEL SYSTEMS

Although it is quite clear that the layout of the distribution channels depends primarily on topographic constraints, the structure of the network will to a large extent be influenced by the choice of the mode of distribution.

The geographical unit which encompasses all plots irrigated by rotation of a single stream size is denominated an **irrigation block.**

The block is supplied with irrigation water by a channel, diversely known as a watercourse, sub-minor or quarternary canal, whose capacity is sufficient to carry the stream size.

This channel is fed through an intake provided with a gate which limits the discharge to a value equal to the predetermined stream size.

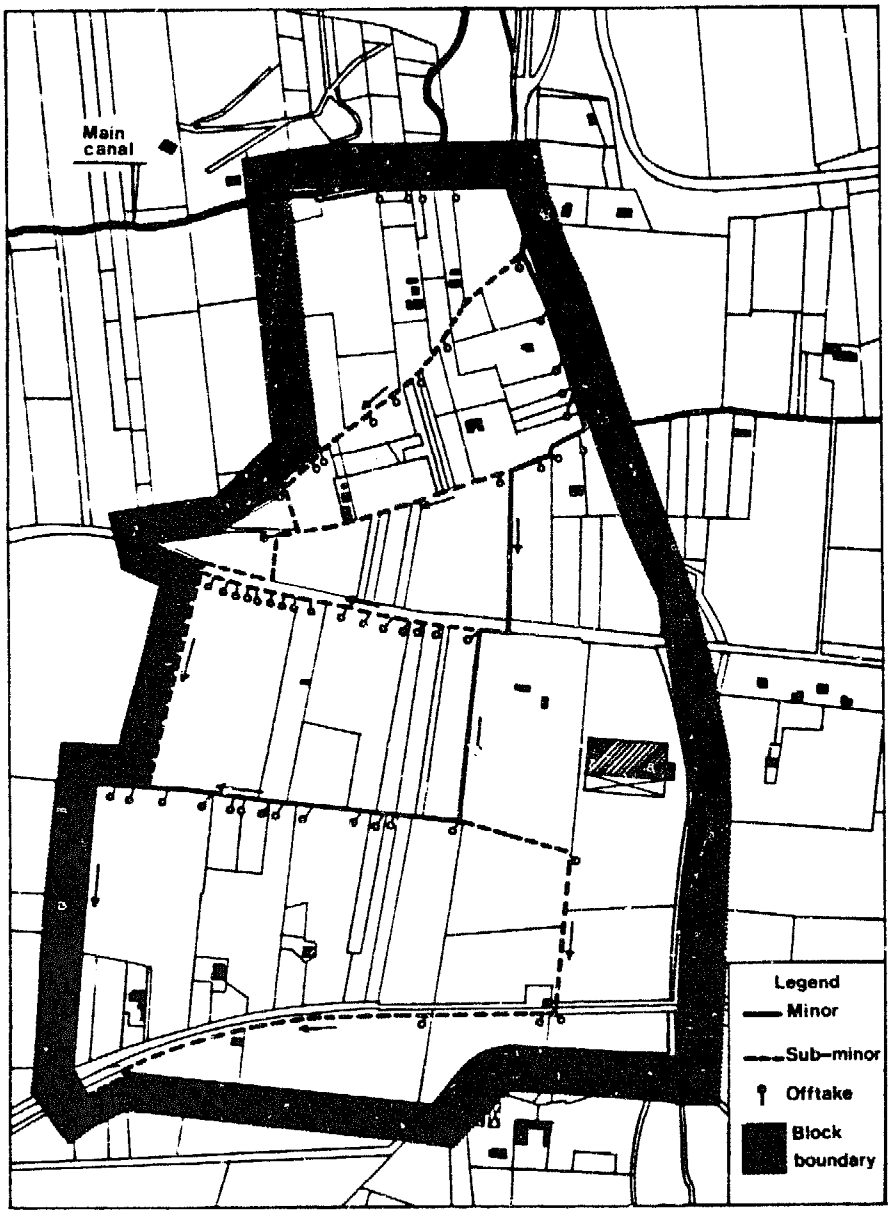


Figure 17 Irrigation block - Layout of unconsolidated holdings

The area under irrigation in each block must satisfy the following condition:

$$A \leqslant m/f$$

where: A = area irrigated in the block (ha)
m = stream size (l/s)
f = water duty in peak demand period (l/s/ha) the flow assumed to be constant over 24 h

When this condition is satisfied, the organization of water distribution within the block involves the rotation of a single stream size (m) to each farmer in turn.

In certain irrigation schemes, the size of a block does not meet the above condition, with the result that water distribution entails the simultaneous rotation of two or even three different stream sizes.

Such a situation tends to make a rotation more complex and calls for the provision of flow dividers.

In general it is preferable to restrain the size of a block both as regards the area to be serviced and the number of farmers receiving water: the block is the basic water management unit and the users should as far as possible be able to take charge of the rotation.

The division of the irrigation area into blocks will to a large extent be influenced by the options taken relative to the landholding structure within the area.

If the existing plot distribution is longstanding the pattern is often irregular, not only because of the shape of the plots, but also due to the variation of their size. In these circumstances the blocks will inevitably be irregular too (Fig. 17).

Such a situation leads to the layout of sub-minors which zigzag between the plots with branches leading the water to the highest point of each plot. The disadvantages of such a layout are twofold:

- excessive length of sub-minors
- complexity of the organization of the rotation.

In the example given in Fig. 17 the users do not manage the rotation on their own: employees (water-bailiffs, ditchriders) of the network operator not only set the sub-minor intake gate according to schedule but also the dividers at the branches of the sub-minor.

A more rational organization of water distribution is possible when the plot layout can be entirely replanned as shown in Fig. 18. The rectangular layout is designed so as to irrigate (whether by furrow or by basin) along the steepest slope of the natural ground.

The irrigation network is always provided with a network of drainage ditches and channels to evacuate rainfall runoff, flow from plot drains and accidental spill of the irrigation sub-minors.

Drainage channels remove water from the block drainage ditches. They discharge into secondary drainage channels which also receive any spill from the minors. The secondary drainage channels lead to the natural drainage system which also evacuates overflow from the distributaries.

Access roads to the block and individual plots are also laid out

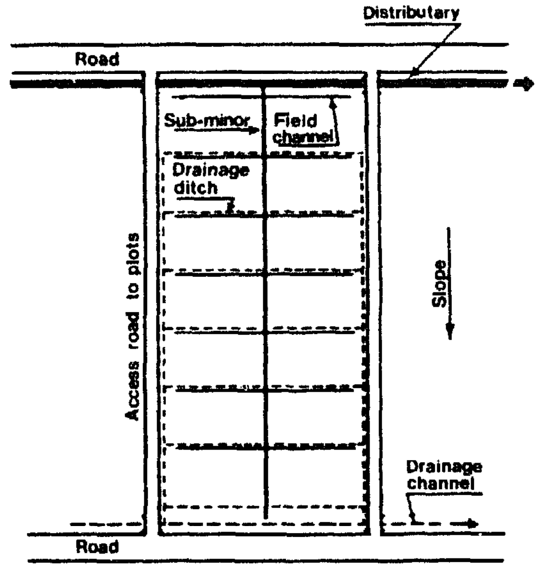


Figure 18 Rational layout of an irrigation block

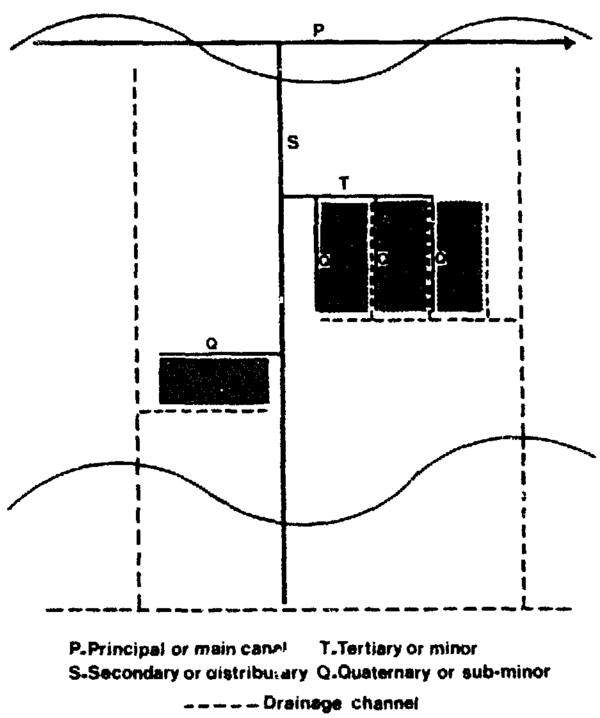


Figure 19 General layout of the distribution network

to follow the rectangular pattern of irrigation canals and drainage channels.

The four-tier structure of a network (main, distributary, minor and sub-minor - also denoted main, secondary, tertiary and quaternary canals) is of course not rigid. The number of successive derivations will depend upon the size of the irrigation scheme or sector, upon the degree of irregularity of the topography and upon the distribution of the blocks within the sector.

As an example, in the case of small schemes which stretch along a river bank, the network may be reduced to a single series of channels which branch off the main canal. This is the case illustrated in Fig. 17 where the second-order canals are in fact sub-minors on which the rotation of the stream size is based.

3.3 STRUCTURE AND LAYOUT OF PRESSURE DISTRIBUTION NETWORKS

Pressure systems consist mainly of buried pipes and are therefore relatively free from topographic constraints.

The aim is to connect all the hydrants to the source by the most economic network. The source can be a pumping station on a river, a canal or a well delivering water through an elevated reservoir or pressure vessel.

Only branching networks will be considered since it can be shown that their cost is less than that of looped networks. Loops are only introduced where it becomes necessary to reinforce existing networks or to guarantee the security of supply.

3.3.1 Design of a Network for On-demand Irrigation

i. Layout of hydrants

Before commencing the design of the network the location of the outlets on the irrigated plots has to be defined. Individual outlets can be grouped in clusters of four or six, depending upon the type of hydrant that is to be installed.

The location of the hydrants is a compromise between the wishes of the farmers, each of whom would like a hydrant located in the best possible place with respect to his plot, and the desire of the water management authority to keep the number of hydrants to a strict minimum so as to keep down the cost of the collective distribution network.

In order to avoid excessive head losses in the on-farm equipment, the operating range of an individual outlet does not normally exceed 200 metres in the case of small farms of a few hectares and 500 metres on farms of about ten hectares.

The location of hydrants and the regrouping of several outlets on a single hydrant is influenced by the location of the plots. In the case of scattered small-holdings the hydrants are situated as far as possible at plot boundaries so as to service up to six users from the same hydrant. When the holdings are large the hydrant is situated preferably at the centre of the area which is to receive water.

ii. Layout of branching networks

a. Principles

On-demand distribution imposes no specific constraints upon the layout of the network: where the land-ownership structure is heterogeneous, the plan of the hydrants represents an irregular pattern of points, each of which is to be connected to the source of water.

For ease of access and to avoid purchase of rights of way it may be decided to lay the pipes along plot boundaries, roads or tracks but since a pipe network is laid in trenches at a depth of about one metre, it is often found advantageous to cut diagonally across properties and thus reduce the length of the pipes and their cost.

A method of arriving at the optimal network layout is described in paragraph 3.3.3. It involves the following three step iterative process:

- **"proximity layout"** or shortest connection of the hydrants to the source
- **"120° layout"** where the proximity layout is shortened by introducing junctions (nodes) other than the hydrants
- **"least cost layout"** where the cost is again reduced, this time by shortening the larger diameter pipes which convey the higher flows and lengthening the smaller ones

The last step presupposes a knowledge of the pipe diameters. A method of optimizing these diameters is described in Chapter 4.

b. Fields of application of pipe network optimization

- case of a dispersed land tenure pattern

A search for the optimal network layout can lead to substantial returns. An in-depth study (ICID 1971) of a network serving 1000 ha showed that a cost reduction of nine percent could be achieved with respect to the initial layout. This cost reduction was obtained essentially in the range of pipes having diameters of 400 mm or more.

In general it may be said that the field of application of network layout optimization mainly concerns the principal elements of the network (pipe diameters of 400 mm and upwards). Elsewhere land tenure and ease of maintenance (accessibility of junctions, etc.) generally outweigh considerations of reduction of pipe costs.

In support of this assertion it is of interest to note that in the case of a 32 000 ha sector, which forms a part of the Bas-Rhône Languedoc (France) irrigation scheme, pipes of 400 mm diameter and above account for less than twenty percent of the total network length. In terms of investment, however, these larger pipes represent nearly sixty percent of the total cost (ICID 1971).

- case of a rectangular pattern of plots

In the case of schemes where the land tenure has been totally redistributed to form a regular checkwork pattern of plots, the pipe network can follow the same general layout with the average plot representing the basic module or unit.

The layout of the pipe network is designed so as to be integrated with the other utilities, such as the roads and the drainage system.

3.3.2 Design of a Network for Irrigation by Rotation

When irrigation water is distributed by rotation, management and supervision of the network should be taken into account at the network layout design stage.

Due consideration must be given to the need to strictly enforce irrigation water rotation. In open channel distribution systems, a farmer cannot draw water without there being repercussions in the immediate neighbourhood. Abusive use of water is therefore detected at once by the rightful user whose supply vanishes in so far as the area of the block is not excessively large.

With pressure distribution systems however the situation is very different: in the lower lying areas, a farmer can open an outlet without affecting his neighbours and yet he may deprive a rightful user who is situated far away on higher ground. If the network has not been laid out to take this constraint into account, supervision of the rotation will weigh heavily on the management authority.

A satisfactory solution to the problem can be found by organizing the rotation in one of the two following ways:

- Rotation at hydrant level: each hydrant of the network is supplied with the duty of water corresponding to the total area served by the hydrant. This discharge or stream size is then rotated through the hydrant outlets to the individual plots in turn and for a period of time proportional to their size.

  This is equivalent to a rotation with variable stream size, the hydrant having the same function as the sub-minors of an open-channel irrigation system.

  Each hydrant is fitted with a flow regulator common to all the outlets and the supervision of the rotation is straightforward.

- Rotation at branch level: in the case of small estates, the stream size equivalent to the duty of the area served by one hydrant might prove to be insufficient. This situation can be overcome by grouping several hydrants on a given branch. It is the duty corresponding to the area serviced by the branch which is then rotated to each hydrant in turn. A flow regulator corresponding to the stream size is placed at the head of the branch. Organized in this way, the branch has the same function as the sub-minors of an open-channel irrigation system.

  The general structure of the network must be designed to allow for a division of the sector into blocks each of which is serviced by a specific branch. No hydrant may be connected directly to the network upstream of the branches which supply the blocks. The layout of the upstream components of the network can be optimized.

3.3.3 Optimization of the Layout of Branching Networks

i. Methodology

The method commonly used (Clément and Galland 1979) involves three distinct stages:

- 1: proximity layout
- 2: 120° layout
- 3: least-cost layout

Stage 1: Proximity layout

The aim is to connect all hydrants to the source by the shortest path without introducing intermediate junctions here denominated nodes. This may be done by using a suitable adaptation of Kruskal's classic algorithm from the theory of graphs.

If a straight line drawn between hydrants is called a link and any closed circuit a loop, then the algorithm proposed here is the following:

Proceeding in successive steps a link is drawn at each step by selecting a new link of minimum length which does not form a loop with the links already drawn. The procedure is illustrated in Fig. 20 for a small network consisting of six hydrants only.

In the case of an extensive network, the application of this algorithm becomes impractical since the number of links which have to be determined and compared increases as the square of the number of hydrants: $(n^2 - n)/2$ for n hydrants. For this reason it is usual to use the following adaptation of Sollin's algorithm:

Selecting any hydrant as starting point, a link is drawn to the nearest hydrant thus creating a 2-hydrant subnetwork. This subnetwork is transformed into a 3-hydrant subnetwork by again drawing a link to the nearest hydrant. This in fact is an application of a simple law of proximity, by which a sub-network of n-1 hydrants becomes a network of n hydrants by addition to the initial network. This procedure, which considerably reduces the number of links which have to be compared at each step, is illustrated in Fig. 21.

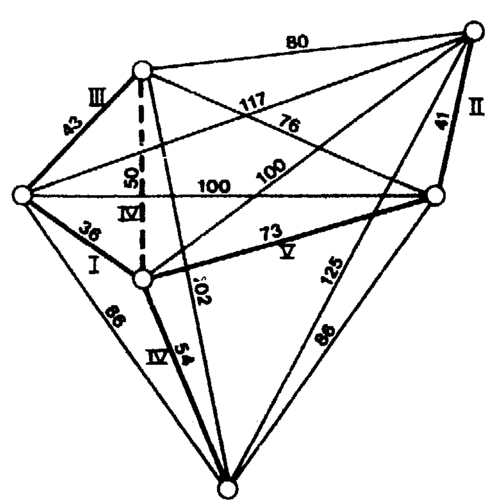


Figure 20 Proximity layout - application of Kruskal's algorithm

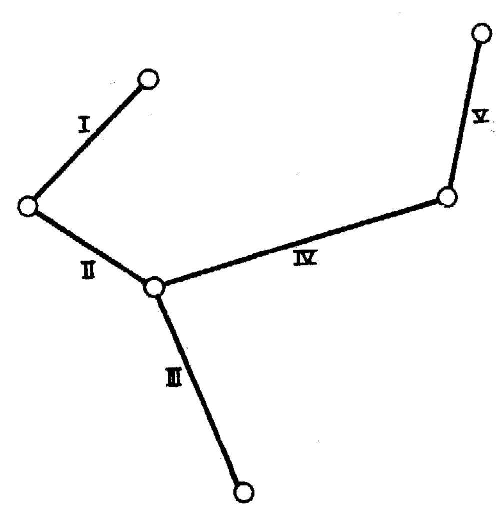


Figure 21 Proximity layout - application of Sollin's algorithm

Stage 2: 120° layout

By introducing nodes other than the hydrants themselves, the proximity network defined above can be shortened.

- Case of three hydrants

Consider a sub-network of 3 hydrants A, B, C linked in that order by the proximity layout (Fig. 22)

A node M is introduced whose position is such that the sum of the lengths MA + MB + MC is minimal.

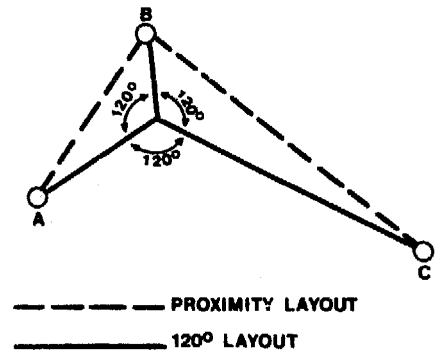


Figure 22 120° layout - case of three hydrants

Let $\vec{i}$, $\vec{j}$, $\vec{k}$ be the unit vectors of MA, MB and MC and let dM be the incremental displacement of node M.

When the position of the node is optimal then

$$d(MA + MB + MC) = (\vec{i} + \vec{j} + \vec{k})dM = 0$$

This relation will be satisfied for all displacements dM when

$$\vec{i} + \vec{j} + \vec{k} = 0$$

It follows therefore that the angle between vectors $\vec{i}$, $\vec{j}$, $\vec{k}$ is equal to 120°.

The optimal position of the node M can readily be determined by construction with the help of a piece of tracing paper on which are drawn three converging lines subtending angles of 120°. By displacing the tracing paper over the drawing on which the hydrants A, B, C have been disposed, the position of the three convergent lines is adjusted without difficulty and the position of the node determined.

It should be noted that a new node can only exist if the angle ABC is less than 120°. When the angle is greater than 120°, the initial layout ABC cannot be improved by introducing a node and it represents the shortest path. Conversely, it can be seen that the smaller is the angle ABC, the greater will be the benefit obtained by optimizing.

- Case of four hydrants

The 120° rule applies to the case of a four-hydrant network ABCD (Fig. 23).

The layout ABC can be shortened by introducing a node M_1 such that links M_1A, M_1B and M_1C are at 120° to each other.

Similarly the layout M_1CD is shortened by the introduction of a node M_1' such that $M_1'M_1$, $M_1'C$ and $M_1'D$ subtend angles of 120°. The angle AM_1M_1' is smaller than 120° and the node M_1 is moved to M_2 by the 120° rule, involving a consequent adjustment of M_1' to M_2'.

The procedure is repeated with the result that M and M' converge until all adjacent links subtend angles of 120°.

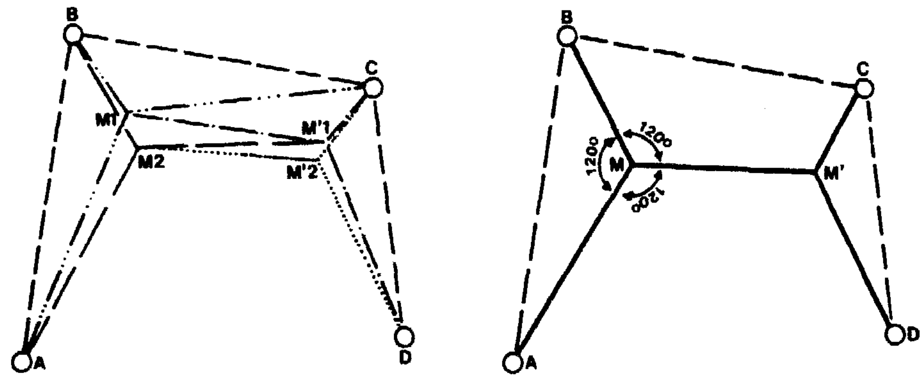


Figure 23 120° layout - case of 4 hydrants

In practice, the positions of M and M' can readily be determined manually with the assistance of two pieces of tracing paper on which lines converging at 120° have been drawn.

A different configuration of the four hydrants, such as the one shown in Figure 24, can lead to a layout involving the creation of only one node since the angle ABM is greater than 120°.

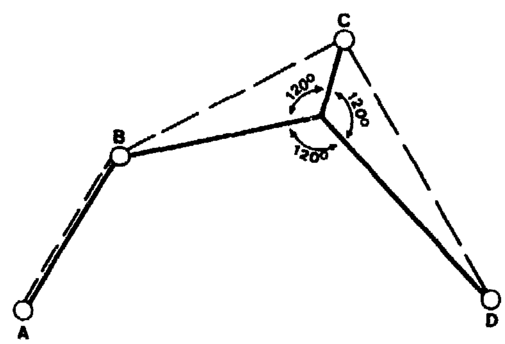


Figure 24 120° layout - case of 4 hydrants (different configuration)

- Case of n hydrants

The above reasoning can be extended to an initial layout consisting of n hydrants. It can be shown that the resulting optimal layout has the following properties:

- the number of nodes is equal to or less than n-2
- there are not more than three concurrent links at any node
- the angles between links are equal to 120° at nodes having three links and greater than 120° when there are only two links.

In practice it is impractical to deal manually with the construction of a network consisting of four or five hydrants, involving the introduction of two or three adjacent nodes, even with the help of tracing paper. Several geometric construction procedures have been devised to facilitate such layouts, but these are rather cumbersome and the problem can only be resolved satisfactorily with the assistance of a computer.

In actual fact it rarely happens that it is necessary to create more than two or three consecutive nodes. It should also be noted that the benefit to be gained by optimizing decreases as the number of adjacent links to be examined increases.

Stage 3: Least-cost layout

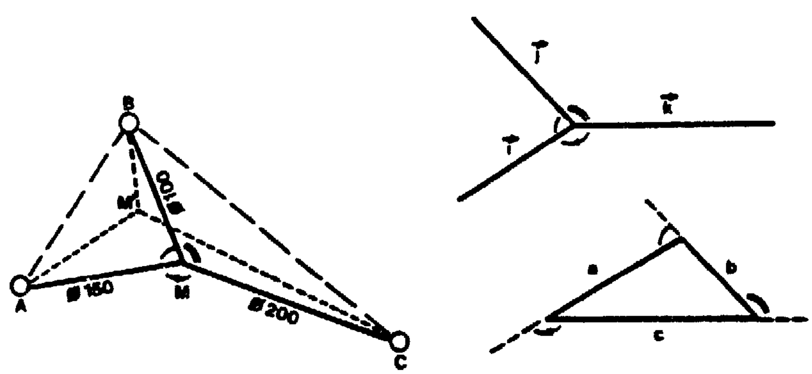
Although the layout which results from applying the 120° rule represents the shortest path connecting the hydrants, it is not the solution of least cost since no account is taken of pipe sizes. The total cost of the network can further be reduced by shortening the larger diameter pipes which convey higher flows whilst increasing the length of the smaller diameter pipes which convey smaller flows. This will result in a modification of the angles between links at the nodes.

Going back to the three hydrant sub-network A, B, C in Figure 22 the position of the new node M' will satisfy the vectorial relation:

$$a\vec{i} + b\vec{j} + c\vec{k} = 0$$

where a, b and c are the prices per unit length of the pipes connecting the node to the hydrants A, B and C.

The angles of the pipes converging at the new node M can therefore be determined by constructing a triangle the length of whose sides are proportional to a, b and c. The position of M' can be adjusted as before with the help of tracing paper on which suitably orientated converging lines have been drawn (Fig. 25).



Section	Diameters	Unit price
MB	100	200
MA	150	300
MC	200	400

Proximity layout
120° layout
Least cost layout

Figure 25 Least-cost layout - case of 3 hydrants

The least-cost layout resembles the 120° layout but the angles joining the pipes are adjusted to take into account the cost of the pipes.

It should be noted moreover that the step which leads from the 120° layout to the least-cost layout requires a knowledge of the pipe sizes and it can therefore only be taken once the pipe sizes have been optimized as discussed in Chapter 4.

ii. Application of the method

There is no doubt that the 120° layout is an improvement on the initial proximity layout and that the least-cost layout is a further refinement of the 120° layout. It is not certain however that the complete process produces the best result in all cases.

The optimum attained is relative to a given initial layout of which the proximity layout is only the shortest path variant. It could be that a more economic solution might be found by starting with a different initial layout, differing from that which results form proximity considerations, but which takes into account hydraulic constraints.

In practice, by programming the methods described above for computer treatment, several initial layouts of the network can be tested. The first of these should be the proximity layout. The others can be defined empirically by the designer, on the basis of the information available - elevation of the hydrants and distance from the source - which enables potentially problematic hydrants to be identified.

By a series of iterations it is possible to define a "good" solution, if not the theoretical optimum.

iii. Example of layout

The layout of a small network designed to supply irrigation water to 240 ha (net) is shown on Figures 26 to 29. The successive design phases produced the following results:

	length	cost	figure
proximity layout	105.9		26
120° layout	100	108.2	27
least-cost layout	104.6	100	28

A network layout, connecting the same hydrants empirically by following roads, tracks and plot boundaries was found to have a length of 126.9 for a cost of 107.6 (Fig. 29).

It should be noted that the above estimates are based on the cost of engineering works only. They do not include the purchase of land, right-of-way or compensation for damage to crops which might occur during construction, all of which would increase the cost of the optimum network.

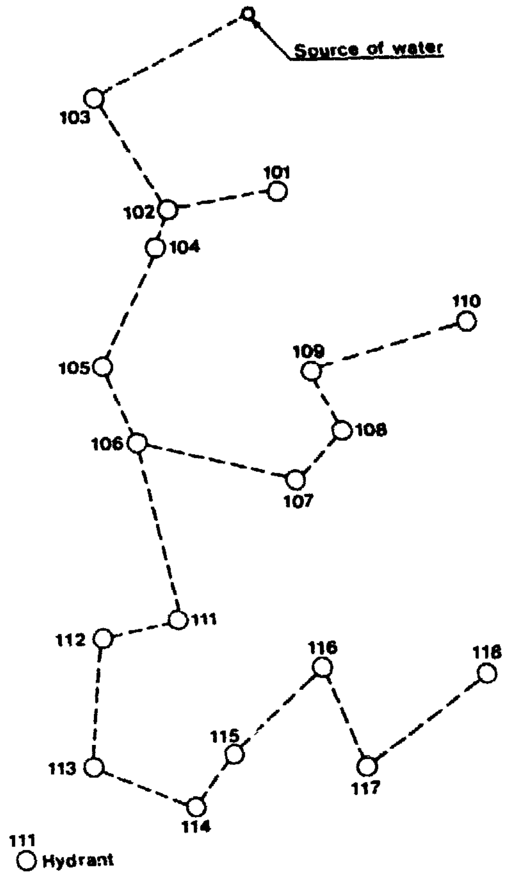


Figure 26 Proximity layout

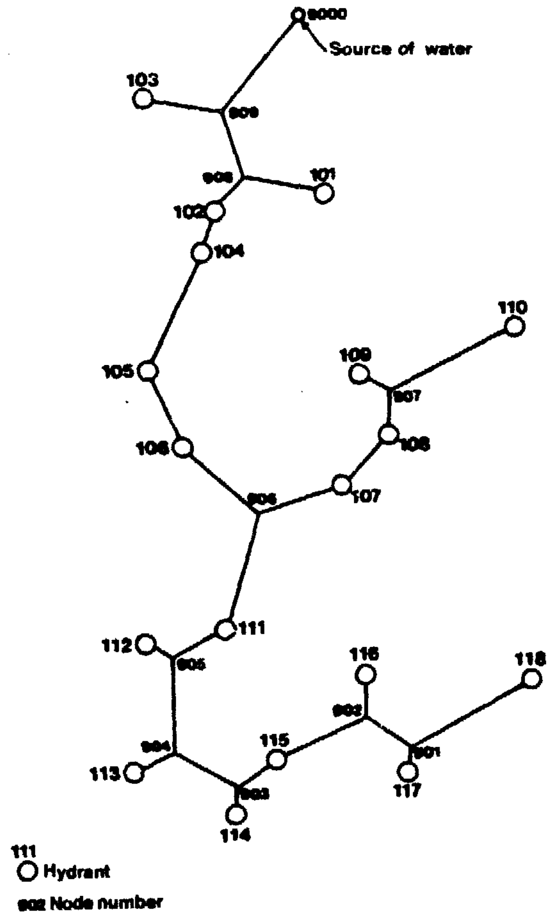


Figure 27 120° layout

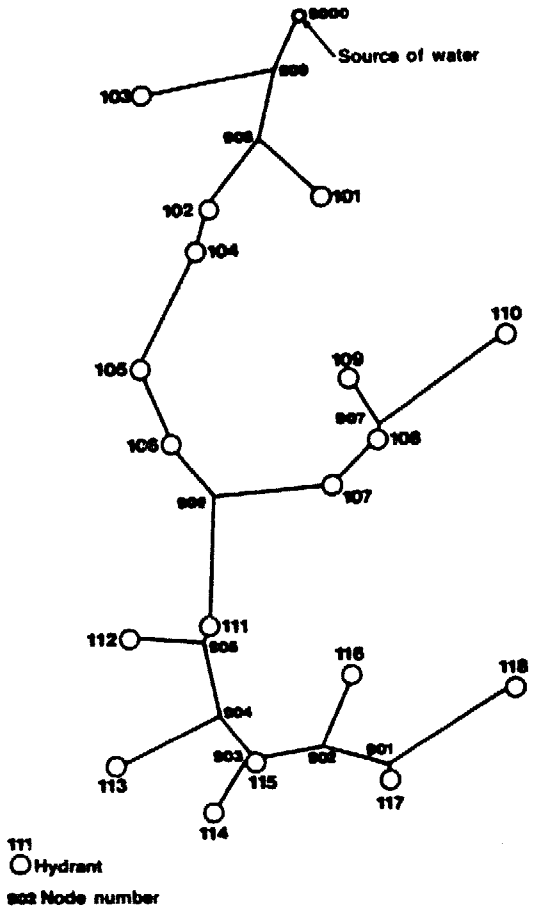


Figure 28 Least-cost layout

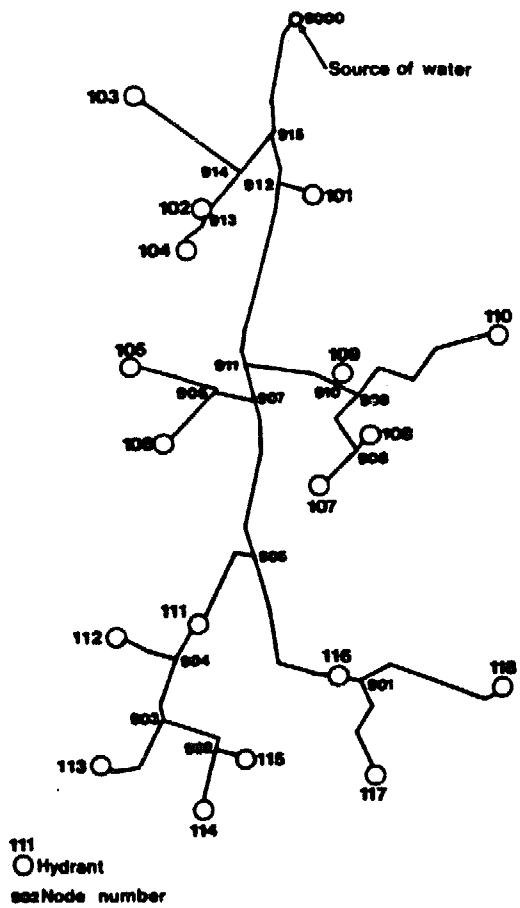


Figure 29 Layout best suited to field conditions

3.4 WATER SUPPLY BY ROTATION AND DETERMINATION OF FLOWS

Open-channel irrigation systems are generally operated on a rotational basis. In the following paragraphs the general principles involved in the concept of a rotation and their impact upon flows in the system will be examined. The specific case of pressure networks will be dealt with separately.

3.4.1 Fixed Stream Size Rotation

i. Stream size

The irrigation stream size is the flow rate which the network can deliver at the farm outlet. The value of the stream size is generally less than the maximum flow which a farmer is capable of handling taking into account his level of competence and the type of irrigation practice. The stream size is greater than the minimum flow required by the on-farm irrigation method adopted.

The stream size generally ranges from 20 to 80 l/s with the lower values reserved for furrow irrigation and the higher values for border or basin irrigation. Market gardening may require stream sizes as low as 10 l/s whereas large pastures may require as much as 200 l/s.

ii. Rotation

When a fixed stream size is rotated each farm is supplied with the stream size during a fixed period of time according to a pre-established schedule.

If a = plot irrigated area (ha)
v = peak period water requirement on a 24 h basis (l/s/ha)
θ = time interval between the commencement of two successive irrigations (hours)
m = stream size (l/s)
t = duration of each irrigation (hours)

then $$v\,a\,\theta = mt$$

The stream size (m) and the irrigation interval (θ) having been determined, the stream size is made available to each farmer during a time (t) which is proportional to the area to be irrigated

$$t = v\,a\,\theta/m$$

The system operator determines each year and for each irrigation block the time during which the stream size is made available to each farmer for one complete rotation during the period of peak plant water requirements and taking into account the nature of the crops. An irrigation schedule is established which details the order of rotation of the stream size or stream sizes to the different farm outlets.

The schedule drawn up for one rotation during the peak demand period is generally retained for the whole irrigation season. By selecting a value of the irrigation interval (θ) which includes a fraction of a day, it is possible to avoid the systematic attribution of the least practical times (essentially the night hours) to the same farmers throughout the irrigation season. The desired irrigation interval to attain this objective is obtained by a slight adjustment of the optimal interval and irrigation dose within the tolerance limits.

During the peak demand period each sub-minor will operate at maximum design discharge during an interval of time less than or equal to the duration of a rotation (θ).

EXAMPLE 7 - DESIGN DISCHARGE AND OPERATING SCHEDULE OF SUB-MINORS

Peak period water requirement (v)	1.1 l/s/ha
Stream size (m)	33 l/s/ha
Total area irrigated in 1 block (A)	50 ha
Irrigation interval (θ)	6¼ days = 150 hours

The flow in the sub-minor will be

1 stream size during time θ = 150 hours
2 stream sizes during time θ' with

$$(1 \times 150 + \theta')33 \times 3.6 = 150 \times 1.1 \times 3.6 \times 50$$

which yields θ' = 100 hours.

This duration must be increased to allow for the time during which water is moving in the sub-minor from one outlet to the next without being distributed.

The irrigation schedules of the blocks which make up a sector should be planned so as to ensure continuity of flow in the minors and canals of higher order.

A detailed discussion of the method of establishing a rotation will be found in Section 3.4.4.

iii. Design discharge

The risk of the most demanding crop being concentrated in one block should be examined at the **project formulation stage**.

- In the case of **sub-minors** which service relatively small areas, the **duty of the most demanding crop** v* (l/s/ha) is normally determinant. Thus if it is decided to operate with a single stream size m (l/s) in each sub-minor, the irrigated area of the block will be less than than m/v*.

- In the case of **main canals** which serve large areas it can be assumed that the most demanding crops are distributed evenly over the project area. The main canals are therefore calibrated to convey the **peak** water requirements of the **average cropping pattern** during a dry year.

- In the case of **distributaries and minors**, the duty to be taken into consideration will increase proceeding downstream, starting with the **average cropping pattern** water requirements and ending with those of the **most demanding crop.** This is to allow for the increasing risk of concentration of high demand crops as the area to be served decreases. The design discharge of a reach is based on the duty of the areas downstream of the reach, rounded off to the next higher number of stream sizes.

  The design discharge of a reach must be increased to allow for the water lost during conveyance. The conveyance efficiency depends not only on the nature of the canal (lined or unlined) but also on the type of regulation.

3.4.2 Reduction of Constraints Associated with Distribution by Rotation

Although extremely simple to operate, the fixed stream size rotation suffers from two drawbakcs:

- need to adjust the flows diverted to each block during the rotation requiring considerable involvement on the part of the system operating staff;
- lack of flexibility of irrigation conditions and therefore ill-adapted to the simultaneous production of crops with varying rooting depths and very constraining for the farmer.

i. Rotation of a variable stream size

Irrigation with variable stream sizes reduces the work load of the water bailiffs (ditchriders). Since the stream size can vary within a fairly wide range, the sector is divided into blocks supplied by sub-minors so that

$$m_1 \leqslant A_i \cdot v/e \leqslant m_2$$

where: m_1 and m_2 = lower and upper limits of stream size (l/s)
A_i = irrigated area of block (ha)
v = peak duty of plot (l/s/ha)
e = sub-minor conveyance efficiency

Having satisfied the above condition by a suitable division of the sector into blocks, **each block is attributed a specific stream size (m)** whose value is

$$m = A_i \; v/e$$

and which must be **continuously** available in the block sub-minor.

The farmers operate the rotation on the sub-minor according to a schedule determined by the system operator. The staff is not required to adjust gate positions since the network operates continuously during the period of peak water requirements up to the intakes of the sub-minors.

In practice the stream size required by each block is calculated annually by the system operator who takes into account the actual irrigated areas (or water subscriptions) or the specific water requirements.

ii. <u>Rotation with variable irrigation interval</u>

Distribution with a **variable interval** results in increased flexibility of irrigation, an interval of θ_1 being selected for shallow-rooted crops and $\theta_2 = 2\theta_1$ for deep-rooted crops.

EXAMPLE 8 - ESTABLISHMENT OF VARIABLE IRRIGATION INTERVAL

In an irrigation sector two crops are predominant.

- Market gardens requiring 350 m³/ha at 4-day intervals with a duty of 1 l/s/ha
- Forage crops requiring 780 m³/ha at 9-day intervals with a duty of 1 l/s/ha

The following practical intervals will be adopted:

θ_1 = $3\frac{3}{4}$ days and 325 m³/ha for the market gardens
θ_2 = $7\frac{1}{2}$ days and 650 m³/ha for the forage crops

In each block the first rotation (0 to $3\frac{3}{4}$ days) will irrigate all the market gardens and half the area under forage crop. During the following rotation ($3\frac{3}{4}$ to $7\frac{1}{2}$ days) all market gardens will again receive water as will the remaining half of the area under forage crops. This arrangement is carried through the whole irrigation season.

iii. <u>Suppression of night-time irrigation</u>

Night irrigation is very unpopular with farmers and very constraining in the case of furrow irrigation which calls for continuous presence.

In order to keep down the size and cost of the canal system it is usual to avoid night irrigation at all times **except during the peak water requirement period.** In practice, high flexibility of use offered to the farmers and ambitious objectives as regards saving of water are very costly.

a. A simple but also inefficient method is often found to be in use in old irrigation networks. The rotation is fixed once and for all at the start of the irrigation season on the basis of the crop peak water requirements. The irrigation interval selected does not correspond to a whole number of days.

   In the foregoing example plots with shallow-rooted crops received water every $3\frac{3}{4}$ days (90 hours). During a 15-day period it can be seen that a farmer receives water three times during the day and once at night. Except during the period of peak water requirements a farmer can avoid irrigating at night by simply refusing his turn once out of four rotations.

If this type of solution requires no particular measures on the part of the system operator, other than to ensure the evacuation of the unused stream size at the end of the sub-minor, it leads to very large losses of water except during the peak demand period. For this reason the method should only be retained when the geographical location of the sector allows for recovery of the unused water for use further downstream.

b. A second solution consists in the establishment at the start of the season of several rotations each corresponding to a stage in crop growth. In its simplest form, this consists of two rotations: one for the peak period and the other for off-peak.

The off-peak rotation, if established for the same stream sizes and intervals as for the peak, results in reduced duration of use and the schedule can be adjusted to avoid night irrigation.

With this type of organization considerable attention must be given to the problem of **regulation**. When networks are large, conveyance times in open channels can often be of the order of 10 hours. In such conditions it is only possible to cater for the simultaneous opening in the morning and closing in the evening of sub-minor and/or minor outlets by the provision of suitably located **storage reaches**. In this way the upstream conveyance channels operate in near steady flow conditions whereas the operation of the downstream channels is discontinuous.

c. A third and even more flexible solution consists in the continuous recalculation of the rotation according to the expressed needs of the farmers.

The farmers indicate one week ahead to the system operator's staff the time during which they will use the stream size. On the basis of these requests the operator establishes the coming rotation in such a way as to avoid night irrigation where possible.

Clearly this type of organization makes for a maximum of flexibility in the management of the water on the part of the farmers. The accounting procedure is also simplified, the water being charged for according to the actual time of use of the stream size by each individual.

On the other hand it involves the system operator in a considerable amount of work and calls for a large and competent field staff to ensure contact with the farmers, note their requirements and establish the modified rotation. Moreover, these variable rotations, which as far as possible avoid night irrigation, call for an **elaborate form of flow regulation** in the canal system in order to control the movement of water.

3.4.3 Case of Pressure Networks Operated by Rotation

The organizational constraints which are inherent to a pressure network operating on a rotation basis have been discussed in Section 3.3.2. The determination of the flows to be conveyed by the system is quite straightforward:

If the rotation is organized at hydrant level all that is needed is to define the design discharge of each hydrant and then to summate the discharges of all hydrants downstream of the link.

Where the rotation is organized at branch level it is essential to acurately define the individual units upon which the rotation is to be organized. Each unit should service plots which are as homogeneous as possible. The total area of each unit should be such that the corresponding stream size is acceptable to the smallest as well as the largest plots within the unit.

When this type of distribution is adopted it is essential to define the layout of the network at the same time as the layout of the units on which the rotation is based.

To each unit corresponds a stream size which is conveyed by all the links of the corresponding distribution branch.

The flow in the network which feeds these branches is determined by summating, from downstream to upstream, the stream sizes which are to be delivered.

It is obviously possible to imagine a more complex system with minors which convey two or more stream sizes in order to shorten the total length of the network. This will result, however, in making more difficult the organization of the rotation as well as its supervision.

The flow at the head of a network operated by rotation is therefore, in theory, equal to the duty required by the total irrigated area. In practice, however, even if the on-farm equipment is well suited to the continuous use of the flow made available during the authorized period, certain interruptions are inevitable due to change of lines and it is safer to calculate the flow to be distributed by taking a value of the duty which is slightly greater than its strictly theoretical value.

3.4.4 Establishment of a Fixed Stream Size Rotation

i. Irrigation interval

The irrigation interval is defined here as the time interval between two successive irrigations of the same plot. The length of this interval is the only parameter which can be adjusted by the system operator once the distribution network is in place.

The simplest case is where the irrigation interval corresponds to a whole number of days but this means that a plot always receives water at the same hour of the day whether or not this hour is convenient to the farmer.

This handicap can be avoided quite simply by choosing an interval which includes a fraction of a day. An interval of $n + 1/p$ days for example with $p = 4$ ensures that for successive rotations the farmer will start irrigating in the morning, in the middle of the day, in the evening and at night. The rotation will occur at the same time of day after p rotations.

Within each rotation each plot is attributed an irrigation time (t) such that

$$t = v\,a\,\theta/m$$

where v is the nominal discharge (l/s/ha) corresponding to the plot water requirement
A is the plot irrigated area (ha)
θ is the rotation interval (hours)
m is the stream size provided by the network (l/s).

ii. Irrigation schedule

In order to establish the irrigation schedule for each plot, the duration of flow in each sub-minor must first be determined. The next step is to organize the sequence of operation of the sub-minors. Lastly, the order in which water is attributed to each plot is decided. Stated in this manner, the procedure appears to be straightforward. In practice, however, the complexity of the problem may be such that an optimal solution cannot be determined manually.

a. Duration of flow in a sub-minor

The duration of flow in a sub-minor is determined on the basis of a single irrigation interval (θ). During this interval, the duration of flow (T) is equal to the sum of irrigation times (t) of each plot.

The duration (T) must be increased by an amount equal to the sum of the conveyance delays. The water authority must take into account the delay which occurs from the moment that water is diverted to a sub-minor and the time when irrigation can actually start on the farthest plot.

b. Scheduling the operation of sub-minors

A number of hydraulic and phsyical constraints have to be satisfied when organizing the sequence of operation of the sub-minors.

- The hydraulic constraints

The first hydraulic constraint concerns the management of the upstream canals.

The flow into the sub-minors must be timed in such a way that the demand can be met at all times: a momentary excess of demand at one point can lead to failure to satisfy the water requirements downstream; a temporary refusal of water can in certain conditions result in a spill. It is essential to plan the withdrawal of water from the main canal in such a way that the main canal operates as closely as possible under steady conditions, with flow decreasing regularly in a downstream direction.

The second hydraulic constraint concerns the conveyance of water in the sub-minors.

A time delay occurs between the moment water is allowed to flow into a sub-minor and the moment when water is available at the last turnout. This delay is known as the conveyance interval. Under normal circumstances the water bailiff accompanies the water and ensures that each outlet receives the correct flow value, removing obstacles where necessary.

In practice this hydraulic constraint means that the water bailiff is fully occupied during the time that water has to be accompanied in the sub-minors. It is therefore advisable to organize their operation in such a way that there is no overlapping of conveyance intervals in the whole of the sector under the water bailiff's jurisdiction.

- The physical constraints

The water bailiff's duties involve the execution of a certain number of tasks such as the opening of sub-minor gates, accompanying the water, closure of the turnouts. When a network with many branches operates on a fixed stream size these tasks are so numerous that the water bailiff is fully employed. When planning the sequence of operation of the sub-minor it should be remembered that the bailiff is single-handed and that time must be allowed for his movements from gate to gate. The sequence should therefore be planned in order to minimise his movements.

c. **Block scheduling**

The sequence of irrigating plots within a block can be planned in several ways. In each case the aim is to reduce time lost during intervals when water is available but not distributed.

- Hydraulic sequencing

Water is distributed to plots in either an upstream to downstream order or the contrary. This type of sequence is simple to organize but is not to the advantage of farmers who cultivate widely dispersed plots.

Scheduling from downstream to upstream has the merit of facilitating the supervisory task of the water bailiff: a farmer opens the turnout to his plot and closes the sub-minor at the level of his turnout. If he fails to reopen it at the end of his turn the next user upstream is in no way penalized since all he has to do to obtain water is to close the sub-minor at his level and divert water to his plot. If the sequence had been organized from upstream to downstream, he would have had to re-position his neighbour's flashboard.

- Sequencing by farms

Water is distributed successively to all plots belonging to a single farm wherever they are located, on one or more sub-minors. The optimal distribution sequence involving the least conveyance intervals becomes very difficult to establish in this case. However far the optimization is pushed, conveyance intervals are invariably longer than is the case with hydraulic sequencing. Moreover users must show a high degree of responsibility, making sure that they do not retard the flow in the sub-minor on completion of their turn.

d. Preparation of the irrigation time sheets

In the first instance the system operator collects the water requirements data relative to each plot, determines the irrigation durations and establishes the sequence of operation of the sub-minors. At this stage it is possible to attribute to each plot the irrigation timings for the whole season.

The establishment of an irrigation schedule of this type calls for the preparation of several documents. Although each water authority has its own methods, three basic documents are commonly used.

- Irrigation roster

The irrigation roster is retained by the system operator and indicates the sequence of irrigation of the plots in each block. The operator records each year the duration of supply of the stream size, correcting the conveyance intervals where necessary. The duration of supply of the stream size and the conveyance intervals are defined at this stage.

On the basis of these values, the distribution of the individual stream sizes is organized, taking into account the constraints discussed above. This is conveniently done by drawing a bar-chart for the first period. The next period follows by simple staggering.

Once established the time and day of distribution of each stream size, the time of opening and closure of each plot turnout can be entered in the irrigation register, allowance being made for the conveyance intervals. Only the timings of the first cycle are recorded in the irrigation register.

- User's timetable

The user's timetable is prepared for each plot. It is a daily calendar on which are entered the irrigation dates with the time of opening and closure of the outlets for the year. A single form is issued for each plot which means that in some cases a farmer will receive several forms. The inconvenience which this entails dissappears however if the operator accepts to sequence the rotation by farms as described above. The timetable in that case is prepared on a single sheet and indicates the starting and finishing times for the farm as a whole and the order in which the water is distributed to each plot.

- Water bailiff's time sheet

The water bailiff's time sheet is based on the diagrams in the irrigation roster. The time sheet indicates the times of opening and closure and of accompanying the water distribution for the duration of the irrigation season.

3.5 FLOW IN PRESSURE NETWORKS OPERATING ON-DEMAND

By definition, a farmer receiving water on-demand is provided with a gated outlet or hydrant which is connected to the collective distribution network and which he is free to operate at any time without having to inform the system operator.

Generally this form of distribution is restricted to pressure systems. In the case of open-channel networks technical and economic considerations tend to impose "upstream control" at least at the level of the lower order canals (minors and sub-minors). It is then necessary either to adopt a rotation system or to ensure that there be a dialogue between the farmer and the network management before operating an outlet gate.

3.5.1 Flow at Farm Outlets

Although a farmer supplied by an on-demand system is free to use his outlet at any time, a physical constraint is nevertheless imposed as regards the maximum flow he can draw. This is achieved by fitting the outlet with a flow regulator.

The stream size attributed to each outlet is defined according to the size and crop water requirements of the plot. It is always greater than the duty (the duty is the flow based on peak period water requirement on a 24-hour basis) so as to give the farmer a certain degree of freedom in the management of the irrigation.

The ratio between the stream size attributed to each outlet and the duty is a measure of the "degree of freedom" which a farmer has to arrange his irrigation.

The wide variety of agronomic situations is reflected by the wide range of the value of the degree of freedom found in practice:

- high degree of freedom: family holdings with limited labour, low crop water requirements, small or scattered plots, low investment level in on-farm equipment;

- low degree of freedom: large size plots, large scale farming, abundant labour, high investment level in on-farm equiment.

Since the maximum flow at outlets is fixed by flow regulators it is usual to opt for a standard range of flows. In south-eastern France for instance, a range of six outlets has been standardized, corresponding to the following flows:

Class of outlet	0	1	2	3	4	5
flow in m^3/h	7.5	15	30	50	75	100
flow in l/s	2.1	4.2	8.3	13.9	20.8	27.8

3.5.2 The Demand Formula

The characteristics of the network having been established (location of outlets, flow at each outlet, approximate layout) the next step is to define the flow in each link of the network. In the following discussion the term "node" includes both hydrants and junctions of two pipes whereas the term "link" is used to describe the pipe connecting any two nodes.

For on-demand irrigation the flow attributed to each outlet considerably exceeds the duty. In other words the duration of flow through an outlet is well below 24 hours each day. As a result it is most improbable that all outlets are open at the same time and it would not be reasonable to dimension the network to convey a flow equal to the sum of the outlet capacities.

The probabilistic approach to the determination of the flow in a network which is described below was originally developed by Clément (1966).

i. Case 1: Equal flow at all outlets

Let us examine the flow in any one link of the network and assume, at first, that the network downstream of this link has (R) outlets of equal discharge (d). The maximum flow in the link under study will occur during the peak irrigation period for which the following conditions hold:

- duration of peak: T (peak month or 10-day peak)
- duration of operation of the network during the peak period: T'
- use-coefficient of the network: $r = \frac{T'}{T}$

  The significance of this parameter will be examined at a later stage.
- flow on a continuous basis in the link during the peak period: D

It follows then that the mean duration of flow at each outlet (t') is given by:

$$t' = \frac{\text{volume of water required}}{\text{discharge}} = \frac{DT}{dR}$$

The model is based on the hypothesis that the outlets of the network are independant and that they operate in a random manner during the period of operation (T') of the network. Hence the frequency (F) or probability (p) of operation of each outlet is:

$$\frac{1}{F} = p = \frac{t'}{T'} = \frac{t'}{rT} = \frac{D}{rRd}$$

The probabilities, in a population of R outlets, that any one outlet is either open or closed are:

$$p(\text{open}) \quad \text{and} \quad q(\text{closed}) = 1 - p$$

The number of open outlets is therefore a variate (or random variable) with a binomial distribution having a mean value Rp and a variance Rpq.

The probability (P_q) that among the R outlets of the network there are N outlets operating simultaneously is:

$$P_q = \sum_{i=1}^{i=N} C_R^i \; p^i \; q^{R-i} \qquad (59)$$

where C_R^i is the number of combinations of R objects taken i at a time.

When R is sufficiently large, probability theory can be used to demonstrate that the binomial distribution approximates to the normal or Gauss distribution.

To the discharge Q, for which the link is designed, is associated a "quality of operation" P_q which is defined as the probability that the demand does not exceed the discharge Q.

The link must therefore be designed to convey a discharge:

$$Q \quad = \quad Rpd + U(P_q)(Rpqd^2)^{\frac{1}{2}} \qquad (60)$$

where $U(P_q)$ is obtained from tables of the reduced variable of the normal distribution. For the levels of quality of operation usually adopted, $U(P_q)$ has the following values:

P_q	$U(P_q)$
0.99	2.324
0.95	1.645

It is worth noting that the first term of Eq (60) corresponds to the **mean** discharge available during the peak period T':

$$Rpd \quad = \quad R\frac{D}{rRd}d \quad = \quad \frac{D}{r} \qquad (61)$$

The second term includes, within the second bracket, the variance of the discharge.

ii. Case 2: Unequal flow at the outlets

The demand formula can readily be extended to cover the case where the flows at the farm outlets are not equal.

The outlets are arranged in homogeneous groups according to the flow class to which they belong (cf. 3.5.1).

For each class of discharge d_i into which R_i outlets fall, the probability (p_i) of operation of the outlets is calculated.

The peak flow is given by the **demand equation:**

$$Q \quad = \Sigma_i R_i p_i d_i \; + U(P_q)(\Sigma_i R_i p_i q_i d_i^2)^{\frac{1}{2}} \qquad (62)$$

iii. Values of the parameters

The application Eq (62) involves the parameters r and $U(P_q)$.

- **the network coefficient (r)** has no real physical significance. It can be assimilated to a factor of safety or adjustment with respect to the model of operation of the farm outlets. It caters for the non-random behaviour of the farmers.

  Normally the values selected for this parameter lie between 16/24 (0.67) and 22/24 (0.93). As will be seen later, the analysis of the performance of existing networks is the most reliable approach to the selection of the coefficient best suited to a given irrigation context (CTGRF 1977).

- **the parameter $U(P_q)$** defines the "quality of operation" of the network; it normally has a value of 0.99 or 0.95. It is hardly possible to go below a value of 0.95: detailed analyses of network operation (Galand et al. 1975) indicate that in the case of on-demand systems, failures are concentrated in certain unfavourably situated branches of the network. A significant reduction of this parameter beyond these values can lead to the occurrence of inacceptable failures to satisfy the demand in certain parts of the network.

In view of the hypotheses made when formulating the demand equation it is recommended that a deterministic approach be adopted at the extremities of the network by cumulating the flows at the outlets when their number falls below a certain value which in practice lies between four and ten.

iv. Some orders of magnitude

In order to illustrate the order of magnitude of the flows conveyed by a network operating on-demand, a network of homogeneous outlets will be taken as an example and a relation estalished between

$\frac{Q}{D}$: the ratio of the network peak flow to its flow assumed on a continuous basis, a ratio which is a measure of the overcapacity of the network and which is characteristic of on-demand operation,

and

$x=\frac{Rd}{D}$: the ratio between the flow at the outlets and the continuous flow, a ratio which defines the freedom afforded to farmers to organize their irrigation.

The demand equation (60) can be written:

$$Q = Rpd + U(Rpqd^2)^{\frac{1}{2}}$$

By substituting for p and q their values as defined in (i), then

$$\frac{Q}{D} = \frac{1}{r} + U\left[\frac{1}{rR}\left(x - \frac{1}{r}\right)\right]^{\frac{1}{2}}$$

For values of $U = 1.645$ ($P_q = 0.95$) and $r = 0.75$, the ratio Q/D has the following values:

R	for x = 2 Q/D	for x = 4 Q/D
10	1.82	2.31
20	1.68	2.03
100	1.49	1.64

It can be seen that for on-demand systems, the ratio of the peak flow in the network to the assumed continuous flow increases as the number of outlets decreases. With outlet capacities two to four times greater than the duty, the peak flow in a network having 100 outlets is only 50 to 65 percent greater than the continuous flow.

The values of the peak to continuous flow ratio (Q/D) quoted above refer to a network designed to supply equal flows at all outlets. When the outlet design flows are unequal, the values of the ratio are slightly greater.

Nevertheless, whether the outlets are homogeneous or not, taking into account the probability of the demand being spread results in a network peak design flow which is very much smaller than that which would be obtained by summating the flows at all outlets.

v. Determination of the continuous flow

In order to be able to apply the above methodology it is necessary to know the value of the continuous flow (D) in the network downstream of the link under consideration. Its value can readily be determined when:

- The cropping pattern is identical throughout the area. If this is so, the unit continuous flow v (l/s/irrigated ha), estimated by giving due weight to each of the crops, holds good for every farm and all branches of the network under consideration.

- The cropping intensity is identical throughout the area. When this is so, the ratio (K) of the net irrigated area to the gross area also holds good for every holding and all parts of the network under study.

  The continuous flow (D) in a given link is then simply a function of the gross area (Ag) downstream of the link:

$$D = v\ K\ Ag \qquad (64)$$

  This simplification is often used. Some designers however prefer to allow for a certain degree of variability of the values of the unit cotinuous flow (v) and of the irrigated/equipped ratio (K). Hence the unit continuous flow (v) at the lower extremities of the network will be adjusted to the water requirements of the most demanding crop whereas upstream its value will correspond to the standard cropping pattern adopted for the whole network. In the same way, the net/gross ratio (K) will tend towards unity at the network extremities whereas the mean value estimated for the whole area will be used upstream. In practice both the unit continuous flow and the cropping intensity values will tend to vary with the areas serviced.

3.5.3 Field of Application of On-demand Irrigation Networks

A large number of on-demand irrigation networks, dimensioned according to the methodology described above, have been developed and successfully operated over the past fifteen years.

In spite of this, the flow determination model is often misunderstood and inadequately applied. For this reason, many of the criticisms levelled against on-demand systems are attributable to shortcomings in the project design.

Experience shows that for on-demand systems to give satisfactory results, three basic premises must be satisfied:

i. Well-managed family-holdings

On-demand systems were at the onset placed at the disposal of small family holdings on which diversified crops were grown.

These farmers had a high standard of farm management and were eager to make the best possible use of this new tool which allows for a more flexible integration of irrigation practice amongst other farming activities.

This type of system is probably less suited to other social environments such as very large farms with abundant hired labour or very small holdings run by unskilled farmers.

In the case of the very large farms with abundant labour, the flexibility afforded by an on-demand system would appear to be superfluous since detailed planning of irrigation tasks entrusted to personnel exclusively engaged in this work is possible. Distribution on a rotation basis seems better suited to such a situation, provided that the stream size is sufficient.

On-demand systems may not be best suited to the needs of very small holdings if the farmers have a low level of skill. Here the existence of a rotation can provide a strict framework which assures a proper understanding of the quantity of water to be used, as well as the proper frequency of application.

Due attention must therefore be paid to the selection of a type of irrigation flow control which is coherent with the socio-economic context of the project.

ii. Volumetric water rates

Irrigation water must be sold on a volumetric basis if on-demand systems are to operate economically. If water meters are not installed, the farm outlets will remain open for durations greater than was allowed for at the design stage and the system will not operate satisfactorily.

iii. Correct estimation of design parameters

a. **Degree of freedom (x)**

As was stated above, the degree of freedom that is to be afforded to farmers (ratio of the flow at the outlet to the duty) is selected according to criteria such as size and dispersion of plots, availability of labour, type of on-farm equipment, frequency of irrigation.

Outlets with capacities of one and a half to twice the value of the duty correspond to the lowest feasible degree of freedom. With smaller values, the probability of an outlet being open becomes too great for the demand model to apply.

Conversely, outlet capacities should not exceed six to eight times the value of the duty. This corresponds to a very high degree of freedom.

b. **Use coefficient (r) and quality of operation (P_q)**

It must be pointed out that the theoretical approach, like all other models, only offers a schematic representation of an actual network. The model must be adjusted or calibrated by introducing field data relative to existing networks.

The use coefficient (r) has no physical significance. It is an adjustment coefficient used to account for the non-random behaviour of the farmers. Values of this parameter should be, whenever possible, selected for homogeneous regions and for particular crops.

In the specific context of south-east France it has been found that a high quality of network operation has been obtained with values of r = 0.75 and P_q = 0.95. This region is characterized by irrigation sectors consisting of small family holdings growing diversified crops and which have the benefit of a high degree of freedom (x).

The methodology developed in connection with these networks (Galand **et al.** 1975; Fattah 1981; CTGREF 197?) could be usefully applied elsewhere to ensure that project designs are appropriate to their environment.

3.5.4 Limited On-demand Irrigation

On-demand irrigation systems afford a high degree of latitude of use to the farmers. In some cases, however, the irrigation authority may consider that the provision of an on-demand system goes beyond the real needs of the farmers and that it entails the conveyance of peak flows which are excessive, particularly at the downstream extremity of the network.

A compromise can be found between the flexibility of an on-demand system and the rigidity of a rotation distribution. Such is the case of "limited on-demand irrigation". A system of this type can be designed by accepting the following constraints:

- Outlet use restricted to fixed times. Outlets can be grouped into even and odd outlets and their use restricted to even and odd days.

- The design flow at an outlet is proportional to the plot size. The flow is just sufficient to allow the farmer a certain freedom of action within fixed times. The design flow at the outlet can for example be fixed at a value that is equal to four times the duty for the plot.

In these conditions, a farmer is free to irrigate on-demand on alternate days during an average period of twelve hours. A solution of this type alleviates the rigidity of irrigation on a rotation basis.

By judiciously distributing the even and odd outlets, particularly towards the downstream extremities of the network, it is possible to reduce the peak flow in the system to a value which is substantially less than would be the case for true on-demand irrigation for the same outlet design flows.

Distinct network conveyance calculations must be undertaken for each period of operation. In the example discussed above, flows must be determined for even days and for odd days.

3.6 DISCHARGE IN CONTINUOUS FLOW SYSTEMS

In a continuous flow system water is made available at the outlet to each plot throughout the irrigation system at a rate which is equal to or slightly greater than the peak period duty required by the crop.

EXAMPLE 9 - DETERMINATION OF FLOWS IN AN ON-DEMAND NETWORK

Determine the flow conveyed by the network illustrated in Figure 29 designed to service an irrigation block having the following characteristics:

Equipped area : A_e = 237.5 ha

Peak duty : v = 0.6 l/s/irrigated ha

Irrigation ratio : K = A_i/A_e = 0.5

The hydrant layout follows the general outlines discussed in section 4.3.1 with an outlet for each plot and regrouping of outlets on one hydrant where the plots are small. The network has 18 hydrants and 20 farm outlets. Each outlet supplies water to 12 ha (equipped) or 6 ha (irrigated).

The data relative to each plot is listed in Table 9.

The flow attributed to each plot is based upon on a high degree of freedom of demand. The discharge attributed to each plot has an average value of about 10 m³/hour/irrigated ha.

Table 9 PLOT AREAS AND OUTLET CAPACITIES

Plot reference	Equipped ha	Irrigated ha	Outlet capacity m³/h
A	11.0	5.5	50
B	9.0	4.5	50
C	30.0	15.0	125
D	15.0	7.5	70
E	17.0	8.5	90
F	10.0	5.0	50
G	6.0	3.0	30
H	16.0	8.0	90
I	15.0	7.5	70
J	12.0	6.0	70
K	10.0	5.0	50
L	5.0	2.5	30
M	10.0	5.0	50
N	10.0	5.0	50
O	22.0	11.0	125
P	8.0	4.0	50
Q	4.0	2.0	20
R	6.0	3.0	30
S	15.0	7.5	70
T	6.5	3.2	30
Total	237.5	118.7	1200

Seven outlet capacities have been selected from the range of sizes offered by manufacturers. The choice has been determined by the need to maintain the flow attributed to each plot within the limits of 8 to 12 m^3/h/irrigated ha.

These outlets are attributed to the plots according to their irrigated areas as follows:

Table 10 OUTLET CLASSES AND IRRIGATED AREAS

Outlet class m^3/h	Irrigated area ha
10	less than 1
20	1 to 2.5
30	2.5 to 4.0
50	4.0 to 6.0
70	6.0 to 8.0
90	8.0 to 11.0
125	11.0 to 15.0

The total length of the network amounts to 7 675 m or 32 m/equipped ha.

The discharge is determined on the basis of the following design parameters:

- use coefficient r = 0.75
- quality of operation P_q = 0.95 (U = 1.645)
- number of outlets summated at branch endings N_o = 4

The results of the discharge calculation are listed in Table 11. Each link is identified by the number of the node situated immediately downstream and its respective calculation line includes all the outlets which it supplies. The links are examined in the following order: starting at any downstream extremity, the network is followed in the upstream direction, starting again at the next downstream extremity whenever a junction is encountered.

For each link, the following are calculated:

- the irrigated area : A_i (ha)
- the nominal discharge : $D = 0.6 A_i$ (l/s)
- the number of offtakes supplied N_i for each class of outlet d_i
- the total design discharge $\Sigma_i N_i d_i$
- the mean probability of operation of an outlet $p = D/0.75\ \Sigma_i N_i d_i$
- the peak discharge $Q = (D/0.75) + 1.645\ (pq\ \Sigma_i N_i d_i^2)^{\frac{1}{2}}$

In the case of links below which the number of outlets is either equal to or less than four, the peak discharge is obtained by summating the discharge of each of the outlets.

In certain instances, it may happen that the calculated discharge of a link of rank n serving five or six outlets is less than that of a link of rank n-1 serving four outlets whose flows have been summated. In this case the flow in the link of rank n will of course be made equal to the flow in the link of rank n-1.

Table 11 DETERMINATION OF THE FLOW IN THE NETWORK (ON-DEMAND)

Number of d/s link	Ai	D	Number of outlets served N_i							Total link capacity	Mean probability p	Peak flow Q
			10 m^3/h	20 m^3/h	30 m^3/h	50 m^3/h	70 m^3/h	90 m^3/h	125 m^3/h			
	ha	l/s	2.8 l/s	5.6 l/s	8.4 l/s	13.9 l/s	19.5 l/s	25 l/s	34.8 l/s	l/s		l/s
118	15	9.0							1	34.8	–	34.8
117	4.50	2.7				1				13.9	–	13.9
901	19.50	11.7				1			1	48.7	–	48.7
116	27.00	16.2				1	1		1	68.2	–	68.2
114	8.50	5.1						1		25.0	–	25.0
115	5.50	3.3				1				13.9	–	13.9
902	14.00	8.4				1		1		38.9	–	38.9
113	5.00	3.0				1				13.9	–	13.9
903	19.00	11.4				2		1		52.8	–	52.8
112	8.00	4.8						1		25.0	–	25.0
904	27.00	16.2				2		2		77.8	–	77.8
111	30.00	18.0			1	2		2		86.2	0.279	77.8*
905	57.00	34.2			1	3	1	2	1	154.4	0.295	89.9
106	7.50	4.5					1			19.5	–	19.5
105	6.00	3.6					1			19.5	–	19.5
906	13.50	8.1					2			39.0	–	39.0
907	70.50	42.3			1	3	3	2	1	193.4	0.292	105.1
107	11.00	6.6							1	34.8	–	34.8
108	5.00	3.0				1				13.9	–	13.9
908	16.00	9.6				1			1	48.7	–	48.7
110	7.50	4.5			1	1				22.3	–	22.3
909	23.50	14.1			1	2			1	71.0	–	71.0
109	5.00	3.0				1				13.9	–	13.9
910	28.50	17.1			1	3			1	84.9	0.269	71.0*
911	99.00	59.4			2	6	3	2	2	278.3	0.285	137.2
101	6.00	3.6		1		1				19.5	–	19.5
912	105.00	63.0		1	2	7	3	2	2	297.8	0.282	142.8
104	3.25	1.95			1					8.4	–	8.4
102	7.50	4.5					1			19.5	–	19.5
913	10.75	6.45			1		1			27.9	–	27.9
103	3.00	1.8			1					8.4	–	8.4
94	13.75	8.25			2		1			36.3	–	36.3
915	118.75	71.25		1	4	7	4	2	2	334.1	0.284	156.3

* Case where the peak flow is less than the flow of the u/s link (4 outlets summated).

Each farmer organizes his own internal rotation according to his needs. When the sector consists of small holdings, this type of system is not very practical since the flow available at the outlet may well be insufficient to be used directly. When this is so, the farmer must invest in a storage tank from which a convenient stream size is released. He is then free to irrigate every day if he wishes for durations which are inversely proportional to the rate of outflow from the reservoir.

This type of distribution system can be applied equally well to open channel or pressure networks. The flow to be conveyed in each link is determined by summating from downstream to upstream throughout the network. The channels or pipes are continuously supplied and their dimensions are minimal.

4. DESIGN AND OPTIMIZATION TECHNIQUES OF PRESSURE DISTRIBUTION NETWORKS

4.1 INTRODUCTION

This section deals with the selection of the technical specifications and dimensions of the various items which together constitute the network for conveying irrigation water from the source to the farm outlets.

It is assumed that the network is of the branching type as opposed to looped and that the following characteristics have already been defined:

- location and elevation (Z) of the hydrants;
- minimum design pressure at each hydrant;
- network layout including length of each section;
- discharge to be conveyed by each section;
- temperature of water at the source;
- the series (s) of pipes (i) which are available and for each of which the following are known:
  - price (p_i) per metre of length
  - equivalent roughness height (k) in mm
  - maximum permissible pressure (p_s) in m
  - minimum (v) and maximum (V) permissible velocity for each pipe in m/s.

As regards the price of pipes, either the price of the pipe alone can be used or that of the pipe layed, including the cost of the trench. In the latter case the cost varies with the nature of the ground and the pipe material since the techniques involved in laying PVC pipes differ from those used when laying cast iron.

Minimum velocities are selected in order to ensure that there is no deposition of solids. Maximum velocities are chosen to reduce the consequences of water hammer and to limit pumping costs. The values normally retained are 0.3 m/s and 3 m/s respectively but these limits can vary with the diameter of the pipes.

Since pumping costs are proportional to head losses and decrease as the pipe diameter increases, whereas at the same time the cost of pipe increases with the pipe diameter, it is often worthwhile to select the most suitable pipe by an optimization process, the optimum discharge and network layout having been selected by procedures such as those described in Chapter 3.

The optimization procedure for pipe diameters involves three distinct phases:

Phase 1. Construction of the "lower envelope curve" for a section. This indicates the minimum price of the section as a function of the head loss in the pipe (4.4).

Phase 2. Ascent of the network: moving upstream section by section, the lower envelope curves of the individual sections are summated either in groups or in series (4.5) to obtain the network characteristic curve. This curve represents the total cost of the network as a function of the hydraulic head at the upstream extremity.

Phase 3. Descent of the network: moving downstream section by section, for a given hydraulic head at the upstream extremity selected as a function of pumping or reservoir costs, the diameter of each section is defined together with the hydraulic head at each node.

4.2 NETWORK DESCRIPTION - NUMBERING OF COMPONENTS

4.2.1 Introduction

In order to demonstrate the principles of network component numbering, reference will be made to the network illustrated in Figure 30. Originally, the components of this network have been identified arbitrarily by attribution of the names of localities or owners as node and hydrant labels.

Nodes can be either hydrants or points of departure of other sections. Furthermore, a node can receive water from only one section since the network is not looped.

The geometry of a network is completely determined when the upstream extremity of the network and the extremities of each section are defined.

In the case of Figure 30, the upstream extremity of the network is the point numbered 9000 whereas the sections are (G,L), (80, G) etc. but these could also be referred to as (L, G), (80, G), etc.

Such a system is obviously cumbersome and totally incoherent from the point of view of modelling for optimization.

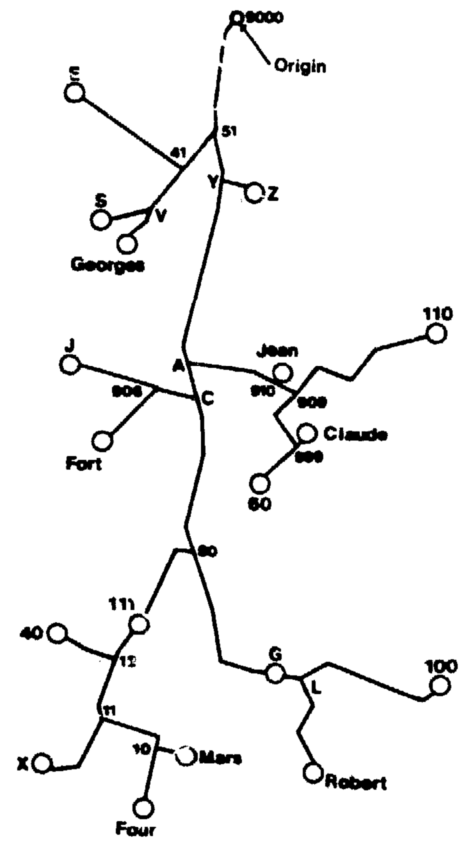


Figure 30 Original network numbering

4.2.2 Principles of Rational Numbering

The aim is to attribute a number to each section so that the calculation of the diameter of each section can be performed unequivocally on the basis of what lies immediately upstream and downstream of the section.

This is achieved as follows: Moving downstream to any extremity, the last section is numbered (1). The section that precedes it is numbered (2) and so forth. When a node is encountered one moves downstream to the next extremity, the last section of this branch being attributed the next number. The node or hydrant at the downstream end of a section is given the section number but without brackets.

In order to reduce possible overload of the computer memory, branches which carry the greatest number of ramifications should be numbered first. Referring to the network in Figure 30 when node 80 is reached on the initial descent from 9000, the network should be followed in the direction of node 111 rather than towards node G. In the same way at node 11, the descent is towards node 10 and not towards hydrant X.

Whilst proceeding with these successive ascents and descents, the Network Description (Table 12) is completed. This table consists of eight columns, the last of which is subdivided.

Table 12 NETWORK DESCRIPTION

1	2	3	4	5	6	7	8
Section (n)	u/s section (n+1)	Hydrant	Length (m) $L_{(n)}$	Discharge (m^3/s) $Q_{(n)}$	Elev. (m) $Z_{(n)}$	Hydrant head (m)	d/s section numbers

u/s : upstream section
d/s : downstream section

The network description contains the following information:

Column 1: The number of the section under consideration (n).

Column 2: The number of the section immediately upstream (n+1).

Column 3: 1 if the downstream end of the section is a hydrant or hydrant node; 0 if this is not the case.

Column 4: The length (m) of the section under consideration.

Column 5: The discharge (m^3/s) in the section under consideration. The discharge at hydrants alone is required. Elsewhere it is computed automatically unless it is imperative to impose a discharge in a particular section as a result of applying the demand formula.

Column 6: The elevation (Z) of the node or hydrant at the downstream end of the section under consideration.

Column 7: The hydraulic head required at the hydrant (m of water) when the downstream end of the section carries a hydrant.

Column 8: The sub-columns indicate the numbers of the sections downstream of the section under consideration in descending order. 0* indicates that either there is no downstream section or that there are no more downstream sections.

A very simple table consisting of four columns can be drawn up to indicate the correspondence between the original and the new numbering systems for quick reference:

Column 1: The section number, as entered in Column 1 of Table 12.

Column 2: The name of the section or node at the downstream end of the section.

Column 3: The name of the section or node which preceeds the section.

Column 4: The number of the upstream section as entered in Column 2 of Table 12.

4.2.3 Example of Network Numbering

As a practical example of rational numbering, the network illustrated in Figure 30 will be renumbered and the network description and correspondence tables drawn up (Tables 13 and 14).

Starting at the upstream end of the network an arrow indicates the path which is followed (Fig. 31). The sections, nodes and hydrants are numbered with and without brackets as described above. To avoid overloading the computer memory, the smallest numbers are attributed to the branches having the greatest number of ramifications. Although not compulsory, this device is recommended.

The network description (Table 13) is drawn up as follows: section numbers from (1) to (34) are entered in the first column. Starting with section (1), the number of the preceding section (3) is entered in column 2 and 1 in column 3 since section (1) carries a hydrant at its downstream end. Columns 4 to 7 are filled in, the discharge being that which has been calculated by applying the demand formula. In the first sub-column of column 8, 0* indicates that section (1) is a terminal section.

Section (2) is dealt with in the same manner, 0 and * are entered in the first two sub-columns of column 8. Section (3) is preceded by section (5) as shown in column 2 and 0 in column 3 indicates that there is no hydrant at the downstream end of the section. The numbers 2, 1 and 0 are entered in the first three sub-columns of column 8 and * in the fourth indicates a termination. It is clearly apparent that section (3) which has node 3 at its extremity gives rise to a subnetwork consisting of sections (2) and (1). In the same way, section (5) can be seen to be at the origin of a subnetwork which consists of sections (4) (3) (2) and (1) whereas section (33) is at the origin of the whole network. The table of correspondence between the original numbering and the new can also be drawn up if desired (Table 14).

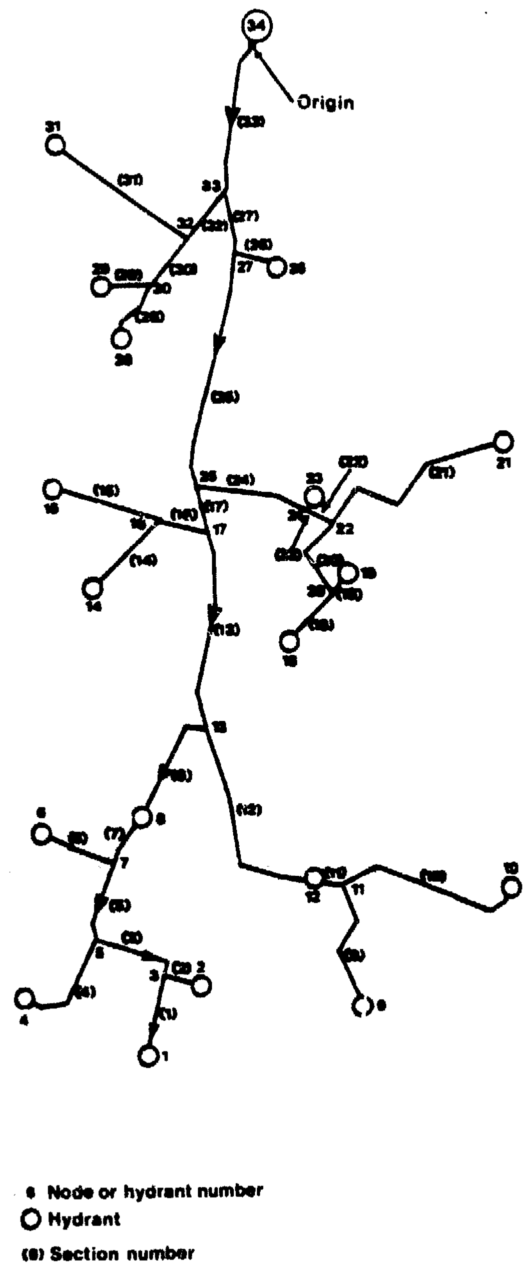


Figure 31 Rational network numbering

Table 13 DESCRIPTION OF NETWORK ILLUSTRATED

1 Section (n)	2 u/s sect. (n+1)	3 Hydr.	4 Length (m)	5 Dis. (m^3/s)	6 Elev. (m)	7 Hydr. head (m)	8 d/s section numbers
1	3	1	195	0.025	505	30	0 *
2	3	1	90	0.0139	500	30	0 *
3	5	0	220	0.0389	0	0	2 1 0 *
4	5	1	290	0.0139	515	30	0 *
5	7	0	190	0.052	0	0	4 3 2 1 0 *
6	7	1	190	0.025	515	30	0 *
7	8	0	130	0.0778	0	0	0 *
8	13	1	300	0.0778	505	30	7 6 5 4 3 2 1 0 *
9	11	1	330	0.0139	520	30	0 *
10	11	1	485	0.0348	500	30	0 *
11	12	0	70	0.0487	0	0	10 9 0 *
12	13	1	530	0.0682	510	30	11 10 9 0 *
13	17	0	490	0.0899	0	0	12 11 10 9 8 7 6 5 4 3 2 1 0 *
14	16	1	225	0.0195	500	30	0 *
15	16	1	280	0.0195	505	30	0 *
16	17	0	130	0.039	0	0	0 *
17	25	0	120	0.1052	0	0	16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 *
18	20	1	160	0.0348	495	30	0 *
19	20	1	50	0.0139	505	30	0 *
20	22	0	220	0.0487	0	0	19 18 0 *
21	22	1	540	0.0223	515	30	0 *
22	24	0	85	0.071	0	0	21 20 19 18 0 *
23	24	1	10	0.0139	510	30	0 *
24	25	0	290	0.071	0	0	23 22 21 20 19 18 0 *
25	27	0	575	0.1372	0	0	24 23 22 21 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 *
26	27	1	110	0.0195	540	30	0 *
27	33	0	145	0.1429	0	0	26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 *
28	30	1	135	0.0084	515	30	0 *
29	30	1	10	0.0195	535	30	0 *
30	32	0	170	0.0279	0	0	29 28 0 *
31	32	1	400	0.0084	515	30	0 *
32	33	0	140	0.0363	0	0	31 30 29 28 0 *
33	34	0	370	0.1565	0	0	32 31 30 29 28 27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 *

Table 14 CORRESPONDENCE BETWEEN ORIGINAL AND RATIONAL NUMBERING

1 number of section	2 name of section	3 name of section	4 number of section	1 number of section	2 name of section	3 name of section	4 number of section
(1)	Four	10	(3)	(18)	60	999	(20)
(2)	Mars	10	(3)	(19)	Claude	999	(20)
(3)	10	11	(5)	(20)	999	909	(22)
(4)	X	11	(5)	(21)	110	909	(22)
(5)	11	12	(7)	(22)	909	910	(24)
(6)	40	12	(7)	(23)	Jean	910	(24)
(7)	12	111	(8)	(24)	910	A	(25)
(8)	111	80	(13)	(25)	A	y	(27)
(9)	Robert	L	(11)	(26)	z	y	(27)
(10)	100	L	(11)	(27)	y	51	(33)
(11)	8	G	(12)	(28)	Georges	v	(30)
(12)	G	80	(13)	(29)	s	v	(30)
(13)	80	C	(17)	(30)	v	41	(32)
(14)	Fort	910	(16)	(31)	E	41	(32)
(15)	J	906	(16)	(32)	41	51	(33)
(16)	906	C	(17)	(33)	51	9000	(34)
(17)	c	A	(25)				

4.2.4 Use of the Network Description Table

The purpose of the network description table (Table 13) is twofold:

- identification of sections located downstream of a given section or of a subnetwork located below a given section;
- automatic determination of discharges.

i. Identification of a subnetwork downstream of a section

Let (n) be the number of any one section. This implies that n is its downstream extremity. To (n) is associated a second number m(n) such that:

- if n is a termination then m(n) = n
- if n is not a termination then m(n) is equal to the smallest number given to the sections which lie downstream and

$$m(n)=\min[m[d_{k(n)}]]$$

where $d_{k(n)}$ is the kth section below section (n)

Taking an example from Table 13, to section (1) is associated 1 since

$$n = 1 \longrightarrow m(n) = n = 1$$

Similarly 2 is associated with section (2) but 1 is associated with section (3) since 1 is the lowest number attributed to these sections which are (1) and (2), hence 1=min(2.1)

It follows that a subnetwork consists of the section whose number r satisfies

$$m(n) \leqslant r \leqslant n - 1$$

there being of course no subnetwork if there is no value of r. Hence, in the present case, there are no subnetworks below 1 and 2 but for n = 3 the subnetwork satisfies

$$1 \leqslant r \leqslant (3-1) = 2$$

thus sections (1) and (2) form a subnetwork.

ii. Automatic determination of discharge

It is necessary to distinguish between two situations: either all hydrants have the same discharge or all hydrants have different discharges. It is assumed that the demand formula (Clément) has been used to determine the discharge where need be.

a. Equal discharge at hydrants

If $\varepsilon_{k(n)}$ signifies the presence or absence of a hydrant downstream of the kth section of (n) then

$\varepsilon_{k(n)} = 1$ if there is a hydrant
$\varepsilon_{k(n)} = 0$ if there is no hydrant

If Q = discharge of the hydrant (m^3/s)
m(n) = number attributed to section (n)
$Q_{(n)}$ = discharge of section (n)

then

$$Q_{(n)} = Q \left[\Sigma_{k=m(n)}^{(n)} \varepsilon_{k(n)} \right] \qquad (65)$$

Referring once more to Table 13:

if (n) = 24, the associated number m(n) = 18 then the discharge in section (24) will be

$$Q_{(24)} = Q \left[\Sigma_{k=18}^{24} \varepsilon_{k(n)} \right]$$

$$= Q\,(1 + 1 + 0 + 1 + 0 + 1 + 0)$$

$$= 4Q$$

b. Different discharges at hydrants

Let $Q_{k(n)}$ = discharge (m^3/s) at the kth hydrant downstream of section (n)
$\varepsilon_{k(n)}$ = presence or absence of a hydrant downstream of the kth section of (n)
m(n) = the number attributed to section (n)
$Q_{(n)}$ = the discharge (m^3/s) in section (n)

Then

$$Q_{(n)} = \Sigma_{k=m(n)}^{(n)} \left[\varepsilon_{k(n)} \; Q_{k(n)} \right] \qquad (66)$$

4.3 CONTENTS OF THE TABLE OF SUITABLE PIPES

The preparation of the table of suitable pipes is a relatively simple matter but is of considerable importance in the optimizing procedure. The table of pipes provides information concerning not only the price per unit length (p_i) of a pipe but also on the unit head loss (j_i), the permissible pressure (Pa), the minimum (v_i) and maximum (V_i) allowable velocities and the discharges which correspond to these velocities. When entering pipes in the table of pipes (Table 15) for a given material, the low pressure pipes are entered first.

Table 15 TABLE OF SUITABLE PIPES

1 pipe (i)	2 diameter (D_i) (mm)	3 head loss j_i (m/km)	4 permissible pressure (Pa) (m)	5 min vel v_i (m/s)	6 max vel V_i (m^3/s)	7 disch. Q_{vi} (m^3/s)	8 disch. Q_{Vi} (m^3/s)	9 unit price (-/m)
1								
2								
3								

Table 15 is completed as follows:

Column 1: A number (i) is attributed to each pipe starting with 1 for the first pipe.

Column 2: The inside diameter of the pipe (mm) in decreasing order of size, due attention being paid to the galvanic series.

Column 3: The head loss per unit length (j_i) calculated by the Colebrook-White formula. The head loss is computed for the minimum and maximum permissible velocities, taking into account the temperature of the water at the source.

Column 4: The maximum permissible pressure (Pa) in the pipe (m of water). There may often be two pipes of the same material and diameter but the wall thickness can vary. The thin-wall pipe is attributed a lower pipe number and entered first.

Column 5: The minimum permissible velocity (v_i) in the pipe (m/s).

Column 6: The maximum permissible velocity (V_i) in the pipe (m/s). The choice of velocity results from a compromise between a high velocity to reduce the diameter and cost of the pipe and the higher cost of water hammer protection devices associated with the higer velocities.

Column 7 & 8: The discharges Q_{vi} and Q_{Vi} (m^3/s) corresponding to the minimum (v_i) and maximum (V_i) permissible velocities.

Column 9: The price (p_i) per metre length of pipe.

4.3.1 Choice of Velocity

As a guide, the range of minimum and maximum permissible velocities adopted for the design of a 72 000 ha irrigation scheme in France are indicated in Table 16 for pipe diameters varying from 100 mm to 1 m.

Table 16 MINIMUM AND MAXIMUM PERMISSIBLE VELOCITIES IN PIPES

Diameter (mm)	Minimum velocity (v_i) (m/s)	Maximum velocity (V_i) (m/s)
1000	0.50	3.10
900	0.50	3.10
800	0.50	3.10
700	0.50	3.10
600	0.50	3.10
500	0.50	2.85
450	0.50	2.85
400	0.50	2.50
350	0.50	2.30
300	0.40	2.25
250	0.40	2.15
200	0.35	2.05
150	0.25	1.95
125	0.25	1.85
100	0.20	1.80

4.3.2 Application of the Table of Suitable Pipes

The table of pipes (Table 15) is the starting point for the network optimizing procedure. The table is entered into the computer memory or, if the operation is manual, a list of pipes [l(n)] is extracted from the table for each section.

The algorithm defined in Figure 32 is used to select a pipe which meets the discharge and hydraulic head requirements. It is a mathematical representation of the steps taken implicitly in the designer's approach.

In a cylindrical pipe the discharge passing a cross section may be written:

$$Q = 10^{-6}\,\pi D^2 V/4 \qquad (67)$$

where: Q = discharge (m^3/s)
D = pipe diameter (mm)
V = velocity (m/s)

A pipe i is therefore selected to satisfy the following relations:

(1) $$10^{-6}\,\pi D_i{}^2 v_i/4 \leqslant Q_{(n)} \leqslant 10^{-6}\,\pi D_i{}^2 V_i/4 \qquad (68)$$

(2) $$Pa \geqslant Z_{(r)} - Z_{(n)} \leqslant Pa_c \qquad (69)$$

where: $Z_{(r)}$ = the tank elevation (m)
$Z_{(n)}$ = elevation of d/s end of section (m)
Pa = permissible pressure in thin wall pipe (m of water)
Pa_c = permissible pressure in thick wall pipe (m of water)

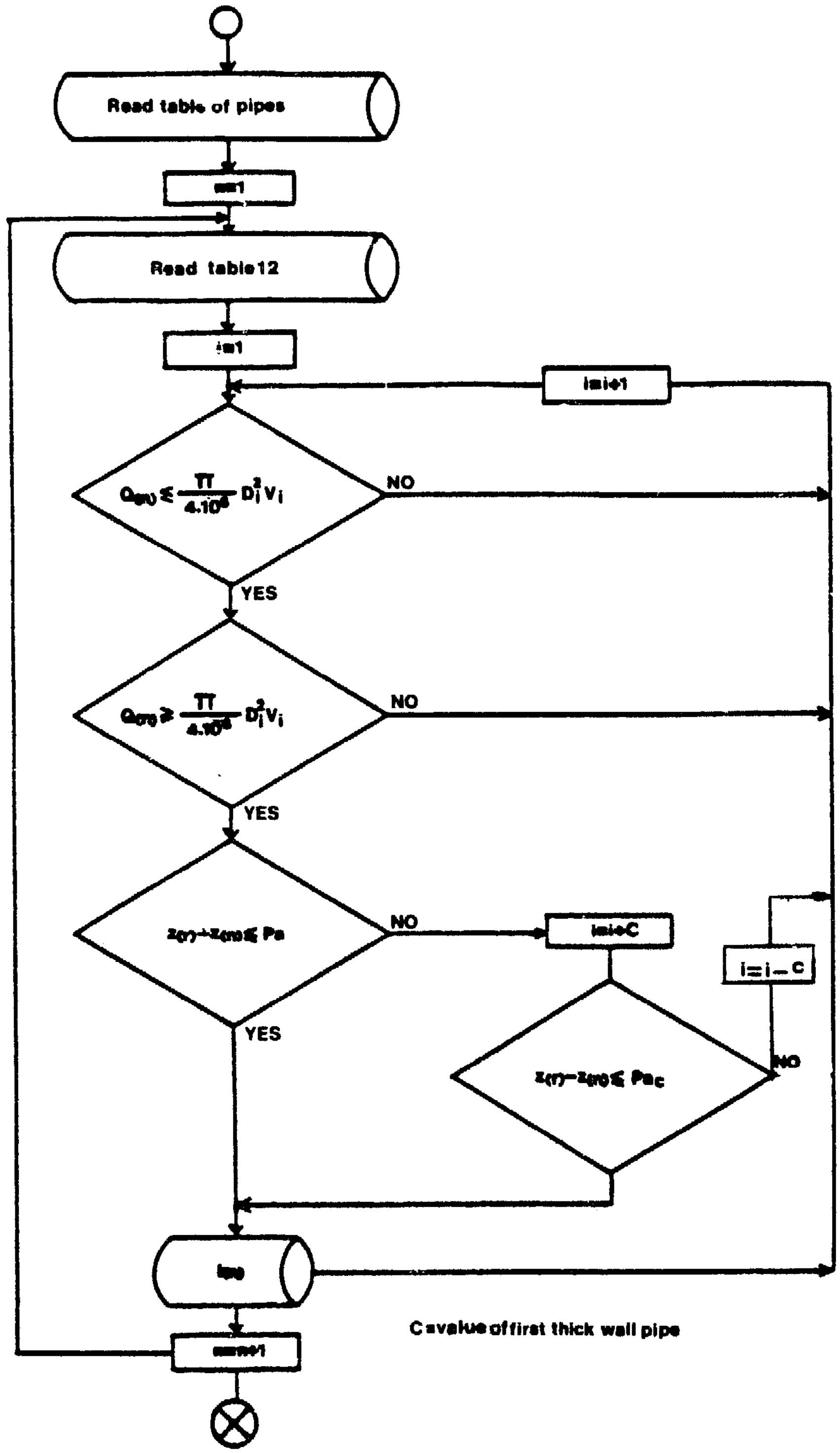


Figure 32   Algorithm for pipe selection

If the pipe is found to be suitable it is entered in a list of pipes suitable for the section (Table 17).

4.3.3 List of Pipes Suitable for a Given Section

This list [l(n)] is drawn up for each section using data contained in the table of suitable pipes (Table 15). The price and the head loss which are entered refer to the complete section. The pipes are entered in the order of increasing head losses. This is the same as entering the pipes in the order of decreasing diameters if all pipes are of the same material. If several materials are competitive it may happen that the order of increasing head losses does not always correspond to decreasing diameters throughout the list due to the incidence of different roughness heights.

Table 17 LIST OF PIPES SUITABLE FOR A GIVEN SECTION [l(n)]

1 pipe number	2 pipe no. in table of pipes (Table 15)	3 diameter D_i (mm)	4 total head loss H_i (m)	5 total price $P_{t(n)}$
1 (n)				
2 (n)				
etc.				

In Table 17, the total head loss entered in Column 4 is the head loss in the section when fitted with pipe i (expressed in metres of water):

$$H_i = j_i L_{(n)}$$

and the total price is the price of the section when fitted with pipe i:

$$P_{t(n)} = p_i L_{(n)}$$

4.4 MINIMUM PRICE OF A SECTION

4.4.1 Introduction

In order to simplify the explanation of the method used to optimize the pipe diameter it will first be demonstrated that for any given value of head loss, the lowest cost of a section is found when the section carries not more than two pipes drawn from the list of pipes suitable for that section (Table 17).

This demonstration will be followed by the development of the expression for the lowest cost of a branching network.

4.4.2 Not More than Two Pipes Per Section

Let a compound section (n) carry three diameters of length x_1, x_2 and x_3. Then

- length of section $L_{(n)} = x_1 + x_2 + x_3$
- section head loss $H_{(n)} = j_1\, x_1 + j_2\, x_2 + j_3\, x_3$
- section cost $P_{(n)} = p_1\, x_1 + p_2\, x_2 + p_3\, x_3$

The section length and section head loss being given, there is an infinite number of values of x_1, x_2 and x_3 which will satisfy the first two equations, the section cost varying with these values. By eliminating x_2 and x_3 in the three equations a relation is obtained which can be written

$$P_{(n)} = A\, x_1 + B$$

where A and B are constants which are independant of x_1, x_2 and x_3 but which are functions of the unit head loss.

Since $P_{(n)}$ is a linear function of x_1, it is possible to vary the value of x_1 in order to reduce the value of $P_{(n)}$ as long as A is greater than zero or, in other words, as long as one of the lengths x_1, x_2 or x_3 is not equal to zero. When one of these lengths becomes zero, a solution is obtained consisting of a compound pipe with only two diameters whose cost is less than the original compound pipe with three diameters.

The cost of the compound pipe with two diameters cannot be reduced when $L_{(n)}$ and $H_{(n)}$ are given since the relations:

$$L_{(n)} = x_1 + x_2$$

$$H_{(n)} = j_1\, x_1 + j_2\, x_2$$

determine the values of x_1 and x_2.

There is then only one solution, the cost of which is determined by the relation

$$P_{(n)} = p_1\, x_1 + p_2\, x_2$$

The above solution is illustrated in Figure 33 where pipe 1 has a diameter D_1 throughout the length of section (n). This pipe has a cost

$$P_1 = p_1\, L_{(n)}$$

and a head loss

$$H_1 = j_1\, L_{(n)}$$

(The figurative point for this pipe is D_1 in Figure 33.)

Similarly, pipe 2 has a diameter D_2, a head loss H_2 and its figurative point is D_2.

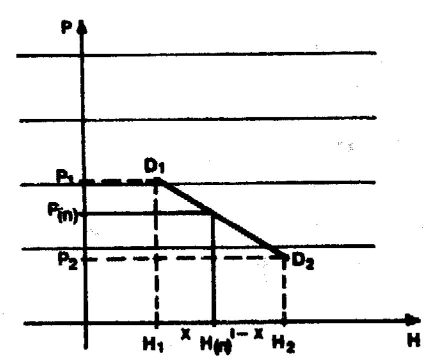


Figure 33 Minimum cost of a compound section

If the permissible head loss, $H_{(n)}$ in the section lies between H_1 and H_2 then the section is fitted with

a length x of diameter D_1 and
a length 1-x of diameter D_2

The minimum price of the section is

$$P_{(n)} = x P_1 + (1-x) P_2$$

The head loss in the section is

$$H_{(n)} = x H_1 + (1-x) H_2$$

where x: length of pipe 1

$$x = \left[\frac{H_2 - H}{H_2 - H_1}\right] L_{(n)}$$

and 1-x: length of pipe 2

$$1-x = \left[\frac{H - H_1}{H_2 - H_1}\right] L_{(n)}$$

It is of importance to realize however that in practice, 90 percent of the sections carry a single diameter, the remainder being compound pipes consisting of two diameters and principally located at the extremities. This is due to the fact that in general lengths of less than 50 m are not fitted with a second pipe of larger diameter.

4.4.3 Practical Application of the List of Pipes Suitable for a Given Section

i. Construction of the lower envelope curve

It was shown in 4.4.2 that the curve of the price of a compound section expressed as a function of the head loss has a negative slope. If this list of pipes suitable for a given section (Table 17) is used and all the figurative points are plotted as in Figure 33, a series of segments of negative slope can be drawn which represent a group of combinations. The slope (ß) of a segment representing pipes i and i+1 is given by

$$ß_{i,\,i+1} = \frac{P_{i+1} - P_i}{H_{i+1} - H_i} < 0$$

The curve of the minimum price of a section as a function of the permissible head loss in the section is called the "lower envelope curve" P(H). In the case illustrated in Fig. 34, the diameters which are to be retained are D_1, D_3, D_4, D_5 and D_7 since their segments have the steepest slopes. The other diameters correspond either to pipes of different material or belong to the same commercial range but are not competitive because of their price. In practice it is convenient to pivot a straight-edge about point D_1 then point D_3 and so forth to draw the lines of greatest slope. It is preferable, however, to calculate the slope $ß_{i,i+1}$.

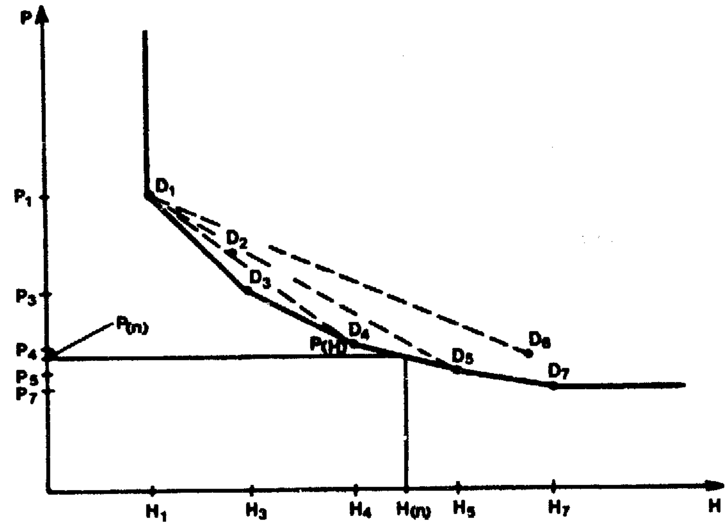


Figure 34 P(H): Lower envelope curve or minimum price of section

The lower envelope curve P(H) is extended at each end by straight segments

- a vertical segment rising from the point of minimum head which indicates that the discharge cannot be conveyed with a smaller head loss
- a horizontal segment extending from the point of minimum price and which indicates that the cost of the section cannot be reduced even if excess head is available.

It can be seen that for a given head loss $H_{(n)}$ the optimum diameter(s) can easily be determined in the same way as in section 4.4.2. The benefit obtained by accepting an increase of head loss can also be calculated.

ii. Schematic construction

It is possible to avoid constructing a lower envelope curve for each section by using a horizontal axis where the choice of optimal diameter can easily be determined after calculating the greatest slopes.

The schematic construction technique is illustrated in Figure 35

- below the horizontal axis is shown the head loss $H_{(i)}$ which corresponds to a section of a single pipe. The head loss need not be drawn to scale, the axis is there only for the purpose of visualization. The equivalent price of a section equipped with the left hand diameter is indicated between two head losses
- above the horizontal axis is indicated the diameter of the pipe (D_i) corresponding to the head loss (H_i) and the slope of the segment ßi, ßi+1 between each pipe diameter. From 0 to H_1 the value of the slope (ß) is infinite; from the last

pipe (diameter D_7) to the end of the axis, the slope is zero. For a given head loss situated between $H_4 < H_{(n)} < H_5$ the formula of paragraph 4.4.2 still holds.

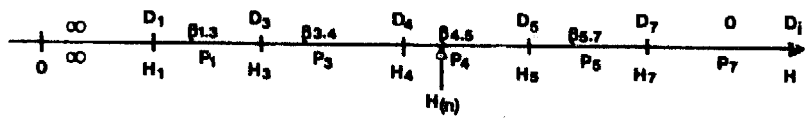


Figure 35 Schematic construction of lower envelope curve

iii. Algorithm for the lower envelope curve

The algorithm for constructing a lower envelope curve for a network of N sections, drawn from lists of pipes $l_{(n)}$ for each section is shown in Figure 36.

4.4.4 Special Cases

i. End sections

End sections are defined here as sections downstream of which it is essential to satisfy a minimum piezometric head either because of the presence of a hydrant or because a branch or subnetwork originates at the end of the section.

The least-cost section, as a function of the upstream piezometric head is obtained by shifting the lower envelope curve through a distance Z_m equal to the imposed minimum piezometric head as shown in Figure 37. The values Z_1 to Z_7 represent the required head at the upstream end of the section for head losses H_1 to H_7 where $Z_1 = Z_m + H_1$ and $Z_7 = Z_m + H_7$.

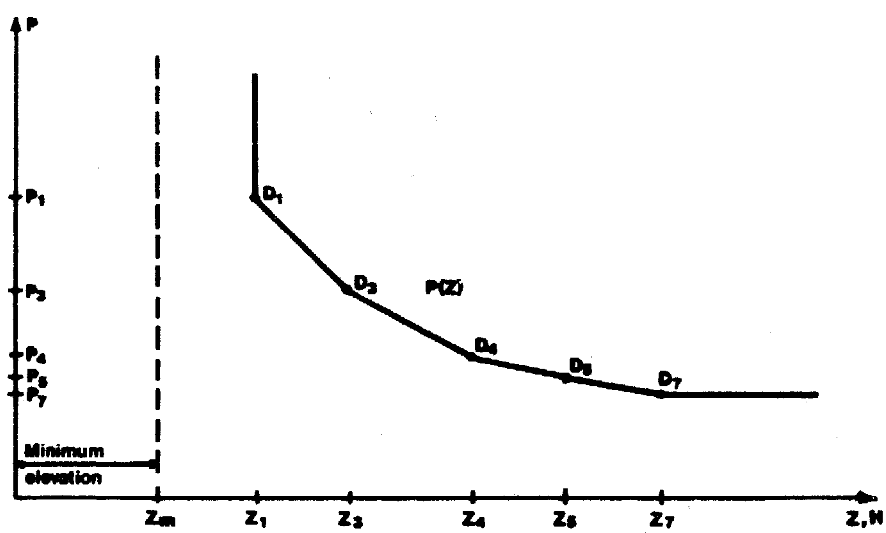


Figure 37 Lower envelope curve for end section

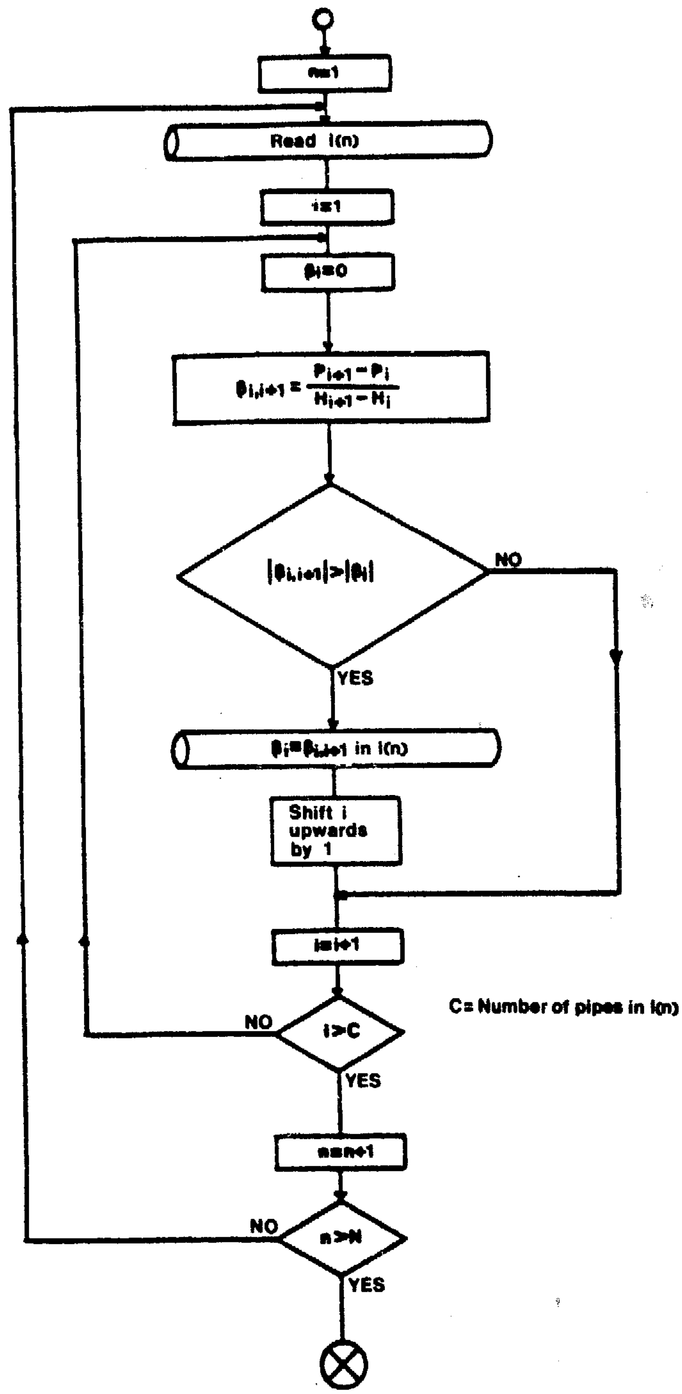


Figure 36 Algorithm of lower envelope curve

EXAMPLE 10 - CALCULATION OF PRICE OF SECTION

Assume that section 17 conveys a discharge of 145 l/s. The pipes which are commercially available are listed in Table 18.

Table 18 TABLE OF PIPES

ENTRIES STORED ON DISK UNDER NAME: TABLE 14
NUMBER OF PIPES IN THIS LIST: 14
WATER TEMPERATURE (°C): 10

Pipe (no)	Dia. (mm)	Roughness (mm)	Velocity (m/s) Min.	Velocity (m/s) Max.	Discharge (m^3/s) Min.	Discharge (m^3/s) Max.	Head loss (m/km) Min.	Head loss (m/km) Max.	Maximum pressure (m)	Cost (FF/m)
1	1000	0.025	0.50	2.00	0.393	1.571	0.180	2.367	0.00	285.97
2	900	0.025	0.50	2.00	0.318	1.272	0.204	2.679	0.00	231.13
3	800	0.025	0.50	2.00	0.251	1.005	0.235	3.077	0.00	182.73
4	700	0.025	0.50	2.00	0.192	0.770	0.275	3.600	0.00	144.13
5	600	0.025	0.50	2.00	0.141	0.565	0.331	4.318	0.00	100.46
6	500	0.025	0.50	2.00	0.098	0.393	0.412	5.357	0.00	72.45
7	400	0.025	0.50	2.00	0.063	0.251	0.538	6.980	0.00	48.34
8	350	0.025	0.50	2.00	0.048	0.192	0.632	8.182	0.00	38.34
9	300	0.025	0.50	2.00	0.035	0.141	0.762	9.832	0.00	28.26
10	250	0.025	0.50	2.00	0.025	0.898	8.950	12.226	0.00	25.58
11	200	0.025	0.50	2.00	0.016	0.063	1.246	15.977	0.00	17.69
12	150	0.025	0.50	2.00	0.009	0.835	1.772	22.594	0.00	11.12
13	100	0.025	0.50	2.00	0.004	0.016	2.920	36.938	0.00	7.66
14	80	0.025	0.50	2.00	0.003	0.010	3.852	48.494	0.00	6.25

The pipes which can be used for the section are listed in Table 19 whereas Table 20 and Figure 38 represent the lower envelope curve for this section.

Table 19 LIST OF PIPES SUITABLE FOR SECTION 17 (220 m)

No.	Pipe Number	Diameter (mm)	Velocity (m/s)	Total Head Loss (m)	Total Cost (FF)
1	5	600	0.51	0.076	22 101.20
2	6	500	0.74	0.185	15 939.00
3	7	400	1.15	0.550	10 634.80
4	8	350	1.51	1.059	8 434.80

Table 20 LOWER ENVELOPE CURVE FOR SECTION 17

No.	Pipe Number	Diameter (mm)	Total Head Loss (m)	Total Head (m)	Cost (FF)	Slope (Bi)
0	5	600	0.08	0.076	0.00	-9 999 999.00
1	5	600	0.08	0.076	22 101.20	-56 701.58
2	6	500	0.18	0.185	15 939.00	-14 529.34
3	7	400	0.55	0.550	10 634.80	-4 319.24
4	8	350	1.06	1.059	8 434.80	0.00
5	8	350	000000.00	000000.000	8 434.80	0.00

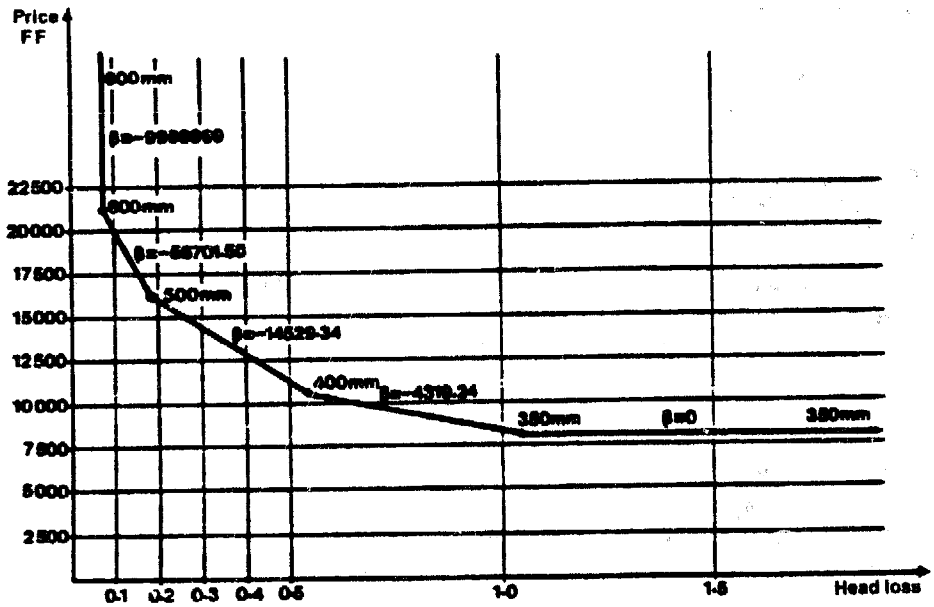


Figure 38 Lower envelope curve for section 17

The schematic representation of the lower envelope curve of section 17 is shown in Figure 39. The drawing is not to scale, it merely serves to locate the different points of the curve on an axis.

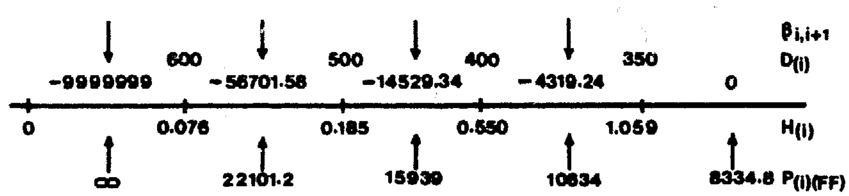


Figure 39 Schematic representation of lower envelope curve for section 17

When the schematic representation is used, the minimum head is shown below the origin and the corresponding piezometric head is indicated below each head loss. If a single diameter (D_1) is used for the section then $Z_1 = Z_m + H_i$, as shown in Figure 40 ($Z_1 = Z_m + H_1 \ldots Z_7 = Z_m + H_7$). The slope $\beta_{i,i+1}$ of the segments which form the lower envelope curve is not affected (Fig. 37) and there is no need to modify the schematic representation.

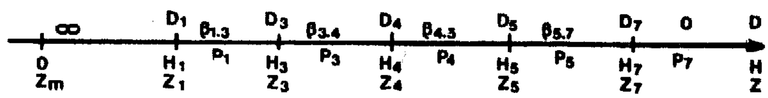


Figure 40 P(Z) schematic representation in the case of an end section

ii. <u>One diameter only per section</u>

It was shown in (4.4.2) that the optimum solution is obtained when a compound section consists of not more than two diameters. Experience shows that after optimization, 90 percent of the sections have only one diameter. Compound pipes are usually found in terminal sections made up of the smallest diameter allowed on the network and the next larger diameter.

It sometimes happens that the design specification calls for a single diameter on each section. In this case it is no longer possible to refer to a lower envelope curve since there is only one head loss corresponding to a single diameter D_i for a cost P_i. The figurative points are determined as follows:

The first step is as before: preparation of the list of pipes which can be used for each section ,arranged in order of increasing head losses. The competitivity of a pipe is defined by its cost. Having selected the first pipe, a check is made to verify that no other pipe has a higher or equal cost for a higher head loss:

$$P_1 > P_i, \quad (i > 1)$$

Pipes which fail to satisfy this condition are eliminated. The same procedure is repeated for the second pipe, or the third pipe if the second pipe has been eliminated. In more general terms:

$$P_i > P_{i+x} \qquad (1 \leqslant x \leqslant d-i)$$

where d is the number of the last pipe on the list.

A graph of the type shown in Figure 34 can be plotted (Fig. 41) making use of the same pipes as in Figure 34.

It may be seen that the above equation leads to the elimination of pipe D_6 only. There is no need to draw the zero slope curve with the figurative points of the pipes which have been retained since, by definition, the section can only be fitted with a pipe of one diameter.

For a given head loss $H_{(n)}$, situated between H_4 and H_5, it can be seen that the section must have a pipe of diameter D_4. The excess head is catered for by a pressure-reducing valve.

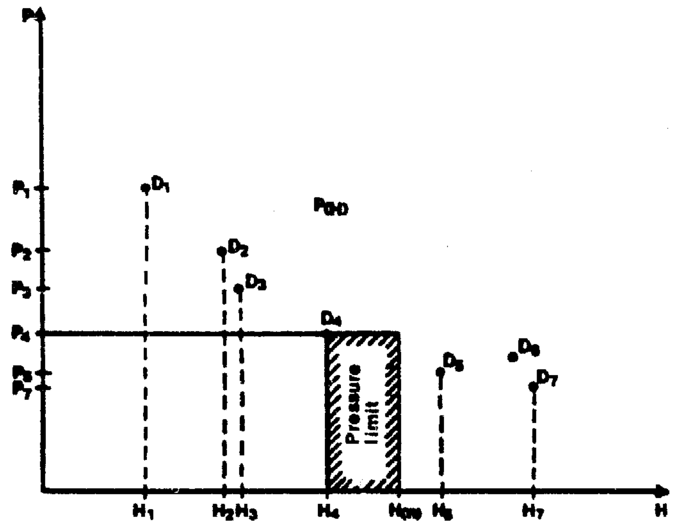


Figure 41 Step function P(H) with only one diameter allowed per section

The basic premise of the schematic representation still holds good when a single diameter is imposed. There is, however, no need to plot the slope $s_{i,i+1}$ since only the cost p_i is determinant in this case. Figure 42 illustrates the principle and it can be seen that for a head loss $H_{(n)}$ the diameter D_4 situated immediately to the left of $H_{(n)}$ must be selected.

The same procedure is followed for a terminal section, the head loss being added to the imposed piezometric head by shifting the point of origin (Fig. 41).

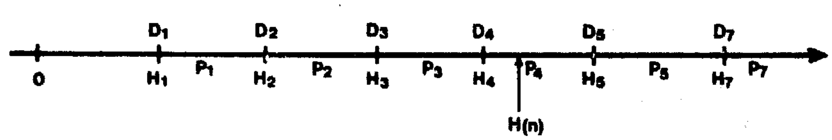


Figure 42 Schematic representation P(H) with only one diameter allowed per section

4.5 BASIC STEPS FOR THE CALCULATION OF THE MINIMUM COST NETWORK

4.5.1 Introduction

The method used to calculate the minimum cost of a section having been demonstrated (4.4) it will now be shown how to optimize the elevation of the tank at the head of the network.

In the first place, the lower envelope curve upstream of the network is determined by a succession of elementary additions in derivation (4.5.2) or in series (4.5.3) of the lower envelope curves of the sections.

The upstream piezometric head having been fixed, the diameters and piezometric heads of each section can be determined step by step, using the lower envelope curves of each section.

In order to facilitate the understanding of the procedure it may be stated here that when adding in series, a section whose cost is known only as a function of the head loss (H) is added to a subnetwork whose cost is known as a function of the piezometric head (Z) or in other terms a P(Z) curve is added to a P(H) curve. When adding in derivation P(Z) or P(H) or P(Z) and P(H) curves are summated.

4.5.2 Adding Sections in Derivation

i. Introduction

Addition in derivation (Fig. 43) occurs when one or more subnetworks can consist of either a single section, in which case the section is a terminal section, or of a branching subnetwork. In both cases the lower envelope curve has been determined on the basis of a minimum piezometric head. The resulting lower envelope curve is of the P(Z) type, the price being expressed as a function of the piezometric head. The head loss in each pipe of the lower envelope curve can readily be determined by subtracting the minimum piezometric head (Z_m) from the piezometric head (Z).

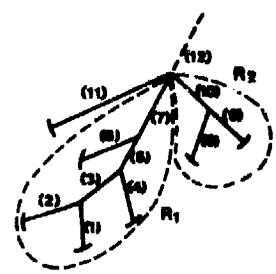


Figure 43 Addition in derivation

ii. Addition in derivation

Consider two terminal sections (1) and (2) having a common node (3) as shown in Figure 44. The procedure is illustrated in Figure 45. Let $P_{(1)}(Z)$ and $P_{(2)}(Z)$ be the lower envelope curves of sections (1) and (2). The minimum cost of the subnetwork situated downstream of node 3 is then:

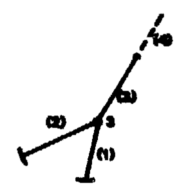


Figure 44 Addition in derivation of terminal sections

$$P_R(Z) = P_{(1)}(Z) + P_{(2)}(Z)$$

Since the curves $P_{(1)}(Z)$ and $P_{(2)}(Z)$ consist of a series of segments, each intersection of these segments on either of the two initial curves will give rise to an intersection on the lower envelope curve of node 3. Similarly, each segment forming the curve $P_R(Z)$ will have a slope equal to the sum of the slopes of the two initial segments:

$$s_{2,3} = s_{1,3} + s_{2,4}$$

It should be clear that for section (2), the only solution is for the piezometric head $Z = Z_2$. The minimum piezometric head Z_m of the lower envelope curve at node 3 cannot be less than Z_2 which leads to the elimination of $Z_1 < Z_2$. For a piezometric head Z' at the node, the cost of sections (1) and (2) is P(Z').

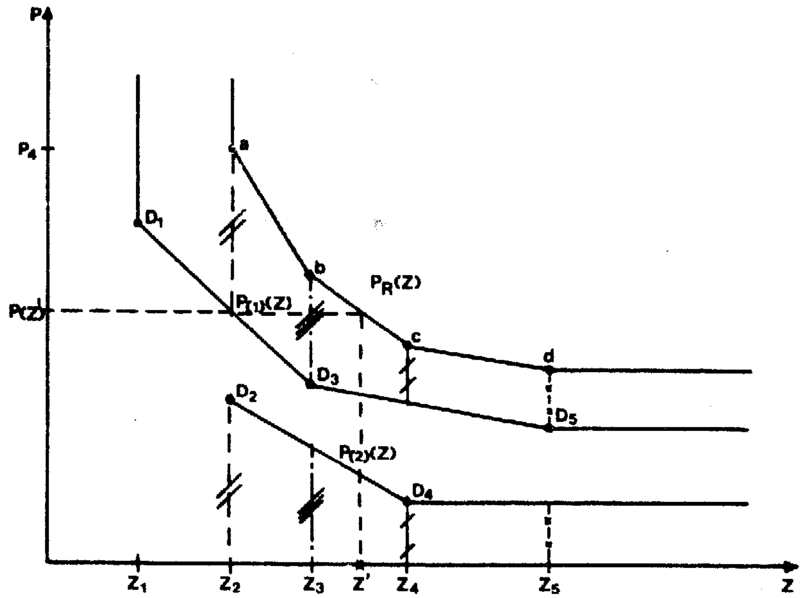


Figure 45 Addition in derivation of sections (1) and (2)

The pipe diameters for use on sections (1) and (2) for a piezometric head of Z' at node 3 can easily be obtained from Figure 45: section (2) will have a pipe of diameter D_2 and length:

$$L_{D2} = \left[\frac{Z_4 - Z'}{Z_4 - Z_2}\right] L_{(2)}$$

and a pipe of diameter D_4 and length:

$$L_{D4} = L_{(2)} - L_{D2} = \left[\frac{Z' - Z_2}{Z_4 - Z_2}\right] L_{(2)}$$

whereas section (1) will have a pipe of diameter D_3 and length:

$$L_{D3} = \left[\frac{Z_5 - Z'}{Z_5 - Z_3}\right] L_{(1)}$$

and a pipe of diameter D_5 and length:

$$L_{D5} = L_{(1)} - L_{D3} = \left[\frac{Z' - Z_3}{Z_5 - Z_3}\right] L_{(1)}$$

The schematic representation of the addition in derivation of these sections offers an equally simple solution (Fig. 46). Each segment (represented by two piezometric heads) has an equivalent segment whose slope is the sum of the slopes of the segments which give rise to this new segment. Heads smaller than the highest minimum piezometric heads of the two sections are discarded (Z_1 in the present case).

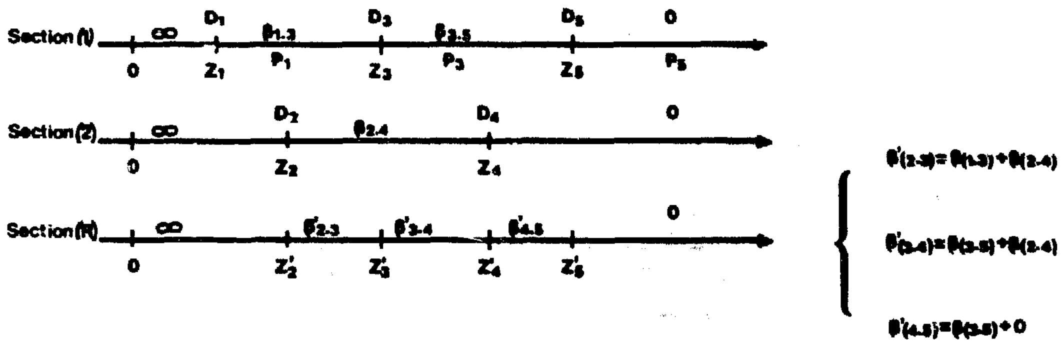


Figure 46 Schematic representation of an addition in derivation

NB: The schematic representation does not directly indicate the cost of a section for a given piezometric head. The length of each pipe must first be determined, this length being multiplied by the unit price of the pipe. The resultant cost is the sum of the cost of the individual pipes. It can be seen that this type of representation is better suited to the case where a piezometric head having been selected it is required to determine the pipe diameters to be used for each section. The determination of the lengths of each pipe is as described above (Fig. 45).

The algorithm for addition in derivation is shown in Figure 47.

4.5.3 Adding Sections in Series

i. Introduction

There is addition in series when a section is added to a subnetwork (Fig. 48). In the example shown, the problem is to define the lower envelope curve at node 8 as a function of the piezometric head at node 7 and of the head loss in section (7).

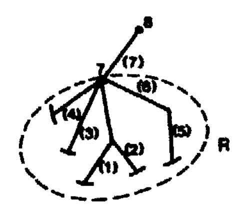


Figure 48

Addition in series

ii. Addition in series

As an example (Fig. 49) section (3) is added to the subnetwork formed by sections (1) and (2) added in derivation. The procedure is illustrated in Figure 50. If $P_{(R)}(Z)$ is the lower envelope curve with respect to the piezometric head of the subnetwork and $P_{(3)}(H)$, the lower envelope curve with respect to the head loss in section (3), it follows that the minimum piezometric head at node 4 is the sum of the minimum head loss in section (3) and the piezometric head of the subnetwork (1+2) whilst the cost amounts to:

$$P_4(Z) = P_{(3)}(H_1) + P_{(R)}(Z_1)$$

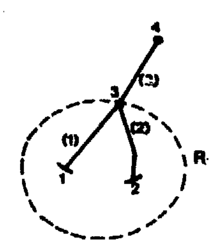


Figure 49

Addition in series in terminal sections

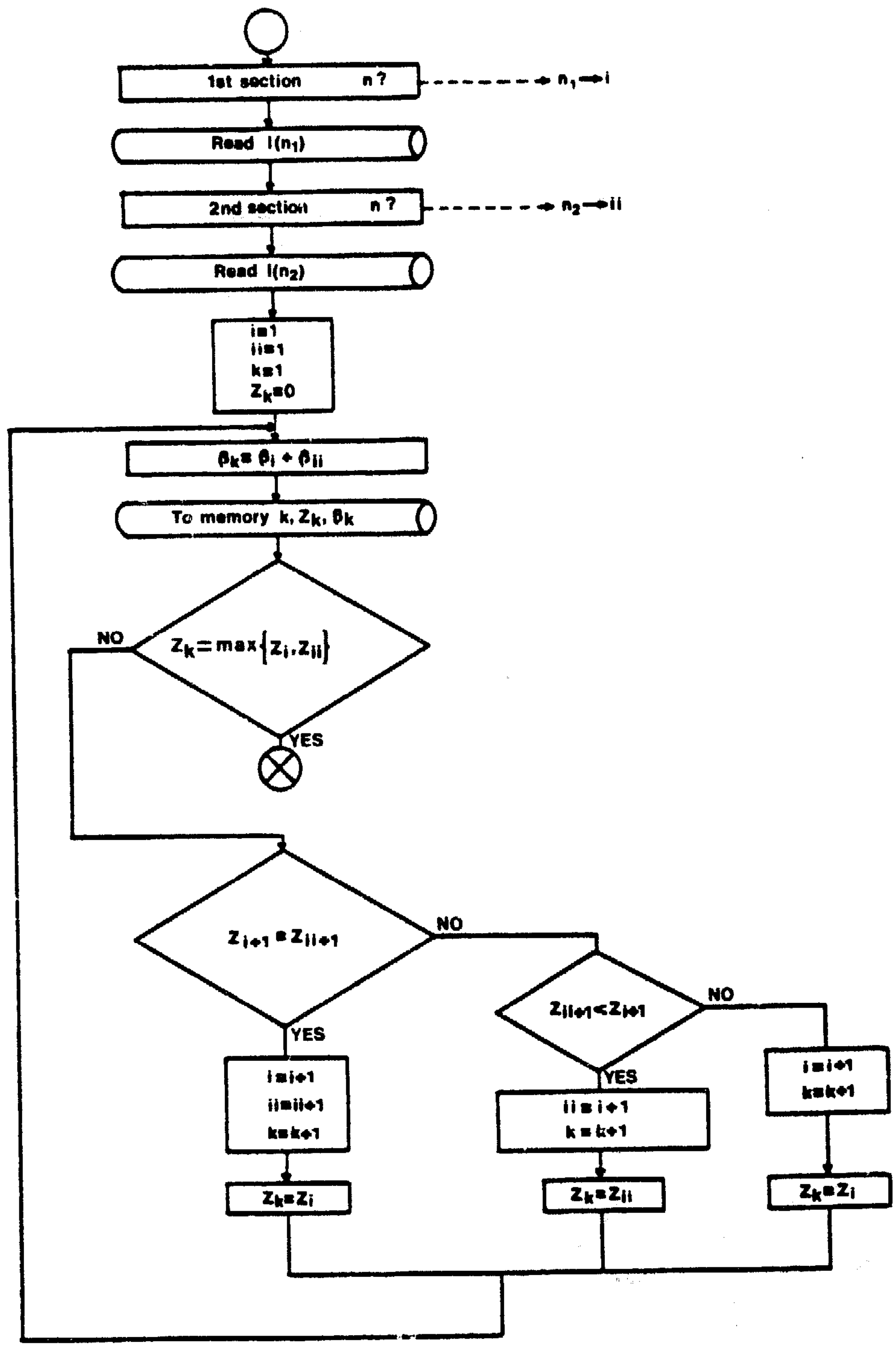


Figure 47 Algorithm of addition in derivation

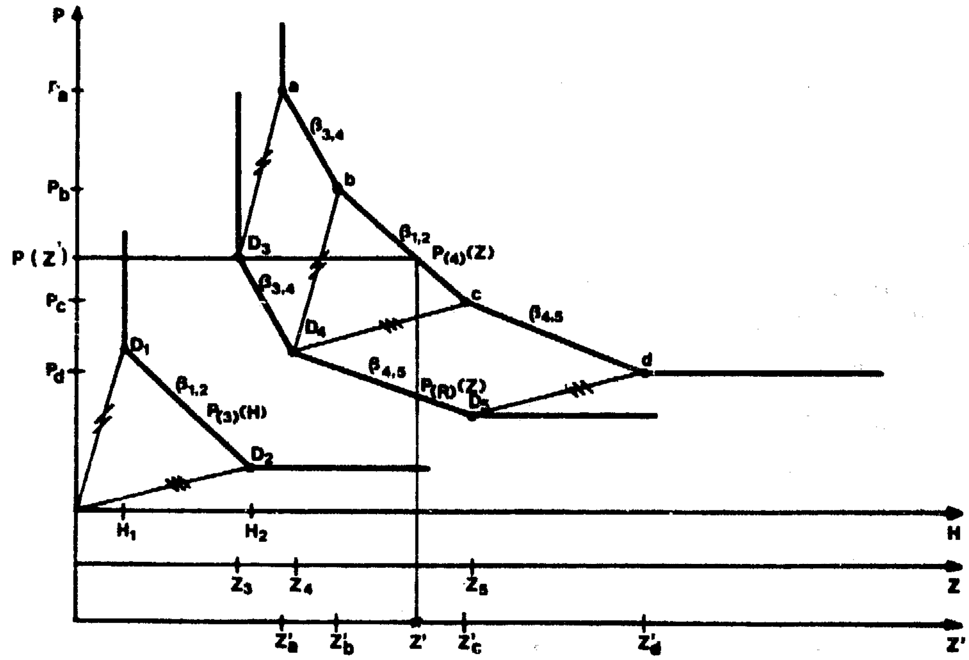


Figure 50 Addition in series

The remainder of the curve is determined by arranging all the $P_{(3)}(H)$ and $P_{(R)}(Z)$ segments in increasing order of magnitude and plotting them in this order from the point of minimum head, here $Z'a$.

In this case, the $P_4(Z)$ curve does not directly indicate the diameter of pipe to be fitted: it is first necessary to determine to which subnetwork the line belongs. In Figure 50 the segments belonging to the subnetwork $P_{(R)}(Z)$ are labelled $\beta_{3,4}$, $\beta_{4,5}$; those of section $P_{(3)}(H)$, $\beta_{1,2}$ and the intersections by a, b, c, d. For the sake of clarity three x axes have been drawn. The first of these represents the head loss (H); the second, the piezometric heads (Z) and the third, the piezometric heads (Z') resulting from the addition in series.

If a piezometric head Z' is selected, this head is situated on the line $\beta_{1,2}$, between the heads Z'_b and Z'_c. The cost of section (3) is therefore P(Z').

Since the piezometric head Z' is located on the line $\beta_{1,2}$, the section will be made up of a first pipe of diameter D1 and length:

$$L_{D1} = \left[\frac{Z'_c - Z'}{Z'_c - Z'_b}\right] L_{(3)}$$

where $L_{(3)}$ is the length in metres of section (3), and a second pipe of diameter D_2 and length:

$$L_{D2} = L_{(3)} - L_{D1} = \left[\frac{Z'_3 - Z'_b}{Z'_c - Z'_b}\right] L_{(3)}$$

The effect of altering the piezometric head upstream of section (3), in this case at node 4, will now be examined. Starting at the minimum head $Z'_a = Z_3 + H$, an increase of head to Z'_b involves a displacement along segment $\beta_{3,4}$. The change in head Z' is equal to the change of piezometric head Z at the node ($\Delta Z' = \Delta Z$) and the head loss (H) in section (3) remains constant.

If the piezometric head continues to increase from Z'_b to Z'_c, the displacement is along segment $\beta_{1,2}$, the variation in piezometric head Z' is equal to the head loss in the section ($\Delta Z' = \Delta H$), and the piezometric head, Z, at node 3 remains constant.

The use of the schematic representation for adding in series will now be examined. The slopes of the segments are arranged in increasing order of magnitude and inscribed in this order (Fig. 51).

The piezometric head Za' is the sum of H_1 and Z_3; Z'_b is the sum of Z'_a and $(Z_4 - Z_3)$; Z'_c is the sum of Z'_b and $(H_2 - H_1)$ etc. The cost for a given piezometric head cannot be calculated until the costs of subnetwork (R) and section (3) have been determined for this head. As in the case of addition in derivation, use of the schematic representation is particularly convenient when it is required to determine the pipe diameters for a given piezometric head. The algorithm of the series addition is shown in Figure 52.

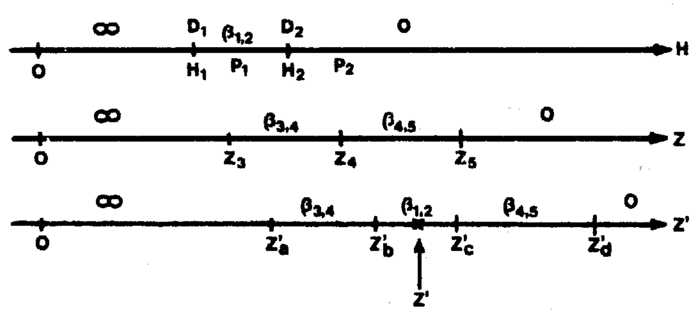


Figure 51 Schematic representation of addition in series

4.6 DETERMINATION OF THE MINIMUM COST OF A NETWORK

4.6.1 First Step: Ascending the Network

i. Determination of the parameters and variables

The suggested method of numbering involves the preparation of a descriptive table of the network (Table 12) which indicates for each section (n):

- the component immediately upstream of the section (column 2);
- when the section is terminal it is identified by 0* (column 8);

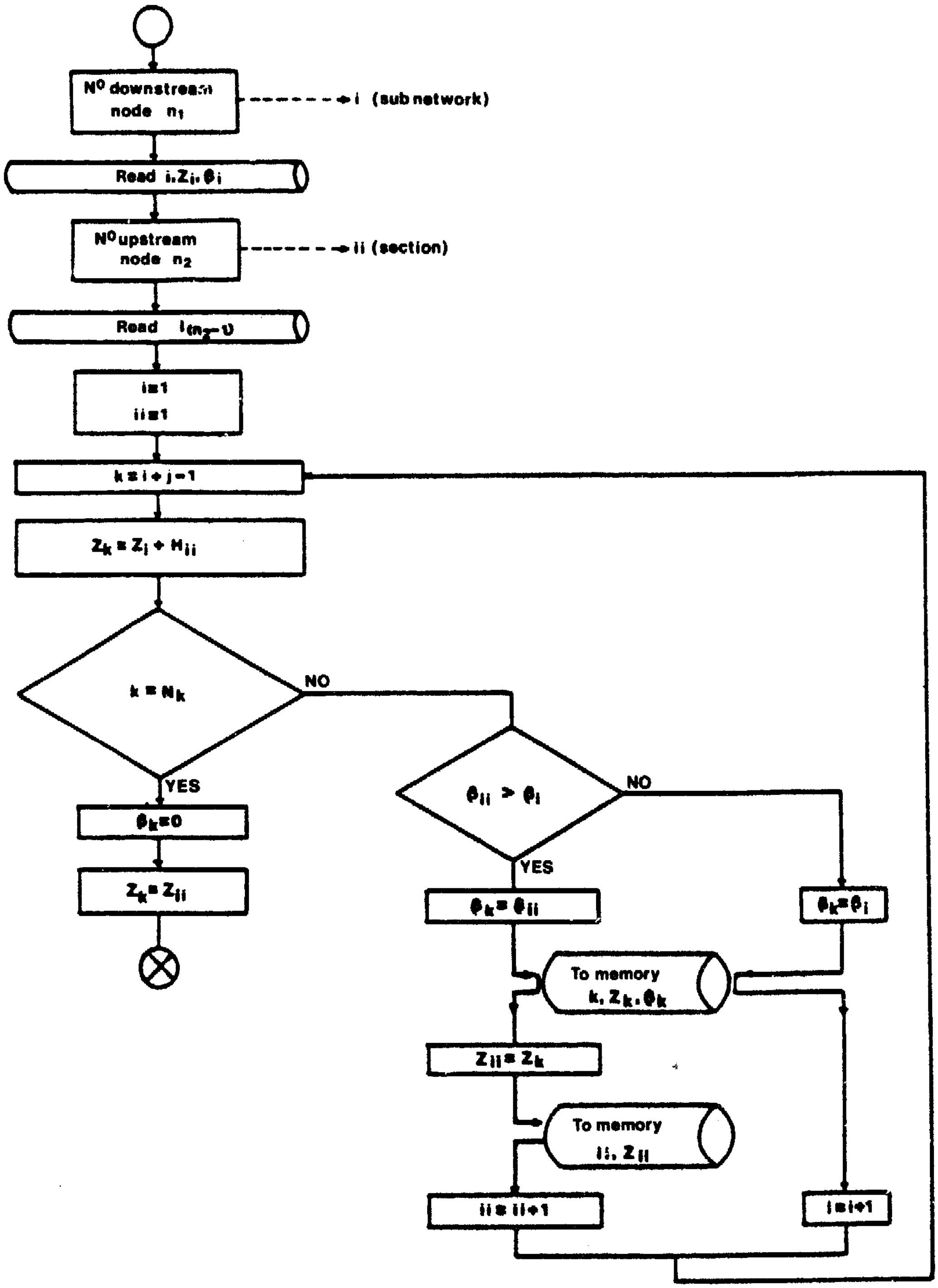


Figure 52 Algorithm of addition in series

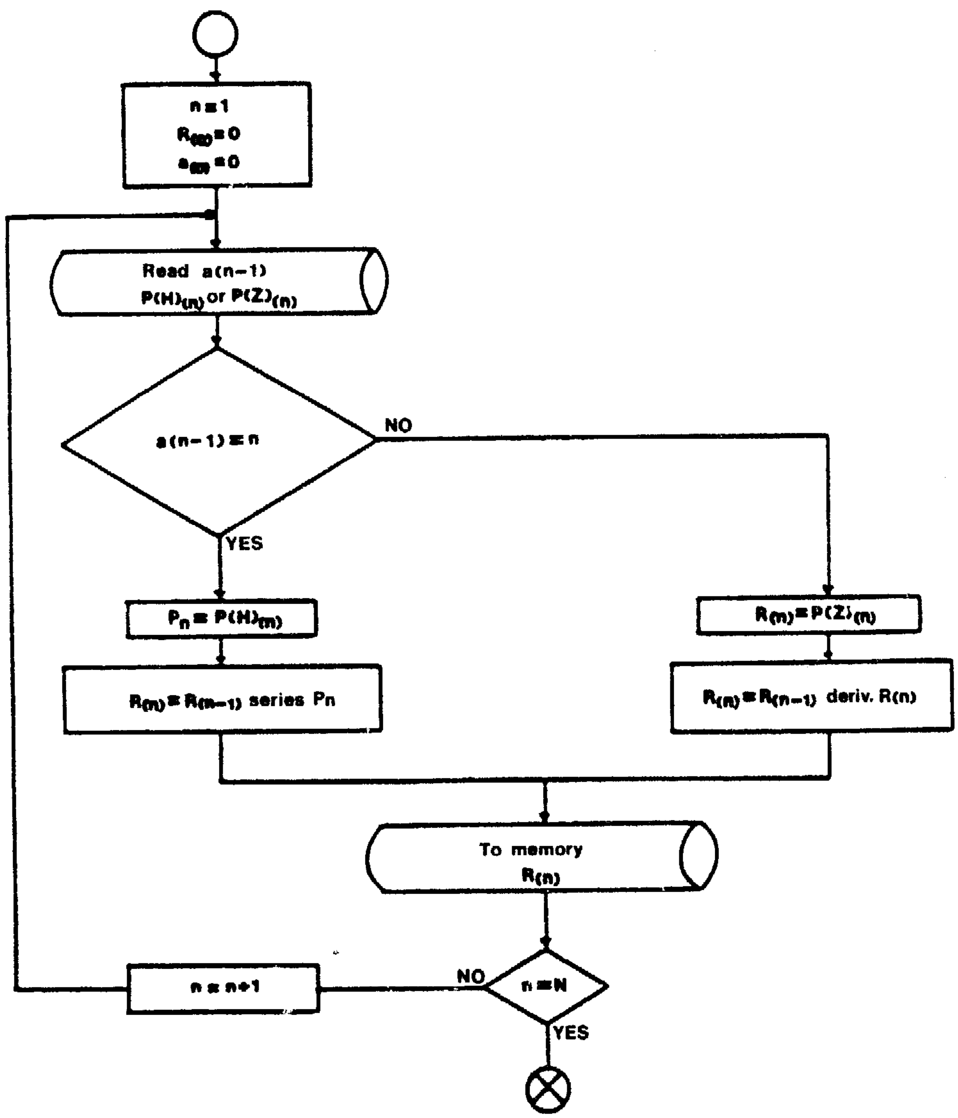


Figure 53 Algorithm for ascending the network

- when the section is followed by several components, these are indicated in decreasing order of numbering.

A list is established (Table 17) for each section which recapitulates the head losses and the total cost of the section when fitted with the various pipes listed.

The following designations will be used:

- $R_{(n)}$, the curve resulting from either an addition in series or an addition in derivation;
- a(n-1), the component immediately upstream of a section (n-1);
- N, the number of sections in the network.

It will be recalled here that an addition in series is defined as the addition of a P(Z) curve, relation between the cost and the piezometric head, and a P(H) curve, relation between the cost and the head loss; an addition in derivation is the addition of two P(Z) curves.

ii. Algorithm for ascending the network

The algorithm for ascending the network (Fig. 53) is designed to determine the $P_R(Z)$ function or relation between the cost of a network consisting of N sections and the piezometric head.

The terms

$$R_{(n)} = R_{(n-1)} \text{ series } R_{(n)}$$

$$\text{and} \quad R_{(n)} = R_{(n-1)} \text{ deriv. } R_{(n)}$$

respectively signify additions in series and in derivation. It is here that the respective algorithms are to be placed.

4.6.2 Determination of the Upstream Piezometric Head

i. Introduction

When ascending the network no assumption is made other than that the minimum piezometric head is known. The P(Z) lower envelope curve indicates the range of diameters of pipes of least cost for each head.

There are therefore several possible ways to determine the piezometric head at the upper end of the network:

- adoption of an elevation based upon considerations of convenience, environmental impact or topography;
- optimization of the overall supply network system if the source is a pumping station.

Three types of water supply systems are commonly found:

- a pump supplying the network directly;

- a pump supplying the network in conjunction with an elevated tank;

- a pump supplying the network in conjunction with peak-demand shedding.

It will be seen that generally the minimum cost of a pumping station follows an ascending curve. For the algorithms, only the angular coefficients (slope ß) $ß = -ß'$ where $ß'$ is the slope of a segment of the curve of the minimum pumping cost, need be considered.

ii. Optimization of direct supply

The cost of a pumping station is generally assumed to be a linear function: for a given discharge (peak network discharge) the cost is proportional to the piezometric head downstream of the pumping station. The costs of a network and of a pumping station are however only comparable when referred to the same life span: total costs must therefore be actualized.

Energy costs are proportional to the pump lift and must also be actualized.

The total actualized cost of the system, for a given piezometric head at the upstream end of the network can be calculated by adding the network, pumping station and energy costs.

The variation of the total cost of a pumping station for direct supply is illustrated in Figure 54 where the optimum can be seen to occur for a head Z_o, minimum point of the total cost curve.

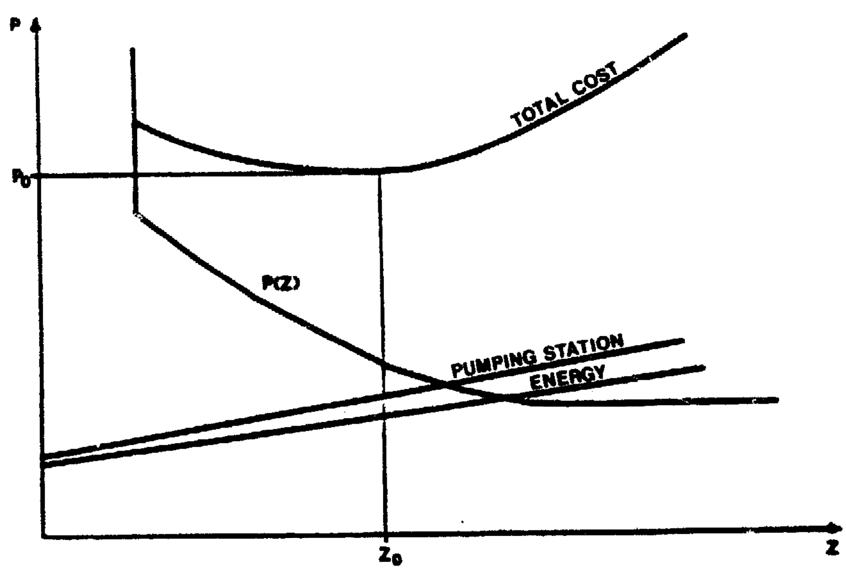


Figure 54 Optimization of direct supply

iii. Optimization of supply with an elevatea tank

The problem is similar to the preceding one if the tank is represented by a fictitious section whose cost is proportional to the pumping lift. In general the cost curve starts to rise steeply above a certain height, for a given volume of tank, as the civil engineering involved becomes more sophisticated. Figure 55 shows the optimum lift for the case of an elevated tank.

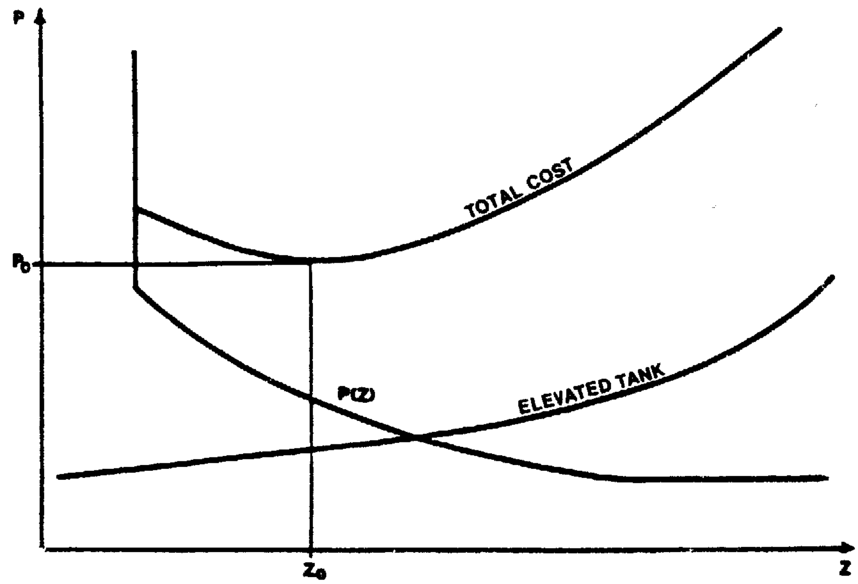


Figure 55

Optimization of supply with elevated tank

iv. Optimization with shedding of peak demand

With this type of installation, the height of the tank is usually imposed by the topography. The network and supply are optimized independently in the same manner as for the case of direct supply (Fig. 56). The network lower envelope or P(Z) curve is replaced by the lower envelope curve of the supply pipe, the curve of the total actualized cost being replaced by the curve of the actualized cost of the supply.

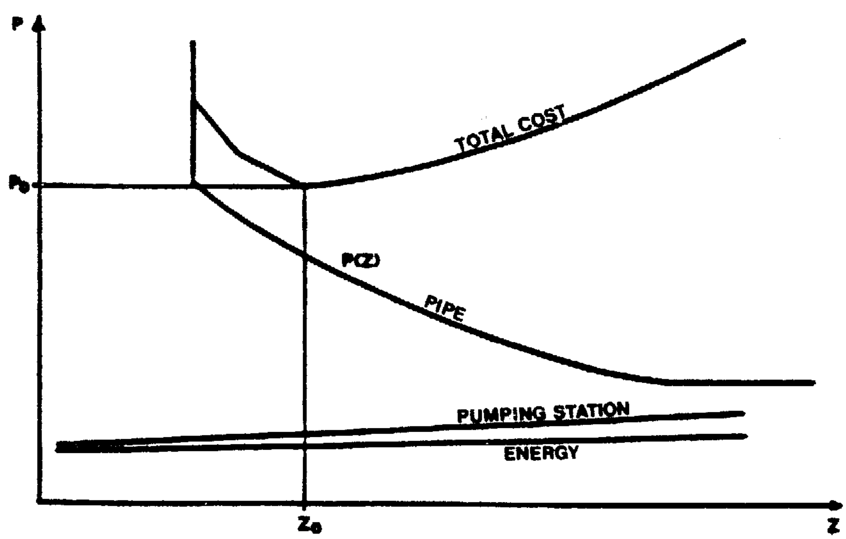
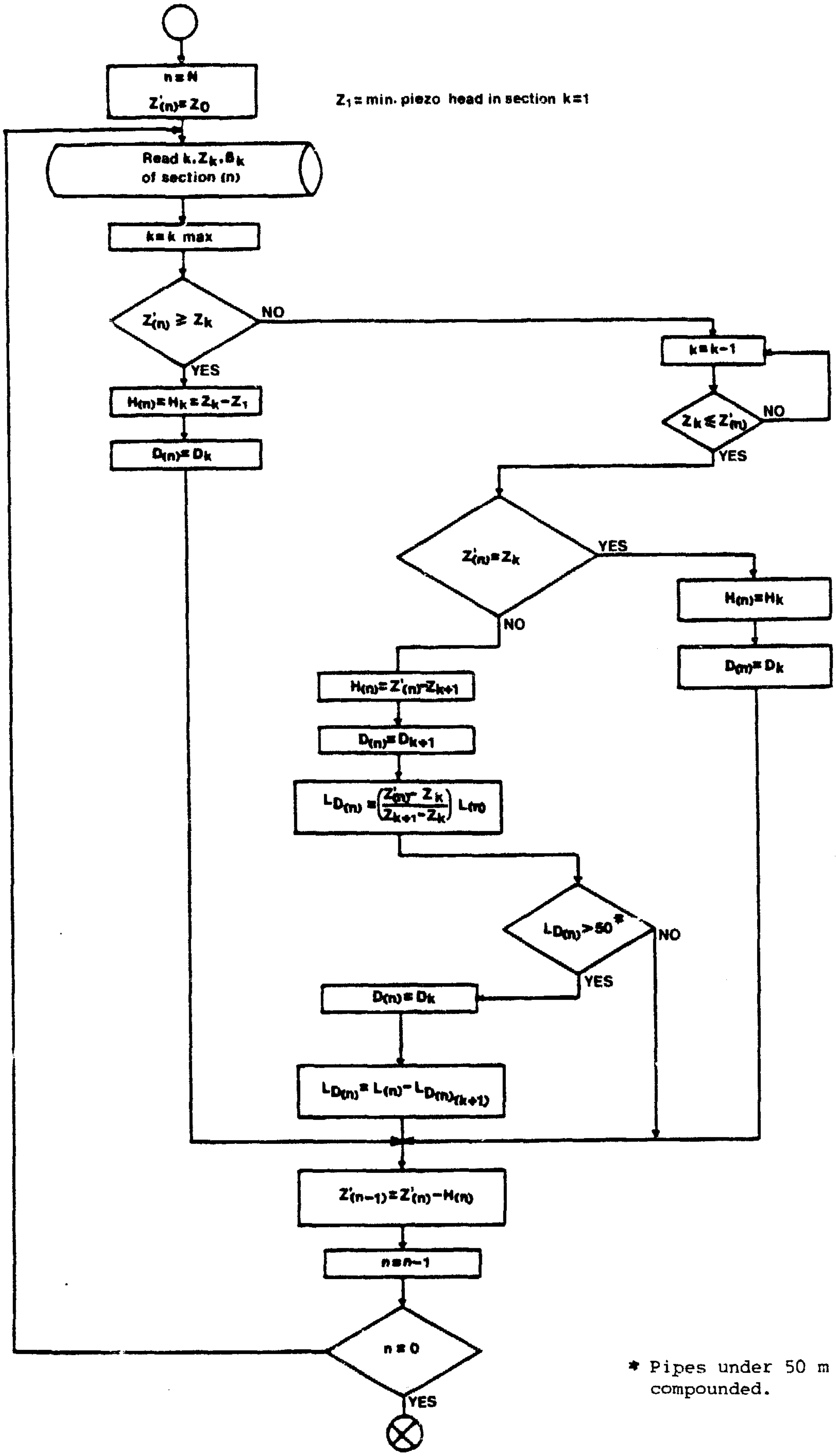


Figure 56

Optimization of supply with peak demand shedding



* Pipes under 50 m not compounded.

Figure 57 Algorithm for descending the network

4.6.3 Second Step: Descending the Network

i. Introduction

The optimum piezometric head at the upstream end of the network having been determined, the next step is to decide upon the head loss in each section. This is done by reading off the piezometric head from the P(Z) lower envelope curve drawn for each node. The procedure for determining the diameter and length of each pipe from the lower envelope curves was described in section 4.5.

It may be noted here that in certain cases it is advantageous to install boosters at certain nodes in order to raise the piezometric heads of the subnetworks further downstream. A comment on the field of application of boosters will be found in section 4.7.2.

When a section has a length of less than 50 m or where compounding (two pipes of different diameters on one section) leads to fitting lengths of less than 50 m, it is not unusual to discard compounding and to resort to fitting a larger diameter pipe.

ii. Algorithm for descending the network

The algorithm for descending the network is presented in Figure 57.

4.6.4 Case where a Single Diameter is Allowed

i. Introduction

It was seen that it is sometimes desirable to restrict certain parts of the network to a single diameter. When this is the case the procedures for ascending and descending the network remain as described above.

The curve defined by iteration for the minimum cost network as a function of the piezometric head is a step function (consisting of segments of zero slope) which can be described as a "lower step" curve for each section, P(Z).

The basic procedures of addition in derivation and in series are slightly modified however.

ii. Addition in derivation

The diagrams for adding in derivation differ from those established when compounding is allowed in that the ordinates of the steps instead of the slope (β_i) represent the cost (p_i) for the piezometric head (Z_i) at the left hand extremity of the step.

In the algorithm for addition in derivation (Fig. 47) the slope β_i must be replaced by p_i, the cost of the pipe of diameter i.

The procedure for adding lower step curves in derivation P (Z) to P (H) to determine the resultant $P_R(Z)$ lower step curve is shown in Fig. 58.

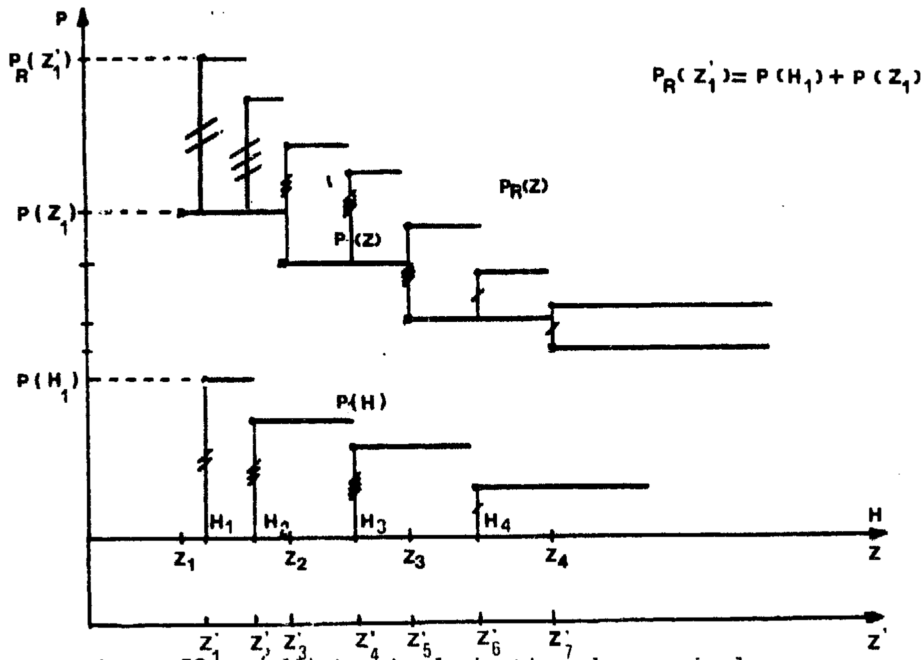


Figure 58 Addition in derivation when a single diameter only is allowed

iii. Addition in series

a. **Description**

The algorithm for adding in series has a different configuration when a single diameter is allowed: the figurative points P(H) of a step function (Fig. 41) are added to a P(Z) lower step curve (Fig. 59).

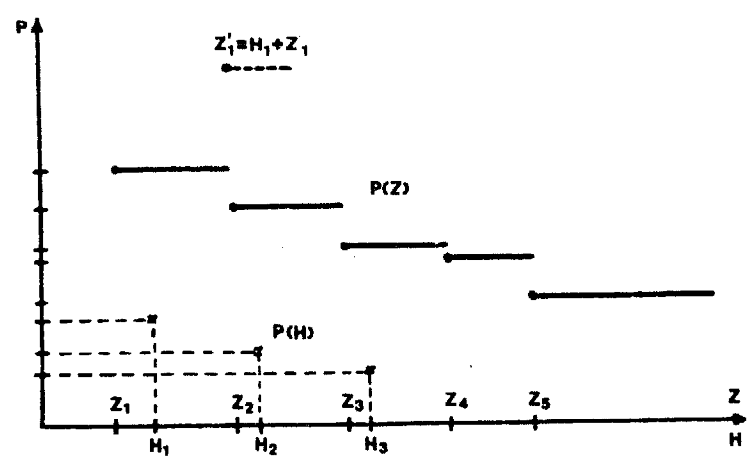


Figure 59 Addition in series when only one diameter is allowed

The construction of the resulting lower step curve is a little more complex: to each piezometric head of the P(Z) curve are added the figurative points of the step function P(H). The same procedure is used for the costs. A series of couples (Z'_k, P'_k) result, where:

$$Z'_k = Z_i + H_j$$
$$P'_k = P_i + P_j$$

and where the subscript i refers to the lower step curve P(Z) and the subscript j to the step function P(H).

These couples are arranged in increasing order of magnitude of piezometric heads Z'_k. The smallest value is:

$$Z'_1 = Z_1 + H_1$$

The lower step curve must satisfy the relation:

$$P'_k > P'_{k+u} \text{ for all values of } u > 1$$

The piezometric heads which fail to satisfy this relation are discarded and the remaining couples are renumbered. Clearly, this type of addition can give rise to a large number of segments and it is preferable to limit their number.

b. **Algorithm for addition in series when only one diameter is allowed**

An algorithm for addition in series when only one diameter is allowed is presented in Figure 60.

It is important to note that the variable Z'_k includes the following information:

$$Z'_k = (Z_i + H_j, Z_i, P'_k)$$

therefore when $P'_k > P'_{k+u}$ is discarded for all values of $u > 1$, the three dimensional variable Z'_k is also eliminated.

4.7 SPECIAL FEATURES

4.7.1 Limiting the Number of Segments on the Lower Envelope Curves

When the number of sections in a network is large, the number of segments that make up the minimum cost curves of subnetworks becomes unmanageable. Clearly, a minimum cost curve consisting of one thousand segments gives rise to excessively cumbersome computations which are not justified in view of the fact that the accuracy of the result obtained is satisfactory with one hundred segments only.

The following method of simplifying the P(Z) envelope curve can be used. It is convenient that the new curve be a polygon inscribed within the P(Z) curve since this ensures that the slope of the resulting

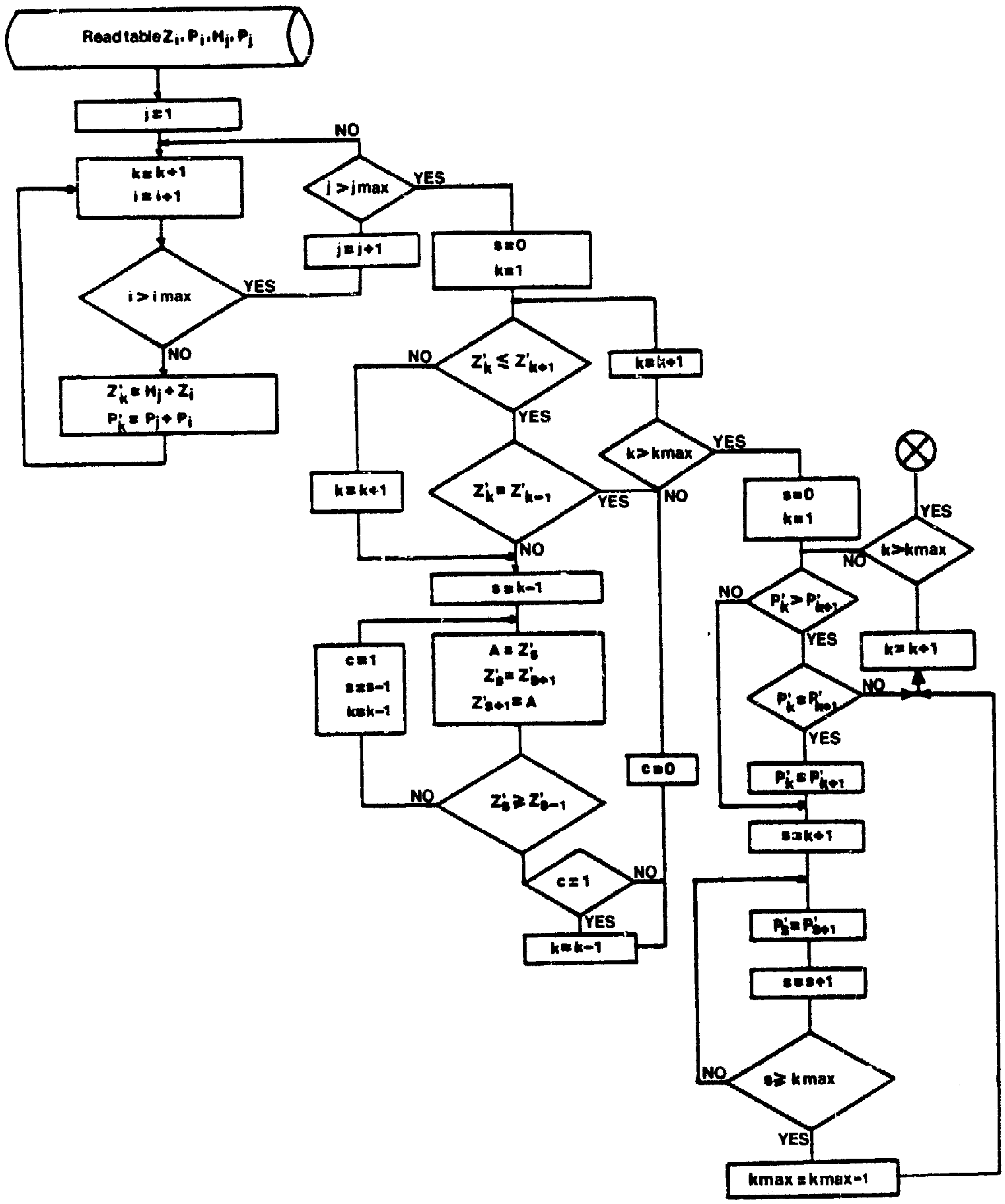


Figure 60 Algorithm for addition in series

curve will automatically decrease with Z. This implies that the extremities of the original curve and those of the new curve are common (points A and B in Fig. 61).

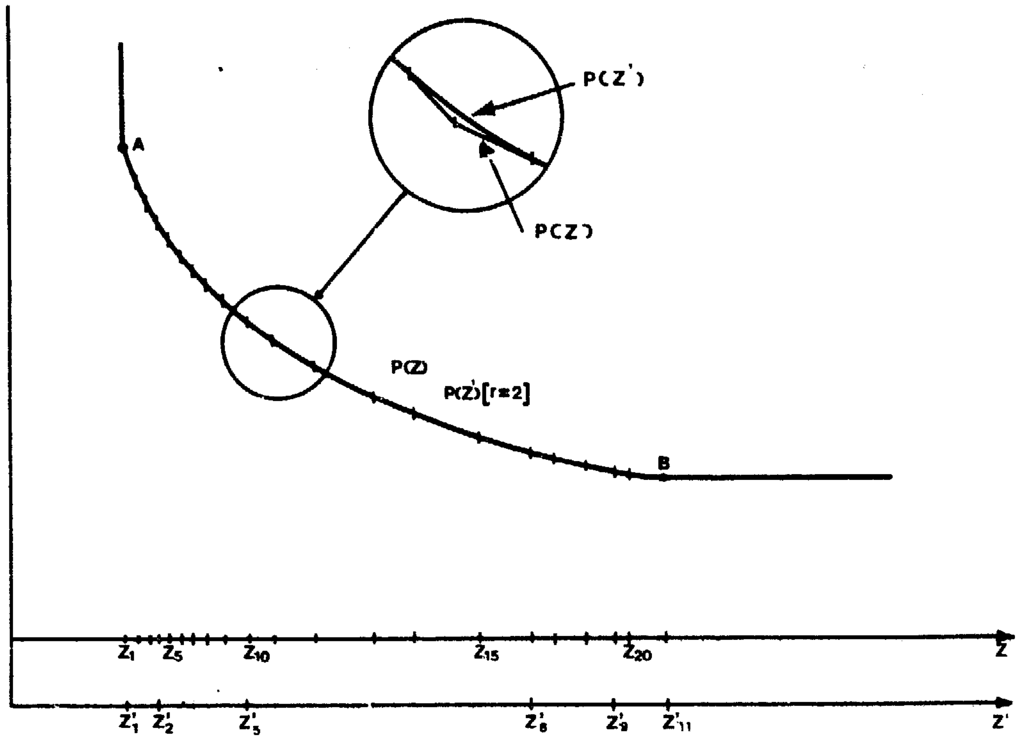


Figure 61 Limiting the number of segments on a curve

It is common practice to replace a number of segments by a single segment when the number of sections in a network exceeds a certain value. The reduction factor (r) which is adopted varies in the range of 2 to 5 but little information is available as a guide to the choice of this factor. A French design office is known to systematically apply a factor of 3 when the number of segments exceeds 90.

Let Z be the piezometric head downstream of each original segment and Z' the piezometric head downstream of each new segment after reducing their number. ß is the slope of each original segment and ß' the slope of the new segments. If an upper limit is attributed to the number of segments and if this limit is a multiple of r, the reduction factor, then when the upper limit is exceeded the number of segments will no longer necessarily be a multiple of r.

The slopes of the new segments are:

$$\beta'_{i,i+1} = \beta'_i = \frac{\sum_{u=1}^{u=r-1} \beta_{ir+u}\left(Z_{ir+u+1} - Z_{ir+u}\right)}{Z'_{(i+1)r} - Z'_{ir}}$$

If the number of segments is not a multiple of the reduction factor then:

$$\sum_{u=1}^{u=r-1} \text{ must be replaced by } \sum_{u=1}^{u=(m-ur)-1}$$

where m = number of sections.

The minimum piezometric head of the new segments remains the same whereas the right hand end of the first segment is the (r + 1)th piezometric head replaced by this segment. If a segment has a slope $\beta'_{4,5}$ or β'_4, the piezometric heads at its extremities are:

$$Z'_{(4-)} = Z_{ir+1-r}; \quad Z'_{(4+)1} = Z_{ir+1}$$
$$= Z_{4,3+1-3}; \quad = Z_{4,3+1}$$
$$Z'_{(4-)} = Z_{10}; \quad Z'_{(4+)} = Z_{13}$$

where 4- and 4+ are the piezometric heads at the left hand and right hand extremities of the segment respectively.

4.7.2 Installation of a Booster

i. Introduction

When examining the "descent" of the network it was pointed out that it was in some cases advantageous to place a booster at a node in order to increase the piezometric head. A booster can be thought of as a section which has a negative head loss and whose cost increases as the delivery head. It may also be represented by a curve of positive slope or a curve of negative slope if it is assimilated to a negative head loss with a positive cost ($x < 0$, $y > 0$) in Fig. 62.

The booster can therefore be introduced when optimizing the network whilst, at the same time, addressing the question regarding the desirability of raising the piezometric head in order to be able to fit a pipe having a smaller diameter.

ii. Determination of the booster pressure range

The relation between the cost of a booster and the delivery head is such that it is not always possible to immediately determine the optimum booster pressure: the booster curve may not always be concave upwards (Fig. 62). When adding in series, the optimum lower envelope curve (the dotted line in Fig. 62) is no longer concave upwards throughout. Thus for a piezometric head Z_o a booster should not be installed since a solution of lesser cost exists. The ascent algorithm introduced earlier cannot be used since it calls for a curve which is concave upwards.

This means that a range of booster pressures must be estimated in order to locate the segment about the expected operating level. The result yielded by optimizing can give rise to the adoption of a slightly different range of pressures in order to approach the true lower envelope curve more closely.

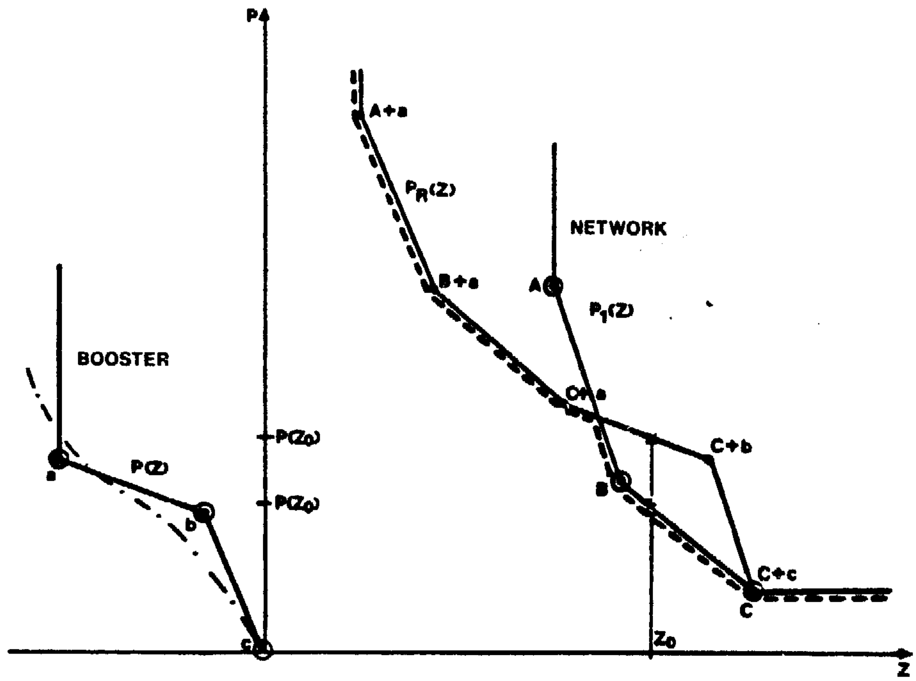


Figure 62 Addition in series of the price curve of a subnetwork and a booster

iii. Recommendation regarding the optimization of the delivery pressure of a booster

The optimization procedure described above implies that the optimum solution is that of least cost. Since it is not always possible to include the solutions to technological problems in economic functions (water hammer protection, pumps with special characteristics, etc.) solutions other than the optimum should also be examined since these may well be technically sounder. The curves representing the cost of pumping stations as a function of the delivery head may indicate a discontinuity due to the need to change from one type of technology to another. The curves of total optimization (network + booster) therefore present discontinuities which are not taken into account by the optimization procedure which calls for continuous curves. It may therefore occur that a mathematical optimum is identified (Z'_o) whereas in actual fact there may exist a solution having a lesser cost (Z_o) as shown in Fig. 63.

In the example shown in Fig. 63, two curves are compared, one with a booster and the other without and it is required to determine if the difference of piezometric head can be arrived at by resorting to a cheaper technology. The cost of energy and maintenance required for a booster station must be taken into account when the economic study is undertaken.

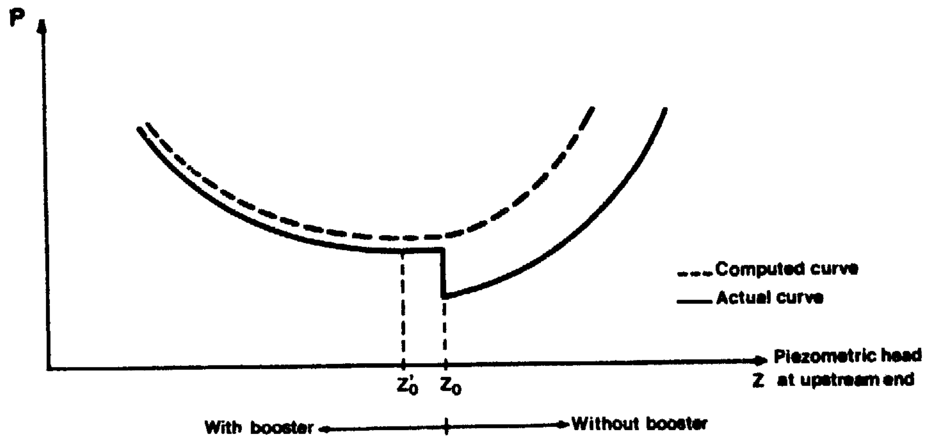


Figure 63 Comparison of optimization curves with and without booster

4.8 REGULATION OF PUMPING STATIONS SUPPLYING PRESSURE NETWORKS

4.8.1 Introduction

Irrigation distribution systems must be conceived to provide water whenever required.

In systems designed for an on-demand supply, the discharge of the network varies considerably with time.

Whereas gravity irrigation systems consisting of closed conduits are self-regulating providing suitable equipment is installed to absorb pressure variations created by the sudden increase or decrease of flow, irrigation networks provided with pumping stations equipped with sets of pumps with fixed discharge rates require regulating devices enabling the flow demand to be satisfied at any moment.

Depending on the specific requirements and constraints of an irrigation perimeter, a choice must be made between manual or automatic regulation.

4.8.2 Manual Regulation

The manual regulation of pumping stations is suitable for small irrigation systems providing the volume of the balancing tank is not economically prohibitive. Two manual regulation methods can be used; in both cases the balancing tank and the operator are the main components of the system.

i. Simple method

The operator starts the pumps at a **fixed** time and these operate until the balancing tank is full. Operation may occur once in twenty-four hours or any other time interval, depending on the demand and the capacity of the balancing tank.

The presence of the operator is required only during the period of operation of the pumps.

ii. Pump regulation by the method of water level observation

The operator is continuously present in the pumping station to operate the pumps according to a predefined sequence requiring continuous observation of the water level in the balancing tank. This method is similar to the automatic method discussed later, with the difference that the pumps are started and stopped by the operator rather than by an electrical or electronic device. With this method an elevated tank is placed on high ground or on a tower between the pumping station and the distribution system (see Fig. 64), or at the far end of the distribution system (see Fig. 65). The capacity of the balancing tank will be discussed under 4.8.3. The tank is equipped with a water level indicator which can be seen from the pumping station.

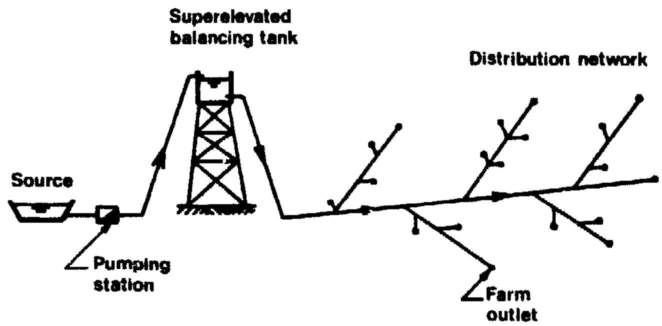


Figure 64 In-between balancing tank (one-way flow in main conduit)

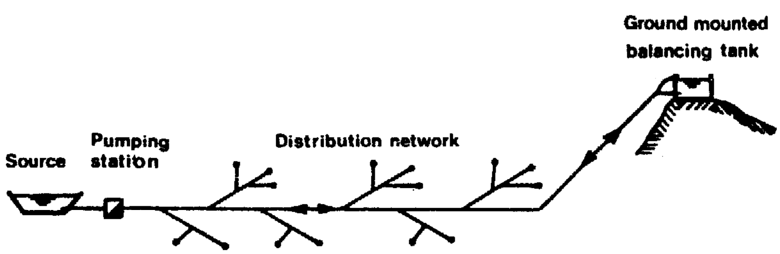


Figure 65 End balancing tank (two-way flow in main conduit)

Irrigation starts with the regulating tank full and all pumps turned off. As irrigation proceeds the water level in the tank drops until a predetermined level is reached and the operator starts the first pump. If the water level continues to drop the operator starts the second pump. If the demand is less than the supply to the tank the water level in the tank starts to rise and when it reaches a predetermined level the operator stops the first pump. When the tank is full the operator stops all the pumps and the procedure starts again from the beginning.

This type of pump regulation is simple but calls for the continuous presence of the operator at the pumping station.

In many pumping stations two pumps are installed whilst one is a stand-by. The length of the cycle depends on the nature of the power available (electrical or mechanical). More details concerning these parameters will be found in the section dealing with automation.

This type of pump regulation is suitable for schemes where in the absence of electric power the pumps are driven by diesel engines. In such situations the length of the cycle is chosen much larger than would be the case with electric pumps. This increases the required capacity of the regulating tanks.

4.8.3 Automatic Pump Regulation

The automatic methods which are described in this section are widely used.

Automatic pumping stations have been introduced in modern irrigation systems in order to reduce manpower requirements. They involve high capital costs and require specialized personnel for their installation and maintenance. Provision must be made at the design stage for manual regulation in case of breakdown or emergency.

The automatic regulation systems used in modern irrigation networks are the following:

- regulation by an elevated open reservoir;
- regulation by an air pressure vessel;
- regulation by flowmeter associated with an air pressure vessel.

The discussion that follows will be limited to regulation systems as such and do not include protection against water hammer, shortage of water in suction pipes, or protection against interruption of pump operation due to power failure. It will be assumed that **each pump delivers a flow of water which varies within narrow limits.** Variable speed pumps is another technique of regulation.

The objective is always to satisfy the varying demand by suitable regulation of a number of pumps in a pumping station.

i. Regulation by means of an elevated open tank

Two methods are used: (i) pressure-sensor regulation and (ii) water-level regulation. In either case the balancing tank may be located at either end of the network (Figs. 64 and 65).

a. **Pressure-sensor regulation**

This system is suitable for small irrigation projects with one pump only and where simplicity and economy are sought. It requires an elevated balancing tank, a pressure sensor mounted on the delivery main of the pumping station, a pressure controller which stops the pump at a predetermined pressure and a time delay switch which starts the pump after a preset time period has elapsed from the moment the pump was stopped. For the proper operation of this system the main delivery pipe is fitted with a float valve at the tank inlet as shown in Figures 66 and 67. Operation is as follows:

Assume that the pump is working and the water level in the tank is at an intermediate level. If the demand is less than the supply from the pumping station, then the water level in the tank starts to rise. When the water level reaches the design maximum the float valve closes and the pressure in the delivery pipe increases until a preset value is attained and the sensor causes the pump to stop. After a preset interval, the time switch starts the pump again.

If the pressure in the main is less than the preset value for stopping the pump, the pump starts to pump water and continues to do so until the tank is again at top level. The float valve then closes causing the pressure switch to stop the pump. This cycle is repeated after a time interval chosen for the system.

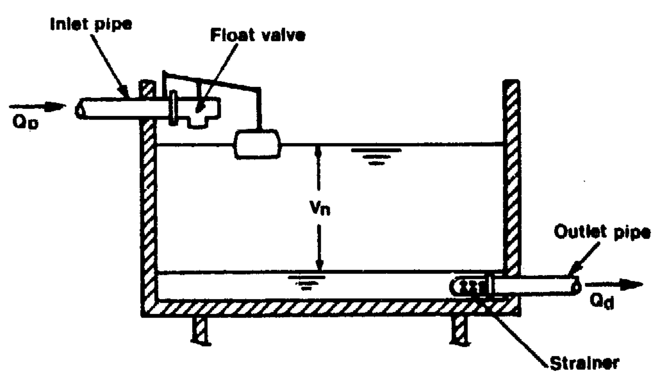


Figure 66

In-between balancing tank for pressure-switch regulation

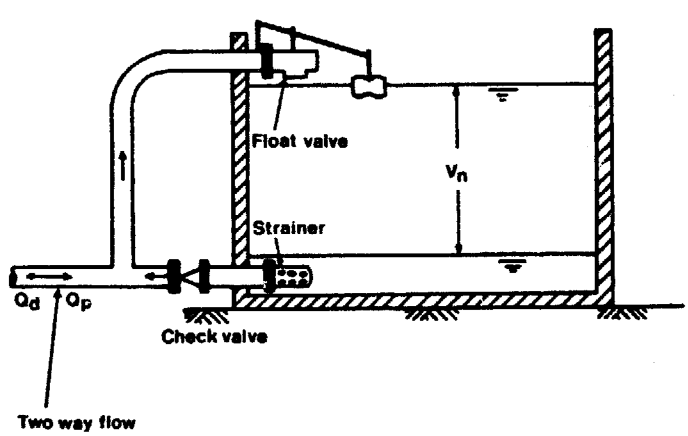


Figure 67

End balancing tank for pressure-switch regulation

The net capacity of the balancing tank is a function of the time interval and the peak demand of the irrigation system:

$$V = Q\,T \tag{70}$$

where: V = net volume of balancing tank (m^3)
Q = peak flow demand of the network (m^3/hour)
T = time interval. This is the interval of time during which the pump is at rest (hours).

The above equation shows that the balancing tank capacity is proportional to the peak demand and the rest interval. The rest interval is chosen by considering the pump motor and starter characteristics. Usually this parameter is given by the manufacturers and may vary from 10 to 20 minutes depending on the type of electromechanical equipment.

It is a cheap solution suitable for networks where the balancing tank is far away from the pumping station but it offers the disadvantage of depending on the correct closure of the float valve.

b. **Water-level method**

This method of pump regulation requires an elevated balancing tank (in-between or end) equipped with float switches connected to the pumping station by line or radio. The pumping station is provided with a set of pumps of equal or different nominal discharges for programming the starting and stopping of the pumps according to the water level in the regulating tank (Fig. 68).

Obviously the frequency of starts and stops depends on the volume of the balancing tank and the variability of the demand. Very frequent starting and stopping is detrimental to the pump electrical and hydraulic equipment as well as to the electric supply network. The capacity of the balancing tank should therefore be designed to ensure that the pump sets do not cut in and out more than a certain number of times in a given period. The capacity of the tank is calculated as follows:

Case of a single pump in the station

q_p = pump discharge
Q^p = peak flow demand of the irrigation network at any given moment
V = capacity of the reservoir (net) between the start and stop float switches

The flow demand Q varies between o and q_p by a factor α lying between 0 and 1 and $Q = \alpha q$.

The time required for the tank to be drained

$$t = \frac{V}{Q} = \frac{V}{\alpha q_p}$$

The time required for the tank to be filled

$$t' = \frac{V}{q_p - Q} = \frac{V}{(1-\alpha)q_p}$$

The length of a cycle is therefore

$$T = t + t' = \frac{V}{q_p}\left[\frac{1}{\alpha} + \frac{1}{1-\alpha}\right]$$

The minimum value of T is obtained for $\alpha = \frac{1}{2}$ and therefore

$$T = \frac{4V}{q_p} \qquad (71)$$

or

$$V = \frac{Tq_p}{4} \qquad (72)$$

Case when the pumping station has several pump sets of equal or similar capacities

If the peak dlow demand Q lies between the flow provided by the operation of n pumps and (n+1) pump (and this is always feasible), then n pumps will work permanently and are regarded as the base group while the (n+1)th pump will operate intermittently, and can be regarded as the regulating set.

The following relationship therefore holds

$$Q = nq_p + \alpha q_p$$

From the point of view of the reservoir, the situation is as if only the (n+1)th pump existed; a flow nq_p merely passing through the reservoir.

The net volume of the balancing tank will therefore still be:

$$V = \frac{Tq_p}{4} \qquad (73)$$

where: V = net volume of the balancing tank (m^3)
q_p = the capacity of the largest pump (m^3/hour) or of one set if capacities are equal
T = lapse of time between two successive starts of the pump (hours).

Since the cost of the balancing tank is a function of its volume, the most economic solution would be obtained by reducing as much as possible the pump capacity (q_p) and time (T). These two parameters can be analysed as follows.

The flow delivered by each pump is a function of the total flow and the number of pumps. Since the total flow is defined by the area commanded by the system, the cropping patterns and other parameters related to the distribution system and the mode of operation, the only way to minimize the unit flow is to install a large number of individual pumps.

If the number of pumps is increased, the number of individual electrical installations is increased accordingly which makes the cost prohibitive. The technical constraints such as reliability and safe operation of the system cannot be disregarded. Irrespective of the fact that an optimization analysis has to be made, the general practice is to install two to three pumps for a supply of up to 100 l/s, and four or five pumps for higher flows. Of course the number of pumps to be installed will affect to a certain extent the volume of the balancing tank, this depending mainly on the pattern of the float level installation, as explained later.

The second parameter which can be reduced is the start-to-start cycle of one pump. This parameter is mainly dependent on the type of starter and other electrical equipment. It is usually defined by the manufacturer of the electrical equipment installed at the pumping station who gives the maximum number of starts in one hour. In order not to overstress the equipment the hourly switch-on frequency should have approximately the following values:

Power of the pump motor	Maximum number of starts in one hour	Minimum interval between consecutive starts
<50 HP	10	6
50 to 200 HP	6	10
200 to 400 HP	3	20
>400 HP	1	60

To overcome this constraint the so-called "rotational" start and stop of the pumps has been introduced which permits a further reduction of the time interval (T). This procedure suffers from the drawback that it affects the power supply lines which are subjected more frequently to high voltage drops. This matter is very important and must be discussed in detail with the power supply authority and the electrical equipment suppliers.

Equation 73 gives the net volume of the balancing tank which is required to keep the operation of the pumping station within safe limits. This is the volume between the float switches as shown in Figure 68.

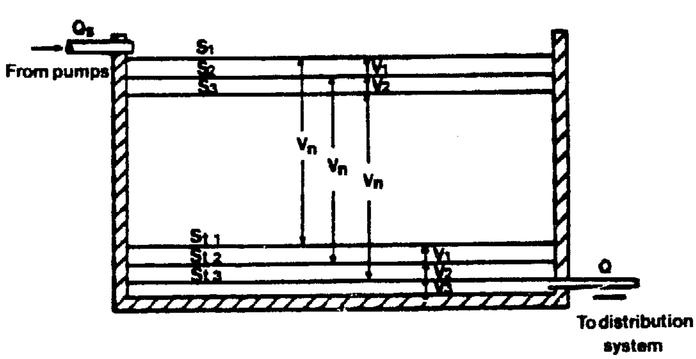


Figure 68

Arrangement of float-switches in a balancing tank (3 pumps)

For correct operation with more than one pump an extra volume has to be provided which depends on the float-switch arrangement in the tank. The most widely accepted and advantageous method of float-switch installation is the "stepped" type where the float switches are installed as shown in Figure 68. This means that pump No. 1 will start when the water level reaches level St 1. The volume between levels St 1 and St 2 is the volume of water that the network draws from the balancing tank before pump No. 1 is put into operation. It is calculated as follows:

$$V_1 = Q\ t_1 \qquad (74a)$$

$$V_2 = Q\ t_2 - q_1\ t_2 \qquad (74b)$$

where: V_1 = volume of water between switches St 1 and St 2 (m^3)
q_1 = discharge of pump No. 1 (m^3/hour)
Q = network peak demand (m^3/hour)
t_1 = lapse of time between receipt of starting signal and start of No. 1 pump (hours). This interval is comparatively small but may be as much as 30 seconds for large pumps. Its value is also dependent on the pipe hydraulic characteristics, allowance being made for water hammer)
V_2 = volume of water between switches St 2 and St 3
t_2 = lapse of time between receipt of starting signal and start of No. 2 pump

The total additional volume of the balancing tank is therefore given by the general equation:

$$Va = \sum_{i=1}^{n} \left[Q\ t_i - q_{i-1}\ t_i \right] \qquad (75)$$

where: n = number of pumps delivering water to the tank and q_i and t_i are as defined above.

Water-level control by float switches has the following advantages:

- The pressure head in the network varies very little and in the case of the in-between balancing tank the pumps may be selected to operate practically at a single point on their characteristic curve with optimum output.

- No extra head is required for the regulation of the tank which makes the system more attractive as regards pumping costs as compared with air vessels.

- The volume of the balancing tank is the minimum and the absence of complicated equipment guarantees reliable operation.

- It is a very cheap system when the regulating tank is very close to the pumping station, since it only involves electric cables for signal transmission.

The disadvantages are the following:

- It is expensive if the balancing tank is situated far from the pumping station.

- It requires that the balancing tank be installed on high ground, not always present in irrigated areas. In their absence, relatively expensive super-elevated tanks have to be built. Since the cost of the tower is the major item and not the tank, this solution is suitable for large irrigation systems but prohibitive for small areas unless the presence of high ground makes the tower unnecessary.

In view of what has been said, this type of pump regulation is generally suitable for irrigated areas of 1000 ha or more and for smaller areas provided that there is a natural high point close to the pumping station.

ii. Regulation by means of a pressurized air-vessel

With this method of pump regulation, which is the second most common one, an air-pressure vessel replaces the elevated tank. The air-pressure vessel is connected to the main and the pump is regulated by the pressure of the air in the vessel.

The operation of the system is as follows: the pump is set to start and stop at pressures H_o and H_i respectively in the pressurized air vessel (Figs. 69 and 70). Assume that the pump is in operation and that the air vessel is full under pressure H_i. When a farmer opens an outlet, water is delivered to the system from the air vessel. As water leaves, the pressure in the air vessel drops until it reaches the value H_o and the pump starts.

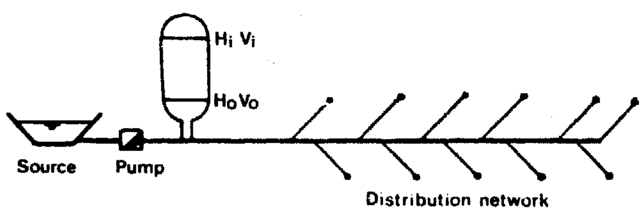


Figure 69

Air-pressure vessel regulation

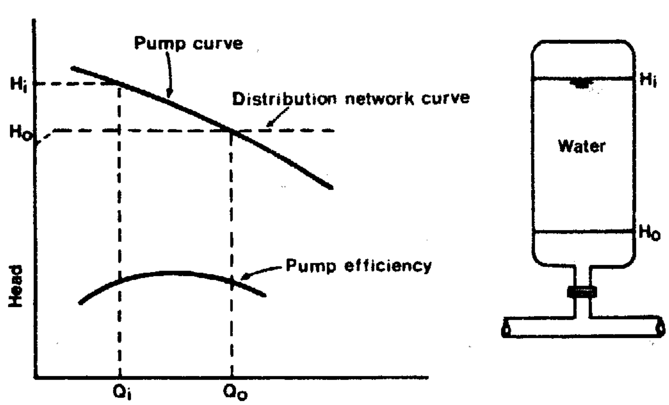


Figure 70

Pump, distribution system and air-vessel operating characteristics

If the demand is less than the pump discharge, water is stored in the air-pressure vessel with a resulting rise of pressure. When the pressure attains the value of H_i, the pump is switched off once more.

The operation is similar to that of an elevated tank and can accommodate more than one pump as shown in Figure 71, where three pumps are connected to a common air-pressure vessel, each pump operating over a particular range of pressures.

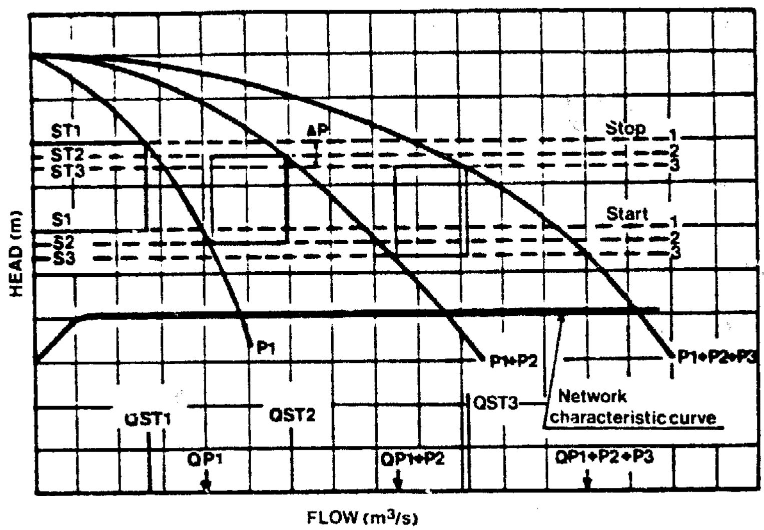


Figure 71 Air-pressure vessel regulation pump operating pressures

The net capacity of the air-pressure vessel is a function of the discharge of the largest pump, the start-to-start cycle time and the pressure at which the pumps are switched on and off. The air in the vessel is assumed to change pressure and volume at constant temperature.

The volume of the air pressure vessel is given by:

$$V = \frac{qT}{4}\left[\frac{H_i}{(H_i - H_o)}\right] \tag{76}$$

where: V = net volume of air pressure vessel (m^3)
q = mean discharge (m^3/s) of largest pump operating in pressure range H_o to H_i (Fig. 70) and Eq. (76)
H_i = maximum pump switch-off pressure (m)
H_o = minimum pump switch-on pressure (m)
T = lapse of time between two successive starts (s).

As can be seen, the net volume of the air-pressure vessel is proportional to the average pump discharge, the lapse time and the highest pressure, and is inversely proportional to the so-called bracket pressure (H_i-H_o). Since the cost of the air-pressure vessel is usually high and is a function of its volume, it should be kept as small as possible. Each parameter is examined below to show how this can be attained.

The capacity of the largest pump is a function of the irrigation demand and the number of pumps to be installed at the pumping station. The reasoning underlying the choice of pump capacity is similar to that described in the case of the water-level regulation method.

The first parameter is the average discharge or the number of pumps. Since the differential pressure (P, Fig. 71), and hence the maximum operating pressure increases with the number of pumps accompanied by an increase in pumping costs and size of pressure vessel (Equation 76) it is important to reduce the number of pumps as far as possible. The number of pumps has to be kept to a minimum, since in the present case the differential pressure P is very important, a large number increasing the size of the air pressure vessel and the highest pressure for pump trip-off, reducing the efficiency of the system due to the incidence of pumping costs.

Assuming that the characteristic of the pump is a parabola, the average discharge (q) of a pump is given by the following expression:

$$q = {}^{2}/_{3} \left[\frac{Q_o^2 + Q_o\, Q_1 + Q_1^2}{Q_o + Q_1} \right] \tag{77}$$

with Q_o and Q_1 as in Figure 70.

The second parameter is the lapse time (T). Although its minimum value is fixed by the manufacturer, correct planning of the pump starting sequence can further reduce the value of the lapse time. It must however be kept above a minimum value acceptable to the power supply authority.

The third parameter is the pump maximum switch-off pressure (H_i). This parameter is very important since it affects the size of the air pressure vessel as well as the operating pressure of the pump. When selecting H_i due consideration must be given to the distribution system pipe class and the extra cost of pumping.

The fourth parameter is the minimum operating pressure H_o at which the pumps switch on. This pressure is dictated by the value of the pressure which must be maintained at the hydrants.

In addition the volume given by Equation 76, an extra volume is required which is a function of the number of pumps and the sensitivity of the pressure sensors. The pressure sensors available on the market generally have a sensitivity of from 0.2 to 0.3 bars.

As an example, for a system requiring a minimum operating pressure (H_o) of 35 m, the maximum pressure (H_i) at the pumping station consisting of three pumps is determined as follows:

nominal working pressure (H_o)	35 m
start to stop pressure range	15 m
sensor sensitivity 3 x 2.5	7.5 m
Total pressure (H_i)	57.5 m

The excess pressure with respect to the nominal pressure required by the network, inherent with this type of regulation, is dissipated in the hydrant pressure regulators.

This method of pump regulation has the following advantages:

- It is comparatively simple, the only control mechanisms being the pressure sensors and the pressure switches.

- It has a low capital cost provided the pump discharge is small and the pressures are low, and that the air-pressure vessel has sufficient capacity to act as a protection against water hammer.

The disadvantages are the following:

- It is not economically feasible for large distribution systems where the unit pump capacity is large.

- The pump operates at higher pressures than required by the network which is wasteful of energy and may necessitate pipes and fittings belonging to a higher pressure class.

iii. Regulation by flowmeter

This very simple system of pump regulation makes use of the following components:

- An electrical flowmeter is mounted on the discharge main. This instrument must be of high accuracy and capable of operating over a wide range of flows.

- A "base" pump of low power which provides a low discharge to make up losses in the system and satisfy low demands. This pump starts and stops according to the pressure in a small air vessel.

- A number of main pumps having flat H-Q curves and capable of satisfying the demand over the whole range of flows. These pumps start and stop according to the demand measured by the flowmeter.

The characteristic curves of a battery of four pumps, one base pump and three principal units (two duty and one stand-by), are shown in Figure 72.

In periods of no demand or very small demand and to make up for water losses, the small base pump piloted by the air-pressure vessel maintains the system under pressure. The cycle of automatic stopping and starting of the main pumps is controlled by the rate of flow in the discharge main, whereas that of the base pump is controlled by the pressure in the network.

The programming of the starting and stopping of the pumps is in accordance with the pump and network characteristic curves. An overlapping of flow rates avoids the danger of oscillations causing frequent starting and stopping of a pump. This overlapping of flows is shown on Figure 72, where pump P2 overlaps pump P1 in the range of flow rates Q'2 and Q2. To ensure overlapping, the characteristic curves of the pumps must be flat so that the range of flows is covered by a small number of pumps.

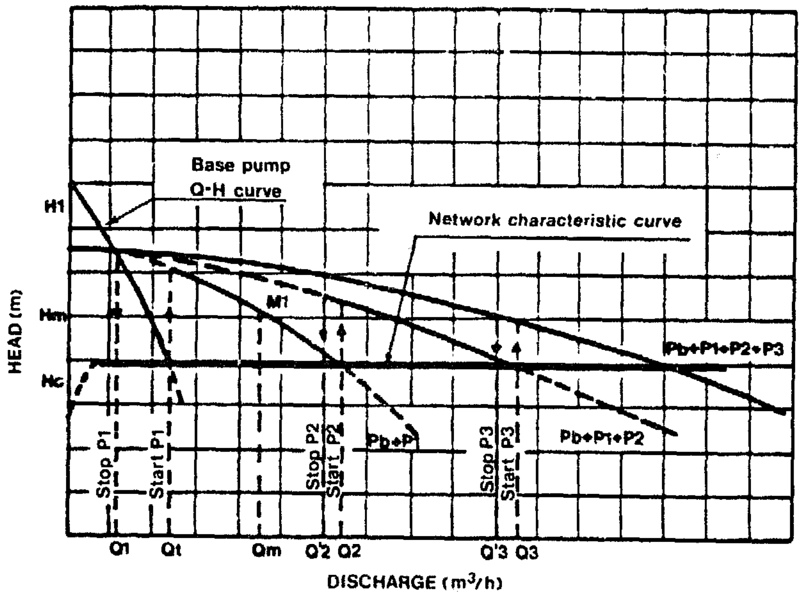


Figure 72 Flowmeter regulation operation of four pumps (overlapping)

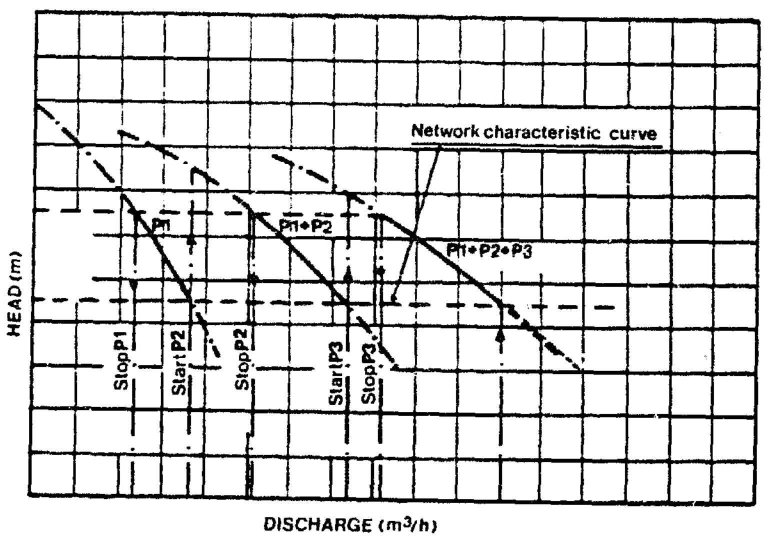


Figure 73 Flowmeter regulation operation of pumps (non overlapping)

The base pump is installed to satisfy up to about 10 percent of the total network demand. The capacity of the regulating air vessel associated with this pump is calculated according to the method described in ii. above.

Pump regulation by flowmeter is simple and enables the selection of the pumps to be made in accordance with the network characteristic curve with a possible reduction in power consumption. Moreover the pumps operate continuously when the demand is constant whereas with the other methods of regulation this is not so. This has the advantage of reducing maintenance costs and increasing the life of the equipment.

The flowmeter must have a wide measuring range and a high accuracy to avoid unnecessary starting and stopping of the pumps.

If a pump fails to start then the flow in the main never reaches the threshold value required for the next pump to start, which results in total failure of the regulating system. For this reason, a stand-by pump is programmed to come into operation should one of the main pumps fail to start.

An innovation has been introduced which is based on both the flow rate and the pressure in the network. A range of pressures and a range of flows are defined for each pump. Once a pump is set into operation, it responds only to pressure; as the flow increases, the flowmeter detects the need for a second pump to be put into operation, placing the pump on the alert without giving the signal to start. If the demand in the system exceeds the capacity of the first pump, the pressure in the system falls below the preselected pressure and the pressure controller starts the second pump. This improved system does not require a very accurate flowmeter and the overlapping of the pump characteristics is not necessary (Fig. 73).

The advantage of this method of pump operation is that it does not require a balancing tank. This is particularly advantageous where such a tank has to be elevated. Provided that the pumps are suitably selected, there follows a reduction of pumping head as compared to regulation by means of an air pressure vessel.

It should be noted that very accurate flowmeters designed to operate under difficult conditions are comparatively expensive.

Generally speaking, flowmeter regulation is suitable for all sizes of irrigation systems provided that the pumps and the flowmeter are suitably selected.

4.8.4 Variable speed pumps

The pumps now in use in irrigation systems have fixed rotational speeds of 1500 or 3000 rpm. The idea here is that by varying the speed of a pump while keeping the manometric head constant, the discharge can be increased with the pump efficiency within high limits.

This method, long-used in domestic water supply systems, has recently been applied to irrigation networks in France. Three possible applications may be considered as follows.

With diesel engine driven pumps the method of regulation with variable speed is possible since the speed of diesel engines may be varied in the range situated between the nominal speed and 60 percent below without a great reduction of efficiency.

The second possibility is to vary the frequency of the power supply to electric motor driven pumps in order to vary the motor speed.

The third is to use diesel engine generators to drive electric pumps and to vary the diesel generator speed. The frequency of the power produced and the speed of the pump motors can in this way be regulated to meet the demand.

4.8.5 Conclusions

To conclude the above discussion on the regulation of pumps, the fields of application of each method are summarized as follows.

Manual regulation: This is applicable where automation cannot be provided and labour is both available and cheap.

Pressure-switch regulation: This is well adapted to small systems where the balancing tank is very far away from the pumping station.

Water-level regulation: This is the best and most reliable method and it is suitable for large irrigation networks.

Air-pressure vessel regulation: This is suitable for small areas where elevated reservoirs are costly.

Flowmeter regulation: This is a good system for any size distribution system.

Variable speed pump regulation: This has recently been introduced in modern irrigation systems and will probably find a wide field of application.

EXAMPLE 11 - CALCULATION OF THE CAPACITY OF AN ELEVATED BALANCING TANK AND AIR VESSEL AND SELECTION OF AIR VESSEL PUMPS AND FLOWMETER FOR FULLY AUTOMATIC PUMP REGULATION

An irrigation network has a demand flow (Q) of 100 l/s which is met by three pumps, two of which are operational and one is a stand-by.

The total manometric head is 35 metres. The pumps start-to-start cycle time (T) is 10 minutes and the lapse of time to start after receiving the starting signal is 10 seconds for all three pumps (t_1, t_2, t_3).

Solution

1. Elevated Tank Regulation

Since there are two operational pumps for 100 l/s, each pump delivers 50 l/s or q_p = 180 m³/hour.

$$T = \frac{10}{60} = 0.167 \text{ hours}$$

$$V = \frac{Tq_p}{4} = \frac{0.167 \times 180}{4} = 7.5 \text{ m}^3$$

Suppose that the electrodes are installed in a stepwise manner, then:

$$V_1 = Qt_1 = 360 \times \frac{10}{3600} = 1.0 \text{ m}^3$$

$$V_2 = Qt_2 - q_1t_2 = 360 \times \frac{10}{3600} - 180 \times \frac{10}{3600} = 0.5 \text{ m}^3$$

For two pumps the additional tank capacity is 1 + 0.5 = 1.5 m³.

The total capacity of the elevated tank is 9.0 m³. The pumps will operate under a constant manometric head ranging between 35 and 38 m, for a variation of water level in the elevated tank of 3 m (Fig. 74).

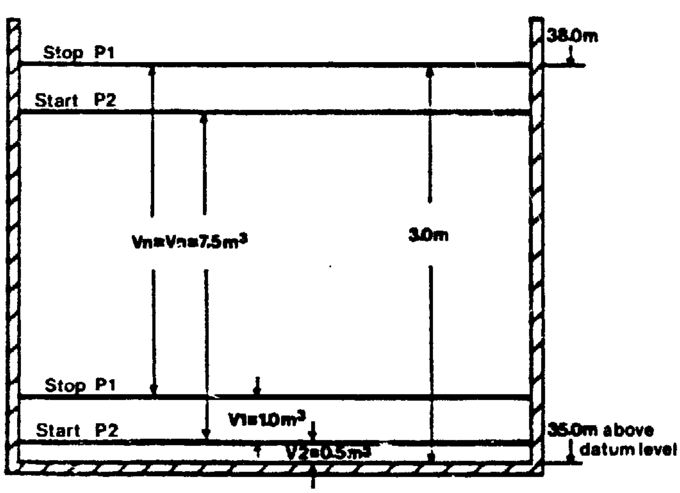


Figure 74

Volume of balancing tank and arrangement of water level control switches

ii. Air Pressure Vessel Regulation

From i. above, q_p = 50 l/s = 180 m³/hour
T = 10 minutes = 0.167 hours
H_o = 35 m

Assume that two pumps are selected whose characteristic curves pass through the following points:

Start: H_o = 35 m, Q_o = 180 m³/hour

Stop: H_i = 55 m, Q_i = 120 m³/hour

Applying equation 77:

$$q = \frac{2}{3}\left[\frac{180^2 + 180 \times 120 + 120^2}{180 + 120}\right] = 152 \text{ m}^3\text{/hour}$$

Applying equation 76, the volume of the air vessel is given by:

$$V = \frac{qT}{4}\left[\frac{H_i}{H_i - H_o}\right] = \frac{152 \times 0.167}{4}\left[\frac{55}{(55-35)}\right] = 17.5 \text{ m}^3$$

The volume of the air vessel for operation with two pumps should therefore be 20 m³. This volume may be reduced by installing four pumps instead of two, each with the following characteristics:

Start: H_o = 35 m, Q_o = 90 m³/hour

Stop: H_i = 50 m, Q_i = 66 m³/hour

Applying equation 77, q = 78.6 m³/hour.

Applying equation 76 for H_i = 50 m H_o = 35 m

$$V = \frac{qT}{4}\left[\frac{H_i}{H_i - H_o}\right] = \frac{78.6 \times 0.167}{4}\left[\frac{50}{(50-35)}\right] = 10.94 \text{ m}^3$$

Using the pump characteristic curves and the distribution system curves, the regulation diagram as shown in Figure 75 was drawn. The values of the starting and stopping pressures of each pump are the following:

Pump No.	Pressure range (m)	Discharge range (m³/hr)	Average discharge (m³/hr)	Average head (m)
P1	56 - 41	54 - 87	71.8	48
P1 + P2	54 - 39	123 - 174	149.0	46
P1 + P2 + P3	52 - 37	192 - 270	233.3	44
P1 + P2 + P3 + P4	50 - 35	270 - 360	317.3	42

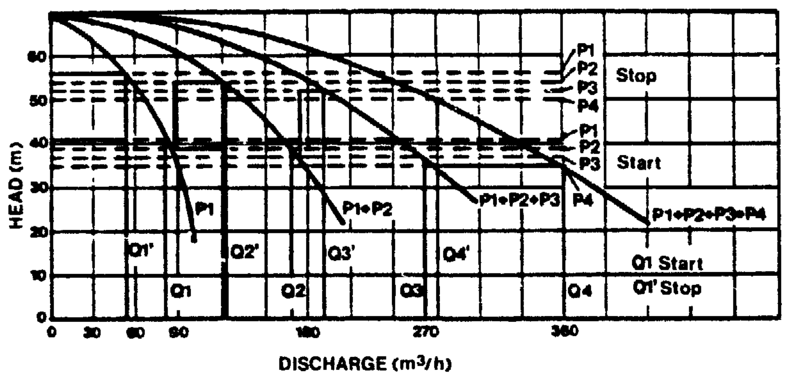


Figure 75 Air-pressure regulation. Pump operation curves

It can be seen that the average operating heads of the pumps are approximately 10 m above the minimum network pressure of 35 m.

iii. Flowmeter Regulation

Four pumps are selected, one base pump and three principal pumps having the characteristics shown in Figure 76. The base pump is selected for a flow equal to 10 percent of the total demand, i.e. 36 m^3/hour at a pressure of 35 m.

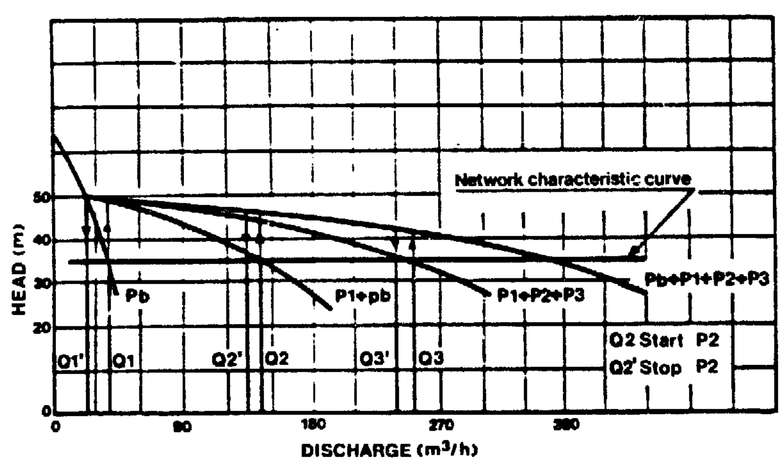


Figure 76 Pumps and network characteristic curves for flowmeter regulation

Referring to Figure 76, operation is as follows:

Pump No.	Range of flow (m^3/hr)	Pressure range (m)	Average discharge (m^3/hr)	Average head (m)
Pb	0 - 36	50 - 35	24.0	50.0
Pb + P1	36 - 144	49 - 35	100.8	41.5
Pb + P1 + P2	144 - 252	44 - 35	203	40.0
Pb + P1 + P2 + P3	252 - 360	42 - 35	309	38.5

It can be seen from the table above that flowmeter regulation requires smaller heads than would be the case with an air pressure vessel regulation.

5. DESIGN AND OPTIMIZATION TECHNIQUES OF OPEN CHANNEL DISTRIBUTION NETWORKS

5.1 INTRODUCTION

This chapter deals with the layout of open channel irrigation networks from the source of the water resource to the farm gate. The theory of open channel hydraulics is applied to the design of conveyors and distributors whether lined or unlined, carrying sediment-laden or clear water.

Methods which can be used to design stable channels in erodible soils whilst transporting bed load diverted with the water at the headworks are described in some detail. Great care is required when designing such regime canals and the advice of engineers having experience in this specialized field should be sought wherever possible before undertaking the actual construction of costly networks.

Siphon, tunnel, flume, drop structures, stilling basin and chute designs are proposed in this chapter. More information concerning these will be found in "Small Hydraulic Structures", FAO (1975).

5.2 DESIGN AND OPTIMIZATION OF CONVEYORS

5.2.1 Open Channel Conveyors

i. Definition

Open channel conveyors carry flow under the action of gravity from a source to the project area and provide the water supply for the irrigation distribution system. The conveyor usually runs normal to the slope of the project lands and along the upper contours or on ridges so as to command as much of the project area as feasible. If the elevation of the source, topography or other consideration makes it impracticable to command the entire project area from the conveyor, then lands above the conveyor can be served by a parallel branch supplied by pumping water from the conveyor.

A canal, lined or unlined, forms the principal component of an open channel conveyor. Free flow tunnels may be required to negotiate topographic obstructions which cannot feasibly be crossed by an open cut and bench flumes may be required along hillsides which are too steep for a feasible cut and fill canal section. Structures common to open channel conveyors include: inverted siphons or elevated flumes for crossing streams or cross drainages which either cannot pass under the canal through culverts at fill sections or pass over the canal in overchutes at cut sections; drops or fall structures, chutes and stilling basins or other types of energy dissipators at drops in grade; pumping plants if a higher level branch is to be supplied from the conveyor; bifurcation structures if the canal is split into branches; checks or regulators, either as separate structures or in combination with other structures at strategic points for water level control; turnouts or offtake structures to supply distributary or lateral canals; wasteways at strategic locations. Other structures which may be required include measuring flumes or rating sections, settling basins or sediment ejectors, bridges or culverts for road or railroad crossings, and crossing structures for animal or pedestrian use.

ii. Characteristics and general requirements

Capacity

Preliminary values for the required discharge capacities at the head and critical intermediate points of the conveyor need to be determined for purposes of field location and preliminary design. Preliminary values based on an increase of thirty percent of the flow required at the farm level to meet the peak irrigation demand are usually sufficient for purposes of layout and preliminary design of the canal system. Adjustments are made to the required discharge capacities as the canal location becomes established, the actual irrigable area under command of the conveyor becomes firm, and field data becomes available for a better evaluation of probable seepage losses in the canal. The final adjusted capacity should be based on the flow needed to meet the network conveyance losses and farm irrigation requirements during the cycle of peak irrigation demand. The cycle can have a duration of one week, 10 days, or some intermediate period that best reflects the scheduled interval between irrigations to meet the crop water requirements during the peak demand period.

The required discharge capacity at the various points of the conveyor are determined by taking into account the peak flows at the farm level and the irrigated area commanded by the conveyor at the location under consideration and adding to those computed discharges the appropriate allowances to account for conveyance losses and operation wastes in the conveyor downstream of the point under consideration.

EXAMPLE 12 - DETERMINATION OF CONVEYOR CAPACITY

Irrigated area commanded by conveyor		20 000 ha
Peak period demand at farm level		1.2 l/s/ha
Allowance for conveyance losses and operational waste:		
distribution canals		15%
conveyor		12%
Discharge components		
Peak period farm level demand	20 000 ha x 1.2 l/s/ha =	24 m^3/s
Distribution canal losses	0.15 x 24 m^3/s =	3.8 m^3/s
Conveyor losses	0.12 (24 + 3.6) =	3.3 m^3/s
Required capacity of conveyor	=	30.9 m^3/s
Peak period demand at headworks	=	1.5 l/s/ha

Losses

The conveyance losses include seepage losses, evaporation losses from the canal water surface and operational wastes, and are expressed in terms of cubic metres per day per square metre of wetted ares, or millimetres per day. Seepage rates may vary from 30mm/day for lined canals to 20 times that value or more for unlined canals in sands or gravels. Seepage losses in unlined canals are estimated by dividing the canal into reaches of similar seepage rates on the basis of judgement, taking into account similarity of soil conditions and results of seepage tests. The loss of flow in each reach is computed as follows:

$$Q_s = q_s PL/86\ 400$$

where: Q_s = flow lost to seepage in canal reach (m^3/s)
P = wetted perimeter (m)
L = length of canal reach (km)
q_s = rate of infiltration (mm/day = l/m^2/day)

The following values of q_s, based on US Bureau of Reclamation data for unlined canals, give an indication of approximate seepage rates in various types of soils. They are helpful, when used in conjunction with soil boring data and field seepage tests, in assessing the seepage loss that is likely to occur in a canal reach.

Table 21 INFILTRATION RATES IN UNLINED CANALS

Type of soil	q_s (mm/day)
Cemented gravel and hard pan with sandy loam	100
Clay and clayey loam	120
Sandy loam	200
Volcanic ash	210
Volcanic ash with sand	300
Sand and volcanic ash or clay	360
Sandy soil with rock	510
Sandy and gravelly soil	670

Evaporation losses are usually small in comparison to seepage losses. The loss by evaporation is computed as follows:

$$Q_E = ETL/86\ 400$$

where: Q_E = evaporation loss (m^3/s)
E = evaporation rate (mm/day)
T = width of canal water surface (m)
L = length of canal reach (km)

Operational wastes result during the peak demand period from changes in gate settings and the practical inability to balance the flow and demand perfectly at all points in a long canal system. A minimum allowance of two percent should be added to the canal design discharge to allow for losses due to operational waste.

Layout

The selection of a route for the conveyor is an important element of the planning and preliminary design phases. Usually the potential irrigable area has been studied and defined and the source or sources of supply identified beforehand. The process involves selection of the best route among alternatives between the source and the irrigable area from the standpoint of technical feasibility, cost and irrigable area that can be commanded within the overall framework of the proposed project development and project economics. Normally this selection is made during the planning and feasibility study phases of a project.

The selection process usually involves studies of topographic maps and aerial photos, if available, to identify possible routes. Field reconnaissances and preliminary surveys of the identified routes are then made and preliminary design and cost estimates are prepared to select the most favourable route for the conveyor. Boreholes and test pits are made at intervals to identify the nature of the material to be crossed and infiltration tests are carried out to appraise the seepage potential and need for canal lining.

Normally after a project is authorized for construction, additional field surveys and investigations are made early in the design phase, to obtain data for the design of the conveyor. These surveys include a survey of the centreline profile, strip topography along the canal and site topography for major structures. The field investigations usually include boreholes or test pits at 100 to 200 metre intervals along the canal alignment, foundation investigations at major structure sites, soil sampling and tests, investigations of sources for sand, gravel, stone, and fill materials for construction of the conveyor. Procedures for field investigations and sampling and for field infiltrometer tests for canal seepage potential are given in field manuals such as the Earth Manual published by the US Bureau of Reclamation.

Curvature

The allowable curvature in earth canals is affected by the nature of the soil, the flow velocity and the capacity of the canal. Canals constructed in alluvium consisting mainly of silts and fine sands are susceptible to erosion at bends and require gentle curves. The following minimum radii of curvature indicated in Table 22 should be adhered to for earth canals constructed in silt or fine sand:

Table 22 MINIMUM RADIUS OF CANAL BENDS

Capacity (m^3/s)	Minimum radius (m)
over 90	1500
30 to 90	1000
15 to 30	600
5 to 15	300

The minimum radius for earth canals constructed in less erosive soils should not be less than 5 to 8 times the water width in the canal. The lower figure can be used for the design of canals conveying 5 m^3/s or less. The following range is suggested: canals conveying 5 to 15 m^3/s, 6 times the water width; 15 to 50 m^3/s, 7 times the water width; over 50 m^3/s, 8 times the water width.

If the canal is concrete lined or if a flume is used, a minimum radius of curvature of 3 times the water width should be used to avoid bend losses. If the canal is lined with brick, masonry, rip-rap, stone pitching or other hard material resistant to erosion it should also have a minimum radius of 3 times the water width.

Cross drainage

Where a conveyor cannot be located on a ridge it is designed to follow the approximate contour of the high part of the irrigation project and to cut across the streams and natural drainages situated along its axis. The flood flows of these natural drainages have to pass safely under the canal in culverts or over the canal in overchutes. If this is not feasible the canal is carried over the stream in an elevated flume or under the canal in an inverted siphon. Where topography and right-of-way permit, it is often expedient to combine drainages by a system of diversion drains and dykes. Smaller drainage may be taken into the conveyor through drain inlet structures if it has excess capacity during the runoff season and a wasteway is provided. Drain inlets should be avoided for steep drainages or erosive channels that are likely to carry sediment and debris into the canal. Drain inlets are best adapted for draining flat areas lying above the canal which flood temporarily and create pondage.

The determination of the design flood discharges for these various crossings is an important element of the early design phase. Canal cross drainage structures should be designed to handle a 50-year flood safely. In the case of large canals, where a breach could result in extensive damage to the project area below the canal, the cross drainage structures should be designed to pass a 100-year flood without overtopping the canal banks. In cases where the cross drainage is intercepted and diverted above the conveyor by a separate extensive system of channels and protective dykes, such systems should also be designed to handle a 100-year flood without overtopping.

It is usually necessary to develop cross drainage design floods by analytical methods based on watershed characteristics and frequency analyses of rainfall intensity. Such analyses should be supplemented by estimates of historic floods, based on field observations of high water marks in the channel, bridges, ford crossings or information from residents in the area. The velocity can be estimated by the Manning formula from average channel slope measurements, the hydraulic radius, and suitable values of the roughness coefficient for natural streams in full flood. Representative valies of Manning's roughness coefficient for natural streams are listed in Table 23.

The flood discharges thus determined can be used for correlation and to substantiate the flood flow calculated analytically. The design floods of the individual watersheds can be compared on the basis of the square root or other power of the drainage areas by plotting the results on logarithmic paper.

The area and shape of the watershed, length of the channel,

Table 23 RUGOSITY OF NATURAL STREAM BEDS

Channel characteristics	n
Straight, clean, free of shoals and deep pools	0.030
Same as above with stones and weeds	0.035
Clean, winding with pools and shoals	0.040
Same as above with stones and weeds	0.045
Mountain stream with gravel and cobbles and few boulders	0.040
Mountain stream with cobbles and large boulders	0.050

difference in elevation from the top of the watershed to the crossing and the nature of the vegetal cover can be determined from aerial photographs and topographic maps. Rainfall intensity is an important factor. Annual maximum daily or storm rainfall depths at rainfall stations in the general area of the project are ranked and plotted on probability paper to determine the probable intensities for recurrence intervals of up to 100 years. Rainfall intensity curves can then be prepared (Fig. 6.1, FAO (1981)).

The flood discharge may be calculated by any of the empirical formulae that abound in the literature, amongst these the McMath formula which is suited to very small watersheds, is written:

$$Q = CIS^{\frac{1}{5}} A^{\frac{4}{5}} /11 \qquad (78)$$

where:
- Q = discharge in m^3/s
- I = rainfall intensity during an interval equal to the time of concentration and for the selected frequency (mm/hour)
- S = fall of main channel between the farthest contributing point and the canal crossing (m/km)
- A = area of watershed (km^2)
- C = runoff coefficient (varies from 0.2 to 0.75)

The value of the runoff coefficient (C) depends on vegetation, soils and topography. The runoff coefficient increases as the vegetation becomes sparser, the soil becomes heavier and the slope becomes steeper. Table 24 indicates drainage basin factors which when summated give the value of the coefficient (C).

Table 24 DRAINAGE BASIN FACTORS TO ASSESS RUNOFF COEFFICIENT

Runoff condition	Vegetation		Soils		Topography	
Low	0.08	(well grassed)	0.08	(sandy)	0.04	(flat)
Moderate	0.12	(good coverage)	0.12	(light)	0.06	(gently sloping)
Average	0.16	(good to fair)	0.16	(medium)	0.08	(sloping to hilly)
High	0.22	(fair to sparse)	0.22	(heavy)	0.11	(hilly to steep)
Extreme	0.30	(sparse to bare)	0.30	(heavy to rock)	0.15	(steep)

The following nomograph (Fig. 77) used by the US Bureau of Reclamation is convenient for estimating concentration time.

A cross drainage map should be prepared which shows the watershed boundaries and the computed design floods for each drainage area. This map is used when defining the dimensions of the cross drainage structures.

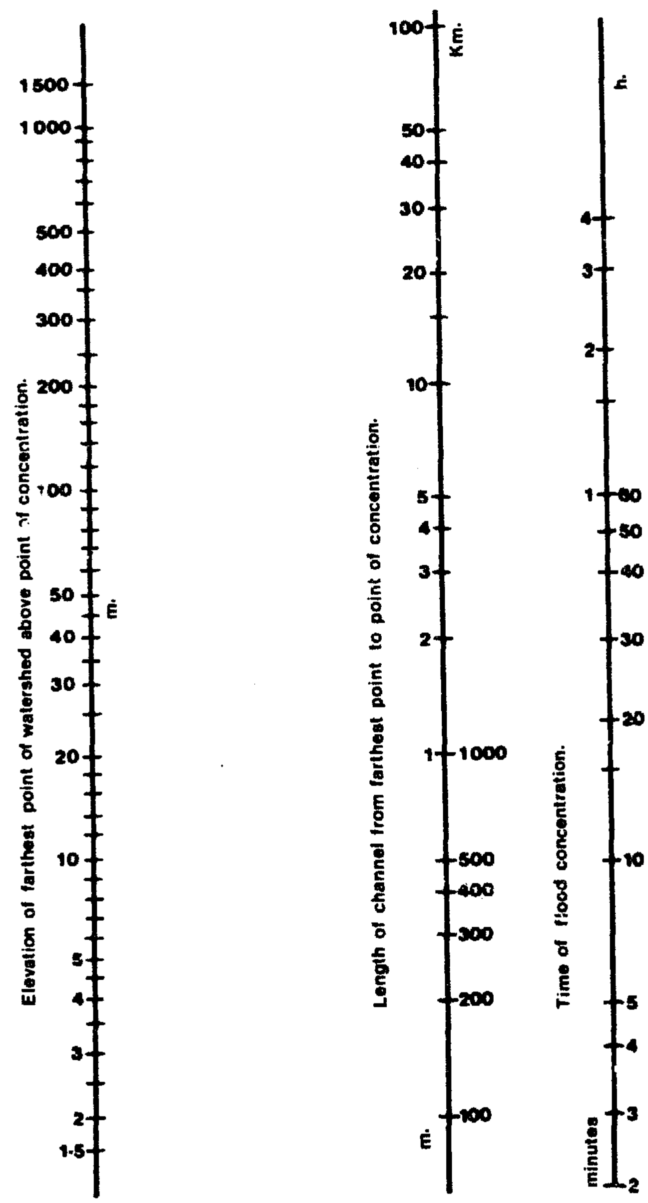


Figure 77 Nomograph for estimating time of flood concentration

EXAMPLE 13 - EVALUATION OF DESIGN FLOOD

Making use of the McMath formula (equation 78), calculate the design flood of a watershed having an area of 10 km² of medium soils with fair to sparse vegetation on sloping to hilly relief. The 100-year rainfall intensity is assumed to attain 60 mm per hour during the time of concentration whereas the slope has a value of 10 m/km.

According to Table 24:

$$C = 0.16 + 0.22 + 0.08 = 0.46$$

then with I = 60 mm/hour, s = 10 m/km, A = 10 km²

$$Q_{100} = CIS^{\frac{1}{5}} A^{\frac{4}{5}} /11 = 25 \ m^3/s$$

Freeboard

Conveyors require freeboard between the canal water surface and the top of the banks to contain water level fluctuations caused by surges, check structures and stormwater inflow. The following heights of lining and bank above the canal design water surface should provide adequate freeboard.

Table 25 FREEBOARD AND HEIGHT OF CANAL BANKS

Design capacity (m^3/s)	Freeboard, hard surface or membrane lining (cm)	Freeboard, earth lining (cm)	Height of bank above water surface (cm)
1	15	15	40
2	20	15	55
5	25	15	70
10	35	20	85
50	60	35	120
100	70	45	140
250	80	55	165

Freeboards suggested by the US Bureau of Reclamation (1974) are indicated in Figure 78.

Canal banks and berms

Conveyors require an operation and maintenance road for access to the structures and the canals. The road should have a minimum width of 6 metres and be located on the right bank of the canal. If the conveyor follows the crest of a ridge and has offtakes to distribution canals, it requires a road on both sides. If there is a rainy season, the roadway should have gravel or other surfacing. The canal road in many developing areas becomes a main road for the project area. In such cases the roadway should be made at least 7.5 metres wide and provided with a suitable surfacing. A canal bank, or berm in cut sections, having a minimum width of 3 metres should be provided on the other bank.

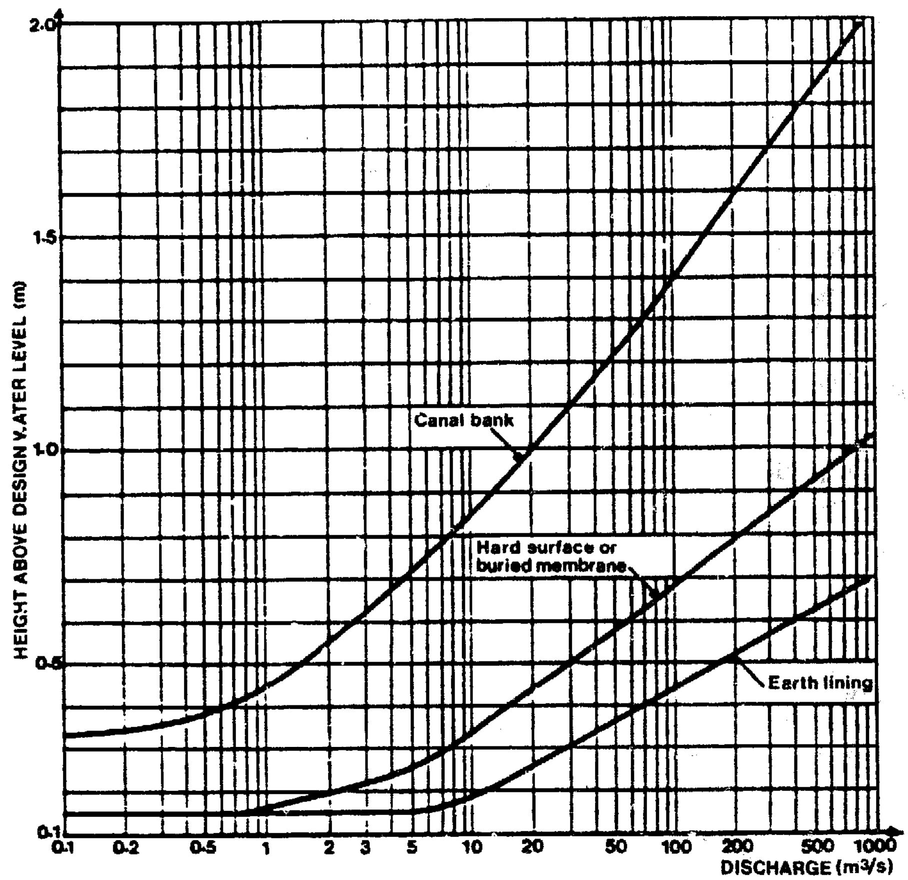


Figure 78 Freeboard for hard surface, buried membrane and earth linings

On large wide canals a 6-metre wide bank, or berm, should be provided for maintenance on the embankment when it does not carry a road.

Need for lining

An early decision has to be made as to whether the conveyor is a concrete lined canal or basically an earth canal, with local lining in areas where seepage is a problem. Frequently a concrete lined canal is adopted for a project as a policy matter or after consideration of a number of reasons other than seepage reduction. Some of these reasons could be the saving in earthwork cost because higher permissible velocities and the resulting reduced section helps defray a good portion of the concrete lining cost; the water supply may be limited and require transportation over a long distance from the source to the project and the smaller section with low seepage rates is desired for efficient utilization; reduced maintenance and better weed control may be an important consideration; also environmental considerations such as better control of snails that spread schistosomiasis (bilharzia) could be an important factor. Outside the above mentioned considerations in favour of concrete lining, and to a lesser degree for other types of hard surface linings such as brick, shotcrete, soil cement or bituminous asphalt paving, the decision as to whether or not to line a canal is economic. The economic justification is based on the present value of the water lost to seepage and the additional cost of the lined canal. The lining might be justified only in high seepage areas and an earth lining might be the most economical way of reducing the loss of water.

The question of whether the conveyor is to have a concrete lining (or other hard surface lining), have an earth lining throughout or only in selected areas, or be unlined, should be resolved in the planning or feasibility phases of the project. The type of canal to be used for the conveyor must be established prior to final location and hydraulic design because the hydraulic parameters of a concrete lined canal or other hard surface lined canal differ from those of an unlined canal or earth lined canal. For more details reference is made to FAO (1977).

iii. Design of the canal

a. General

The design of the canal requires prior field work, preliminary design and planning as discussed earlier. The cost of alternatives such as lined and unlined canal, or various field locations can be based on the preliminary design studies. The route is established after completion of field surveys and exploratory work to obtain data on soils to be crossed, suitability of materials for fills, and identification of potential problem areas for seepage losses or stability.

The design of the canal must be compatible with particular local constraints such as the need to transport sediment-laden or clear water, depending upon whether the source is a river or stream or a clear water source such as a reservoir or well; the canal may be in material sensitive to erosion such as silts and fine sands; the canal may be in gravels, or it may be lined with concrete or other hard surface material.

b. Earth canals in silts and sands carrying bed loads

Canals constructed in silts and sands are sensitive to bank erosion. If the canal is supplied by direct diversion or pumping from a river or stream carrying sediment, a portion of the sediment will be diverted to the canal even though structures are incorporated in the headworks to exclude the coarser material of the bed load. If the sediment inflow exceeds the sediment carrying capacity of the canal, deposition will occur, and the discharge capacity of the canal will be seriously reduced in a matter of a few years. On the other hand if the sediment carrying capacity of the canal exceeds sediment inflow the canal banks will be attacked, meandering will start and costly bank protection will be needed to stabilize the canal. The design objective is to select a slope and geometric dimensions such that during an annual cycle the sediment inflow to the canal is equal to the sediment flowing out of the canal. Since the sediment load carried by a river varies over the year, the sediment concentration of the water entering the canal will also vary with resulting periods of bed deposition and periods of scour. Over a period of one year these should largely offset each other. Canals which are required to operate in such conditions are generally referred to as regime canals.

The Lacey regime equations

On the basis of work carried out in India, Lacey (1929; 1934) developed a series of equations for regime canals which relate channel dimensions and slope to a silt or sediment factor (f) such that:

$$f = 1.76\ d_m^{\frac{1}{2}} \qquad (79)$$

where: d_m = mean diameter of the sediment (mm)

The sediment factor (f) may be refined when observations can be made of existing canals in the neighbourhood which convey a similar sediment load and which are known from experience to have stable channels. These observations are used to determine:

$$f_{(RS)} = 285\ R^{\frac{1}{3}}\ s^{\frac{2}{3}} \qquad (80)$$

$$f_{(VR)} = 2.46\ V^2/R \qquad (81)$$

from which $$f = \left[f_{(RS)}\ f_{(VR)}\right]^{\frac{1}{2}} \qquad (82)$$

where: R = hydraulic radius (m)
s = channel slope
V = velocity (m/s)

Then the three basic equations from Lacey's first two papers may be written as:

$$P = 4.84\ Q^{\frac{1}{2}} \qquad (83)$$

$$R = 0.473\ (Q/f)^{\frac{1}{3}} \qquad (84)$$

and $$s = f^{\frac{5}{3}}\ /3311\ Q^{\frac{1}{6}} \qquad (85)$$

where P = wetted perimeter (m).

When the sediment factor (f) cannot be determined by reference to existing canals its value is established from sediment samples and the use of equation (79) above. In this case it is advisable to increase the value of the sediment factor by about 15 percent for the first 10 km of canal below the headworks, 10 percent for the next 10 km, and 5 percent for the following 10 km.

If the canal is to be subjected to an inflow of wind-blown sand, the sediment factor (f) should be increased by some ten percent in the exposed section and downstream. Additional gradient will then be available to cope with sediment transport should the value of the sediment factor determined by sampling be on the low side. Should the gradient in later years prove to be excessive, it can be corrected by adjusting the check or head regulator structures.

Sediment samples should be taken from the river or stream at the headworks during different seasons of the year to determine the mean diameter of the sediment. The headworks should be designed to exclude the bed load from the canal (FAO 1982).

EXAMPLE 14 - INCIDENCE OF SEDIMENT FACTOR ON SLOPE

Since $s = f^{\frac{5}{3}} / 3311Q^{\frac{1}{6}}$ (85)

then the slope will vary with the sediment factor and discharge

sediment factor f	discharge Q (m^3/s) 100	20
0.80	0.0000966	0.000126
0.92	0.0001220	0.000159

This means that if checks are located at 10 km intervals, they will require height adjustments of only 0.25 m for the 100 m^3/s canal and 0.33 m for the 20^3 m/s canal to correct the gradient for a change of sediment factor (f) from 0.92 to 0.80. Provision for small adjustments can readily be incorporated in the design of the checks or head regulator.

In the natural conditions prevailing in the region where the Lacey equations were established, the natural side slopes tend to be steep with a value of ½:1 (z = 0.5) and the bottom width equals the difference between the surface width of the channel and the water depth of the channel (b = T-y).

After selection of the bottom width, the canal is usually excavated with 1:1 side slopes (z = 1) below the water surface. A berm is provided at the level of the water surface. The width of this berm should be equal to one and a half to twice the depth of water. The berm allows for some widening of the channel as it develops, without affecting the stability of the canal bank.

It is of the utmost importance to remember that the Lacey equations were developed empirically in the specific conditions of India where they have been widely applied with success. Great care should be taken to avoid using this method in different conditions of sediment nature and concentration. In fact, to quote a statement reported by Blench (1957), "if all Lacey channels are regime channels, all regime channels are not Lacey channels".

In the prototype conditions for which the Lacey equations were developed it would appear that "channel sides carry little or no sediment and do not contribute to either rugosity or regime in the same manner as does the bed with its non-cohesive material moving in dune formation" (Blench 1957). The sediment transported is coarse and the total load can be defined as medium. In general, the observations on which the Lacey regime equations are based were made in wide and relatively shallow channels such that the value of the powers suggested do not vary if mean width and mean depth are used in place of wetted perimeter and hydraulic radius.

Empirical rules used in USSR

The code of rules established for the construction of stable canals in the USSR (Guidrotekn 1936) relates the basic velocity to the grain size of non-cohesive bed material.

EXAMPLE 15 - DESIGN OF LACEY REGIME CHANNEL

Determine the cross-section and slope of a channel to convey 100 m³/s in material having a mean grain size (d_m) of 0.21 mm.

Applying Eq. (79) the sediment factor is

$$f = 1.76\, d_m^{\frac{1}{2}} = 0.81$$

increasing the value of the sediment factor (f) by 15% for the initial reach of 10 km, then f = 0.93.

Applying Eq. (83) and (84) to determine the cross section

$$P = 4.84\, Q^{\frac{1}{2}} = 48.40 \text{ m} \qquad \text{and} \qquad R = 0.473(Q/f)^{\frac{1}{3}} = 2.25 \text{ m}$$

hence A = PR = 108.9 m² and V = Q/A = 0.92 m/s.

Since $P = b + 2y(1+z^2)^{\frac{1}{2}}$ and $A = (b+zy)y$

if z = 0.5 for a Lacey channel, then the cross-section has a depth (y) = 2.48 m and bottom width (b) = 42.85 m.

Inserting V = 0.92 m/s in Eq. (81) as a check

$$f_{(VR)} = 2.46V^2/R = 0.925$$

The slope (s) is determined from Eq. (85)

$$s = f^{\frac{5}{3}} / 3311Q^{\frac{1}{6}} = 0.000124$$

Inserting s = 0.000124 in Eq. (80) as a check

$$f_{(RS)} = 285\, R^{\frac{1}{3}}\, s^{\frac{2}{3}} = 0.930$$

Substituting in Eq. (82)

$$f = \left[f_{(RS)}\, f_{(VR)}\right]^{\frac{1}{2}} = 0.93$$

which corresponds to the selected value.

Rearranging the Manning equation the value of the roughness coefficient (n) is found to be

$$n = R^{\frac{2}{3}} \cdot s^{\frac{1}{2}}/V = 0.021$$

A canal excavated with 1:1 side slopes (z=1), a bottom width (b) = 42 m and a flow depth (y) = 2.5 m would have a wetted perimeter (P) = 49 m and a hydraulic radius (R) = 2.27 m which satisfy the above determined regime conditions.

With a slope s = 0.000124 and n = 0.021 the flow velocity (V) would be

$$V = R^{\frac{2}{3}}\, s^{\frac{1}{2}}/n = 0.92 \text{ m/s} \qquad \text{and} \qquad Q = VA = VPR = 102 \text{ m}^3/\text{s}$$

which value is within 2 percent of the design discharge.

Assuming that in the second reach (km 10 to km 20) the value of the sediment factor (f) drops to 0.81 then as above

$$R = 0.473(Q/f)^{\frac{1}{3}} = 2.35 \text{ m}$$

$$s = f^{\frac{5}{3}} / 3311Q^{\frac{1}{6}} = 0.0000987$$

In order to satisfy the regime condition the hydraulic radius of the second reach must be increased to R = 2.35 m which entails increasing the depth to y = 2.6 m for a constant bottom width of 42 m whilst the slope is reduced to 0.0000987.

In these conditions n = 0.020 and A = 115.96 m² whereas Q = VA = 102 m³/s, which satisfies the design discharge.

Other reaches further downstream are defined in the same manner, allowing for further reductions of the value of the sediment factor and for discharge diversions.

Table 26 MAXIMUM PERMISSIBLE MEAN VELOCITY RELATED TO GRAIN SIZE

Bed material		d (mm)	V (m/s)	Bed material		d (mm)	V (m/s)
Silt		0.005	0.15	Cobbles	small	15	1.2
		0.05	0.20		medium	25	1.4
Sand	fine	0.25	0.30		large	40	1.8
	medium	1.00	0.55			75	2.4
	coarse	2.50	0.65			100	2.7
Gravel	fine	5.00	0.80		very large	150	3.3
	medium	10.00	1.00			200	3.9
	coarse	15.00	1.20				

The above permissible velocities are adjusted according to the depth of flow by applying the following correction factors.

Table 27 CORRECTION FACTOR FOR MEAN VELOCITY (TABLE 26) AS A FUNCTION OF DEPTH

Mean depth (m)	0.3	0 6	1.0	1.5	2.0	2.5	3.0
Correction factor	0.8	0.9	1.0	1.1	1.15	1.2	1.25

The Blench regime equations

To overcome the limitations of the Lacey equations Blench proposes three dimensionally satisfactory equations which account for the different hydraulic behaviour of the bed and the side-slopes (Blench 1957). For practical purposes these can be written:

$$B = (F_b\ Q/F_s)^{\frac{1}{2}} \qquad (86)$$

$$y = (F_s\ Q/F_b^{2})^{\frac{1}{3}} \qquad (87)$$

$$s = \frac{F_b^{\frac{5}{6}}\ F_s^{\frac{1}{12}}\ \nu^{\frac{1}{4}}}{3.63(1+aC)gQ^{\frac{1}{6}}} \qquad (88)$$

where: F_b = bottom factor = V^2/y (89)
F_s = side factor = V^3/B (90)

with: B = A/y or width at half depth
ν = kinematic viscosity
g = acceleration of gravity
a = 1/233
C = concentration of dry bed load in parts per 100 000 by weight

For practical artificial channels the term aC is negligible unless the load is abnormal and equation (88) may be written (in metric units) for water at 20°C:

$$s = F_b^{\frac{5}{6}}\ F_s^{\frac{1}{12}}\ /1147Q^{\frac{1}{6}} \qquad (88a)$$

The factors F_b and F_s can be evaluated from existing canals known to be in regime, operating under project conditions of sediment load by repeated measurement of width, depth and discharge in several cross-sections.

In the absence of direct evaluations from field observations, Blench recommends working side factors of 0.01, 0.02 and 0.03 for loams of very slight, medium and high cohesiveness with higher values for indurated clays and shales initially, reducing to 0.03 or less after some years of drying and wetting.

No rules for defining the bed factor from soil characteristics are at present available and only experience can guide the choice if actual canal observations are not possible. For very small charges ($F_b = F_{bo}$) the following equation can be used:

$$F_{bo} = 0.58 d_m^{\frac{1}{2}} \qquad (91)$$

where d_m = mean diameter of sediment (mm).

If the bed load is sufficient to form dunes, the bed factor, for material up to and including fine gravel, can be written:

$$F_b = F_{bo}(1+0.12C) \qquad (92)$$

where C is the bed load concentration of equation (12).

The general behaviour of regime channels, illustrated in Table 27, is that according to Blench:

- large channels run flat; small channels run steep;
- large channels are relatively shallow; small channels are relatively deep;
- for the same bed and side materials respectively, large channels run fast whilst small channels run slow.

EXAMPLE 16 - DESIGN OF BLENCH REGIME CHANNELS

Determine the cross-section dimensions and slope of regime channels to convey discharges of 300, 30, 3 and 0.3 m^3/s with a bed factor of 0.3 and a side factor of 0.02 for water at 20°C. Making use of equations (60), (61) and (62a) the following dimensions are found:

Table 28 REGIME CANAL DIMENSIONS (F_b = 0.3, F_s = 0.02, t = 20°C)

Description	Discharge (Q) m^3/s	Slope m/km	Breadth (B) m	Depth (y) m	Velocity (V) m/s	B/y
Main	300	0.09	67	4.0	1.08	16.7
Distributary	30	0.13	21	1.9	0.74	11.1
Minor	3	0.19	6.7	0.87	0.50	7.7
Sub-minor	0.3	0.28	2.1	0.38	0.40	5.3

Conclusion

The above discussion of the design of regime channels gives a brief overview of a small part of the methodology available to the designer who is encouraged to further reading, notably of Blench (1957) and Leliavsky (1955), on a subject whose complexity is well described by the following quotation (Chow 1959) "The behaviour of flow in an erodible channel is influenced by so many physical factors and by field conditions so complex and uncertain that precise design of such channels at the present stage of knowledge is beyond the range of theory".

c. Earth canals in silts and sands carrying relatively clear water

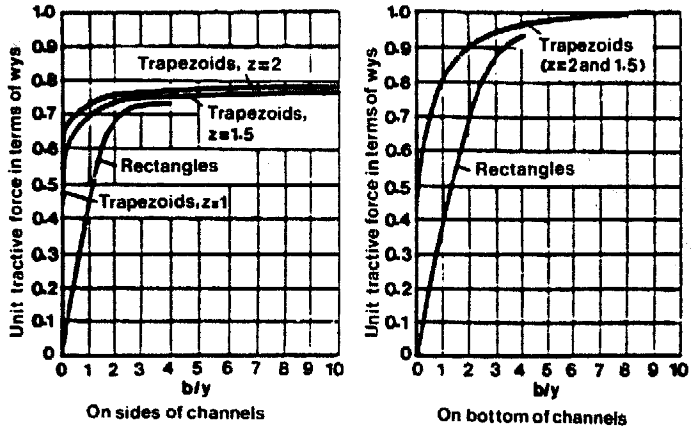
If the source of supply is a reservoir, well field or the middle or lower reaches of a canal, the water may have some fine sediment in near-colloidal suspension but will be relatively clear and practically free of sediment load. Water that is relatively clear has a natural affinity for sediment particles. Silts and fine sands are particularly sensitive to erosion by flowing water and such soils are frequently encountered in desert regions.

Earth canals carrying relatively clear water and designed for construction in soils consisting of silt and fine sand require careful selection of geometry, velocity and slope to avoid scour of the canal prism. Earth canals under those conditions should be designed by the tractive force method. Empirical formulae, such as the Kennedy formula as modified by the US Bureau of Reclamation, are also useful for preliminary design.

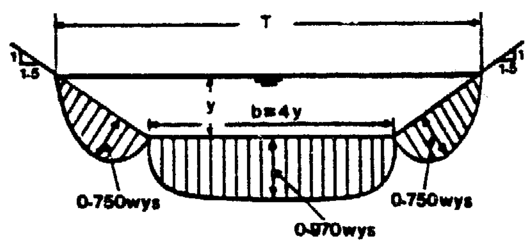
The tractive force per unit area is obtained by resolution of forces along the wetted boundary of the canal, assuming that the force component of the weight of the water acting in the direction of the canal slope is balanced by the shear resistance of the water acting on the wetted perimeter. The tractive force is not uniform along the wetted perimeter unless the channel is very wide or the side slopes are shaped along a cosine curve dependent on the angle of repose of the soil. The soil particles on the sloping sides of trapezoidal canals are subject to tractive force from the flowing water as well as gravity force which tends to cause the particles to roll down the sides.

Unit Tractive Force: The unit tractive force is a multiple of a distribution factor and w y s, where w is the unit weight of the water, y is the depth of water on the bed, and s is the slope in the direction of flow. The distribution factor is a function of the side slopes and the ratio of the bottom width of the canal to the water depth. The distribution factor applied to the sides of the canal has a smaller value than the factor applied to the bottom. Typical distribution factors in terms of w y s for the sides and bottom width over water depth ratios are shown in Figure 79. For the channel sections ordinarily used for earth canals the tractive force on the bed, (τ_b), is approximately equal to w y s and on the sides (τ_s) 0.75 to 0.78 w y s depending on the side slopes.

Permissible Tractive Force: The permissible tractive force is the maximum unit tractive force on the canal bed which will not cause serious erosion of the bed material. Figure 80 shows



MAXIMUM UNIT TRACTIVE FORCES IN TERMS OF wys



DISTRIBUTION OF TRACTIVE FORCE IN
A TRAPEZOIDAL CHANNEL SECTION

Figure 79 Maximum unit tractive forces in terms of w y s

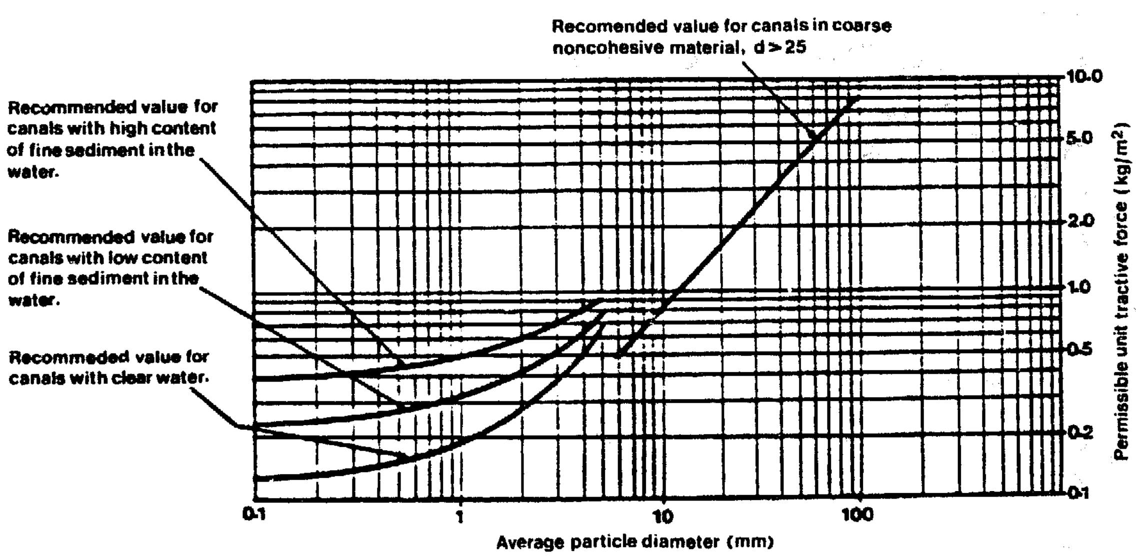


Figure 80 Permissible tractive force for canals in non-cohesive material

permissible unit tractive forces for canals in non-cohesive soils, recommended by the US Bureau of Reclamation. For fine non-cohesive material, smaller than 5 mm particle diameter, the median size or d_{50} particle size is used and for coarser non-cohesive materials, larger than 5 mm particle diameter, the d_{75} particle size is used, size for which 25 percent of the sample (by weight) is larger. Concentrations of suspended or colloidal type fine sediments of the order of 0.1 percent and 0.5 percent are suggested as being representative of low content and high content of fine sediment respectively.

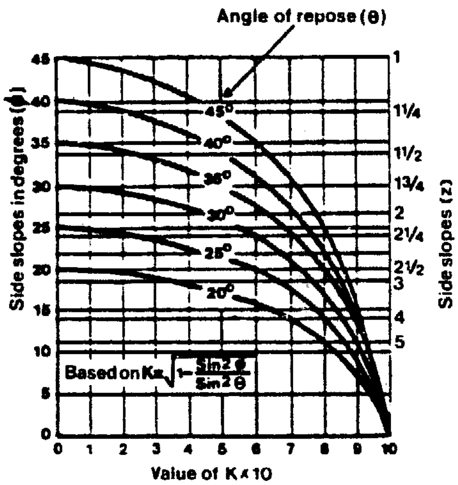


Figure 81 Critical shear stress on inclined slope

The ratio (K) of the tractive force on a sloping surface, τ_s, to the tractive force on a level surface, τ_b, is the tractive force ratio used for design. This ratio is known as the critical shear stress and is expressed as:

$$K = \frac{\tau_s}{\tau_b} = 1 - \frac{\sin^2 Ø}{\sin^2 \theta}^{\frac{1}{2}} \qquad (93)$$

where Ø is the angle of inclination of the sloping side and θ is the angle of repose of the soil. Values of K for various side slope inclinations and angles of repose will be found in Figure 81.

Figure 82 gives recommended values for angles of repose for coarse materials of 5 mm diameter particle size or larger.

<u>Permissible Tractive Force in Cohesive Soils</u>: The permissible tractive force in cohesive soils is based upon the compactness or voids ratio. In cohesive soils the rolling down effect is negligible and only the effect of the distribution of tractive forces (from Fig. 80) is taken into account for design purposes. Permissible unit tractive forces for canals in cohesive soils are indicated in Figure 83. The chart is entered with the void ratio determined from samples of the in-place cohesive soils.

<u>Slope</u>: The slope (s) is determined by rearranging the Manning formula:

$$s = V^2 n^2 / R^{\frac{4}{3}}$$

<u>Roughness coefficient</u>: Table 29 indicates recommended values of the Manning roughness coefficient.

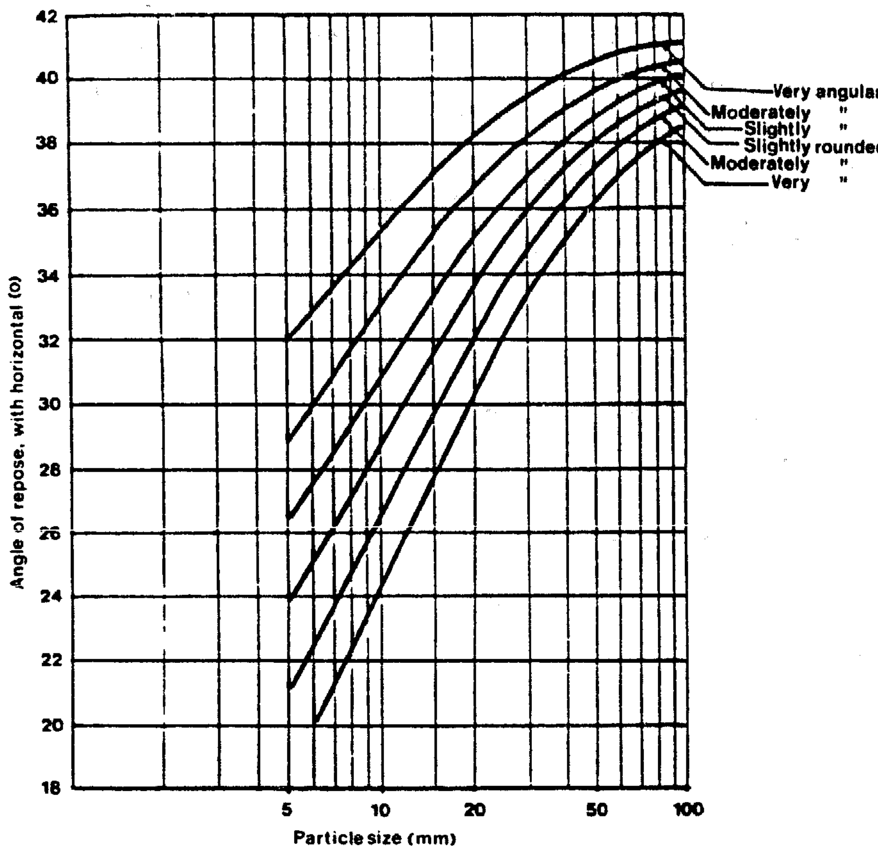


Figure 82 Angles of repose of non-cohesive material

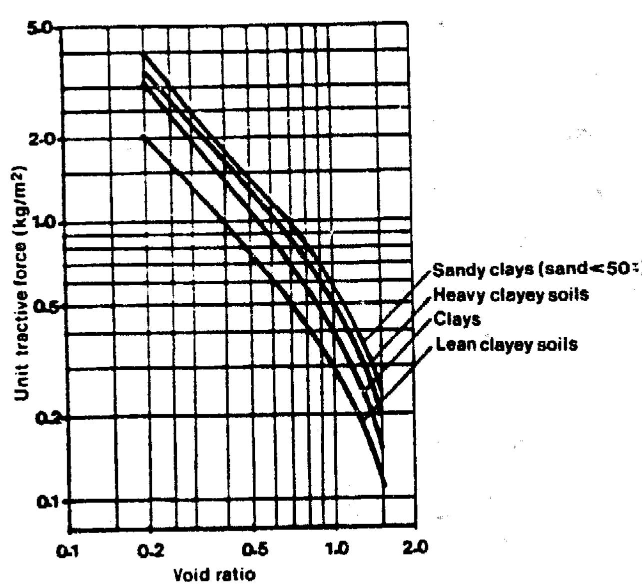


Figure 83 Permissible unit tractive forces for canals in non-cohesive material derived from USSR data on permissible velocities in Chow (1959)

EXAMPLE 17 - DESIGN OF EARTH CANAL MAKING USE OF TRACTIVE FORCE PRINCIPLE

Design an earth canal to carry 100 m^3/s. The material is non-cohesive fine sand with some silt, the supply source is a reservoir providing clear water.

Step 1 Conduct soil explorations and determine the grain size distribution of the material, the angle of repose and side slopes to be used for stability. For the purpose of this example assume that the average particle size (d_{50}) is 0.4 mm, the angle of repose 25°. The side slopes will be inclined at 3:1 for stability (z = 3).

Step 2 Use a value of Manning's n = 0.020 (Table 29), try a design depth (y) of 2.5 m. From Figure 79 the maximum unit tractive force (τ_b) on the bed is 0.97 w y s and as w = 1 it is approximately equal to 25 x 100 x s whereas on the sides (τ_s) it is 0.8 x 2.5 x 1000 x s (kg/m^2).

The permissible tractive force on the bed (τ_b) for a 0.4 mm particle size is 0.15 kg/m^2 (Fig. 80). The tractive force ratio (K) for a 25° angle of repose and z = 3 is given by Eq. (93):

$$K = \left(1 - \frac{\sin 18.4°}{\sin 25°}\right)^{\frac{1}{2}} = 0.66$$

(The value of K = 0.66 can also be read off Figure 81.)

The permissible tractive force on the inclined sides (τ_s) is therefore 0.66 x 0.15 = 0.10 kg/m^2.

Step 3 Determine the allowable slope (s) to avoid exceeding the permissible unit tractive force (τ).

For the sides: τ_s = 0.10 = 2.5 x 0.8 x 1000 s, s = 0.00005

For the bottom: τ_b = 0.15 = 2.5 x 1000 s, s = 0.00006

Step 4 Determine the bottom width (b) with Manning's n = 0.020, Q = 100 m^3/s, y = 2.5 m, z = 3 and s = 0.00005.

Since the area of the cross section is not yet known, the bottom width (b) is determined by trial and error.

Given y = 2.5 m, z = 3, n = 0.02 and s = 0.00005

from $$V = R^{\frac{2}{3}} s^{\frac{1}{2}}/n$$

and in a trapezoidal section $$R = \frac{(b + z y)y}{b + 2y(1+z^2)^{\frac{1}{2}}}$$

then with b = 60 m: Q = VA = 101.7 m^3/s

This value is within 2 percent of the desired discharge of 100 m^3/s.

Step 5 Compare above with a depth (y) of 3 m.

The permissible slope s = 0.10/(3 x 0.8 x 1000) = 0.000042 with b = 47 m:

$$Q = 101.5 \ m^3/s$$

Either section is acceptable but it should be noted that the deeper section has two advantages:

- it requires less right of way
- seepage loss is reduced since the wetted area is diminished by 10 m^2/metre length of canal.

The canal should have 1.5 m freeboard and 6 m wide bank on one side and 4 m wide bank on the other side, see Figure 84.

Table 29 RUGOSITY OF ERODIBLE CHANNELS

n = 0.020	for earth canals discharging 10 m^3/s or more, in fine material with clean banks and straight alignment; and canals discharging 80 m^3/s or more having a medium smooth bottom, average alignment, gravel or grass on the banks, silt deposits at both sides of the bed; or canals built in hardpan in good condition, in clay or lava ash soil
n = 0.0225	for earth canals conveying from 10 to 80 m^3/s under the conditions described under n = 0.020 for canals discharging 80 m^3/s or more
n = 0.025	for earth canals conveying less than 10 m^3/s; larger canals with moss and dense grass at the edges, and noticeable scattered cobbles
n = 0.0275	for cobble-bottom canals where cobbles are well graded and compact
n = 0.030	for canals with heavy growth of moss, irregular banks overhung with dense rootlets; bottom covered with large rock fragments or pitted with erosion

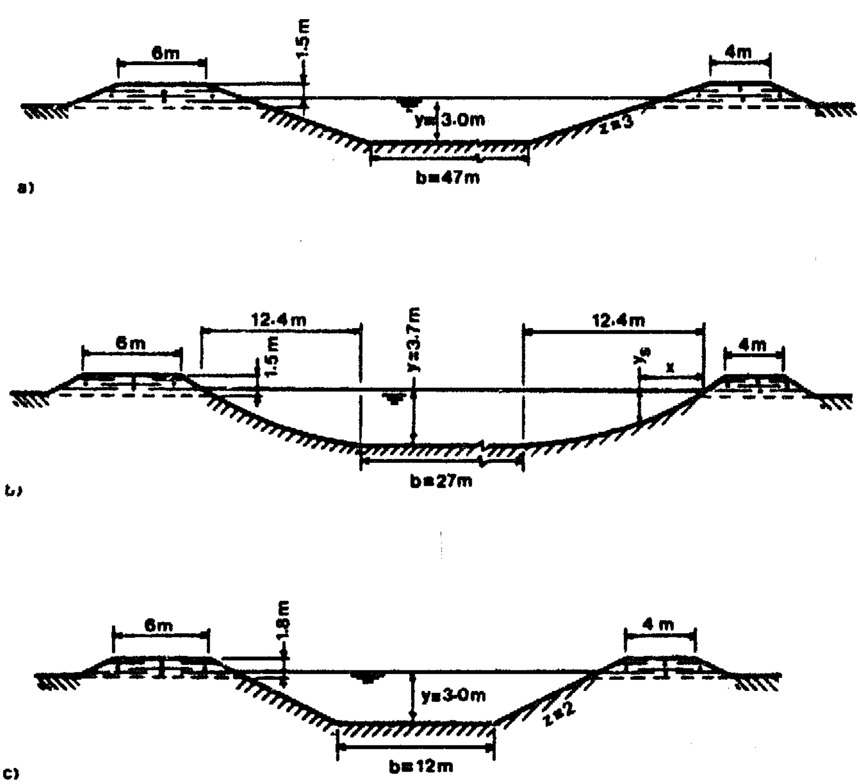


Figure 84 Typical canal sections for a design discharge Q = 100 m^3/s

EXAMPLE 18 - THE MODIFIED SIDE-SLOPE SECTION

Determine the shape of the modified side slope section to replace the trapezoidal section of the previous example.

As before: $\theta = 25$, $\tau_b = 0.15$ kg/m² on the bed and $s = 0.000042$

Step 1 The centre depth (y) is obtained by applying Eq. (95)

$$y = \tau_b/970\ s = 0.15/(970 \times 0.000042) = 3.68\ m$$

The shape of the section is determined by Eq. (68):

$$y_s = y \cos\left[\frac{\tan\theta}{y}\, x\right] = 3.68 \cos\left[\frac{\tan 25°}{3.68}\, x\right] = 3.68 \cos 0.127\, x$$

The top width (T) can be determined by solving the above equation for $y = 0$ noting that the angle of the cosine function is in radians and that x is equal to half of the top width.

When $\cos 0.127\, x = 0$, $0.127\, x = \pi/2$ and $x = 12.37$ m

The top width, $T = 2x = 24.7$ m.

Step 2 Applying Eq. (95), the mean velocity (V) is

$$V = [0.908-0.8 \tan\theta]\ y^{\frac{2}{3}}\ s^{\frac{1}{2}}/n = 0.41\ m/s$$

Applying equation (97), the water area $A = \dfrac{2.04\ y^2}{\tan\theta} = 59.2$ m²

hence $Q = VA = 0.41 \times 59.2 = 24.3$ m³/s

Step 3 Since 24.3 m³/s is less than the design discharge of 100 m³/s, a rectangular centre section must be added using Eq. (99).

$$T'' = \frac{n(Q'' - Q)}{y^{\frac{5}{3}}\ s^{\frac{1}{2}}} = 26.6\ m$$

The total top width is then 24.7 + 26.6 = 51.3 m

and the total area is 59.2 + (26.6 x 3.68) = 157.1 m².

Step 4 The side slope shape is determined by assuming values for y_s and solving for x in the equation

$$y_s = 3.68 \cos 0.127\, x.$$

For $y_s = 1$ m, $\cos 0.127\, x = 1/3.68$ radians and $x = 10.20$ m

Similarly, for $y_s = 2$ m, $x = 7.84$ m
and for $y_s = 3$ m, $x = 4.86$ m, etc.

The resulting canal section is shown in Figure 84.

Modified Side Slope Section: In a trapezoidal section the tractive force is made equal to the permissible value over only a portion of the perimeter. If the side slopes are designed to follow a cosine curve, the permissible tractive force over the entire wetted perimeter can be used. The formula for selecting the side slopes is as follows:

$$y_1 = y \cos\left[\frac{\tan\theta}{y}\, x\right] \tag{94}$$

where: y_1 = depth at distance x from the toe of the bank
y = depth on the bottom of the canal
θ = angle of repose of the material or the slope angle of the section at the water edge of the channel

$$y = \frac{\tau}{0.97 \times 1000\ s} = \frac{\tau}{970\ s} \tag{95}$$

$$V = [0.908 - 0.8 \tan\theta]\ y^{\frac{2}{3}}\ s^{\frac{1}{2}}/n \tag{96}$$

$$A = 2.04\ y^2/\tan\theta \tag{97}$$

where: τ = permissible tractive force in kg/m²

The discharge, Q, of the theoretical section is VA. If the discharge to be conveyed is less than that of the theoretical section, the top width (T) is reduced at the centre by the following formula:

$$T' = 0.96\left[1 - \frac{Q'}{Q}\right]^{\frac{1}{2}} T \tag{98}$$

where: T' = reduction of top width
Q' = discharge to be conveyed (which is less than Q)

If the discharge to be conveyed is greater than Q then the top width is increased to add a rectangular section at the centre by the following formula:

$$T'' = \frac{n(Q'' - Q)}{y^{\frac{5}{3}}\ s^{\frac{1}{2}}} \tag{99}$$

where: T'' = increase of top width
Q'' = discharge to be conveyed (which is larger than Q)

Canals with Gravel Beds: Canals with beds of gravel, shingle or cobbles should be classified according to whether or not there is bed load movement. The design of the canal should be based on the tractive force method using Figures 79, 80, 81 and 82.

The Manning roughness coefficient is expressed as follows:

$$n = (n_g^2 + n_f^2)^{\frac{1}{2}} \tag{100}$$

$$n_g = 0.021\ d_m^{\frac{1}{6}} \tag{101}$$

where: n_g = coefficient for grain roughness
d_m = mean grain size (in mm) of the material forming the boundary
n_f = coefficient for form roughness based only on the irregularities

Values of Manning's roughness coefficient (n) for various mean grain sizes in the boundary material and various form factors are given in Figure 85. The lower values of the form roughness coefficient (n_f) should be used for design of canals with gravel beds if little or no bed load movement is expected. However, since the values of the roughness coefficient (n) in gravel canals are sensitive to bed irregularities, the stability of the bed and sides should be checked for conditions of higher form roughness coefficient (n_f) if a low value of Manning's roughness coefficient (n) is used for design.

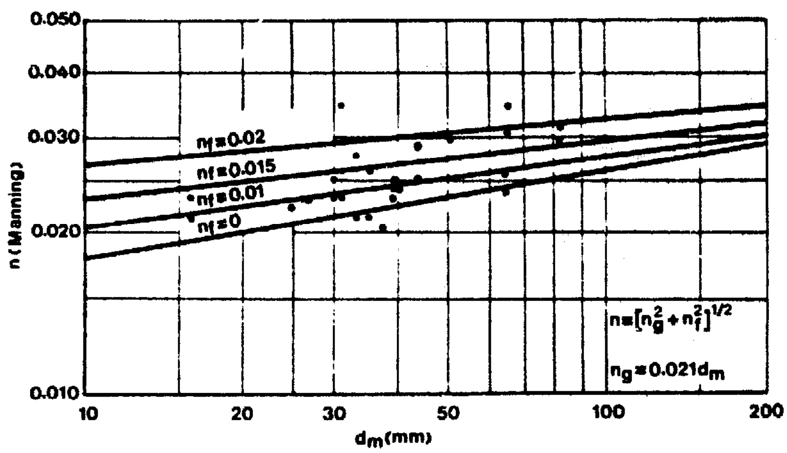


Figure 85 Manning's n for gravel channels

Kennedy Formula: The Kennedy formula was developed to determine the necessary velocity of sediment laden water flowing in a channel having a boundary of similar material. It is written as follows:

$$V_s = 0.55 \ Cy^{0.64} \qquad (102)$$

where: V_s = velocity for channel equilibrium (no siltation, no scour) in m/s
y = depth of water (m)
C = coefficient depending on soil conditions as defined in Table 30

Table 30 VALUES FOR COEFFICIENT C FOR USE WITH THE KENNEDY FORMULA

Extremely fine soils	0.56
Fine, light, sandy soil	0.84
Coarser, light, sandy soil	0.92
Sandy, loamy silt	1.01
Coarse silt or hard soil debris	1.09

For clear water the US Bureau of Reclamation recommends the following modified Kennedy formula:

EXAMPLE 19 - DESIGN OF CANAL HAVING GRAVEL BED WITH NO BED LOAD MOVEMENT

Assume that a canal to convey 100 m^3/s is to be built in soil containing gravel and cobbles. From samples, the d_m75 size of the boundary material has been determined to be 40 mm and the angle of repose 35°. Side slopes will be inclined at 2:1 (z = 2).

Step 1 From Fig. 79 the maximum unit traction on the sides is 0.77 w y s and on the bottom 0.98 w y s. The critical shear stress ratio (K) on the side slope is 0.63 (Fig. 81). The permissible unit tractive force on the bottom (τ_b) is 3.0 kg/m^2 (Fig. 80) and the permissible tractive force on the sides (τ_s) is 0.63 x 3.0 = 1.9 kg/m^2.

Try a design depth (y) of 3.0 m.

The maximum permissible canal slope is then:

for the bottom: τ_b; $3.0 = 0.98 \times 3 \times 1000\ s$

$s = 0.00102$

for the sides: τ_s; $1.9 = 0.77 \times 3 \times 1000\ s$

$s = 0.00082$ (limiting)

Step 2 Select a value of n = 0.025 from Figure 85 and determine the section dimensions for Q = 100 m^3/s and y = 3.0 m and a bottom width (b) = 12 m.

$P = 25.42$ m, $A = 54.0\ m^2$, $R = 2.13$ m

$V = R^{\frac{2}{3}}\ s^{\frac{1}{2}}/n = 1.90$ m/s $\quad Q = VA = 102\ m^3/s$ (within 2% of design Q)

Step 3 Check stability with n = 0.030 and y = 3.3 m.

$\tau_s = 0.77 \times 1000 \times 3.3 \times 0.00082 = 2.1\ kg/m^2$

$V = 1.66$ m/s and $Q = 102\ m^3/s$

The small increase in tractive force should not seriously affect the stability but the freeboard should be 1.8 m. The canal section is illustrated in Figure 84.

$$V_s = 0.55\ Cy^{\frac{1}{2}} \tag{103}$$

The US Bureau of Reclamation recommends the following formula for non-scour velocity when sand and gravel are used for protection of the banks against wave action.

$$V_s = d_{50}^{\frac{1}{3}}\ R^{\frac{1}{6}}/2 \tag{104}$$

The Kennedy formula is useful for preliminary design but the final design of the cross-sections for earth canals in erosive soils carrying little or no sediment load should be based on tractive force considerations.

d. Lined Canals

Lined canals can have three types of lining: hard surface linings, buried membrane linings and earth linings. Hard surface linings include concrete, shotcrete, brick, stone masonry, soil cement, asphalt concrete, and exposed plastic. Buried membrane linings include sprayed asphalt, prefabricated asphalt, plastics and bentonite membranes protected by earth covers. Earth linings include thick compacted earth, thin compacted earth, loose earth blankets and soil bentonite mixtures.

The main reason for lining a canal is to reduce seepage losses. Secondary factors, other than cost and availability of materials, that may influence the choice of lining are hydraulic, maintenance, right-of-way and environmental considerations (FAO 1977).

Hard surface linings

The cost of hard surface linings amounts to a large percentage of the total cost of a canal and a hydraulically efficient canal section is required. Lined canals of this type are usually designed with a base width to water depth ratio of 1 : 2 and side slopes of 1½:1.

Rugosity of hard surface lining

Roughness coefficients recommended for design of hard surface linings are listed in Table 31.

Table 31 RUGOSITY OF HARD SURFACE LININGS

Type of lining	Manning's n
Concrete lining (R up to 1.0 m)	0.014
Shotcrete lining (on earth)	0.017
Asphaltic concrete, machine placed	0.014
Soil cement	0.016
Prefabricated asphalt or plastic linings, exposed	0.015
Brick lining	0.015
Cemented rubble masonry	0.025
Dressed ashlar masonry	0.015

The roughness coefficient for Portland cement concrete lining becomes higher as the hydraulic radius increases. Coefficients recommended for the design of machine placed concrete lining recommended for design are given in Table 32.

Table 32 RUGOSITY OF MACHINE PLACED CONCRETE LININGS

Hydraulic radius R (m)	Manning's n
1.0	0.014
2.0	0.015
3.0	0.0153
4.0	0.0156
5.0	0.0159
6.0	0.0162

The above values are for machine placed concrete lining. If the lining is hand placed, 0.001 should be added to the values of the roughness coefficient listed in Table 32.

The rugosity increases with sinuosity and it is recommended that the coefficients given above be increased by 5 to 10 percent in reaches where the canal alignment is sinuous, depending on the degree of sinuosity or frequency of curves.

The rugosity of concrete lined canals is markedly affected by aquatic growth and algae on the concrete surface. If the canal is in an area subject to aquatic growths and chemical treatment is not feasible, the roughness coefficient for concrete canal lining in such areas should be increased by about 20 percent.

Foundation recommendations

It is essential that concrete, masonry, asphalt cement and other rigid hard surface linings rest on a firm subgrade. These types of linings should not be used where a firm subgrade is not assured. In-place soils of low density should be thoroughly compacted before trimming for lining placement. Soils such as low density silts, loess and expansive clays should be removed and replaced with suitable material before compacting. If this is not practical, rigid type linings should be avoided. Special consideration must be given to gypseous soils, as will be discussed later in this chapter.

Expansive clays are particularly hazardous and rigid linings across such areas require costly foundation treatment such as removal and replacement of material, special drainage blankets over watertight membranes, and watertight membranes under the lining. Fine sands which are susceptible to piping through cracks or joints in rigid type linings lead to the formation of voids behind the lining and subsequent failure. Such fine sandy soils should be thoroughly compacted and covered with a suitable filter layer before the lining is placed to avoid piping and to form a firm subgrade. This foundation treatment increases the lining cost and alternatives such as buried membrane or earth linings should be considered for such fine sandy soils.

Effect of high groundwater level

Groundwater levels can build up behind the lining in irrigated areas and water pressure can rupture the lining during drawdown of the water level in the canal. Where groundwater build-up is likely, drainage provisions are required under the lining. Such provisions normally include a toe drain and a connecting system of finger drains under the lining on the sides. The toe drain consists of a 10 or 15 mm diameter drain tile or perforated plastic pipe set in a gravel filled trench and flap valves or other suitable outlets for the drain pipe. The finger drains consist of trenches filled with free draining gravel filter material. A typical design for a flap valve and drain system is shown on Figure 86.

Hard-surface rigid canal linings when properly installed on a firm subgrade of suitable materials are permanent and will give years of satisfactory service. If a hard-surface rigid lining is being considered, a subsurface exploration programme should be carried out to identify any potential soil problem areas where

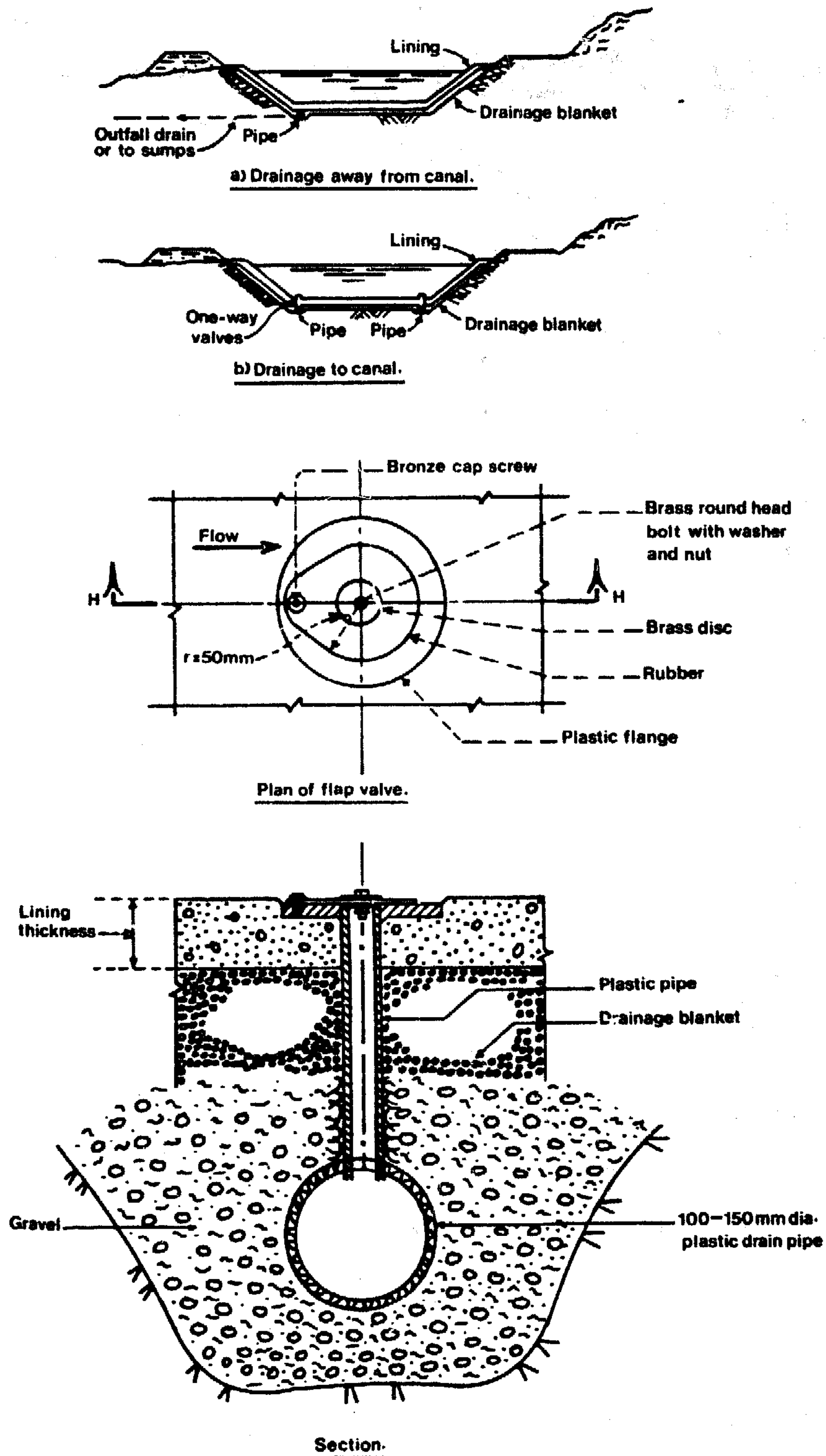


Figure 86 Canal underdrains, flap valves and lining grooves

rigid lining is to be avoided and confirm areas where in-place soil conditions are satisfactory for a hard-surface rigid lining.

Velocity limitations

Hard-surface canal linings permit higher velocities than earth canals. The maximum velocity should be limited to 2.5 m/s to avoid uplift of the slab in the event that velocity head is converted to pressure head at a crack. If the velocity exceeds 2.5 m/s a reinforced concrete lining is required.

Concrete lining

The majority of today's hard-surface canal linings consist of unreinforced Portland cement concrete. Where long distances are to be lined, machines are available for trimming and placing the lining.

Contraction cracking, due to tensile stresses produced in the concrete in hardening or by decreases in moisture or temperature, must be controlled in concrete lining. This is done by incorporating grooves having a depth equal to one third of the concrete thickness to confine the cracking to predetermined planes of weakness. These grooves can be mechanically formed by the lining machine at selected spacings. The shape of the groove is illustrated in Figure 87.

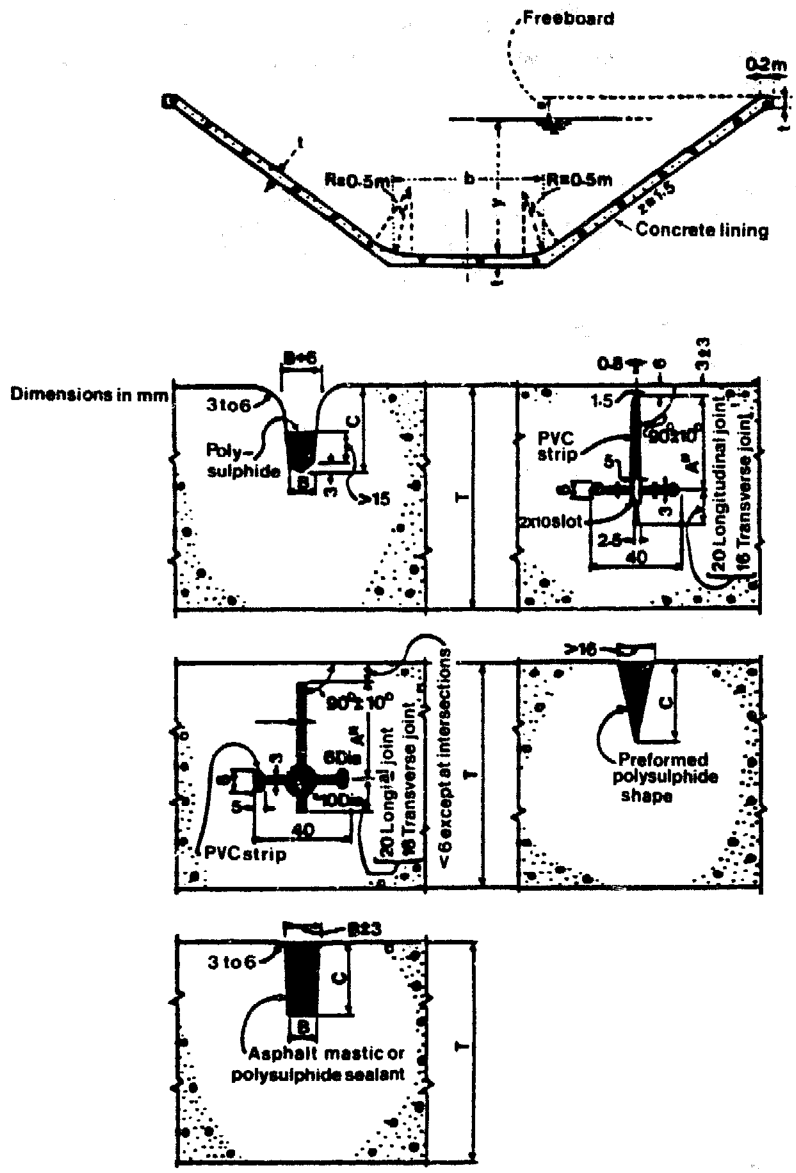
Recommended minimum lining thicknesses for various canal capacities, groove dimensions and groove spacings are given in Table 33.

Table 33 MINIMUM THICKNESS OF CONCRETE LINING

Canal capacity (m^3/s)	Lining thickness (cm)	Groove dimensions b (mm)	c (mm)	Groove spacing (m)
6 or less	6	6 to 9	18 to 21	3.0
6 to 15	7	9 to 12	22 to 24	3.0
15 to 40	8	9 to 12	26 to 28	3.5 to 4.5
40 to 100	9	9 to 12	29 to 31	3.5 to 4.5
over 100	10	9 to 12	32 to 34	3.5 to 4.5

The grooves, or control joints, require treatment with joint sealant or plastic sealing strips to prevent seepage or possible piping of fines through the contraction crack. The plastic sealing strips can be conveniently placed in the longitudinal grooves from the lining machines. Placement of sealing strips in the transverse joints requires special equipment. The sealing strips provide a permanent seal. If the sealing strips cannot be used because of lining thickness restrictions or equipment limitations, then the joint should be sealed with a two-base polysulphide sealant. The polysulphide sealant is applied by a pressure gun to the dry groove following cleaning by sandblasting to remove curing compound, laitance and loose materials.

If the concrete lining is placed by manual methods a sill type joint is effective. The sill is placed in advance and centred at the groove spacings. The surface of the sill is painted with bitumen and the panels are poured.



GROOVE AND PVC STRIP DIMENSIONS AND TRANSVERSE SPACING

Slab thickness (mm)	A PVC strip height (mm)	B Groove width (mm)	C Groove depth (mm)	Approximate groove spacing (m)
50	30 to 40	10 to 15	15 to 20	3
65	30 to 40	10 to 15	20 to 22	3
75	30 to 40	10 to 15	25 to 28	3.5 to 4.5
90	30 to 40	10 to 15	28 to 32	3.5 to 4.5
100	40 to 45	10 to 15	32 to 35	3.5 to 4.5
115	40 to 45	15 to 20	38 to 40	3.5 to 4.5

Figure 87 Concrete lining and joint details

Typical lining and joint treatment details are shown on Figure 87.

Shotcrete lining

Shotcrete lining is Portland cement mortar applied under pressure. It is particularly adapted for placement as a lining over rock in rock cuts. It is also a satisfactory lining when placed on earth for canals up to 15 m^3/s capacity in mild climates. The shotcrete is placed about 3 cm thick and reinforced with wire mesh.

Asphalt concrete

Asphalt concrete may be used for canal lining in regions where it is an economical substitute for concrete. It 'ould be placed about 10 cm thick and the velocity in the canal should be limited to 1.5 m/s. It has one advantage over concrete in that it is more flexible and will adapt to subgrade changes. It has a shorter life expectancy than concrete and it is essential that the subgrade be chemically sterilized before placement since asphalt concrete is subject to damage from weed growth.

Brick linings

Where bricks or brick tiles are locally produced and labour is cheap a brick lining may be an economical alternative to concrete. The bricks or tiles are placed on a firm subgrade in a bed of sand-cement mortar in two layers with a sprayed asphalt or plastic sheeting membrane sandwiched between the brick layers. Brick sills are required at the toe of the side walls and at transverse intervals to support the lining.

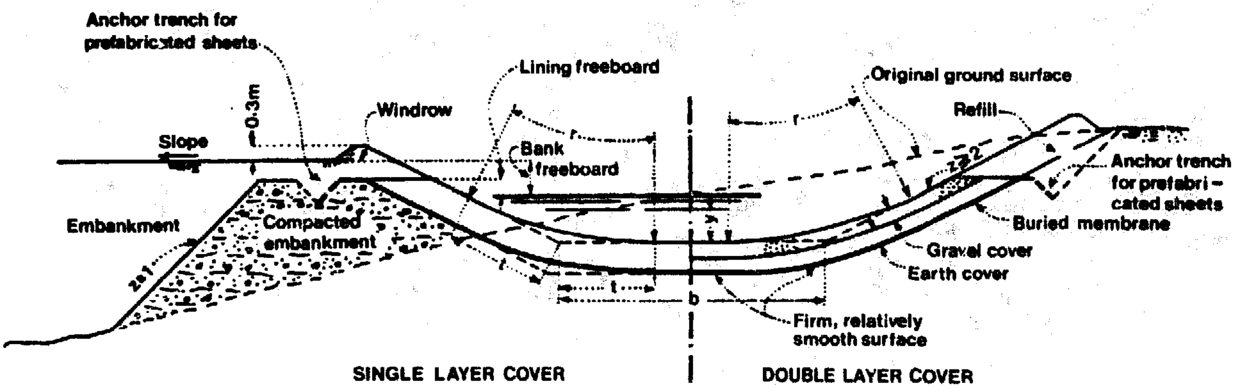
Masonry linings

Where suitable stone is plentiful and labour is cheap, masonry linings should be considered as a substitute for concrete. Masonry linings are permanent and require little maintenance when set in sand-cement grout. Rubble masonry is rough and a Manning's roughness coefficient (n) of 0.025 should be used for design. Dressed stone ashlar masonry however is almost as good hydraulically as concrete.

Soil cement

Soil cement is a mixture of Portland cement and natural soil. In areas where the subgrade or adjacent soils are of a sandy nature and other suitable lining materials are not readily available, soil cement may be economical and should be considered for canal lining. A sandy soil of 20 mm maximum grain size with 10 to 35 percent passing the No. 200 sieve gives the best results. A bituminous coating should be applied after placement for curing.

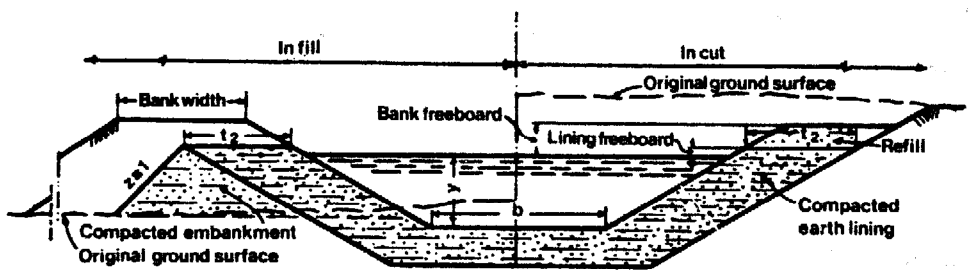
Soil cement can be mixed in place or mixed in a pugmill type mixer as a plastic mix and placed like concrete. Plastic soil cement lining should be 10 cm to 15 cm thick. If the soil cement is to be mixed in place, the side slopes should not be steeper than 4:1. The material should be mixed in place with travelling mixing machines and thoroughly compacted.



c = 0.075 y + 0.25

b (m)	0.50	1.00	2.50	5.0	10.0	15.0
t (m)	0.25	0.30	0.80	1.50	2.50	3.75

(a) Buried membrane lining
(Adapted from FAO 1977)



y (m)	t_1 (m)	t_2 (m)	b/y	z
0.50	0.30	0.90	2	1.5
1.25	0.45	1.20	3	1.5 - 1.75
2.50	0.60	1.80	3.5	2
>2.50	0.60	2.50	4 - 7	2

(b) Thick compacted earth lining
(Adapted from USBR (1963) and
ICID (1957) in FAO (1977))

Figure 88 Typical canal sections

Flexible linings

Lining materials such as prefabricated asphalt, plastic and butyl coated fabric linings have the advantage of flexibility and readily conform to irregularities in the subgrade. They have special applications for temporary use but are susceptible to puncture and are better used as a buried membrane protected by earth cover.

Precast concrete

Where labour is cheap and cement is expensive, precast concrete slab lining may be economical and should be considered. The slabs should be made 5 cm thick, 0.5 m square, and cast in forms to make tongue and groove edges. The slabs are laid by hand on the subgrade and the joints sealed with mastic.

Buried membrane canal lining

Buried membrane linings consist of a thin impervious membrane protected by an earth and gravel cover. The impervious membrane can be sprayed asphalt, plastic, prefabricated asphalt, butyl-coated fabric or bentonite. This type of lining is almost completely watertight as long as the cover material is adequate for protection against weathering, erosion and mechanical damage. The cover material should be at least 0.8 m thick to protect the membrane from the hooves of animals. The cover material should be compacted by rolling, but compaction should be done with care and only after sufficient cover has been placed to protect the membrane.

It is important that the section be stable and the tractive force method should be used for design. A section approximating the modified side slope section shown on Figure 84 should be used to improve the stability. The side slopes should be made 2½:1 or flatter, since the membrane forms a weak plane for sliding.

A band of coarse gravel and cobbles of 75 to 100 mm size should be placed along the water line for protection against wind-generated waves.

A typical buried membrane section is shown in Figure 88.

Earth lining

Effective earth linings include thick compacted earth, gravel-protected thin compacted earth, and gravel-protected bentonite-soil linings.

Thick compacted earth linings perform very well and should be considered when suitable material is available within an economical haul distance. Gravel with sand-clay binder or sand with clay binder are ideal materials but silts and clays and clayey sands can also be used with appropriate velocities. The tractive force method should be used for design. The lining thickness on the sides should be such that the material can be placed and compacted in horizontal layers. The earth lining on the bed should be at least 0.6 m thick.

Thin compacted earth linings should consist of a cohesive soil placed about 0.3 m thick and compacted. The sides are compacted by pulling a roller up and down the slope with a crane or pulling a roller along the side slope, with a tractor working on the canal bank and a tractor on the bed. After compaction, the earth lining is covered with a protective layer of gravel 0.2 to 0.3 m thick. A disadvantage of thin compacted linings is a possible loss of the protective material when heavy equipment is used for canal cleaning under flowing conditions. If a thin compacted earth lining has been used, then canal cleaning should only take place when the canal is dewatered, using hand labour and tractor loaders.

Bentonite-soil is a mixture of bentonite with sandy soils mixed in place, spread over the canal perimeter and compacted to form a lining 8 to 10 cm thick. A protective layer of gravel or stable soil 0.3 m thick is placed over the lining. The same restrictions on canal cleaning apply as for thin compacted linings.

Typical sections for earth canal linings are shown on Figure 88.

Freeboard for lined canals

Recommended minimum freeboard provisions for lined canals of various capacities are given in Table 34.

Table 34 FREEBOARD FOR LINED CANALS

Design capacity of canal (m^3/s)	Height of hard surface or buried membrane linings above water surface (m)	Height of earth linings above water surface (m)	Height of canal bank above water surface (m)
1 or less	0.15	0.15	0.4
2	0.20	0.15	0.6
5	0.30	0.20	0.8
10	0.40	0.25	0.9
20	0.50	0.30	1.0
50	0.60	0.40	1.2
100	0.70	0.50	1.4
200	0.80	0.55	1.6
400	0.90	0.60	1.8
700	1.00	0.70	2.0

Sizes of gravel and cobble for protection against wind generated waves of various heights are listed in Table 35. Protection against wind-generated waves is required at the water's edge for buried membrane, thin compacted earth and bentonite-soil canal linings. The protection should be placed in a band extending from the top of the lining to about 0.5 m below the lowest normal operating water level.

Table 35 GRAVEL SIZE FOR WAVE PROTECTION IN CANALS

Wave height (m)	d_{50} size (mm)
0.10	40
0.15	75
0.20	100
0.30	150

If there is a relift pumping plant in the canal, sudden stoppage of the pumping plant due to power failure or sudden closure of the gates will create a surge with relatively high velocities. In such cases the canal may require additional freeboard and erosion protection.

The velocity and height of the surge can be calculated by applying translatory wave theory as suggested by Jaeger (1956), Chow (1959) or, as follows, by King (1963).

In a wide rectangular channel and neglecting friction forces

$$V_2 = \frac{y_1}{y_2} (V_1 - C) + C \qquad (105)$$

where: y_1 and y_2 are the initial and final depths (m)
$y_2 - y_1$ = height of surge wave (m)
V_1 and V_2 are the initial and final canal velocities (m/s)
C = celerity of the surge wave (m/s)

and where the wave travels upstream

$$C = - \left[\frac{gy_2}{2y_1} (y_2 + y_1)\right]^{\frac{1}{2}} + V_1 \qquad (106)$$

The values of C and y_2 are determined by trial and error by selecting a value of the final depth (y_2) which when inserted in Eq. (106) yields a value of the celerity (C) which satisfies Eq. (105).

When the channel is not rectangular, then:

$$V_2 = \frac{A_1}{A_2} (V_1 - C) + C \qquad (107)$$

and $$C = - \left[\frac{g(A_2\bar{y}_2 - A_1\bar{y}_1)}{A_1(1 - A_1/A_2)}\right]^{\frac{1}{2}} + V_1 \qquad (108)$$

where $\bar{y}$ the distance from the surface down to the centre of area of the wetted cross-section is written:

$$\bar{y} = \left[\frac{b/2y + z/3}{z + b/y}\right] y \qquad (109)$$

Since friction forces have been neglected the actual wave heights will be somewhat less because channel resistance causes a gradual reduction of velocity.

Downstream of the gate a negative surge occurs with a decrease in depth upon closure and a gradual recession of the water surface travelling downstream rather than a well defined wave.

More refined methods are available for calculating the surge height and velocity which take into account friction forces, but it is apparent that if the canal flows directly into a pumping plant and sudden stoppage occurs, a high surface velocity can

EXAMPLE 20 - DESIGN OF LINED CANALS

1. Concrete Lining

Subgrade: Poorly graded gravel-sand mixture
$Q = 100\ m^3/s$, side slopes $z = 1\frac{1}{2}$, slope = 0.0003

Step 1 Try R = 2.4 m, Manning's n = 0.0152 (Table 32)

$V = R^{\frac{2}{3}}\ s^{\frac{1}{2}}/n = 2.04\ m/s$

Step 2 $A = Q/V = 49.02\ m^2$

Step 3 Try: y = 4.1 m, b = 6.0 m

then $P = 20.78$ m, $A = 49.81\ m^2$, T (top width) = 18.3 m,
R = 2.40 m and $Q = 101.74\ m^3/s$

Step 4 Reduce y by 0.8/(18.3 x 2.04) = 0.02 m
[49.84 - 49.02 = 0.8]

Step 5 Use y = 4.08 m, b = 6.0 m, $A = 49.44\ m^2$, R = 2.39 m
and $Q = 100.7\ m^3/s$ (for design section see Figure 89)

2. Buried Membrane Lining

$Q = 100\ m^3/s$
Cover material d_{75} = 8 mm, moderately rounded.
Wind generated wave height 0.2 m.
Use tractive force method for design of section.
Use Manning n = 0.0225 (Table 29).

Step 1 Angle of repose for d_{75} = 8 mm is 24° (Fig. 82).
Use 1:3 side slopes (Fig. 81).

Step 2 Maximum unit tractive: (Fig. 79).
Sides = 0.78 x 1000 ys, bottom = 0.97 x 1000 ys

Step 3 Permissible tractive force: K = 0.63 (Fig. 81) or

$$K = \left(1 - \frac{\sin 18.4°}{\sin 24°}\right)^{\frac{1}{2}}$$

Bottom $(\tau_b) = 0.65\ kg/m^2$ (Fig. 80)

Sides $(\tau_s) = K \times 0.65 = 0.63 \times 0.65 = 0.41\ kg/m^2$

Step 4 Try y (water depth) = 3.0 m to determine canal slope.
Sides: S = 0.41/(780 x 3.0) = 0.000175 (determinant)
Bottom: S = 0.65/(970 x 3.0) = 0.00022

Step 5 Try bottom width, b = 24 m, depth y = 3.0 m, slope S = 0.000175, P = 42.97 m, A = 99.0 m², R = 2.30 m, V = 1.02 m/s, Q = 101.5 m³/s.
Retain b = 24 m to compensate for loss of area due to rounding at toe.

Step 6 Assume seasonal minimum normal operating level is 0.8 m below design depth. For wave height of 0.20 m, the d_{50} size of gravel for wave protection is 100 mm (Table 35). The lining freeboard is 0.7 m. The wave protection extends from 0.7 m above to 1.3 m below the design water surface. For design section see Figure 89.

3. Thick Compacted Earth Lining

Assume Q = 100 m³/s. Lining material is compacted clayey sand, d_{50} size = 2 mm, angle of repose = 30°.
Thickness on bed = 0.8 m
Side slopes to be 2½:1
Manning's n = 0.20

Step 1 Maximum unit tractive force (Fig. 79)
Sides = 0.76 x 1000 ys
Bottom = 0.97 x 1000 ys

Step 2 Permissible unit tactive force: K = 0.68 (Fig. 81)
Bottom τ_b = 0.30 kg/m (Fig. 4)
Sides τ_s = k x 0.30 = 0.68 x 0.30 = 0.20 kg/m²

Step 3 Try y (depth water) = 3.0 m
Bottom: S = 0.30/(970 x 3.0) = 0.000103
Sides: S = 0.20/(760 x 3.0) = 0.000088 (determinant)

Step 4 Try bottom width b = 32 m, y = 3.0 m, S = 0.000088
P = 48.16 m. A = 118.5 m², R = 2.46 m, V = 0.85 m/s
Q = 101.3 m³/s (close to design discharge)

Step 5 Add wave protection as for buried membrane section since the clayey-sand lining is relatively fine. For canal section see Figure 89.

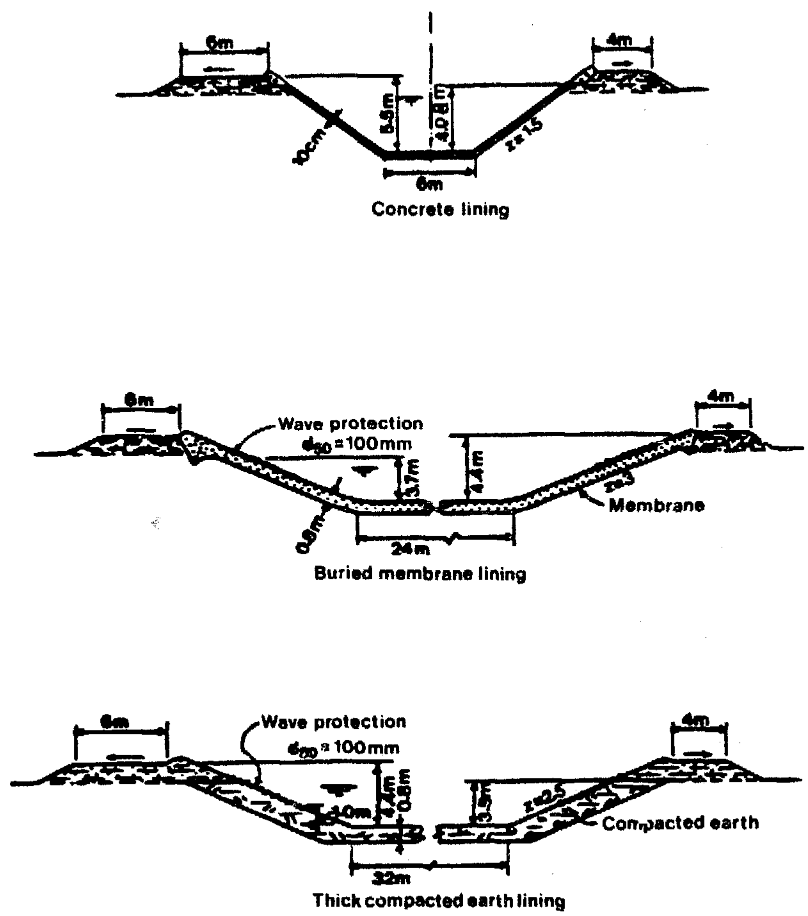


Figure 89 Typical canal sections

result. Provision must be made for such high velocities by including wasteway facilities interconnected with the pumping plant and protection of the canal surfaces that are exposed to the wave.

Hand Placed Rip-rap or Pitching

Approximate stone sizes for hand placed rip-rap or pitching are listed in Table 36 for various flow velocities.

The dry pitching or rip-rap should be placed on a gravel bed 0.15 to 0.20 m thick or on a suitable synthetic fabric filter.

Rip-rap or dry pitching is required in earth canals adjacent to structures in transition zones and in areas subject to high velocity waves or eddies.

Table 36 SIZE OF RIP-RAP FOR CANAL LINING

Flow velocity (m/s)	Stone diameter (m)
1.5	0.10
2.0	0.15
2.5	0.20
3.0	0.25
3.5	0.30
4.0	0.40

EXAMPLE 21 - SURGE WAVE DETERMINATION

1. Calculate the height and velocity of a surge wave resulting from the instantaneous closure of a gate in a rectangular canal conveying water at 1 m/s with a flow depth of 3 m.

   After closure the final velocity (V_2) is zero, then from Eq. (105)

$$0 = \frac{3}{y_2}(1 - C) + C$$

   Selecting a trial value of the final depth (y_2) = 3.6 m

$$C = -\left[\frac{3.6g}{6}(3.6 + 3)\right]^{\frac{1}{2}} + 1 = -5.23 \text{ m/s}$$

   (the sign of C is negative since the wave travels upstream)

   Substituting for C and y_2 in the first equation above

$$\frac{3}{3.6}(1 + 5.23) - 5.23 = 0.03$$

   which is close to zero and confirms the value of y_2 = 3.6 m.

2. For a trapezoidal canal of 20 m bottom width (b) and side slopes of 3:1 (z = 3) conveying water with a depth (y_1) of 3 m and a velocity of 1 m/s, calculate the velocity and height of the surge wave which travels upstream after sudden closure of a gate.

   Selecting a trial value of the final depth (y_2) = 3.5 m

   from equation (109) $\bar{y}_1$ = 1.344 m $\bar{y}_2$ = 1.551 m

   and since A = (b + zy)y, A_1 = 87 m², A_2 = 106.75 m²,

   from equation (108) $C = -5.44 + V_1 = -4.44$ m/s

   Substituting this value for C in equation (107) with V_2 = 0 after closure:

$$0 = 87/106.75\ (1 + 4.44) - 4.44 = 0$$

5.2.2 Conveyance Structures

i. Definition

A canal, or open conveyor, may have tunnels, inverted siphons, drops or fall structures, flumes, or pumping plants. Tunnels are required through topographic barriers that cannot be economically circumvented by an open canal or bench flumes. Inverted siphons are required for crossing depressions or drainages that cannot be economically crossed by a fill section, in combination with culverts or elevated flumes if a stream is to be crossed. Drops or fall structures are required if the canal has to be dropped in grade due to topographic considerations. In larger canals, if the drop is significant, the economics of including a canal drop hydro-electric plant should be considered as an appurtenance to the drop structure.

Flumes may be bench type flumes or elevated flumes. Bench flumes are used to carry the conveyor along steep ground, usually in bench cuts in rock, where a canal section is not feasible. Elevated flumes are used to carry a conveyor across drainages that are subject to torrential flows with much debris movement making culverts unfeasible, or where the magnitude of the flood flow is such that an elevated flume is more economical than culverts. The econmics of an inverted siphon crossing as an alternative to a fill section with culverts or elevated flume should be evaluated with due regard to the economic effect of the loss of command because of the siphon head loss.

If it is necessary to raise the canal bed level because of adverse topography or other reasons, a pumping plant is necessary. Such installations require adequate provisions in the canal to cope with the effects of surge waves generated from sudden stoppages due to power interruptions.

ii. Transitions

Properly designed transitions are required upstream and downstream of conveyance structures to provide smooth inflow conditions at the inlet, minimize head losses, and minimize waves and eddies at the outlet.

Head losses in transitions from the canal section to the structure include velocity head loss and friction loss. The friction loss is usually small compared to the velocity head loss. The velocity head loss in the transition is computed as K Δh_v, where Δh_v is the difference in velocity head between the canal and the entrance to the conduit and K is a coefficient that varies with the type of transition and differs for the inlet and the outlet. Recommended values for K are given in Table 37.

The minimum length of transitions should not be less than the following:

Inlet : T/2 tan 27.5°
Outlet: T/2 tan 22.5°

where T is the change of water surface width.

A streamlined warped transition is designed as follows:

Table 37 TRANSITION HEAD LOSS COEFFICIENT

Type of transition	Inlet K	Outlet K
Open transitions:		
Streamlined warped to rectangular	0.10	0.20
Straight warped to rectangular	0.20	0.30
Broken-back type to rectangular	0.30	0.50
Broken-back type to pipe opening	0.40	0.70
Closed transition:		
Rectangular to round (maximum angle with axis = 7½°)	0.10	0.20

Inlet:

Step 1 Compute difference in water-surface level (drop) between the canal and conduit.

$$\text{Drop} = 1.1\ \Delta h_v$$

$$= 1.1\left(\frac{V_1^2 - V_2^2}{2g}\right)$$

where V_1 = velocity in canal (m)
V_2 = velocity in conduit (m)

Step 2 Divide inlet into 9 or 10 equal segments and compute water surface profile as two parabolic curves (see Fig. 90).

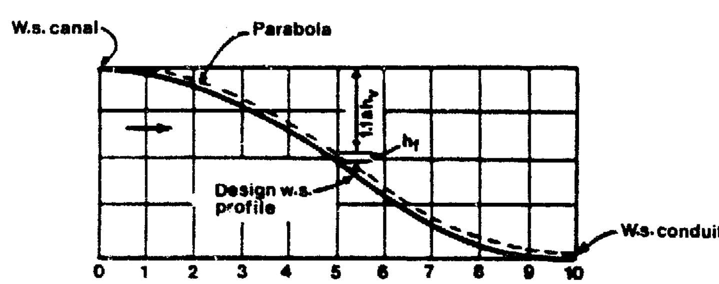


Figure 90 Design of warped transition

Step 3 Proportion transition by adopting bottom width and sidewall slope to match water line (trial and error).

Step 4 Compute friction loss and subtract from water surface in Step 2 to obtain design water surface and adjust the sections to conform.

Typical transitions are shown in Figure 91.

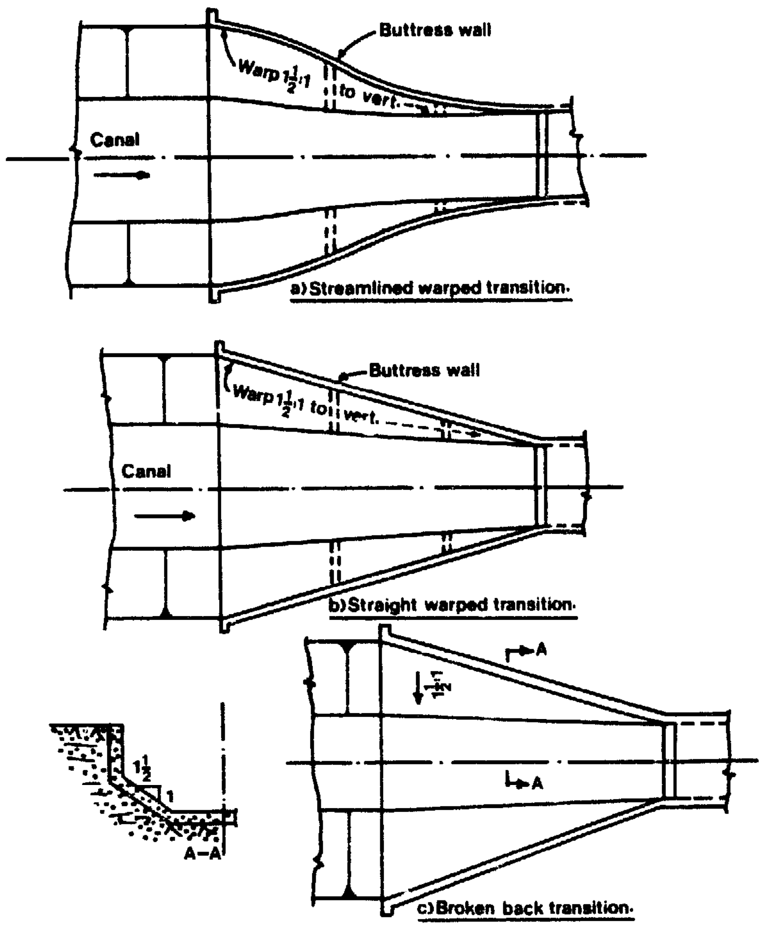


Figure 91 Typical transitions

Outlet:

Same procedure as for inlet, except in Step 1 where 0.8 Δh_v is used to compute difference in water surface (rise) between conduit outlet and canal.

A straight-warped transition (Fig. 91) is designed so that the top and base (at floor level) of the sidewall are straight while the wall is gradually warped from the canal side-slope to vertical. This transition is simpler to construct than the streamlined warped transition and is recommended unless head is critical.

The broken-back type transition (Fig. 91) can be used where the additional headloss is not critical. It is particularly useful for smaller canals. The construction is simpler than for warped transitions since it has a straight vertical wall segment and a sloping segment at the canal side slope angle. The walls for the warped transitions require counterforts (buttresses).

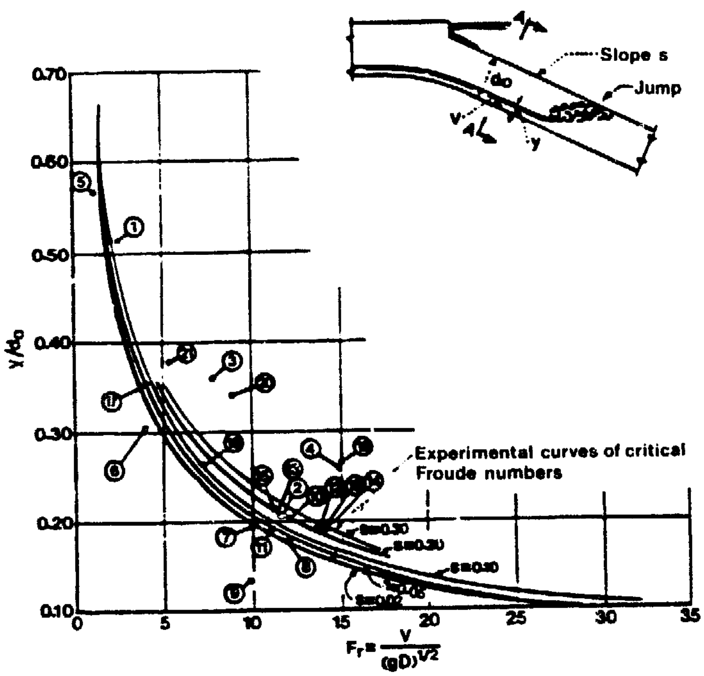
Inlet transitions to closed conduits should be designed to provide a submergence or seal of 1.5 Δh_v, but not less than 75 mm, on the top of the conduit opening. The conduit outlet should not be submerged.

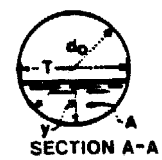
If the inlet to a free-flow conduit is sealed, the discharge should be determined by an orifice equation using an orifice coefficient of 0.6 and a head corresponding to the inlet water surface and the centre of the opening.

Under certain discharges the seal at the inlet to a long conduit, such as a siphon, will be broken and a hydraulic jump will occur at the head of the conduit. To avoid blowback and operational difficulties the conduit should be designed so that the Froude number (F_r) falls below the curves in Figure 92 (ASCE 1943).

$$Fr = V/(gD)^{\frac{1}{2}} \tag{16}$$

where: y = free flow depth in conduit (m)
V = free flow velocity (m/s)
g = acceleration due to gravity (m/s²)
D = hydraulic depth (m) - A/T





NOTES

Siphon inlets marked thus O, have given trouble in operation and air outlets were installed in some cases to relieve the blowing back of air and water. All other siphons have not given trouble in operation. A study made indicates that free flow siphon inlets designed so that Froude number will not fall above the critical curves established by experiments will give satisfactory performance.

Procedure to determine Froude number:

For a given Q, diameter d_o, slope S, and coeff. n_1 calculate the following:

a. y using Manning's Formula
b. A
c. V
d. $T = 2[(d_o-y)y]^{\frac{1}{2}}$
e. D = A/T
f. $F_r = V/(gD)^{\frac{1}{2}}$

Figure 92 Design data for free-flow siphon inlets

Procedure: Calculate V and y, using the Manning formula.

Calculate $T = 2 [(d_o - y) y]^{\frac{1}{2}}$

where: d_o = diameter of conduit (m)

Calculate A = flow cross-sectional area for y in conduit (m²).

Then D = A/T

The transition should be designed to maintain the hydraulic control section within the transition for free flow conditions to avoid erosive upstream velocities. If check gates are combined with the inlet they will serve as a control.

iii. Tunnels

Tunnels in conjunction with canals, or open conveyors, are generally of the free flow or grade type and should be designed to carry the design discharge with a minimum freeboard of 0.5 m. The tunnels are usually of the modified horseshoe shape and concrete lined for hydraulic efficiency and maintenance free operation. If the tunnel is excavated by boring machine a circular shape is used. If the tunnel is in rock the invert may be lined with concrete and the sides and crown lined with shotcrete reinforced with wire mesh, rock bolts being added where required. If the tunnel is in sound rock and weak zones are treated it may be left unlined if some loss of water through the rock joints can be tolerated.

The tunnel portals are usually located where the depth of cover is about twice the diameter but not less than 6 m in rock, or about three times the diameter but not less than 9 m in earth materials. The open cut approach section should transition from the canal or flume section to the rectangular section of the portal. A second transition section is provided from the portal to the tunnel section. The transition walls should be concrete, and the open cut approach section should transition from the canal or flume section to the rectangular section of the portal. A second transition section is provided from the portal to the tunnel section. The transition walls should be concrete or stone masonry, or shotcrete if in sound rock.

The minimum height for a finished tunnel is usually about two metres to provide working space during construction.

The following Manning n values are recommended for tunnels:

Lining	n
Concrete	0.014
Shotcrete with concrete invert	0.024
Rock, unlined	0.040 to 0.045

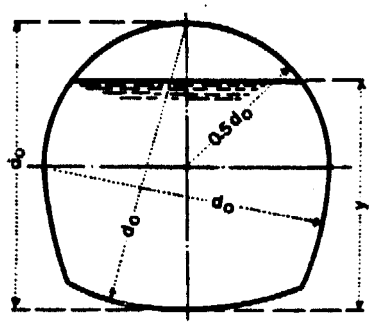
The design velocity for tunnels likely to carry abrasive materials, such as sand and gravel, should be limited to 3.0 m/s. The velocity in tunnels carrying clear water should be a safe margin below critical velocity.

Typical tunnel sections and tables for the computation of partial flows in circular and horseshoe sections are given in Figures °3/1 and 2.

iv. Inverted siphons

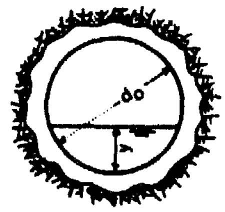
Inverted siphons are conduits designed to flow full and under pressure. The conduits may be single or multiple barrel rectangular sections if the head is 10 m or less. The conduits for large siphons with heads of more than 10 m are usually single or multibarrel monolithic reinforced concrete sections with circular interiors designed to resist the internal pressure and external loads. Smaller siphons may be single or multibarrel pipe sections such as precast concrete, asbestos-cement, steel-encased concrete pipe, and so forth.

Siphons crossing streams should be designed to have at least a 10 percent margin of safety against flotation with the cover scoured away and the pipe empty.



y/d_o	A/d_o^2	P/d_o	R/d_o	y/d_o	A/d_o^2	P/d_o	R/d_o
0.01	0.0019	0.2830	0.0066	0.51	0.4466	1.7162	0.2602
0.02	0.0053	0.4006	0.0132	0.52	0.4566	1.7362	0.2630
0.03	0.0097	0.4911	0.0198	0.53	0.4666	1.7562	0.2657
0.04	0.0150	0.5676	0.0329	0.54	0.4865	1.7763	0.2683
0.05	0.0209	0.6351	0.0329	0.55	0.4865	1.7964	0.2707
0.06	0.0275	0.6963	0.0394	0.56	0.4965	1.8165	0.2733
0.07	0.0346	0.7528	0.0459	0.57	0.5064	1.8367	0.2757
0.08	0.0421	0.8054	0.0524	0.58	0.5163	1.8569	0.2781
0.0886	0.0491	0.8482	0.0578	0.59	0.5261	1.8772	0.2804
0.09	0.0502	0.8513	0.0590	0.60	0.5359	1.8976	0.2824
0.10	0.0585	0.8732	0.0670				
0.11	0.0670	0.8950	0.0748	0.61	0.5457	1.9180	0.2844
0.12	0.0753	0.9166	0.0823	0.62	0.5555	1.9386	0.2864
0.13	0.0839	0.9382	0.0895	0.63	0.5651	1.9592	0.2884
0.14	0.0925	0.9597	0.0964	0.64	0.5748	1.9800	0.2902
0.15	0.1012	0.9811	0.1031	0.65	0.5843	2.0009	0.2920
0.16	0.1100	1.0024	0.1097	0.66	0.5938	2.0219	0.2937
0.17	0.1188	1.0236	0.1161	0.67	0.6033	2.0431	0.2953
0.18	0.1277	1.0448	0.1222	0.68	0.6126	2.0645	0.2967
0.19	0.1367	1.0658	0.1282	0.69	0.6219	2.0860	0.2981
0.20	0.1457	1.0868	0.1341	0.70	0.6312	2.1077	0.2994
0.21	0.1549	1.1078	0.1398	0.71	0.6403	2.1297	0.3006
0.22	0.1640	1.1286	0.1454	0.72	0.6493	2.1518	0.3016
0.23	0.1733	1.1494	0.1508	0.73	0.6582	2.1742	0.3026
0.24	0.1825	1.1702	0.1500	0.74	0.6671	2.1969	0.3036
0.25	0.1919	1.1909	0.1611	0.75	0.6758	2.2198	0.3044
0.26	0.2013	1.2115	0.1662	0.76	0.6844	2.2431	0.3050
0.27	0.2107	1.2321	0.1710	0.77	0.6929	2.2666	0.3055
0.28	0.2202	1.2526	0.1738	0.78	0.7012	2.2906	0.3060
0.29	0.2297	1.2731	0.1804	0.79	0.7094	2.3149	0.3064
0.30	0.2393	1.2935	0.1850	0.80	0.7175	2.3397	0.3067
0.31	0.2489	1.3139	0.1895	0.81	0.7254	2.3650	0.3067
0.32	0.2586	1.3342	0.1938	0.82	0.7332	2.3907	0.3066
0.33	0.2683	1.3546	0.1981	0.83	0.7408	2.4170	0.3064
0.34	0.2780	1.3748	0.2023	0.84	0.7482	2.4440	0.3061
0.35	0.2878	1.3951	0.2063	0.85	0.7554	2.4716	0.3058
0.36	0.2975	1.4153	0.2103	0.86	0.7625	2.5000	0.3050
0.37	0.3074	1.4355	0.2142	0.87	0.7625	2.5000	0.3050
0.38	0.3172	1.4556	0.2181	0.88	0.7759	2.5595	0.3032
0.39	0.3271	1.4758	0.2217	0.89	0.7823	2.5909	0.3020
0.40	0.3370	1.4959	0.2252	0.90	0.7884	2.6235	0.3005
0.41	0.3469	1.5160	0.2287	0.91	0.7943	2.6576	0.2988
0.42	0.3568	1.5360	0.2322	0.92	0.7999	2.6935	0.2903
0.43	0.3667	1.5661	0.2356	0.93	0.8052	2.7315	0.2947
0.44	0.3767	1.5761	0.2390	0.94	0.8101	2.7721	0.2922
0.45	0.3867	1.5962	0.2422	0.95	0.8146	2.8160	0.2893
0.46	0.3966	1.6162	0.2454	0.96	0.8188	2.8643	0.2855
0.47	0.4066	1.6362	0.2454	0.97	0.8224	2.9188	0.2816
0.48	0.4160	1.6562	0.2514	0.95	0.8256	2.9832	0.2765
0.49	0.4266	1.6762	0.2544	0.99	0.8280	3.0667	0.2695
0.50	0.4366	1.6962	0.2374	1.00	0.8293	3.2670	0.2538

Figure 93/1 Area, wetted perimeter and hydraulic radius of partially filled horseshoe conduit sections



y/d_o	A/d_o^2	P/d_o	R/d_o	y/d_o	A/d_o^2	P/d_o	R/d_o
0.01	0.0013	0.2003	0.0066	0.51	0.4027	1.5908	0.2531
0.02	0.0037	0.2838	0.0132	0.52	0.4127	1.6108	0.2561
0.03	0.0069	0.3482	0.0197	0.53	0.4227	1.6308	0.2591
0.04	0.0105	0.4027	0.0262	0.54	0.4327	1.6509	0.2620
0.05	0.0147	0.4510	0.0326	0.55	0.4425	1.6710	0.2649
0.06	0.0192	0.4949	0.0389	0.56	0.4528	1.6911	0.2676
0.07	0.0242	0.5355	0.0451	0.57	0.4625	1.7113	0.2703
0.08	0.0294	0.5735	0.0513	0.58	0.4723	1.7315	0.2728
0.09	0.0350	0.6094	0.0574	0.59	0.4822	1.7518	0.2753
0.10	0.0409	0.6435	0.0635	0.60	0.4920	1.7722	0.2776
0.11	0.0470	0.6761	0.0695	0.61	0.5018	1.7926	0.2797
0.12	0.0534	0.7075	0.0754	0.62	0.3155	1.8132	0.2818
0.13	0.0600	0.7377	0.0813	0.63	0.5212	1.8338	0.2839
0.14	0.0668	0.7670	0.0871	0.64	0.5308	1.8546	0.2860
0.15	0.0739	0.7954	0.0929	0.65	0.5404	1.8755	0.2881
0.16	0.0811	0.8230	0.0986	0.66	0.5499	1.8965	0.2899
0.17	0.0885	0.8300	0.1042	0.67	0.5594	1.9177	0.2917
0.18	0.0961	0.8763	0.1097	0.68	0.5687	1.9391	0.2935
0.19	0.1039	0.9020	0.1132	0.69	0.5780	1.9606	0.2950
0.20	0.1118	0.9273	0.1206	0.70	0.5872	1.9823	0.2962
0.21	0.1199	0.9521	0.1259	0.71	0.5964	2.0042	0.2973
0.22	0.1281	0.9764	0.1312	0.72	0.6054	2.0264	0.2984
0.23	0.1365	1.0003	0.1364	0.73	0.6143	2.0488	0.2995
0.24	0.1449	1.0239	0.1416	0.74	0.6231	2.0714	0.3006
0.25	0.1535	1.0472	0.1466	0.75	0.6318	2.0944	0.3017
0.26	0.1623	1.0701	0.1516	0.76	0.6404	2.1176	0.3025
0.27	0.1711	1.0928	0.1566	0.77	0.6489	2.1412	0.3032
0.28	0.1800	1.1152	0.1614	0.78	0.6573	2.1652	0.3037
0.29	0.1890	1.1373	0.1662	0.79	0.6655	2.1985	0.3040
0.30	0.1982	1.1593	0.1709	0.80	0.6736	2.2143	0.3042
0.31	0.2074	1.1810	0.1755	0.81	0.6815	2.2395	0.3044
0.32	0.2167	1.2025	0.1801	0.82	0.6893	2.2653	0.3043
0.33	0.2260	1.2239	0.1848	0.83	0.6969	2.2916	0.3041
0.34	0.2355	1.2451	0.1891	0.84	0.7043	2.3186	0.3038
0.35	0.2450	1.2661	0.1935	0.85	0.7115	0.2462	0.3033
0.36	0.2546	1.2870	0.1978	0.86	0.7186	2.3746	.3026
0.37	0.2642	1.3078	0.2020	0.87	0.7254	2.4038	0.3017
0.38	0.2739	1.3284	0.2061	0.88	0.7320	2.4341	0.3008
0.39	0.2836	1.3490	0.2102	0.89	0.7384	2.4655	0.2996
0.40	0.2934	1.3694	0.2142	0.90	0.7445	2.4981	0.2980
0.41	0.3032	1.3898	0.2181	0.91	0.7504	2.5322	0.2963
0.42	0.3130	1.4101	0.2220	0.92	0.7560	2.5681	0.2944
0.43	0.3229	1.4303	0.2257	0.93	0.7642	2.6061	0.2922
0.44	0.3328	1.4505	0.2294	0.94	0.7662	2.6467	0.2896
0.45	0.3428	1.4706	0.2331	0.95	0.7707	2.6906	0.2864
0.46	0.3527	1.4907	0.2366	0.96	0.7749	2.7389	0.2830
0.47	0.3627	1.5108	0.2400	0.97	0.7785	2.7934	0.2787
0.48	0.3727	1.5306	0.2434	0.98	0.7816	2.8578	0.2735
0.49	0.3827	1.5508	0.2467	0.99	0.7841	2.9412	0.2665
0.50	0.3927	1.5708	0.2500	1.00	0.7854	3.1416	0.2500

Figure 93/2 Area, wetted perimeter and hydraulic radius of partially filled circular conduit sections

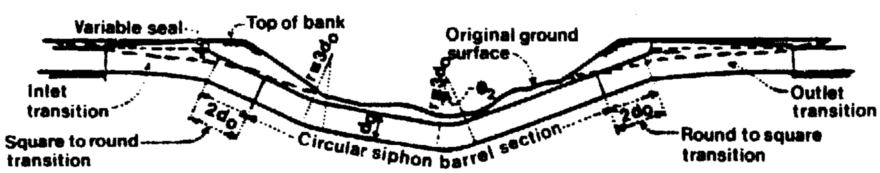
The headloss through the siphon should include transition losses, bend losses, gate or check losses if combined with the siphon, and friction losses through the transitions and barrel. Appropriate values of Manning's roughness coefficient (n) should be used for computing the friction losses. A safety factor of 10 percent is added to all the losses for the establishment of the invert elevations of siphons.

The following values of Manning's roughness coefficient (n) are suggested for siphon conduits:

Material	n
Cast-in-place concrete	0.014
Precast concrete pipe	0.013
Asbestos-cement pipe	0.013

A blow-off valve and access manhole should be provided at a convenient low point in the siphon if it is not practicable to empty the pipe by pumping from the lower end.

A typical siphon is shown in Figure 94.



COMPUTATION OF HEAD LOSSES

Inlet Open Transition (Friction)
Inlet Open Transition (Convergence)
Inlet Closed Transition (Convergence)
Closed Transitions (Friction)
Circular Barrel (Friction)
Barrel Bend θ_1
Barrel Bend θ_2
Outlet Closed Transition (Divergence)
Outlet Open Transition (Divergence)
Outlet Open Transition (Friction)
Total Loss (Energy Gradient)
Add 10% for excess capacity
Total head required is sum of above

Figure 94 Typical siphon

v. Drops and flumes

Drop structures consist of an intake, chute section and stilling basin for dissipating energy in a hydraulic jump. For detailed hydraulic design of the various components of the structures, it is suggested to refer to the specialized literature available on standardization (USBR 1977; 1974; FAO 1975; ILRI 1976). Only a general review will be given below.

A vertical drop may be more economical than an inclined drop for small changes of elevation. A vertical drop has an intake, a vertical wall and a stilling pool for dissipating energy from the free overfall.

Bench flumes are carried on the ground and are used to convey water along steep hillsides. Elevated flumes are carried on piers and are used to convey water across a depression.

A bench flume may consist of a U-shaped section supported on the ground or of an L-shaped section on one side with a cut slope lined with concrete or shotcrete on the other side.

An elevated flume is a reinforced concrete U-shaped section designed so that the walls carry the load between the piers. The floor loads are transferred to the wall by shear reinforcement.

Flumes should be provided with joints and water stops at 8 m to 10 m intervals. The freeboard in flumes should be slightly less than in the canal so that the flume will overtop before the canal.

Flumes have the same requirements for transitions as tunnels and siphons. Hydraulic losses are computed in the same way as for a siphon but the safety factor of 10 percent is not added. The velocity in a flume should be a safe margin below critical velocity. The reinforced concrete flume should be designed using a Manning roughness coefficient (n) of 0.014 but the critical velocity should be checked assuming n = 0.011 and a slope between joints that takes into account construction tolerances.

Vertical drops

Vertical drops should be limited to 2 m drop in elevation. Check gates or an overflow crest are required at the inlet to avoid excessive velocities upstream. Vertical drops usually generate downstream surface waves and extensive rip-rap protection is required to control erosion unless the downstream canal is concrete-lined. The downstream water surface must be low enough for the crest to be submerged by less than 0.6 times the critical depth to permit the jet to plunge.

Inclined drops

Inclined drops are the most frequently used drop structure for large canals. The intake requires check gates, control section or overflow crest to avoid excessive upstream velocities. The intake and sloping chute should be connected by a vertical curve or trajectory.

Stilling basins

Energy is dissipated downstream of chutes and inclined drops by the formation of a hydraulic jump within the confines of the stilling basin.

If the stilling basin discharges into an uncontrolled channel a control section should be provided at the end of the chute to assure jump formation.

If the stilling basin discharges into a canal, or other controlled non-erodible channel, a channel depth corresponding to a 20 percent reduction in Manning's roughness coefficient (n) should be used to compute the downstream energy level.

5.3 DESIGN AND OPTIMIZATION OF DISTRIBUTION NETWORKS

5.3.1 Open Channel Systems

i. Definition

Open channel distribution networks carry flow from the open channel conveyor to the farm system. Distribution or lateral canals usually run down the general slope of the project lands and are spaced at intervals that will enable the various farm systems to be supplied. Sublateral canals which are supplied from the lateral canal frequently branch out on one or both sides of the lateral to reach farm systems between the laterals.

ii. Characteristics and General Requirements

Location

The distribution canal network is usually planned on project topographic maps on which the delivery points to the farm systems and farm boundaries have been marked out. Natural drainages and streams crossing the project area often play a dominant role in fixing the boundaries of the area served by a particular distribution canal. The lateral is normally located to follow the divide or ridge between the drainages. Such a location generally permits the lateral, in combination with sublaterals where required, to deliver water to the farm system canals at the high point of a farm. The delivery rate should correspond to the design capacity of the farm system canal at the farm gate. Procedures for the determination of the farm gate capacity are outlined in Chapter 3.

Capacity

The sizing of a distribution system canal is related to the type of distribution adopted for the project; that is whether the irrigation deliveries are to be supplied by demand, continuous or by rotation. The three types of distribution and recommended peak irrigation season discharge probabilities at the farm gates for each are discussed in Chapter 3. The distribution canal is sized to meet the probable peak season farm gate deliveries plus an allowance for conveyance losses and operational waste in the distribution canals. Evaporation losses are minor in a distribution canal and estimated seepage loss can be used to develop an allowance for conveyance loss.

Losses

The procedure and seepage rates used for determination of seepage losses for open conveyors under section 5.2.1 are also applicable for the determination of seepage losses in distribution canals. Operational waste at the downstream end of a distribution canal is practically unavoidable due to the inability to have a perfect balance between the dispatch and measurement of diversions to the head of the distribution canal and the farm gate deliveries. An allowance of 2 to 5 percent should be made for operational waste when determining the dimensions of the canal system.

For example, assume that an unlined distribution canal 6 km long, built in sandy loam, has a seepage rate of 200 mm/day and that the peak irrigation discharge varies from 1.2 m^3/s at the head to 0.15 m^3/s at the end. If the total wetted area is 15 000 m^2, the resulting seepage losses amount to 0.2 x 15 000 = 3000 m^3/day, or 35 l/s. If the average discharge is 700 l/s, the conveyance loss represents 5 percent. If an allowance of 3 percent is made for operational waste the total estimated distribution canal conveyance loss is 8 percent. The design capacity for the lateral at any point should therefore be equal to the downstream demand plus 8 percent. In hot arid climates an allowance should be made for evaporation by applying the same reasoning to average water surface area.

Cross drainage

Distribution canals are usually located between natural drainages and cross drainage structures are required only in special situations, such as for a sublateral which takes off from a lateral and continues across a natural drain to reach an isolated area; or if the general topography is such that an area can be served economically by continuing the distribution canal across a natural drainageway. Since distribution canals are generally small it is usually economical to carry the canal across the drainage in an inverted siphon rather than pass the drainage under the canal through culverts. The magnitude and frequency of flood flows in a natural channel can be determined by the methods described in section 5.2.1 for open channel conveyors.

Curvature

The capacity of distribution canals seldom exceeds 5 m^3/s and is usually less than 3 m^3/s. Only in cases where the depth and general topographic configuration of a project area is such that a distribution canal could continue for a long distance and serve a relatively wide area between natural drainages would a larger canal be used. Canals of 5 m^3/s capacity or more should be designed using the guidelines described in 5.2.1 for conveyor channels. Unlined canals conveying less than 5 m^3/s should have a radius of curvature of at least five times the water width. A concrete or masonry lined distribution canal should have a minimum radius of three times the water width.

Section dimensions

Distribution canals normally have side slopes of 1½:1. Concrete lined distribution canals carrying discharges of less than 1.0 m^3/s may have 1:1 side slopes in stable soils. Earth distribution canals usually have a bottom width to depth ratio of 3:1 to 4:1 in average soils. If the soils are non-cohesive silts and sands, the stability of the canal section should be checked by the tractive force method as described in Section 5.2.1 for conveyor canals.

Distribution canals lined with concrete, masonry or other hard surface material usually have a bottom width to depth ratio of 1:1.

The minimum freeboard or height above the canal water surface should be as follows:

Canal capacity (m^3/s)	Hard surface or Membrane lining (m)	Earth lining (m)	Top of bank (m)
1 or less	0.15	0.15	0.4
2	0.20	0.15	0.6
5	0.30	0.20	0.8

Design velocities

Maximum mean velocities in earth distribution canals can be determined by using the Kennedy formula, modified for clear water. The maximum velocity in concrete lined or other hard-surface lined canals should be below critical for the following conditions:

Manning n = 0.8 x design n
Grade departure = 0.03 m (subtract from depth)
Manning n = 0.025 for earth distribution canals
0.014 for concrete lined distribution canals

Values of Manning's roughness coefficient for other lining materials will be found in section 5.2.

Canal bank

Distribution canals will require maintenance access on one of the banks unless an access road is available adjacent to the canal. Roads are required for access to the project farms and frequently it is convenient to locate access roads adjacent to the canal right-of-way. If access is not convenient from an adjacent road, then one of the banks should have a minimum top width of 4 m to accommodate maintenance and operational equipment. The opposite bank may require a top width of 1½ to 2 m to accommmodate weed control equipment.

A typical earth canal section is shown in Figure 95. The concrete lined section for a distribution canal would be similar to the canal section shown in Figure 87. Longitudinal grooves or joints are omitted in concrete lining for distribution canals but transverse grooves or joints are provided at 3-4 m intervals.

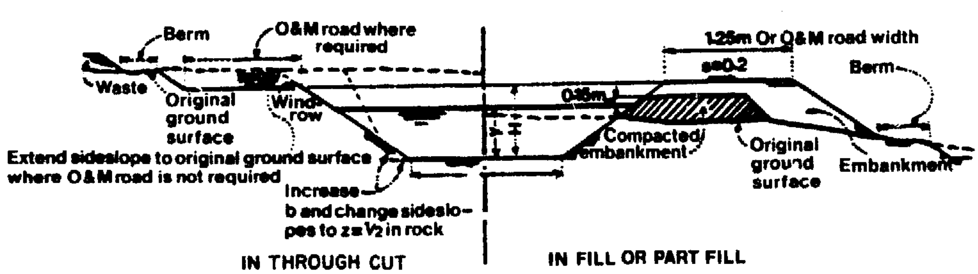


Figure 95 Typical unlined section for canals and laterals

EXAMPLE 22 - CONCRETE-LINED CANAL

Verify that the velocity of flow in a canal of specified characteristics is below the critical velocity.

discharge	$Q = 1\ m^3/s$	side slopes	$z = 1$
bottom width	$b = 0.6\ m$	depth of water	$y = 0.6\ m$
slope	$s = 0.0018$	Manning coeff.	$n = 0.014$

Under the design conditions:

the mean velocity $$V = R^{\frac{2}{3}}\ s^{\frac{1}{2}}/n = \left[\frac{(b + zy)}{b + 2y(1+z^2)^{\frac{1}{2}}}\right]^{\frac{2}{3}} s^{\frac{1}{2}}/n \qquad (48)$$

$$= 1.40\ m/s$$

the discharge $$Q = VA = V(b + zy)y = 1.01\ m^3/s$$

If the actual rugosity is less than expected ($n = 0.8 \times 0.014 = 0.011$) whilst the discharge remains constant at $1\ m^3/s$ then by trial and error the actual flow depth (y) is found to be 0.53 m whilst the velocity (V) increases to 1.67 m/s.

Reducing the flow depth (y) by 3 cm to 0.50 m to allow for a local slope error during construction, the flow velocity is:

$$V = Q/A = 1/(b + zy)y = 1.82\ m/s$$

For the flow to be in the critical state, applying Eq. (17)

$$V_c = (gD)^{\frac{1}{2}} \qquad (17)$$

In the present case of a trapezoidal section in which the flow depth (y) is 0.50 m

$$V_c = \left[\frac{gA}{T}\right]^{\frac{1}{2}} = \left[\frac{g(b + zy)y}{b + 2zy}\right]^{\frac{1}{2}} = 1.84\ m/s$$

It may therefore be seen that with the above safety margin the flow remains sub-critical in the design section.

EXAMPLE 23 - EARTH CANAL

Determine the permissible maximum slope of an earth canal constructed in medium fine light sandy silt to convey $1\ m^3/s$ of clear water with side slopes of 1½:1 ($z = 1.5$) and a bottom width equal to about four times the depth of flow ($b = 4y$).

Applying Eq. (103) as recommended by the US Bureau of Reclamation for clear water conveyed in earth canals:

$$V_s = 0.55\ Cy^{\frac{1}{2}}$$

with $C = 0.84$ (from Table 30), $V_s = 0.462\ y^{\frac{1}{2}}$.

If $y = 0.70$ m, the velocity not to be exceeded (V_s) is 0.39 m/s.

Since $Q = 1\ m^3/s$ and $Q = V_sA$, then $A = 2.56\ m^2$.

In a trapezoidal section:

$$A = (b + zy)y, \qquad \text{hence } b = (A/y) - zy = 2.6\ m$$

The slope of the canal is determined by applying the Manning formula with a roughness coefficient $n = 0.025$ (Table 29)

$$V = R^{\frac{2}{3}}\ s^{\frac{1}{2}}/n \quad \text{and} \quad R = \frac{(b + zy)y}{b + 2y\ (1+z^2)^{\frac{1}{2}}}$$

which yield a value of the slope $s = 0.00024$.

Drops

Where the natural ground slope is steeper than that which can safely be used for the distribution canal, drops are needed to absorb the excess gradient. If the ground in the project area has a relatively uniform slope then vertical drops can be spaced at fairly uniform intervals. The drops should be standardized as much as possible to permit repetitive use of forms. The inlet of each drop should be provided with stop logs, or gates to prevent excessive upstream velocities and to maintain water levels for delivery to the farm gates. The amount of fall at a drop affects the earthwork costs for cut and fill for the canal reach under consideration and the choice of fall should be based on economy. A 1.0 m fall for a vertical drop is usually reasonable. An inclined drop or pipe drop may be more economical if the fall is 2.0 m. Procedures for design of various types of drops suitable for distribution canals are given in FAO (1982). Vertical or inclined drops with stilling basins suitable for particular situations can be designed by using the references given for conveyor canals.

Inverted siphons

For distribution canals, either reinforced concrete pipe, asbestos cement pipe, reinforced plastic mortar pipe or coated steel pipe is generally used as the conduit of an inverted siphon. Rubber gaskets should be used for the joints of reinforced concrete, asbestos cement and reinforced plastic mortar pipes to provide watertightness under pressure and allow for some movement at the joint. The joints of steel pipes should either be welded or have sleeve-type couplings with flexible gaskets to provide for some pipe movement. Buried steel pipe should be coated to reduce corrosion. Availability, useful life and cost factors should be considered and evaluated when selecting the pipe to be used.

The guidelines and procedures described for the design of inverted siphons for conveyor canals should also be followed for the design of inverted siphons for distribution canals, except that the following values should be used for the transition coefficient (K) when computing the inlet and outlet transition losses:

Transition type	K (Entrance)	K (Exit)
Broken-back type between canal and pipe	0.4	0.7
Straight headwall or earth canal to pipe	0.5	1.0

Chutes

Occasionally the general topography along alluvial valleys is such that the project irrigable lands are separated by remnants of older valleys or other steep ground. A distribution canal offtaking from a conveyor along the higher lands frequently has to be carried down the face of such steep areas as it is continued across lower-level project land. The distribution canal flow may be carried down such steep areas in an open chute section to a terminal stilling basin.

The discharge in a distribution canal can vary over a wide range during the year and under such conditions a chute that is not properly designed can generate waves and slug type flows that will cause unstable hydraulic conditions. It is necessary to check a chute design to assure that its operation will be stable over the full range of flow. The procedure for making this check suggested by USBR (1974) is as follows:

Step 1 Determine the uniform flow depth (y_0) on the chute for 20 percent, 50 percent and full design flow. To each discharge corresponds a gradually varied flow profile (S2) passing through the respective critical depth (y_c) at the chute entrance and, if the chute is long or steep enough, the uniform depth (y_0).

Step 2 For each discharge divide the chute into reaches with depths such that the velocity increases by increments of about 10 percent, from the critical velocity (V_c) at the entrance of the first reach, to the uniform velocity (V_o) at the end of the last reach. This is the initial stage of the direct step method for the computation of the gradually varied flow profile.

Step 3 Calculate the average friction slope ($\bar{s}_f$) in each reach for each discharge and hence the Vederhikov number ($\underline{V}$) with

$$\underline{V} = \frac{2}{3}\ \frac{b}{P}\ V/(gD \cos \theta)^{\frac{1}{2}} \tag{87}$$

where: $\theta = \sin^{-1} \bar{s}_f$
and with b, P, V and D at the end of the reach.

Step 4 Calculate the Montuori number ($\underline{M}$) with

$$\underline{M}^2 = V^2/g\bar{s}_f\ \Delta x \cos \theta \tag{88}$$

where: Δ x = length of reach.

Step 5 Refer to Figure 96 and plot the computed values of $\underline{V}$ and $\underline{M}^2$. If they fall in the slug zone, check intermediate points to determine where waves begin to form.

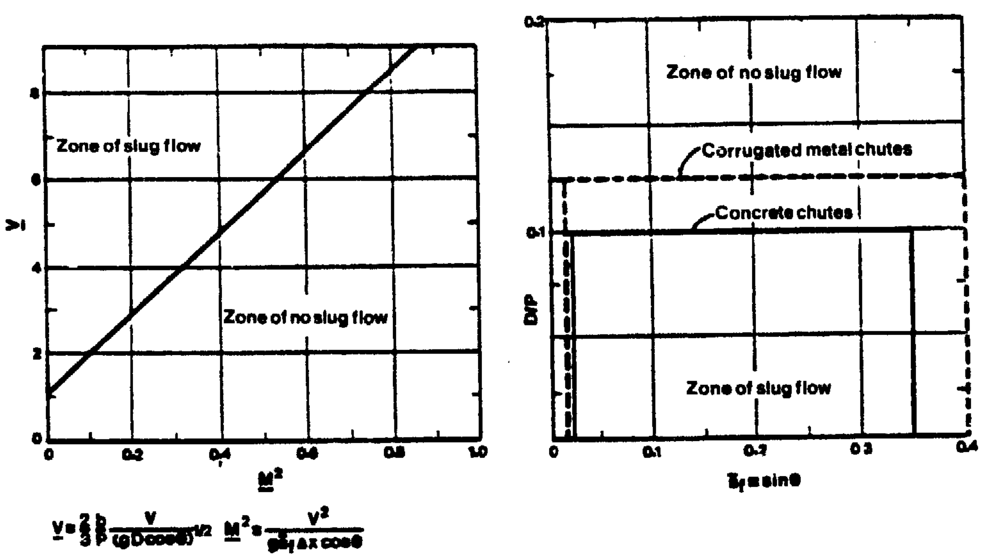


Figure 96 Criteria for slug free flow

Step 6 Calculate D/P, the shape factor for the chute section under consideration, and plot against $\bar{S}_f$. Waves will be generated if points fall within the slug zone of both charts.

If it is found that slug flow is likely to occur then design changes are required which can include steepening of the chute; use of a succession of shorter chutes each with a stilling basin; replacement of the open chute by a pipe or use of a triangular or V-shaped chute section.

5.4 REGULATION OF CANAL NETWORKS

5.4.1 Introduction

In a conveyor or distribution canal, the discharges, hence the water depths, are highly variable with time. The fluctuations originate from increases or decreases of the demands by users because of changes in weather conditions, disturbances in a reach of the canal due to a bank collapse or excessive runoff or any other unpredictable phenomenon that may happen along a canal.

Therefore, to provide timely the required discharges to the users and, at the same time, to avoid unnecessary waste by spillover, there is the need for regulation.

It is beyond the scope of this manual to deal with large canals which are generally of a multipurpose nature but rather limit the review to small canals having a capacity of up to 5 m^3/s.

5.4.2 Regulation of Small Canals

i. Characteristics and general requirements

Definition

Canals having a capacity of up to 5 m^3/s are generally classified as small canals. The majority of distribution canals fall into this classification. They cover the water requirements of irrigation projects up to 5000 hectares in size.

Small canals may receive water from a large canal or be supplied directly by a diversion weir on a stream, by a pumping station located on a river or by pumping from a well field.

The economic feasibility of a small project generally puts ceilings on the investment that can be justified for construction of the canal system and cost frequently becomes an important consideration in planning the system of structures and facilities to be provided for measurement, regulation and protection.

Regulation requirements

Small canals have a low hydraulic radius and require relatively steep bed slopes to maintain acceptable velocities. Small canals also offer little storage potential due to the narrow water width. These factors make regulation of a small canal relatively more difficult to achieve than is the case for large canals. The basic regulation structures which must be provided in a small canal system include gates or stop logs for controlling the

EXAMPLE 24 - CHECK FOR SLUG FLOW IN CHUTE

A chute has the following characteristics: Rectangular section, length $x = 100$ m, bottom width $b = 0.6$ m, rugosity (Manning) $n = 0.010$, discharge $Q = 1$ m³/s, difference in elevation from entry to exit = 24.5 m.

Check that slug flow does not occur for design discharge.

Step 1 The critical velocity (V_c) occurs in a rectangular channel when

$$V_c = (gy_c)^{\frac{1}{2}} \tag{17}$$

hence the critical depth (y_c) at the entrance is

$$y_c = [(Q/b)^2/g]^{\frac{1}{3}} = 0.66 \text{ m}$$

and the critical velocity (V_c) is

$$V_c = Q/A_c = Q/by_c = 2.78 \text{ m/s}$$

The uniform depth (y_o) on the chute can be determined by substituting Q/A for V,

$$R^{\frac{2}{3}} A = nQ/s^{\frac{1}{2}} \tag{48b}$$

and since for a rectangular channel

$$A = by \quad \text{and} \quad R = b + 2y$$

$$0.6y\,[0.6y/(0.6 + 2y)]^{\frac{2}{3}} = 0.01/0.245^{\frac{1}{2}}$$

which is solved by trial and error when $y_o = 0.1545$ m at which depth the uniform velocity (V_o) = 10.79 m/s.

Step 2 The profile of the nappe is of the S2 type. Its shape is determined by arbitrary selection of water depths between the limits $y_c = 0.66$ m at the entrance and y_o = 0.1545 as shown in Table 38.

Steps 3-6 The Vedernikov and Montuori numbers are shown for each reach together with the shape factor in Table 38. Referring to Fig. 96 it may be seen that all the above points fall outside the zones of possible slug flow.

The procedure is repeated for 50 percent and 20 percent of the design discharge and it is found that slug flow does not occur on the chute for these flows.

Table 38 VEDERNIKOV (V), MONTUORI (M) NUMBERS AND SHAPE FACTOR (D/P)
Design discharge Q = 1 m³/s

y (m)	V (m/s)	$\bar{s}_f$	x (m)	x (m)	V	M²	D/P
0.66	2.78						0.34
0.55	3.03	0.008	0.10	0.10	0.3	1171	0.32
0.50	3.33	0.009	0.20	0.30	0.4	609	0.31
0.46	3.62	0.012	0.27	0.57	0.5	429	0.30
0.42	3.97	0.014	0.41	0.98	0.5	271	0.29
0.38	4.39	0.018	0.61	1.59	0.7	174	0.28
0.34	4.90	0.024	0.93	2.52	0.8	108	0.27
0.30	5.56	0.033	1.46	3.98	1.1	65	0.25
0.26	6.41	0.048	2.44	6.42	1.4	36	0.23
0.23	7.25	0.068	3.12	9.54	1.8	25	0.22
0.21	7.94	0.091	3.34	12.88	2.2	21	0.21
0.19	8.77	0.119	5.49	18.37	2.6	12	0.19
0.17	9.80	0.161	11.34	29.71	3.3	5.5	0.18
0.16	10.42	0.203	14.90	44.61	3.7	3.7	0.17
0.155	10.75	0.231	26.36	70.99	3.9	2.0	0.17
0.1545	10.79	0.243	27.91	98.90	3.9	1.8	0.17

diversion of water to the heads of the canals and deliveries to farm turnouts as well as provisions for checking the water surface to assure that the diversions and deliveries can be made over the full range of canal flow.

A higher degree of regulation, approaching that of a fully automated system, can be achieved by increasing the canal freeboard, adding additional checks, automating the gates, adding water level sensors and controls, measuring devices, and a remote control and monitoring system. Project factors to be considered when justifying a system for providing a high degree of regulation and automation include the following:

- An irrigation system of concrete lined canals and/or concrete or steel pipes is planned.

- The water resource is scarce or costly and a high level of control is required for efficient allocation of the available supply with minimum wastage.

- The irrigated land is to be fully developed, making use of efficient on-farm equipment.

- The project is expected to produce high value crops and thus justify the additional irrigation system costs.

Regulation systems

There are three general types of regulation systems for small canals. These include manually operated, semi-automated, and fully automated systems.

In manually operated systems a ditchrider, water bailiff or canal operator adjusts the gates that control the diversions, adjusts the stop logs or gates at checks or regulators, sets the farm turnout gates and makes any necessary adjustments to wasteways in accordance with the deliveries scheduled for the day.

In semi-automated systems the main canal headgate or pumped supply is automatically controlled from sensors at an upstream check and main canal check gates are provided which respond either hydro-mechanically or electrically to upstream and downstream water levels. The turnouts to the distribution canals are of the modular type, and farm turnouts are operated manually.

In fully automated systems remote control and telemetering are added, water level sensors and measuring devices located at strategic locations to monitor water levels and discharges, the check gates in the distribution canals are automatic and respond either hydro-mechanically or electrically to changes in upstream or downstream water levels and farm turnouts are of the modular type.

ii. <u>Manual regulation</u>

The design of a small canal network for manual regulation usually includes the following structures and facilities for water level control, flow control and protection of the systems.

Headworks

Provisions must be made for control at the head of the canal system. If the canal is supplied from a diversion weir across the stream, the headworks may have slide gates or a radial gate to control the flow.

Since streams are subject to flood flow before gate settings can be changed, a wasteway must be provided at the upstream end of the canal. The wasteway can take the form of a simple overflow structure with a return channel to the stream and the control can either be a check with underflow gates or a gated entrance to some other type of structure to ensure orifice flow. Design procedures for gated checks are described in FAO (1982). The wasteway can be a concrete-lined section designed to function as an overflow weir to skim off the excess flow.

A rating section or measuring flume is usually incorporated in the canal near the headworks for measuring the flow.

EXAMPLE 25 - DESIGN EXAMPLE: THE OVERFLOW WEIR

Assume that the headworks gate is set to discharge 3 m^3/s under a head of 1 m and that at design flood level the head increases to 4 m. If the gate setting remains unchanged, the diverted flow will increase to 6 m^3/s.

Assume that the gate at the check control will permit the water level to rise 0.3 m without increasing the downstream discharge by more than 10 percent.

If the overflow is set 0.1 m above the design flow level, the side overflow weir must be about 50 m long.

Pumped supply

If the canal is supplied by pumping, the discharge is regulated by controlling the pump operation.

Check structures

Check structures are required at strategic locations for water-level control to assure deliveries to distribution canals and farm turnouts over the full range of canal discharges.

In main canals, check structures, either as separate structures or in combination with other structures, are usually located just downstream of each turnout to a distribution canal.

Check structures are also installed in distribution canals and at locations in main canals to assure deliveries to the farm turnouts.

The number and locatin of check structures are determined by projecting the checked level water surface upstream from a check location and an additional check is located just downstream of the first farm turnout whose command level falls above the checked level water surface. This procedure is continued until each turnout can be commanded from a checked water surface. The checks have stop logs or slide gates for controlling the level of the water surface. Designs for the check structures are given in FAO (1982).

Turnouts, farm turnouts (or outlets)

Turnouts, or intake structures, are required at the head of the distribution canals and to serve the farm plots. The turnouts have slide gates, module type flumes or module type gates to control the flow. Designs for various types of turnouts, intakes or modules are given in FAO (1982).

A slide gate fitted to an intake structure with a short length of pipe through the canal bank makes a satisfactory turnout. The pipe may be set so that the gate can function as a metering gate for measuring the disharge.

Measurement

Measurement structures such as meter gates, measuring flumes or weirs are usually provided at the head of each distribution canal and at each farm turnout for measuring the flow. Designs for measurement structures are given in FAO (1982).

Wasteways

Wasteway structures are required at the downstream end of each canal. A check structure or stop log controlled opening is frequently used to serve as a wasteway at the ends of canals. If the canal is long a wasteway should be provided with a capacity equal to at least 10 percent of the canal flow. An overflow section should be added to the wasteway if the canal is supplied by diversion from a stream.

iii. <u>Semi-automatic regulation</u>

A small canal system designed for semi-automatic regulation requires the same basic regulatin and control structures as a system designed for manual regulation except that the main control gates are automated. This can be achieved either by means of hydro-mechanical or by electrical electronic devices. The control gates of an earth canal system can be designed for electrical operation if a power supply is conveniently near, but a small canal system, when semi-automated, normally has concrete lined canals since one of the main reasons for providing automatic gates is to reduce the conveyance losses. Hydro-mechanically operated gates should only be installed where the canal system is concrete-lined.

Headworks

If power is available at the headworks, the headworks gate can be electrically operated, with water level sensors and electronic equipment providing automatic control of the gate. If power is not available, the control check and wasteway at the head of the canal for disposal of excess diverted flow through the headworks should be provided with automatic gates. A hydro-mechanically automated gate with downstream control at the control check automatically maintains a constant downstream level. Excess inflow is evacuated by the overflow weir and returns to the stream. A wasteway and control check is usually provided to protect the canal in case of power failure during floods.

Pumped supply

If the source of the irrigation water supply is a pumping station, the pumps can be remote controlled and operated from the upstream check. In this case water level sensors in stilling wells start and stop the pumps in a programmed sequence.

Checks

To maintain a constant upstream level the main canal checks are designed for either hydro-mechanical automation, or electrical operation if power is available along the canal route. The distributary turnouts, or intakes, should be of the module type.

Details of hydro-mechanically operated gates and module-type gates are given in FAO (1982).

Distributary canals

The distributary canals and farm turnouts are normally regulated manually.

iv. Fully automatic regulation

A small canal system designed for fully automatic regulation has the same basic control and regulation structures as a small canal system designed for semi-automatic or manual regulation, except that the terminal reaches of the canals and distributaries are designed with higher canal bank levels to provide some terminal storage.

Remote control and telemetering

A fully automatic system can be designed for remote operation of the supply system (either a headgate or a pumping plant) and remote operation of the distribution canal turnouts, or intakes. The headgate (when the small canal system is supplied by diversion from a stream or parent canal), and the distribution canal intake gates are motorized for electrical operation by remote control from an operation centre.

Stilling wells and water level sensors are placed at the check structures and at other strategic locations and a telemetering system installed for monitoring the entire system from the operation centre.

Checks

The check gates on the main canal of a small network can be motorized for electrical operation. The gates are controlled locally by stilling wells, water level sensors, gate position sensors, timers and micro-switches. Provision is made for override by manual or remote control.

The check gates on the distributaries may be operated either by hydro-mechanical or electrical electronic automation.

Farm turnouts

The farm turnouts can be equipped with module type gates.

5.4.3 Regulation of Pumping Stations Supplying Irrigation Canals

i. Characteristics and general requirements

Definition

Pumping stations supplying irrigation systems may pump water from a river, reservoir, canal, well field or single well to a gravity canal or may relift water from a low level canal to another situated at a higher level.

A pumping station may be operated by an attendant at the plant or by remote control. Irrigation needs frequently vary widely over the year and during the day. Pumping stations must therefore be flexible in order to meet varying discharges. The regulation of pumping stations must respond to varying needs and conditions.

Protective controls

Pumping stations must have automatic controls for protection of the pumps against inadequate submergence. The pumphouse must be protected from flooding should the flow be obstructed in the outlet channel. The protective control against inadequate pump submergence usually consists of a stilling well in the pumphouse intake in which a float and switches are arranged to shut down a pumping unit automatically if the water level in the intake drops to a certain level. In multi-pump installations the switches are arranged to shut down the pumps in sequence with a time interval between unit shutdowns calculated to minimize the flow disturbance. Alarm systems are normally associated with the unit shutdowns to alert the operation staff of the emergency situation.

If the pumps discharge into a canal, a stilling well is provided near the outlet structure with a float and switches arranged to shut down the pumping units automatically in a timed sequence when the water level in the outlet canal rises above a certain level. Alarm systems warn the project operation staff of the emergency conditions.

Flow Measurement

If the pumps discharge into a canal a rating flume or gauging section is frequently located a short distance downstream of the outlet structure for measuring the discharge. A stilling well with a float gauge and sensors can be provided at the measuring flume with visual indication in the pumphouse for the operator if the regulation is manual.

Regulation methods

Pumping stations can be either manually regulated by a pump attendant or automatically regulated with either local or remote control.

Number of pumping units

The number of pumping units in a pumping station should be selected so that the annual variations of canal discharges can be met. If the pumps are of the centrifugal type then valves can be used to throttle the discharge to the desired flow. Centrifugal pumps are usually associated with pressure pipe systems operating under relatively high heads.

If the lift is relatively low the pumps are usually of the propeller or mixed flow type. These cannot be throttled by any significant amount and a large number of pumps are required to meet variations in canal flow. It is convenient to use batteries of pumps of various sizes to facilitate different combinations. For example, if a pumping station is to supply a maximum of 4.9 m^3/s to a canal and the minimum flow in the canal is 1.2 m^3/s, a total of 7 pumps might be installed, two of 1200 l/s, two of 800 l/s and three of 300 l/s each. Such an arrangement allows for combinations which can meet almost all flow variations between the maximum and minimum demand.

ii. <u>Manual regulation</u>

If regulation of the pumping station is to be achieved by manual methods various controls, visual guides and alarms are required to aid the operator.

Communication

A system of telephone or radio communication between the canal operator and the pumping station operator is required to ensure good coordination.

Visual aids

The pump operator must be provided with a visual indication of the water levels and discharge rates in the canal in order that he may adjust pump flows.

iii. <u>Automatic regulation</u>

In automatic operation the canal reach just downstream of the pumping station can be operated as a storage section. The pumps discharge into this reach and the water level is allowed to fluctuate over a range of about 0.5 m. A check gate with downstream control releases the flow to maintain a constant downstream level.

Local operation

In the case of local operation the gate downstream of the storage reach can be manually set to operate at the desired discharge. The pump combination that most nearly approaches the desired discharge can also be selected.

A stilling well with float, switches and controls is provided to start and stop the pumps within the limits of the variation of water level in the storage reach.

As an example let it be assumed that a canal discharge of 3 m^3/s is to be maintained and that two pumps each with a capacity of 2 m^3/s are available to maintain this discharge. The two pumps will operate together until the 0.5 m rise in the storage reach has been attained at which time the float in the stilling well will cause one of the pumps to stop. The inflow then is 1 m^3/s less than the fixed outflow, causing the level in the storage reach to fall until the float starts the second pump again. If the storage reach is one kilometre long and four metres wide the pumps will start and stop about once every hour.

Remote control operation

In remote control operation the discharge setting of the measuring gate downstream of the storage reach can be set by remote control from the control centre. The pumps that are to be operated and applicable controls can also be selected by remote operation.

Water level sensors and telemetering are provided for monitoring the flows and water levels from the control centre.

Remote control allows more flexibility in operation than local control.

6. SPECIAL CONSIDERATIONS - WATER HAMMER, GYPSEOUS SOILS, CORROSION, MATERIALS AND EQUIPMENT

6.1 WATER HAMMER

6.1.1 Introduction

Water hammer, which consists of oscillations of pressure both positive and negative due to the rapid modification of the flow in a closed conduit, is a phenomenon which is difficult to grasp intuitively. The pressure oscillations travel along the conduit from one extremity to the other in periodic movement.

The most frequent causes of water hammer are:

- sudden shutting off of the pump supplying water to the conduit;

- near instantaneous closure of a valve or of an irrigation hydrant;

- presence of pockets of air in the pipe network.

The hazards caused by water hammer are various:

- danger of high pressure: the excess pressure resulting from water hammer is added to the initial pressure. If the total pressure is greater than the maximum permissible pressure in the pipe there is a risk of pipe rupture and/or joint failures;

- danger of negative pressure: negative relative pressures may result from water hammer. If these pressures are below -10 m of water column, a pocket of cavitation or water vapour is created, with possible collapse of the conduit wall or sucking in of the joint seals. The pipe lining material may also be damaged;

- risk of pipe fatigue resulting from rapidly alternating positive and negative pressures which singly may not lead to actual failure of the conduit.

The pressure in a pipe varies along its length. It is therefore necessary to compare along the whole length of the pipe:

- the maximum pressure and the maximum permissible pressure;

- the minimum pressure and the saturated vapour pressure of water.

This leads to the use of two terms which frequently recur when dealing with water hammer:

- "envelope curve of high pressures" or "envelope curve of maximum piezometric heads";

- "envelope curve of low pressures" or "envelope curve of minimum piezometric heads".

These two curves facilitate the determination of the critical points in a pipe, as shown in Figure 97. The curve of maximum pressures is compared to the line of maximum permissible pressures (ground elevation + maximum permissible pressure in the pipe) and the risk of pipe failure is examined. The curve of minimum piezometric heads is then compared to the line of minimum permissible pressures (ground elevation - 10.33 m of water column - safety margin) to evaluate the risk of cavitation.

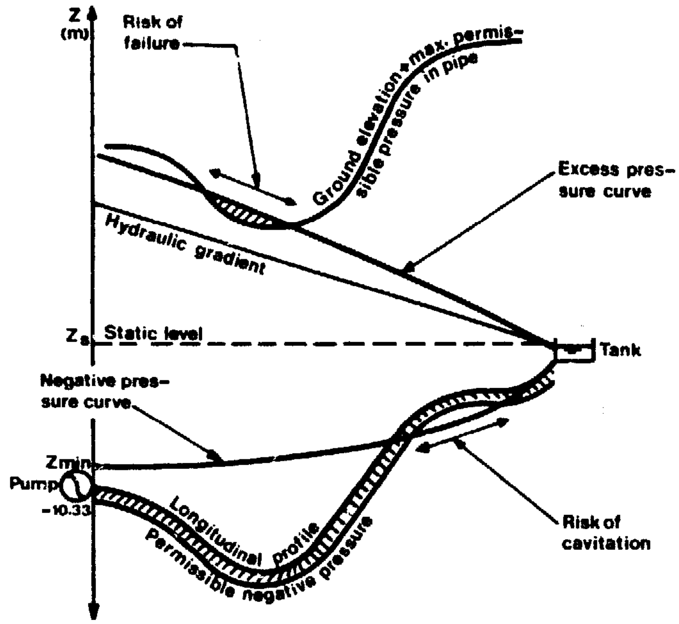


Figure 97 Envelope curves of high and low pressures

The effect of a quasi-instantaneous closure of a valve at the extremity of a pipe may be compared to the behaviour of a spring thrown against a wall on which it remains fixed (Fig. 98a). The following description is based upon the oscillation of a wave, the physical behaviour of the spring being analogous to that of the water.

- In the first place the spring is progressively compressed starting at the point of impact. In the same way the pipe diameter progressively expands starting at the valve due to the effect of the excess pressure P (Fig. 98b).

- Compression of the spring continues until during an instant the whole spring is compressed. The same phenomenon occurs in the pipe until instantaneously its whole length sustains the excess pressure (P_o + P) and the velocity of the water is zero (Fig. 98a).

- The rebound then starts: the spring expands starting at its free end in a direction opposed to that in which it was compressed, at the same velocity as during compresion. The excess pressure in the pipe is dissipated in the reservoir (increase of reservoir water level) and the pipe progressively recovers its initial diameter while the water flows ($V = -V_o$) in the reverse direction (Fig. 98d).

- The spring continues to expand until for an instant it recovers its initial shape. Similarly the whole pipe recovers its initial diameter whereas the water continues to flow in the reverse direction (Fig. 98e).

- The relaxation of the spring continues progressively along its whole length and the anchor is placed under tension. A negative pressure occurs in the pipe in the vicinity of the valve, due to

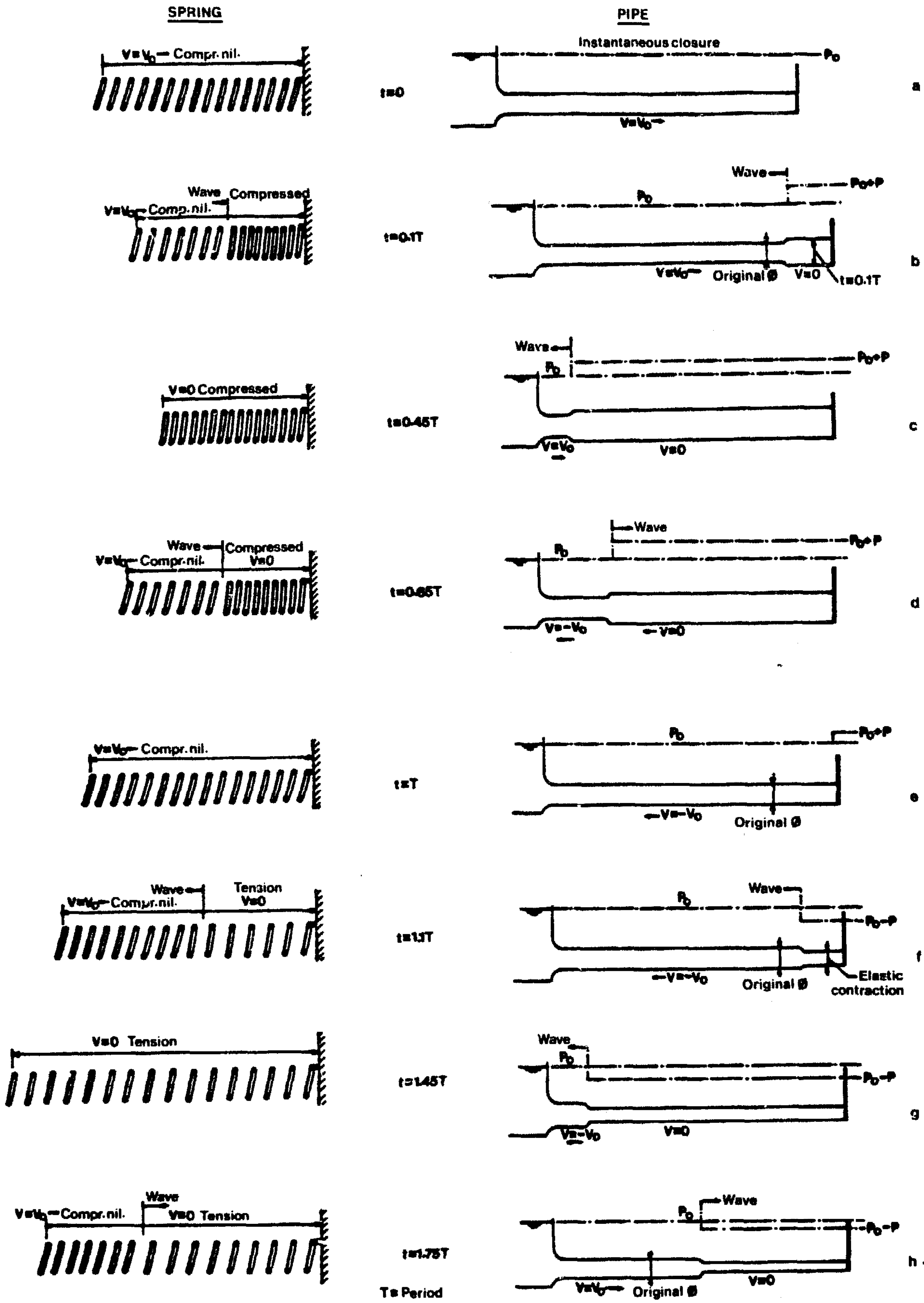


Figure 98 Visualization of effect of water hammer on a pipe

the reverse flow of the water being halted at this point, with a resulting decrease of the pipe diameter (Fig. 98f).

- The spring extends until it is under tension throughout its length. The negative presure and zero velocity extend along the whole length of the pipe, accompanied by a reduction of the pipe diameter (Fig. 98g).

- Starting at its free extremity (Fig. 98h), the tension in the spring is released until it recovers its initial length and the cycle is resumed with the spring being compressed in the direction of the point of anchorage at the initial velocity (Fig. 98b); in the same way , the low pressure wave travels to the tank where the pipe recovers its original diameter with a resulting flow of water from the tank to the valve. The cycle then starts again with a resulting high pressure at the valve when the reflux wave reaches the valve.

The same phenomenon will occur if the pump supplying water to the pipe is shut off suddenly. In this case the water hammer wave travels from the pump to the tank.

6.1.2 Protection Against the Effect of Water Hammer

Before deciding upon the dimensions of the pipes in a network the need for water hammer protection must be examined. Only the protection afforded by an air pressure vessel will be examined here and the reader is advised to consult the specialized literature on the subject for a more complete analysis of the water hammer phenomenon (Meunier 1970; FAO/IBRD 1983; Meunier and Puech 1977).

The following method can be used to determine the magnitude of the low pressures involved and evaluate rapidly the possibility of resorting to air pressure vessels. The method, although approximate, affords a high degree of security.

i. Determination of the wave amplitude

If no account is taken of cavitation or pump inertia upon sudden shut down, the value of the instantaneous low pressure is given by:

$$P_H = C V_o/g$$

where P_H = the resulting low pressure in the pipe (m)
C = the wave celerity (m/s)
V_o = initial velocity of the water (m/s)
g = acceleration due to gravity (m/s²)

The order of magnitude of the wave celerity (C) varies between 150 and 400 m/s in plastic pipes and from 1000 to 1200 m/s in asbestos cement, steel, cast iron and concrete pipes in that order. The wave celerity cannot exceed 1430 m/s which is the velocity of sound in water.

The low pressure travels along a line parallel to the piezometric level (Fig. 99). In view of head losses it can be assumed as a first approximation that the lowest pressures occur at the end of the first outward movement of the wave.

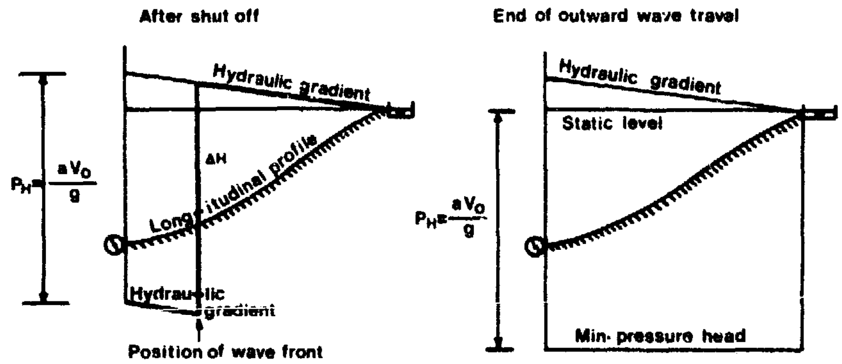


Figure 99 Variation of the theoretical piezometric level in the absence of protection

An elementary form of protection can be assured by means of an auxiliary intake which raises the low pressure to a level equal to that of the intake pipe (Fig. 100). Two distinct levels of intake pipe are indicated. It should be noted however that this solution is only valid in the case where the delivery head is less than 10.33 m. Moreover the intakes must be situated as high as possible. Generally, in irrigation networks, the delivery head is greatly in excess of 10.33 m but this type of protection may be useful in specific situations such as that of a floating pumping station in a river where the stage variations are large.

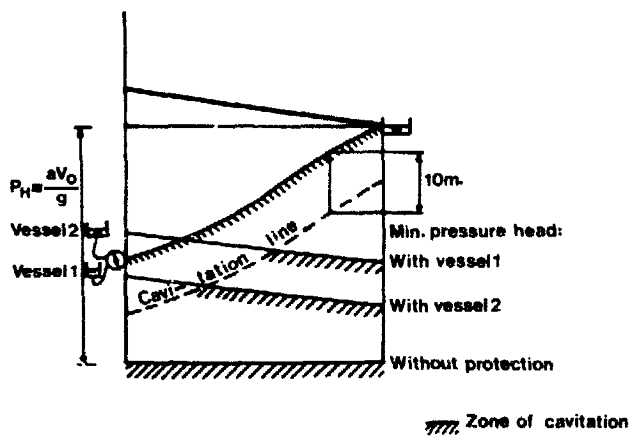


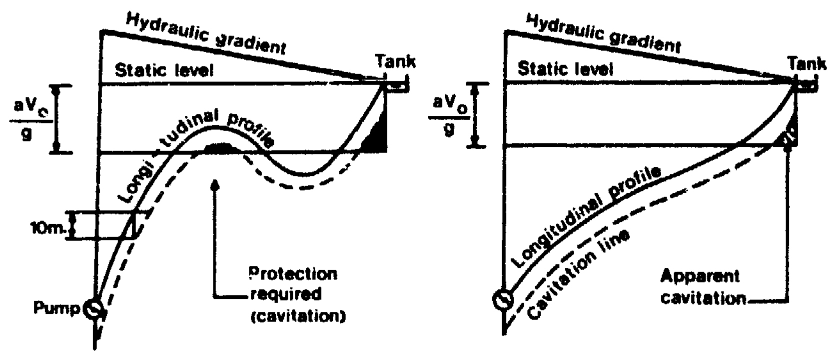
Figure 100

Comparison of minimum piezometric levels

ii. Case of a pump delivering to a reservoir through a pipe

When the delivery head is high and the low pressure which develops in the pipe is greater than the static level (zero discharge), the protection afforded by an auxiliary intake is no longer sufficient because the length of pipe under low pressure is too great (Fig. 100). In this case another form of protection is required, such as an air pressure vessel, provided that there is no risk of cavitation at the pump.

If there is a critical high point in the delivery pipe then some form of protection will be necessary (Fig. 101a). If there is no critical high point then following the reasoning held so far, only the end of the pipe will be subjected to low pressures (Fig. 101b).



(a) (b)

Figure 101 Identification of zones of cavitation

The foregoing reasoning does not take into account:

- pump inertia and linear head losses;
- the return wave resulting from the presence of the reservoir and which tends to rapidly annul the low pressures since the tank behaves like a surge tank.

iii. Case of a branching network

Most networks depart from the above simple case of a pump feeding a reservoir through a single pipe. The problem can be simplified with some loss of accuracy by assimilating the branching network to an equivalent single pipe.

The criterion of equivalence generally adopted is the kinetic energy of the water:

$$E = \frac{1}{2} m V_o^2$$

where m = mass of water
V_o = initial velocity of the water

Since $Q_o = A V_o$

with A = pipe cross sectional area
Q_o = initial discharge in pipe

then

$$L Q_o^2/2 g A = L Q_o V_o/2 g$$

where L = length of pipe from reservoir to pressure vessel

The principle of equivalence usually consists in replacing the branching network by an equivalent pipe whose three parameters, length, discharge and velocity, are the sums relative to each section (n):

$$L\, Q_o\, V_o = \sum_{n=1} L_{(n)}\, Q_{(n)}\, V_{(n)}$$

This approach is approximate and an analysis of the complete network must be made. This analysis is however facilitated by the fact that the order of magnitude of the volume of the air pressure vessel is known.

6.1.3 Equation for Determination of Volume of the Pressure Vessel

The equation which relates the elevation and discharge between the air pressure vessel and the extremity of a pipe (valve, reservoir) may be written:

$$Z_b \pm \frac{C\, Q_b}{g\, A} = Z_e \pm \frac{C}{gA} Q_e + \int_o^L j dx$$

where Z_b, Z_e = piezometric levels of the vessel and pipe extremity respectively (m)
Q_b, Q_e = discharge of the vessel and pipe extremity (m^3/s)
C = wave celerity (m/s)
g = acceleration of gravity (m/s^2)
A = cross section of pipe (m^2)
$\int_o^L$ = head loss over length (L) of pipe (m)

At the air pressure vessel, the continuity equation of the discharges is:

$$\Delta v = Q_o\, \Delta t$$

where Δv = variation of volume in the vessel (m^3)
Q_o = flow in or out of the vessel (m^3/s)
Δt = time interval = L/C (time of propagation of the waves in the pipe

The change of volume of the air in the vessel takes place under thermodynamic conditions situated between the limits of adiabatic exchange ($\gamma = 1.4$) and constant temperature exchange ($\gamma = 1$). A value of $\gamma = 1.2$ can be adopted, thus:

$$Pv^{\gamma} = Pv^{1.2} = \text{Constant}$$

where P = pressure
v = volume of air

If the time interval retained is equal to the time of propagation of the waves in the pipe, the variation of pressure with respect to the initial state may be expressed as:

$$\frac{\Delta P_H}{P_H} = -\gamma \frac{\Delta v}{v_o} = -\gamma \frac{Q_o L}{C v_o}$$

with Δv = variation of volume in the vessel = $Q_o(L/c)$
v_o = initial volume of air in the vessel

The following non-dimensional variables allow a better understanding of the phenomenon of water hammer (see also Fig. 102).

$A = CV_o/gP_s$ is a measure of the wave oscillation or of the instantaneous low pressure without an air vessel with respect to the static pressure (P_s)

$B = LQV_o/gP_Hv_o$ indicates the relative importance of the pipe-vessel inertia and hence the capacity of the vessel to protect the system (inversely proportional to P_Hv, hence the smaller is the value of B the larger will be the vessel)

$K = (P_H-P_s)/P_s$ represents the importance of the head losses in the pipe.

These non-dimensional variables have been used to draw up tables for the dimensioning of air pressure vessels (Meunier 1980).

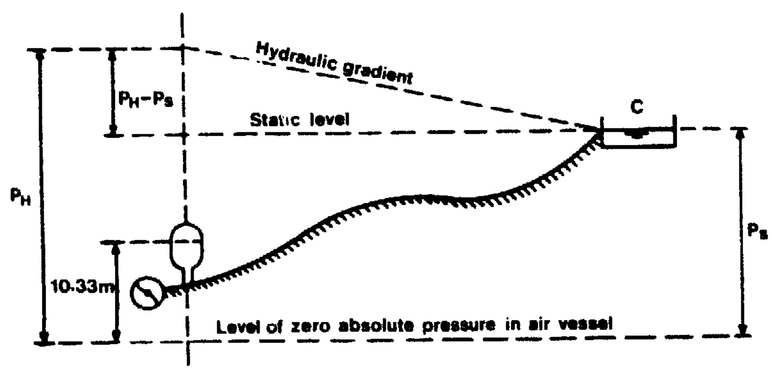


Figure 102 Network protected by an air-pressure vessel

6.1.4 Description and Operation of the Air-Pressure Vessel for Protection Against the Effect of Water Hammer

The air-pressure vessel used to protect networks from the effect of water hammer consists of a shell, generally made of steel, which initially contains a certain volume of water above which is a cushion of air under pressure.

Installed on the pipe, these vessels are frequently fitted with a device which produces a head loss at the entrance (Fig. 103). This head loss is:

- as small as possible in the direction vessel to pipe
- appreciable in the direction pipe to vessel.

Technologically, two types of vessel may be distinguished:

- Membrane vessels: the air and the water are not in contact and the air does not dissolve in the water. The vessel is pressurized at the required pressure prior to placing the network or pipe in operation.

- Non membrane vessels: the air tends to dissolve in the water and a compressor is therefore required to automatically adjust the volume of air in the vessel. These vessels can be manufactured by local artisans.

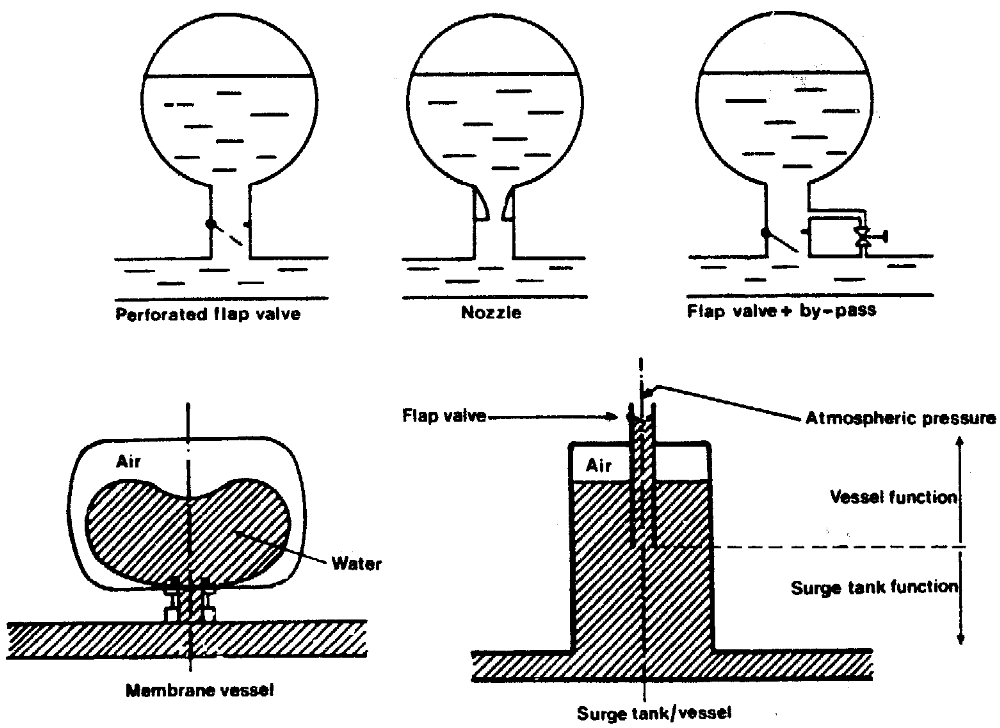


Figure 103 Types of air-pressure vessels for water hammer protection

A third type of vessel which is only suitable for low delivery heads consists of a vessel/surge tank. It operates as a surge tank when the pressure falls below a given threshold value and as an air vessel when the pressure is greater than this value.

6.1.5 Ancillary Protection

Two further types of water hammer protection measures are required for irrigation networks. The first consists of protection against too rapid a closure of valves or irrigation hydrants, the second caters for the evacuation of air, particularly during filling of the network.

i. Closure of valves or hydrants

Rapid closure of a valve gives rise to pressure surges and it is necessary to provide protection against these by installing pressure-relief valves particularly in the case of networks consisting of PVC, concrete or asbestos-cement pipes. These devices should be set to operate when the pressure exceeds the static pressure by five metres of water column. Experience shows that pressure relief valves should be located some 500 m from the extremity of each branch and at 2 km intervals on the main network.

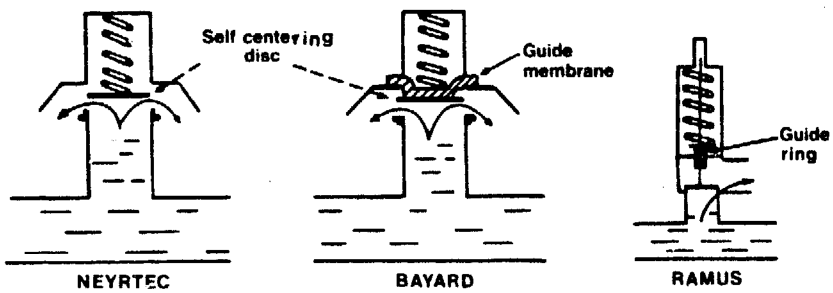


Figure 104 Operating principles of three types of pressure relief valve

Where the network consists of steel or ductile cast iron pipe such protection is not absolutely essential since the maximum operating pressure is of the order of 30 m of water column whilst the static pressure in the network is generally about 12 m.

ii. Evacuation of air

The presence of air in the pipes of a pressure network is undesirable since it increases the head losses and the movement of air pockets may interfere with the flow regime. When the air/water interface reaches a valve there is a sudden reduction in flow velocity which can give rise to severe water hammer. The severity of the water hammer will be notably increased if the air pocket is evacuated by a hydrant since it will lead to rapid filling of the pipe.

In order to obviate the movement of air pockets, bleeder valves can be installed at the high points of the network. The diameter of the air evacuation orifice should not exceed 1/63rd of the pipe diameter. In order to evacuate air during the filling phases a large diameter orifice must be provided. The diameter of this orifice is generally 1/20th of that of the pipe for filling at low discharge (1/10th of the design discharge) or at low velocity (less than 5 m/s). These two constraints have led to the use of double-acting bleeder valves (Fig. 105).

A large orifice caters for high rates of entry and exit of air during filling or drainage of the network whilst a second orifice ensures degassing and evacuation of air pockets during the last stages of filling.

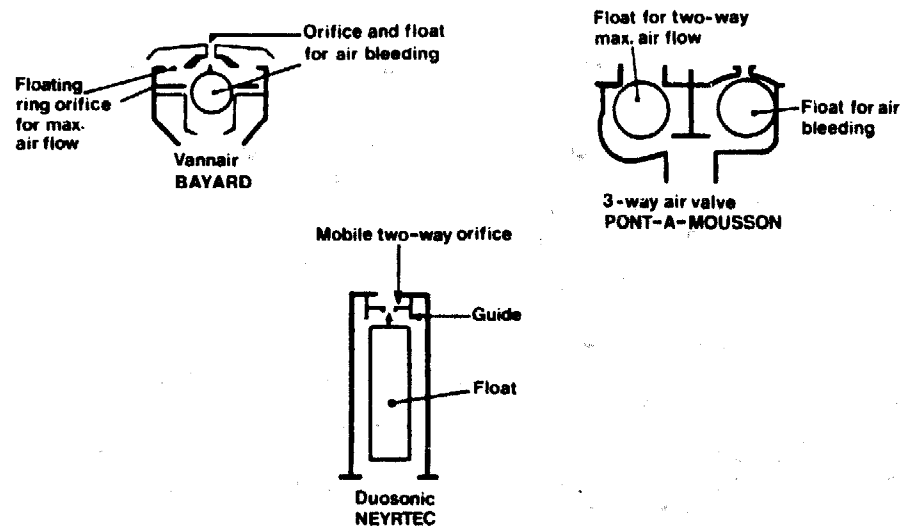


Figure 105 Operating principles of double-acting bleeder valves

6.2 GYPSEOUS SOILS

6.2.1 Introduction

When an irrigation project is planned in an area where the regional geology and rock formations indicate a past or present potential for gypsum deposits, a soils investigation programme should be carried out along the proposed conveyor alignment including visual examination for evidence of gypsum in the soil mass. A close examination of the sides of recently excavated ditches or eroded gullies will often reveal cystals of gypsum in gypseous soils.

Gypsum is composed essentially of calcium sulphate loosely combined molecularly with water in the ratio of two parts of water to one of calcium sulphate. Gypsum is somewhat soluble and a saturated solution contains about 0.25 percent gypsum.

Gypsum in solution is transported by groundwater flow and as the groundwater rises due to capillarity the gypsum is carried upward toward the ground surface and is precipitated in the soil mass due to evaporation. If there is insufficient space in the interstices of the soil to accommodate the crystals the crystallization force will expand the voids to make room.

Rainfall is pure, or slightly acidic, and will dissolve the gypsum on the ground and for a distance below the surface and will tend to undo the work of evaporation. Consequently, if gypsum is present in the parent rock and finds its way into the groundwater, there will be a build up of residual gypsum leading to the formation of gypseous soils in regions where rainfall is sparse.

Gypseous soils are frequently encountered in arid regions of Iran, Iraq, Syria, parts of Russia and other arid areas. Surface soils of low density and vesicular structure are frequently found in gypseous soil regions where the soil has been leached by rainfall or flooding. These soils still have some gypsum in the interstices and exhibit high strength when dry, but tend to collapse when inundated or saturated and load is applied. This is a very deceptive property that has led to collapse of irrigation canals and structures when projects have been put in operation.

Laboratory tests to detect the gypsum content in a sample are usually carried out by adding distilled water to the sample and mechanically mixing for 4 or 5 days. The sample is then chemically analysed for sulphate and calcium content.

6.2.2 Design Criteria and Treatment Procedures

If an irrigation project is planned in an area having gypseous soils and it is impossible to locate the canals outside the gypseous soil area, the following design and construction procedures have proved to be fairly successful. Even so, under gypseous soil conditions some slumping of the banks or settlement failures in isolated areas following filling of the canal with water should be anticipated and repair procedures provided for in the construction contract. The special treatment required in gypseous soils increases the cost of the canal construction and this extra cost should be taken into account when planning the project.

i. Design criteria

The gypsum content of the soil and the density are the two factors to be taken into account in forecasting the mechanical behaviour of the soil with respect to settlement in ground of low plasticity and reduction in cohesion in ground with average to high plasticity if the gypsum is leached from the soil. The criteria that are generally used for evaluation is that settlement be limited to 0.1 metres after the gypsum is dissolved and the residual cohesion after dissolution of the gypsum be at least equal to 0.2 kg/cm^2 to ensure against slip failure in the side slopes.

If these criteria are exceeded then steps must be taken to reduce the entrance of water to the soil. For example if the soil has low dry density **in situ** and high gypsum content, then the canal has to be made almost completely watertight. If the soil has low dry density **in situ** but little gypsum, the soil will require consolidation and improved watertightness (average seepage loss less than 6 mm/day). If the soil has average dry density **in situ** and a moderate amount of gypsum, it requires improved watertightness (average seepage loss less than 6 mm/day).

ii. Treatment procedures

Hydro-consolidation

In areas of low dry-density soils and some gypsum the excavated canal bed should be consolidated by hydro-consolidation before lining. In this procedure the canal is flooded with water and kept full for a specified period to promote consolidation of the soil and permit detection of any pockets of gypsum that may have been missed in the soils investigation. Hydro-consolidation of low density soils and moderately gypseous soils is essential before installing concrete canal lining in such soils. Hydro-consolidation should be avoided, however, in low density soils with high gypsum content.

Earth lining

If suitable soil is available within reasonable hauling distance, compacted earth lining would be preferable to concrete lining, in low density and moderately gypseous soils, since it is flexible and self-sealing. The compacted earth lining should be placed after hydro-consolidation.

Watertight membrane lining

In areas of high gypsum content and low density soils, a watertight lining has to be used. A double layer concrete lining with a butyl rubber-felt or prefabricated asphalt membrane sandwiched in between the concrete layers has been successful.

6.3 AGRESSION OF IRRIGATION NETWORKS BY WATER AND SOILS

Aggressiveness and corrosion phenomena and their consequences on the materials deterioration are fickle due to the complex interplay of material characteristics, water quality parameters and physical factors associated with metal/environment combination. It can be originated by attack from aggressive water or ions in soils.

Prediction of potential aggressiveness is possible on the basis of evaluating certain water/soil quality indices; however, actual rate data for design purposes are elusive. These matters are recognized with the result that corrosion control activity is given high priority to improve lifetime and operational efficiency of the systems. It is beyond the scope of the manual to cover the subject of corrosion in detail; whenever acute problems are foreseen, it is suggested to refer to specialized literature (Neveux 1968; Uhlig 1971; von Fraunhofer 1976; Baeckman and Schwenk 1977; Matthess 1982; Journals of the American Water Works Association).

6.4 MATERIALS AND EQUIPMENT

6.4.1 Introduction

Distribution networks call for a wide range of components such as canals, gates, pipes, valves, water meters and other ancillary equipment. For reliable network operation it is imperative that all its components are adequately designed and correctly operated.

The selection of the right materials and equipment requires a good understanding of their advantages and their functions. In this section will be found a general and in some cases detailed discussion of the materials and equipment used in irrigation networks. For further information, the reader is referred to specialized publications.

6.4.2 Pipes

i. Materials for pipes

The pipes that are used for irrigation networks can be made of steel, ductile iron, asbestos cement or plastic.

Steel pipes

Steel is an ideal material for water conveyance on account of its strength. Other advantages are the possibility of attaining almost any desirable form by welding and the possibility of laying the pipes on the ground or in a shallow excavation. A major drawback, however, is corrosion. Steel pipes may be of the seamless or welded type. Seamless pipes have thicker walls and are generally more expensive; the weld may be longitudinal or in a spiral depending on the manufacturing technique and on the size of the pipe.

Seamless pipes are manufactured in sizes ranging from 63 mm to 450 mm whereas the range of welded pipe extends from 63 to 2000 mm (larger sizes may be obtained on request). There are many systems for coupling steel pipes but most of these are variations of four basic techniques:

- welding
- flanged coupling
- slip-on coupling
- victaulic joints

Large pipes, of 600 mm diameter or over, should preferably be assembled by elding.

Steel pipes laid above ground are subject to thermal expansion and contraction. Care should therefore be taken when selecting the type of coupling to be used. Couplings which lack longitudinal strength, such as the slip-on type, require anchorages in conjunction with expansion or dresser fittings.

The major drawback of steel pipe is corrosion of which two major aspects will be mentioned here. The reader is referred to specialized literature and manufacturers' recommendations regarding protective measures as well as to the discussion of this subject in section 6.2.

- Galvanic corrosion of the outer surface, due to the difference of electric potential in the presence of pipes of dissimilar metals.

- Corrosion of the inner surface of the pipe due to the physical, chemical or bacteriological properties of the water.

When there is a risk of corrosion, steel pipes must be protected either externally or internally or both.

For outside protection, the pipe can be covered with a layer of asphalt, bitumen or coal enamel and wrapped with asbestos paper or fibreglass. The wrapping material used adds to the isolating properties of the cover but serves mainly as a physical protection of the bitumen which becomes brittle and peels off easily at low temperatures and with ageing. Another and probably more efficient technique is cathode protection; the principle of this protection is the createation of a small current which compensates the potential difference and galvanic currents along the pipeline so that an anode "corrodes" instead of the pipe itself.

Internal protection may also be required against chemical corrosion, tuberculation, encrustation, and bacteriological corrosion so as to maintain full pipe capacity.

Various coatings can be used such as bitumen, coal-tar, plastic or mortar. Bitumen and plastic coatings provide protection only against internal corrosion. Mortar coating protects the pipe against external corrosion. With a minimum thickness of about

6 mm it has a certain stability and a resistance against the shearing forces due to internal water pressure which result from the formation of small holes in the pipe wall, caused by external corrosion.

Ductile iron pipes

Ductile iron, one of the three major ferrous alloys used by pipe manufacturers, differs from steel on account of its higher carbon content (2.2 to 4 percent versus 0.10 to 1.50 percent) and the fact that this carbon is uniformly dispersed in the material, in the form of flaws. This gives to the ductile iron material specific mechanical characteristics such as high tensile strength, shock-resistance and high elasticity. Ductile iron pipes perform well on unstable or corroding soils. Standard sizes range from 60 to 1800 mm and can withstand pressures in the range of 25 to 55 bars. In the field they offer the advantage of being easy to cut and drill, etc. Ductile iron pipes, of all diameters, normally have an inside lining of cement mortar.

Different coupling systems are used according to the size of the pipe and the operating pressure, and the reader is referred to the manufacturers' catalogues. Two standard types are described below:

. Flanged joints

  The pipe ends are manufactured with flanges which can be either fixed or mobile. Pipes and fittings are joined together by fitting a flat gasket between the two flanges. Tightening of the bolts compresses the gasket and makes the joint watertight. The flanges are faced and drilled in standard sizes so that pipe fitting and pipe accessories are easily assembled.

  Flanged joints have the following advantages: ease and precision of assembly; thrust blocks are not required since longitudinal forces are accommodated; minimum space of assembly and dismantling possible.

  The disadvantage of the joint is that it does not allow deflection of pipeline route.

. Socket and spigot joint

  The pipe is made with a socket at one end and a spigot at the other. The joint is made by pushing the spigot into the socket. A gasket which is located in the socket ensures watertightness. This type of joint is manufactured in a variety of designs to suit specific conditions.

  These joints have the following advantages:

  - rapid assembly, involving simple operations
  - adaptability to pipeline movements, allowing expansion and deviation of pipeline axis
  - economy due to the reduced amount of excavation necessary for accommodating the joint.

Asbestos cement pipes

The pipe consists of a mixture of cement and asbestos fibres. The cement contributes to resistance to compression and the fibre to tensile or flexural stress. The material is completely free from organic or metallic substances, hence its resistance to rust, electrolysis, and galvanic corrosion. When placed in acid soils, bitumen dipping is recommended.

Particular attention should be given to the stresses that may bear on the pipe as failures have been encountered especially in heavy soils subject to soil movements or bedding.

The stresses that act upon the pipe are:

- internal hydrostatic pressure

  Standard pipes are designed for use at a maximum recommended pressure ranging between 5 and 15 bars.

- flexural or bending stress

  Resulting from both horizontal and vertical soil movement. Standard pipes are designed to sustain a total load ranging from 340 kg to 4600 kg, depending on the class and the diameter.

- compression stress

  Due to weight of pipe and water, earth cover and additional loads such as roads and vehicles etc. Standard pipes are designed with a crushing strength ranging from 6 000 kg/m to 21 000 kg/m, depending on the pipe class.

Asbestos cement pipes are made in classes 100, 150 and 200. (These class numbers indicate working pressure in pounds per square inch.)

Asbestos cement pipes are jointed with cast iron or steel couplings of the slip type which do not take up longitudinal forces. Sound installation procedures therefore call for concrete thrust blocking wherever an underground irrigation main changes direction (at tees, elbows, etc.).

Plastic pipe

There are numerous kinds of plastics, the four major types used in pipe manufacture being:

- Polyvinyl chloride	PVC
- Polyethylene	PE
- Acrylonitrile-butadiene-styrene	ABS
- Polybutylene	PB

Of these, only PVC, PE and ABS are presently used to any great extent in irrigation systems.

A great deal of confusion regarding plastic pipe is caused by the variety of materials, working pressures, wall thickness and pipe sizes available from the manufacturers. Much technical engineering effort has been devoted to the establishment of standards and specifications to aid design engineers in selecting the proper pipes. It is beyond the scope of this manual to go into the details of the various standards and/or recommendations for materials, manufacturing, testing and installation of plastic pipe, because they are numerous and different from one country to another. The reader is therefore referred to manufacturers' catalogues or to the standards and specifications of the country concerned.

ii. Pipeline accessories and control equipment

A large number of different appurtenances are required for the proper operation and pipe distribution systems. Some are used continuously, others intermittently or, like safety equipment, only in cases of emergency. A few of the more important of these accessories are described below:

Valves

- Start or stop valves

  The interruption or start of flow in pipelines is a frequently repeated operation which is performed by means of start or stop valves. These valves are normally in the fully open or fully closed position and their basic requirements are to offer minimum restriction when fully open and be watertight when fully closed. Gate, plug, ball or butterfly valves are all widely used.

- Pipeline protection valves

  Installed within the system to prevent excessive overpressure the most commonly used are the air release valve and the pressure-relief valve.

  Main lines lying over rolling terrain will tend to collect air at high points. Air pockets tend to reduce the capacity of the main and in the case of a closure of the system produce a vacuum which can collapse certain types of pipes. Water hammer may occur in systems in which air pockets can form.

  The air valves are mounted at high points of the pipeline and at points where the discharge of air upon the ventilation of the pipeline is facilitated.

  The pressure-relief valves are used to relieve excessive pressure surge, but are not generally effective in conditions of negative pressure. However, some manufacturers supply pressure-relief valves that also provide air and vacuum relief functions. The valve should be capable of releasing the design flow of the pipeline at a pipeline pressure not exceeding the permissible working head of the line by more than 50 percent. The number and spacing of relief valves on the pipeline are dictated by the grade and the permissible working head of the pipe. One valve, at the lowest point, is adequate for many pipes. If examination of the hydraulic head line shows that permissible pressures could be exceeded, additional relief valves are required.

  Air-release valves and pressure-relief valves are usually installed in conjunction with an isolation valve so that they can be removed without interruption of the flow in the line.

- Pressure-regulating and pressure-sustaining valves

  Pressure-regulating valves maintain a constant pre-set pressure downstream of their position, irrespective of variations of pressure upstream.

  Pressure-sustaining valves on the other hand maintain a pre-set upstream pressure regardless of possible variations of upstream pressure due to fluctuations of demand in the system. This is obtained by placing a valve on the main in order to release pressure to the atmosphere.

- Flow-control valves

  These valves maintain a constant pre-set rate of flow regardless of the changes in the pressure or flow demand of the system. They are installed at the head of the distribution system or at the head of the secondaries, tertiaries or at the hydrants.

- Level-control valves

  They control the water level in the reservoir by detecting the build-up of head in the reservoir. The sensing may be by means of a cock or any other control device other than the valve itself. The valve is installed on the supply side of the pipe and when the water level in the reservoir reaches a predetermined level it closes. The withdrawal from the reservoir is always made through a separate main.

Irrigation hydrants

These are composite valves that integrate in one unit, the pressure-regulating valve, flow-limiting device, the cut-off valve and the water meter. They are installed at the block or plot level. They may have one, two, three or four outlets, each of them servicing a different block or plot with different flow characteristics.

Water meters

Water meters are installed at selected points of the distribution system for the monitoring of the flow both for the record and for billing purposes.

Measurement of flow in pressure systems may be accomplished by various methods:

- Differential-head meters

  The flow of water passes through a constriction with a resulting drop of pressure due to increase of velocity. The drop of pressure is a function of the rate of flow. Constriction meters are the venturi, the flow nozzle and the orifice meter. The difference in pressure is measured with a differential manometer or a pressure gauges.

  Another type of differential-head meter is the pitot tube. This device indicates the velocity head at a point in the pipe cross section from which the velocity and the rate of flow can be calculated.

The differential-head meters have no moving parts and the original accuracy holds as long as they are kept clear. However they are not very practical and at low flows the head differences are not easily measured.

- Mechanical water meters

  Mechanical meters have a limited accuracy which decreases with time because of wear of the moving parts. The range of measurement of each mechanical meter and the accuracy of measurement are specified by the manufacturer.

  Mechanical meters are used to give the instantaneous flow rate and the cumulated volume and can be of two types: the displacement meters which consist of a piston which moves back and forth with the passing fluid, and inferential meters in use in irrigation projects which consist of propellers or turbines whose speed of rotation is a function of the flow velocity.

  When choosing a water meter the following points must be considered:

  . the range of flow for which the meter operates with acceptable accuracy;

  . the actual accuracy of measurement of the flow rate or volume;

  . the loss of head;

  . the working pressure;

  . the installation requirements (horizontal, vertical, flanged or non-flanged);

  . the type of mechanism (removable or closed type);

  . the type of drive (mechanical or magnetic);

  . the type of recording instrument.

  Generally, mechanical water meters require frequent maintenance, repairs and calibration if measurements are to attain the initial accuracy.

- Ultrasonic flow meters

  These flow meters have no moving parts, are completely obstruction-free, highly accurate and can be easily installed on any pipe.

  The measurement is made by a pair of sonic transducers mounted at an angle to the flow of the water through the sensor. For their operation electric power is required which can be provided either by DC or AC power supplies ranging from 24 to 240 volts.

6.4.3 Pump Protection Equipment

Strainers, foot valves with strainers, and check valves are installed either on the discharge or the suction side of the pump. Strainers are installed at the suction end of a pumping system to prevent gravel and other foreign matter from entering the body of the pump and causing damage. They may be fitted with a foot valve in which case the pump remains primed when not in operation.

Check valves are used on the discharge side of a pump or on a pipeline to prevent return flow. Different types of check valves exist, each suited to a specific condition; the most commonly used are the swing check, the lift and the recoil check.

6.4.4 Reservoir Valves

These valves are required to control the inflow to and the outflow from a reservoir. There are two main types: float valves which control inflow, and stop valves controlling the outflow. The float valves are installed in the reservoir and are used to maintain a predetermined water level in the reservoir. They open when the water level falls below a certain limit and close slowly as the water level rises again. Float valves are designed to minimize the possibility of occurence of water hammer in the pipeline upstream of the reservoir. The stop valves are installed at the inlet of a pipeline, in the reservoir, and provide protection against the damaging consequences of a pipe burst. If properly designed and installed they automatically cut the flow when it exceeds the maximum permissible rate.

6.4.5 Open Canals

Reference should be made to Chapter 5 where materials and structures are discussed in some detail.

i. Materials for open canals

The reader is referred to FAO (1977) for detailed information.

ii. Control equipment for irrigation canals

As other open conveyors, irrigation canals are provided with normal control or safety structures such as lateral spillways, siphons, divisors, etc. and reference is made to specialised literature, in particular FAO (1975).

A number of other specific structures and devices have been developed for water management and regulation of irrigation canals. Adjustment of canal discharge to meet the demand is obtained by adjusting water levels.

Duckbill weir cross regulator

The water level in a canal is partially regulated by a weir with a long crest installed across the canal. If the canal discharge upstream of the structure increases the head fluctuation above the crest is minimum - hence the discharge of any intake situated between two such regulators remains almost constant.

Level-control gates

They can be of three types:

- upstream control gates
- downstream control gates
- Neyrpic modules or distributors.

An upstream control gate installed in a canal automatically maintains a constant water level in the upstream reach irrespective of the incoming flow. The gate is nearly closed at low discharge, fully closed at no discharge and opens progressively as the upstream flow increases. These gates are installed in the canal immediately downstream of the offtake for which the water level has to be kept constant. The gate consists of a rigis structure that can freely rotate around a horizontal axis. A cylindrical leaf is provided on the upstream side of the axis with a buoyant compartment whilst a ballast is provided at the downstream side. The whole structure is in equilibrium when the axis and the upstream water level are at the same elevation. Adjustments are made by moving the ballast.

A downstream control level gate operates on the same principle, but controls the downstream water level, irrespective of the water level in the upstream section of the canal.

The Neyrpic modules or distributors are a form of free surface offtake designed to supply water from main canals to secondarty or tertiary channels at constant adjustable flows, even though the water level upstream or downstream varies to some extent.

The distributor consists of a specially shaped sill with a horizontal crest, a tilted and fixed baffle at a predetermined level above the sill and a small gate that can seal or free the opening completely.

At low flows the sill operates at free surface flow conditions and the discharge increases with the water level following the stage-discharge relation of a weir. When the water level reaches the baffle the system works as an orifice and although the velocity increases the discharge decreases sharply. If the water level is further increased, the discharge, which had become stable, rises again and follows the stage-discharge relation of submerged orifices.

Therefore, between two upstream water levels corresponding to the transition between the two hydraulic regimes, the discharge is almost constant. With a correctly adjusted module, the discharge should not fluctuate more than ±5 percent of the average Q.

Some distributors are fitted with two baffles which gives even more accurate regulation.

A large range of discharges can be regulated by placing batteries of modules of different nominal discharges.

ANNEX

USE OF THE PROGRAMME

1. INTRODUCTION

This annex gives a brief description of the programme for the optimization of the pipe diameters of a branching network based upon the method devised by Dr. Y. Labye.

The programme is written in **Basic**.

A more comprehensive document, including the flow charts and listing for each subprogramme, together with the disks and the operation manual for both Apple and IBM-PC is available on application to the Chief, Water Resources, Development and Management Service, Land and Water Development Division, FAO, Via delle Terme di Caracalla, 00100 Rome, Italy.

Users are expected to gain a thorough understanding of the method of optimization described in chapter 4 for a better use of the material and prior to resolving a practical application.

The notes are presented in two sections:

- general layout of programme and preparation of lists of inputs, common to all PCs;
- a case study.

2. GENERAL LAYOUT OF THE PROGRAMME AND PREPARATION OF LISTS OF INPUTS

2.1 General Layout of the Programme

The version of the programme corresponds to DOS 3.3 with GW BASIC. In order to avoid a "disk full" situation, the programme has been designed to record input data and intermediate results on a diskette labelled **"DATA"**. The diskette which stores all programmes (subprogrammes) is labelled **"MAIN PROGRAMME"**.

The **"MAIN PROGRAMME"** diskette is placed in disk drive 1. If the computer has a single drive an instruction with a programme interruption facilitates the necessary changes between diskettes **"DATA"** and **"MAIN PROGRAMME"**. In the case of two drives, the **"DATA"** diskette is set in drive 2.

In order to retain as many memories as possible for the network itself, the programme for optimizing pipe diameters has been subdivided into six components:

- selection programme which gives access to the five other components by pressing the letter or function key corresponding to the required title [**START**];
- preparation of the table of pipes which are technically suitable and available for the network [**TABLE OF PIPES**];
- description of the branching network [**NETWORK DESCRIPTION**];

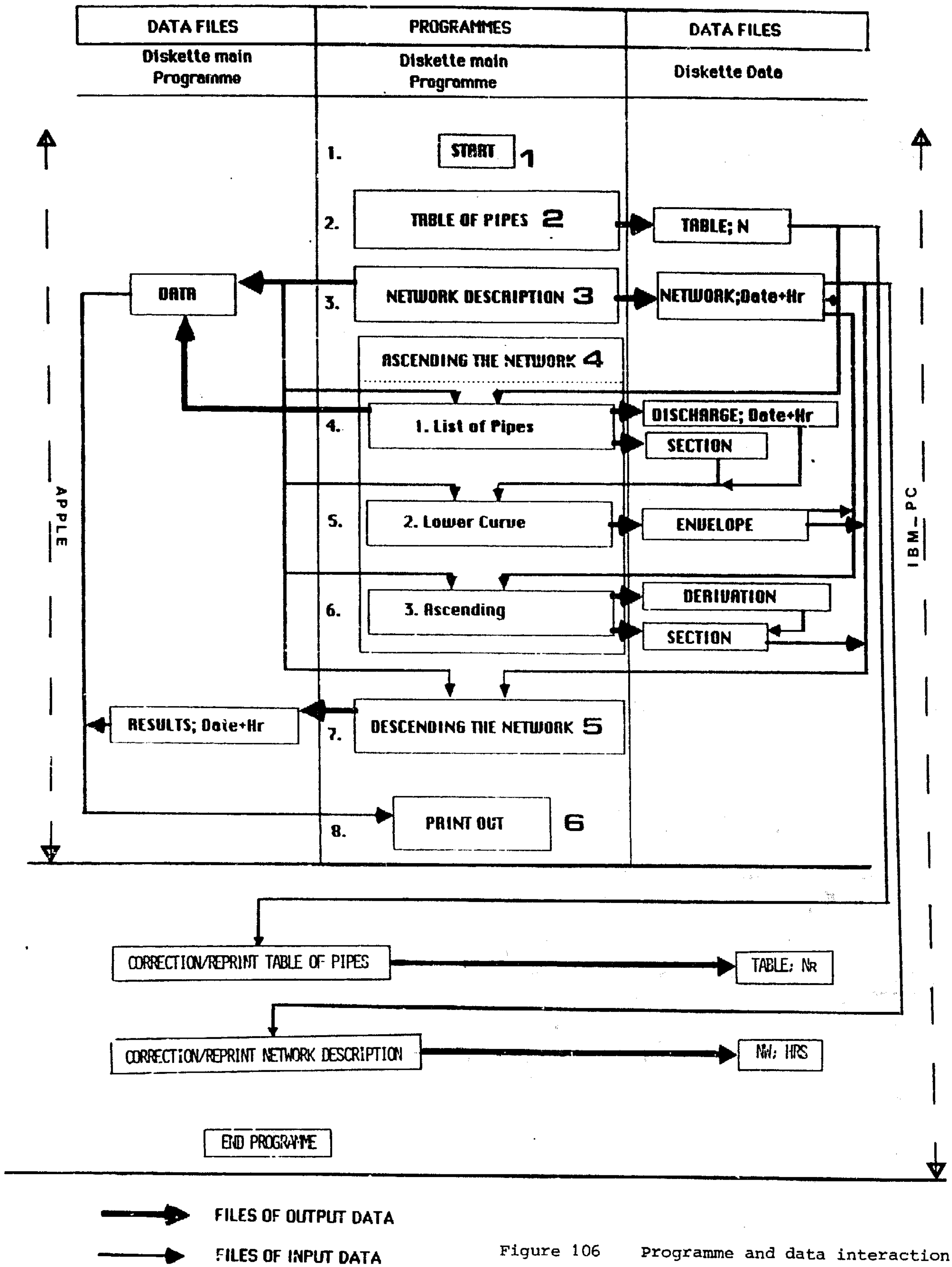


Figure 106 Programme and data interaction

- determination of the lower envelope curve for the network [**ASCENDING THE NETWORK**], which is subdivided into three subprogrammes:

  . **LIST OF PIPES**
  . **LOWER CURVE**
  . **ASCENDING**

- determination of the length and diameter of pipe(s) to be used on each section together with a summary [**DESCENDING THE NETWORK**];

- summary of the results showing the diameters and lengths of pipes to be used on each of the sections, together with a table of the total length of pipe required according to diameter and total cost [**PRINTOUT**].

Programmes and data interaction are represented in Figure 106. [For the IBM-PC two subprogrammes were added to correct-reprint the Table of Pipes and the Net ork Description.]

2.2 Preparation of Lists of Input Data

For a given site location, the design characteristics of a distribution network are subjected to physical constraints such as the discharges and pressures required or permitted at each location, the specifications of the pipes available on the local market, the altitude of the various sections and the costs. These are input data to be entered into the computer memory.

Two of the six programmes will have to include such information; they are:

- Programme - **TABLE OF PIPES**
- Programme - **NETWORK DESCRIPTION**.

2.2.1 Preparation of the list of input data for the Programme - **TABLE OF PIPES**

The list consists of commercially available pipes and the technical specifications are entered into the computer memory in numerical table form in order of decreasing diameters.

The input data are the following:

- pipe diameter (mm)

- roughness height, k (mm)

- minimum and maximum permissible flow velocities (m/s) (these velocities should not be less than 0.3 m/s or greater than 3 m/s; these values will automatically be used by the programme if other values are not entered)

- maximum permissible pressure in pipes (m)

- cost per metre length of pipe; the currency is selected by the user.

After each set of data has been introduced, modification of incorrect entries is possible. The programme then prints out a table as shown in the example developed under paragraph 3.1.

2.2.2 Preparation of the list of input data for the Programme - **NETWORK DESCRIPTION**

For the programme to perform correctly, the network has to be described and each node, hydrant and section is positioned by a number allocated according to the strict procedure described in chapter 4. The following major points should be noted:

- nodes and hydrants are numbered sequentially from 1 to n, with n equal to the total number of nodes and/or hydrants;
- the hydrant number 1 is the most downstream one from the source;
- in the sequence of numbering, a downstream hydrant has priority over the node immediately upstream;
- a section is given the number of the node or hydrant immediately downstream;
- the network must terminate in a single section and not with a node, junction or two or more sections;
- a non-terminal section cannot carry a hydrant; if so, it is necessary to create an imaginary section of minimum length between the hydrant and the section.

The process for the numbering is clarified in the example developed under paragraph 3.

When the numbering has been completed the following information has to be filled in the list for the various components of the network:

- number of the section
- number of the section immediately upstream
- length of the section (m)
- whether the section is terminal or not, (1) and (0) respectively
- the discharge of the section (m^3/s)
- the elevation of the downstream hydrant (m) } piezometric
- the pressure required at the hydrant (m) } level

3. CASE STUDY

Example 9 of chapter 3 is referred to for the basic characteristics of the network. The general layout of the system is represented in Figure 107.

The flow through each section, the discharge and pressure required at the hydrants have been determined. A land survey is available and the elevation of the various components of the network is known. The lengths of the sections have been measured.

The specifications of the pipes, locally available, are also known - diameters, roughness, permissible velocities and pressures, the costs per metre length. Firstly the list of input data for both Programmes, **TABLE OF PIPES** and **NETWORK DESCRIPTION**, are prepared. This information will subsequently be fed into the computer memory.

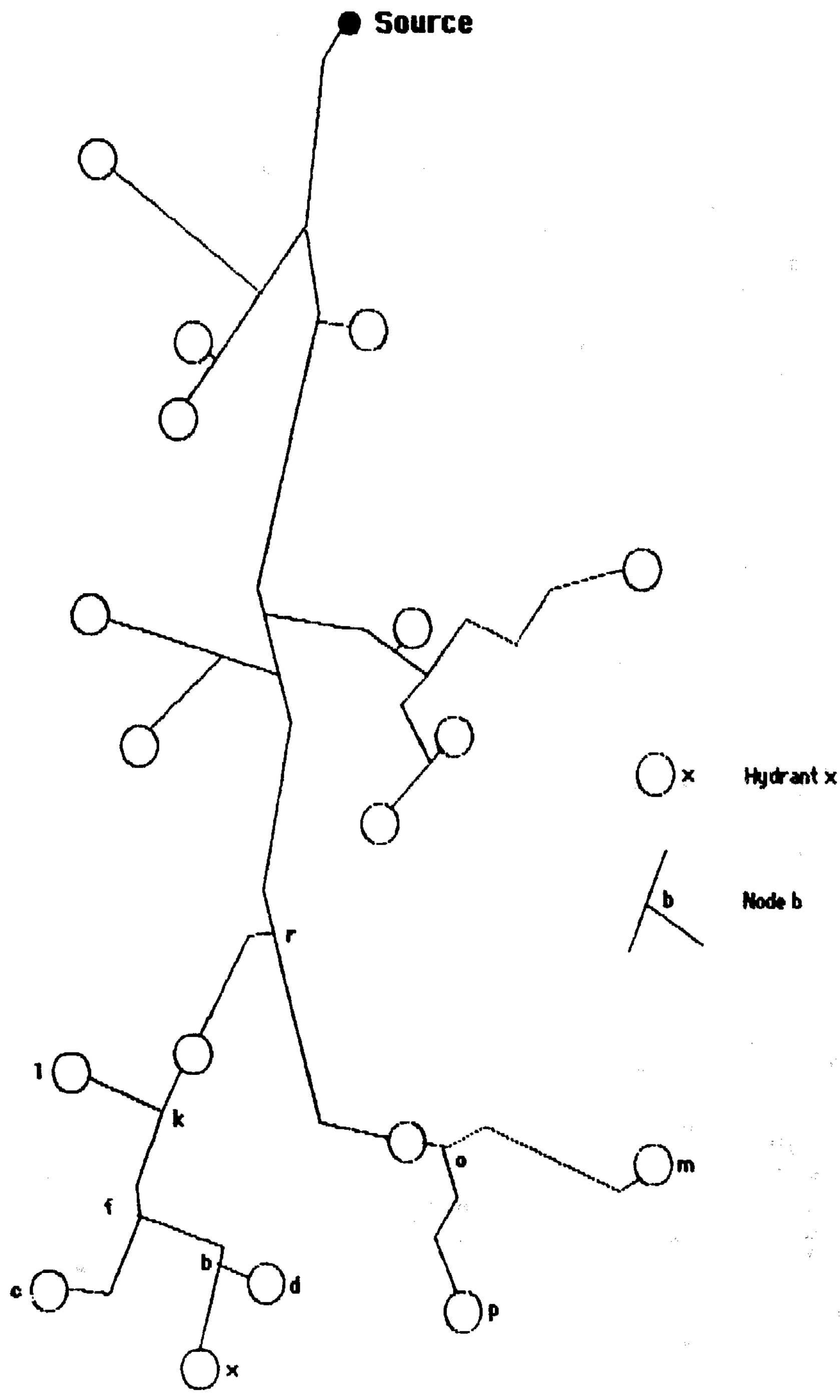


Figure 107 Layout of the network for the case study

3.1 List of Input Data: Programme - TABLE OF PIPES

The programme is designed to enter only one list at a time; we will therefore enter all the pipes which may be suitable for the network and which are available on the local market. The list is shown in Table 39 for 13 pipes.

Table 39 LIST OF PIPES

Pipe No.	Diam. (mm)	Roughness (mm)	Velocity Minimum	Velocity Maximum	Pressure maximum (m)	Cost FF/m
1	800	0.025	0.50	3.10	80	1435.0
2	700	0.025	0.50	2.95	80	1120.0
3	600	0.025	0.50	2.95	80	790.0
4	500	0.025	0.50	2.85	80	580.0
5	450	0.025	0.50	2.85	80	545.0
6	400	0.025	0.50	2.50	80	470.0
7	350	0.025	0.50	2.30	80	400.0
8	300	0.025	0.40	2.25	80	285.0
9	250	0.025	0.40	2.15	80	240.0
10	200	0.025	0.35	2.05	80	195.0
11	150	0.025	0.25	1.95	80	145.0
12	125	0.025	0.25	1.85	80	130.0
13	100	0.025	0.20	1.80	80	112.0

3.2 List of Input Data - Programme NETWORK DESCRIPTION

3.2.1 Numbering the hydrants and nodes

According to the layout of the network, Figure 107, the most downstream hydrant from the source is the hydrant x. It is taken as the origin* and numbered 1 (Figure 108).

From hydrant numbered 1, the network is ascended up to the closest node, which is "b". As the singular point "b" is a node, it is not numbered at this stage (priority rule of hydrant over node). From "b" descending the network the closest hydrant is found to be hydrant "d" which is numbered 2. From hydrant 2 and ascending the network the first singular point met is node "b" which is now numbered 3 (no downstream hydrant to be found). From point 3 and ascending the network node "f" is reached but not numbered at this stage (priority rule), it is necessary first to go down the network to number hydrant "c" which is numbered "4" and up again to "f" which is now numbered 5. From 5 we move up to "k" and then down to "l" which is earmarked 6 and back to "k" which is now given number 7. The process is pursued up to node r. From "r" the network is descended straight to the farthest hydrant "p", which takes number 9. The network is ascended and descended the same way as described earlier to reach the source which is numbered 34.

3.2.2 Numbering the section

Any section will carry the number of its downstream hydrant or node. Hence for the example: section (1) connects hydrant 1 to node 3; section (3) connects node 3 to node 5, etc.

* If there is a doubt between two hydrants, the one to be chosen is the one belonging to the part with the greatest number of branches.

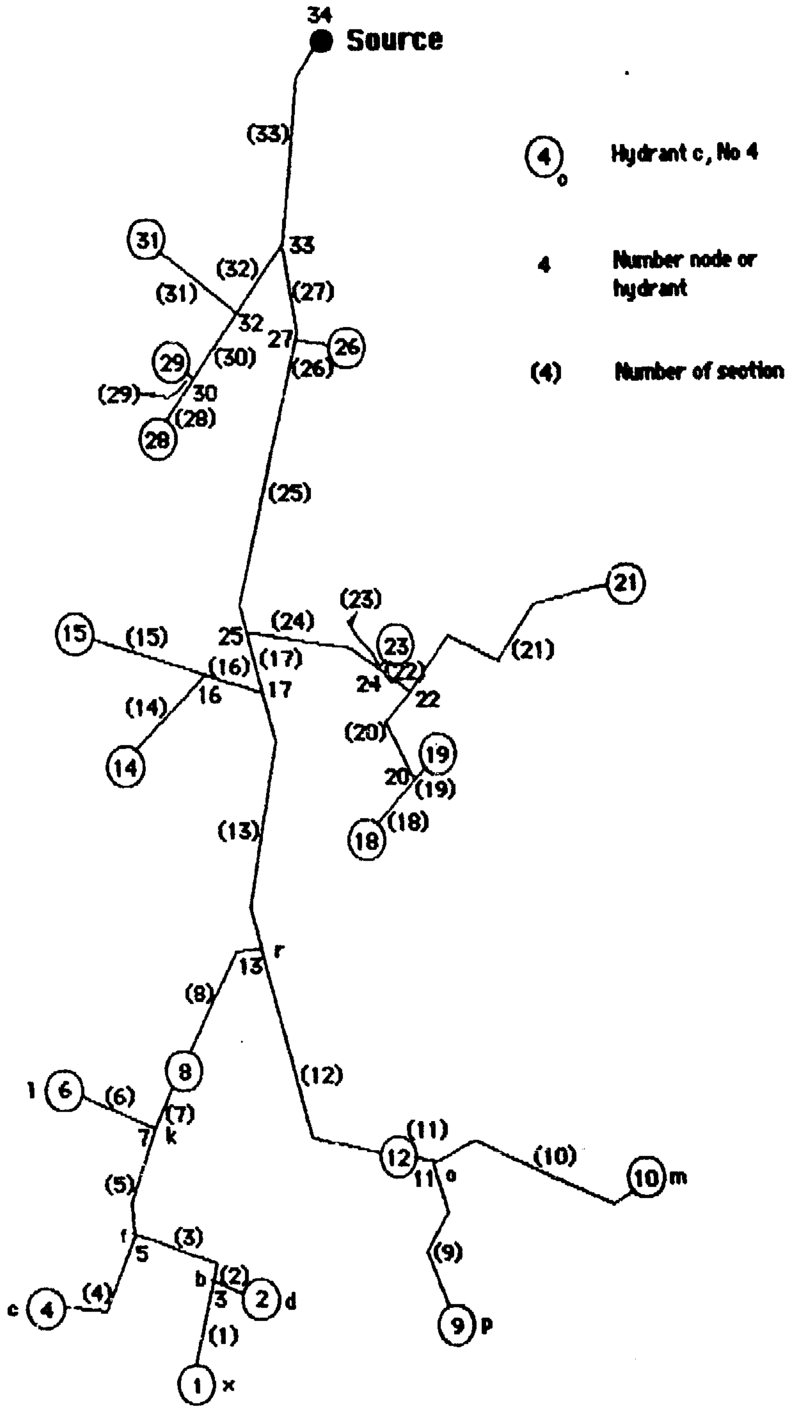


Figure 108 Numbering the network

3.2.3 List of input data

Since other parameters of the network have been determined (see example 9), the tablular list of input data can be prepared (Table 40).

Table 40 DESCRIPTION OF THE NETWORK

ENTRIES STORED ON DISK UNDER NAME: NW 23-83-8713.28
NUMBER OF SECTIONS IN THE NETWORK: 33

Section (No.)	Upstream (No.)	Length (m)	Terminal (Y=1, N=o)	Discharge (m^3/s)	Pressure (m)	Altitude (m)
1	3	195.00	1	0.0250	30.00	505.00
2	3	90.00	1	0.0139	30.00	500.00
3	5	220.00	0	0.0389	0.00	0.00
4	5	290.00	1	0.0139	20.00	515.00
5	7	190.00	0	0.0520	0.00	0.00
6	7	190.00	1	0.0250	30.00	515.00
7	8	130.00	0	0.0778	0.00	0.00
8	13	300.00	0	0.0778	30.00	505.00
9	11	330.00	1	0.0139	30.00	520.00
10	11	485.00	1	0.0348	30.00	500.00
11	12	70.00	0	0.0487	0.00	0.00
12	13	530.00	0	0.0682	30.00	510.00
13	17	490.00	0	0.0899	0.00	0.00
14	16	225.00	1	0.0195	30.00	500.00
15	16	280.00	1	0.0195	30.00	505.00
16	17	130.00	0	0.0390	0.00	0.00
17	25	120.00	0	0.1052	0.00	0.00
18	20	160.00	1	0.0348	30.00	495.00
19	20	50.00	1	0.0139	30.00	505.00
20	22	220.00	0	0.0487	0.00	0.00
21	22	540.00	1	0.0223	30.00	515.00
22	24	85.00	0	0.0710	0.00	0.00
23	24	10.00	1	0.0139	30.00	510.00
24	25	290.00	0	0.0710	0.00	0.00
25	27	575.00	0	0.1372	0.00	0.00
26	27	110.00	1	0.0195	30.00	540.00
27	33	145.00	0	0.1429	0.00	0.00
28	30	135.00	1	0.0084	30.00	515.00
29	30	10.00	1	0.0195	30.00	535.00
30	32	170.00	0	0.0279	0.00	0.00
31	32	400.00	1	0.0084	30.00	515.00
32	33	140.00	0	0.0363	0.00	0.00
33	34	370.00	0	0.1565	0.00	0.00

4. OPERATION OF THE PROGRAMME

The input data having been introduced the programme runs, by itself, through a set of questions and answers.

The final product appears in the form of the following printouts for Apple and IBM.

* RESULTS OF OPTIMISING PIPE DIAMETER*

APPEL

NUMBER OF THE NETWORK: 23-03-8713.20
MINIMUM LENGTH TO COMPOUND PIPES: 1 m
UPSTREAM TOTAL HEAD OR STATIC HEAD SELECTED: 575 m

Section (No)	Upstream (No)	Discharge (m3/s)	Velocity (m/s)	Terminal (1=Y;0=N)	Pipe (No)	Diameter (mm)	Length (m)	Hydr.Level (m)	Head Losses (m)	Velocity Head (m)	Energy Level (m)	Excess Pres. (m)	Tot. Cost (FF)
1	3	0.0250	1.41	1	11	150	195.00	549.57	2.22	0.10	549.670	14.57	28275.00
2	3	0.0139	1.77	1	13	100	90.00	549.24	2.55	0.16	549.401	19.24	10080.00
3	5	0.0389	1.24	0	10	200	220.00	551.79	1.38	0.08	551.869	0.00	42900.00
4	5	0.0139	1.77	1	13	100	290.00	544.96	8.21	0.16	545.120	9.96	32480.00
5	7	0.0520	1.66	0	10	200	190.00	553.17	2.05	0.14	553.313	0.00	37050.00
6	7	0.0250	1.41	1	11	150	190.00	553.06	2.17	0.10	553.164	8.06	27550.00
7	8	0.0778	1.58	0	9	250	130.00	555.23	0.99	0.13	555.356	0.00	31200.00
8	13	0.0778	1.58	0	9	250	300.00	556.22	2.29	0.13	556.348	21.22	72000.00
9	11	0.0139	1.13	1	12	125	246.68	550.00	4.68	0.07	550.065	0.00	41400.32
			1.77	1	13	100	83.32			0.16	550.065		
10	11	0.0348	1.11	1	10	200	485.00	552.20	2.48	0.06	552.263	22.20	94575.00
11	12	0.0487	1.55	0	10	200	70.00	554.68	0.67	0.12	554.602	0.00	13650.00
12	13	0.0682	1.39	0	9	250	530.00	555.35	3.16	0.10	555.447	15.35	127200.00
13	17	0.0899	1.83	0	9	250	490.00	558.51	4.90	0.17	558.680	0.00	117600.00
14	16	0.0195	1.59	1	12	125	225.00	558.62	3.97	0.13	558.748	28.62	29250.00
15	16	0.0195	1.59	1	12	125	280.00	557.65	4.94	0.13	557.776	22.65	36400.00
16	17	0.0390	1.24	0	10	200	130.00	562.59	0.82	0.08	562.670	0.00	25350.00
17	25	0.1052	2.14	0	9	250	120.00	563.41	1.62	0.23	563.647	0.00	28800.00
18	20	0.0348	1.11	1	10	200	160.00	559.70	0.82	0.06	559.759	34.70	31200.00
19	20	0.0139	1.77	1	13	100	50.00	559.10	1.42	0.16	559.257	24.10	5600.00
20	22	0.0487	1.55	0	10	200	220.00	560.51	2.10	0.12	560.636	0.00	42900.00
21	22	0.0223	1.82	1	12	125	540.00	550.36	12.26	0.17	550.527	5.36	70200.00
22	24	0.0710	1.45	0	9	250	85.00	562.62	0.55	0.11	562.724	0.00	20400.00
23	24	0.0139	1.77	1	13	100	10.00	562.88	0.28	0.16	563.041	22.88	1120.00
24	25	0.0710	1.45	0	9	250	290.00	563.16	1.86	0.11	563.271	0.00	69600.00
25	27	0.1372	1.94	0	8	300	575.00	565.03	5.16	0.19	565.220	0.00	163875.00
26	27	0.0195	0.62	1	10	200	110.60	570.00	0.00	0.02	570.020	0.00	21450.00
27	33	0.1429	2.02	0	8	300	145.00	570.19	1.41	0.21	570.402	0.00	41325.00
28	30	0.0084	1.07	1	13	100	135.00	566.95	1.50	0.06	567.007	21.95	15120.00
29	30	0.0195	1.59	1	12	125	10.00	568.27	0.10	0.13	568.399	3.27	1300.00
30	32	0.0279	1.58	0	11	150	170.00	568.45	2.30	0.13	568.574	0.00	24650.00
31	32	0.0084	1.07	1	13	100	400.00	566.39	4.44	0.06	566.445	21.39	44800.00
32	33	0.0363	1.16	0	10	200	140.00	570.83	0.77	0.07	570.894	0.00	27300.00
33	34	0.1565	1.63	0	7	350	139.86	571.60	0.86	0.13	571.734	0.00	121533.67
			2.21	0	8	300	230.14			0.2[illegible]	571.734		

* LENGTH OF PIPES *

PIPES TAKEN FROM TABLE No: 1

Pipe (No)	Diameter (mm)	Cost per metre (FF)	Length (m)	Tot. Cost (FF)
1	800	1435.00	0.00	0.00
2	700	1120.00	0.00	0.00
3	600	790.00	0.00	0.00
4	500	580.00	0.00	0.00
5	450	545.00	0.00	0.00
6	400	470.00	0.00	0.00
7	350	400.00	139.86	55943.19
8	300	285.00	950.14	270790.48
9	250	240.00	1945.00	466800.00
10	200	195.00	1725.00	336375.00
11	150	145.00	555.00	80475.00
12	125	130.00	1301.68	169218.95
13	100	112.00	1058.32	118531.37
		Total:	7675.00	1498137.12

* RESULTS OF OPTIMISING PIPE DIAMETER*

IBM_PC

NUMBER OF THE NETWORK: 134157
MINIMUM LENGTH TO COMPOUND PIPES: 1 m
UPSTREAM TOTAL HEAD OR STATIC HEAD SELECTED: 575 m

Section (No)	Upstream (No)	Discharge (m3/s)	Velocity (m/s)	Terminal (1=Y;0=N)	Pipe (No)	Diameter (mm)	Length (m)	Hydr.Level (m)	Head Losses (m)	Velocity Head (m)	Energy Level (m)	Excess Pres. (m)	Tot. Cost (FF)
1	3	0.0250	1.41	1	11	150.00	195.00	549.57	2.22	0.10	549.67	14.57	28275.00
2	3	0.0139	1.77	1	13	100.00	90.00	549.24	2.55	0.16	549.40	19.24	10080.00
3	5	0.0389	1.24	0	10	200.00	220.00	551.79	1.38	0.08	551.87	0.00	42900.00
4	5	0.0139	1.77	1	13	100.00	290.00	544.96	8.21	0.16	545.12	9.96	32480.00
5	7	0.0520	1.66	0	10	200.00	190.00	553.17	2.05	0.14	553.31	0.00	37050.00
6	7	0.0250	1.41	1	11	150.00	190.00	553.06	2.17	0.10	553.16	8.06	27550.00
7	8	0.0778	1.58	0	9	250.00	130.00	555.23	0.99	0.13	555.36	0.00	31200.00
8	13	0.0778	1.58	0	9	250.00	300.00	556.22	2.29	0.13	556.35	21.22	72000.00
9	11	0.0139	1.13	1	12	125.00	246.68	550.00	4.68	0.07	550.07	0.00	41400.24
			1.77	1	13	100.00	83.32			0.16	550.07		
10	11	0.0348	1.11	1	10	200.00	485.00	552.20	2.48	0.06	552.26	22.20	94575.00
11	12	0.0487	1.55	0	10	200.00	70.00	554.68	0.67	0.12	554.80	0.00	13650.00
12	13	0.0682	1.39	0	9	250.00	530.00	555.35	3.16	0.10	555.45	15.35	127200.00
13	17	0.0899	1.83	0	9	250.00	490.00	558.51	4.90	0.17	558.68	0.00	117600.00
14	16	0.0195	1.59	1	12	125.00	225.00	558.62	3.97	0.13	558.75	28.62	29250.00
15	16	0.0195	1.59	1	12	125.00	280.00	557.65	4.94	0.13	557.78	22.65	36400.00
16	17	0.0390	1.24	0	10	200.00	130.00	562.59	0.82	0.08	562.67	0.00	25350.00
17	25	0.1052	2.14	0	9	250.00	120.00	563.41	1.62	0.23	563.65	0.00	28800.00
18	20	0.0348	1.11	1	10	200.00	160.00	559.70	0.82	0.06	559.76	34.70	31200.00
19	20	0.0139	1.77	1	13	100.00	50.00	559.10	1.42	0.16	559.26	24.10	5600.00
20	22	0.0487	1.55	0	10	200.00	220.00	560.51	2.13	0.12	560.64	0.00	42900.00
21	22	0.0223	1.82	1	12	125.00	540.00	550.36	12.26	0.17	550.53	5.36	70200.00
22	24	0.0710	1.45	0	9	250.00	85.00	562.62	0.55	0.11	562.72	0.00	20400.00
23	24	0.0139	1.77	1	13	100.00	10.00	562.88	0.28	0.16	563.04	22.88	1120.00
24	25	0.0710	1.45	0	9	250.00	290.00	563.16	1.86	0.11	563.27	0.00	69600.00
25	27	0.1372	1.94	0	8	300.00	575.00	565.03	5.16	0.19	565.22	0.00	163875.00
26	27	0.0195	0.62	1	10	200.00	110.00	570.00	0.00	0.02	570.02	0.00	21450.00
27	33	0.1429	2.02	0	8	300.00	145.00	570.19	1.41	0.21	570.40	0.00	41325.00
28	30	0.0084	1.07	1	13	100.00	135.00	566.95	1.50	0.06	567.01	21.95	15120.00
29	30	0.0195	1.59	1	12	125.00	10.00	568.27	0.18	0.13	568.40	3.27	1300.00
30	32	0.0275	1.56	0	11	150.00	170.00	568.45	2.38	0.13	568.57	0.00	24650.00
31	32	0.0084	1.07	1	13	100.00	400.00	566.39	4.44	0.06	566.45	21.39	44800.00
32	33	0.0363	1.16	0	10	200.00	140.00	570.83	0.77	0.07	570.89	0.00	27300.00
33	34	0.1565	1.63	0	7	350.00	139.88	571.60	0.86	0.13	571.73	0.00	121536.70
			2.21	0	8	300.00	230.12			0.25	571.73		

* LENGTH OF PIPES *

PIPES TAKEN FROM TABLE No: 13

Pipe (No)	Diameter (mm)	Cost per metre (FF)	Length (m)	Tot. Cost (FF)
1	800.00	1435.00	0.00	0.00
2	700.00	1120.00	0.00	0.00
3	600.00	790.00	0.00	0.00
4	500.00	580.00	0.00	0.00
5	450.00	545.00	0.00	0.00
6	400.00	470.00	0.00	0.00
7	350.00	400.00	139.88	55953.92
8	300.00	285.00	950.12	270782.90
9	250.00	240.00	1945.00	466800.00
10	200.00	195.00	1725.00	336375.00
11	150.00	145.00	555.00	80475.00
12	125.00	130.00	1301.68	169218.40
13	100.00	112.00	1058.32	118531.90
		Total:	7675.00	1498140.00

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