Chapter 3
Design parameters for the infield works

In order to calculate the design flow of the irrigation system, a number of parameters must be taken into consideration, notably the available moisture, the root zone depth, the allowable moisture depletion, the net peak water requirements, the irrigation frequency and cycle and the irrigation efficiencies. The following surface irrigation schemes will be used to demonstrate the process of using and calculating design parameters:

- Nabusenga irrigation scheme, which is a surface irrigation scheme in Matabeleland North Province in Zimbabwe using a concrete-lined canal system (see Section 4.2 and Figure 19)
- Mangui irrigation scheme, which is an imaginary surface irrigation scheme using a piped system up to field level (see Section 4.3 and Figure 20)

Table 15 shows the given design parameters.

3.1. Crop water and irrigation requirements

The calculation of the crop water and irrigation requirements is discussed in detail in Module 4, to which the reader is referred.

3.2. Net and gross depth of water application

3.2.1. Net depth of water application (d_{net})

The net depth of water application (d_{net}) is the amount of water in millimetres that needs to be supplied to the soil in order to bring it back to field capacity. It is the product of the available soil moisture (FC-PWP), the effective root zone depth (RZD) and the allowable moisture depletion (P), and is calculated as follows:

Equation 5

$$d_{net} = (FC - PWP) \times RZD \times P$$

Where:

d_{net}	=	Net depth of water application per irrigation for the selected crop (mm)
FC	=	Soil moisture at field capacity mm/m)
PWP	=	Soil moisture at the permanent wilting point (mm/m)
RZD	=	The depth of soil that the roots exploit effectively (m)
P	=	The allowable portion of available moisture permitted for depletion by the crop before the next irrigation

Table 15
Design parameters for Nabusenga and Mangui surface irrigation schemes

Parameter	Nabusenga scheme	Mangui scheme
Area	15 ha	2.4 ha
Soil type	Clay loam	Sandy mixture
Available soil moisture (= FC - PWP)	130 mm/m	80 mm/m
Design root zone depth (RZD)	0.70 m for maize	0.75 m for maize
Allowable soil moisture depletion (P)	0.50	0.50
Assumed field application efficiency (E_a)	0.50	0.60
Assumed field canal/ efficiency (E_b)	0.90	1
Assumed conveyance efficiency (E_c)	0.90	1
Farm irrigation efficiency ($E_f = E_b \times E_a$)	0.45	0.60
Distribution system efficiency (E_d) (= $E_c \times E_b$)	0.81	1
Overall irrigation efficiency (E_p) (= $E_c \times E_b \times E_a$)	0.41	0.60
Peak ET_{crop}	6.0 mm/day	6.2 mm/day

3.2.2. Gross depth of water application (d_{gross})

The gross depth of water per irrigation is obtained by dividing the net depth of water (d_{net}) by efficiency, as in Equation 6:

Equation 6

$$d_{gross} = \frac{d_{net}}{E}$$

Example 4

Based on the design parameters given in Table 14, what are the net and gross depths of water application for Nabusenga irrigation project?

d_{net} = 130 mm/m x 0.70 m x 0.50 = 45.5 mm

The gross depths of water application at field and at overall level would be:

$$d_{gross} = \frac{45.5}{0.50} = 91.0 \text{ mm at field level}$$

$$d_{gross} = \frac{45.5}{0.41} = 111.0 \text{ mm at overall level}$$

3.3. Irrigation frequency and irrigation cycle

3.3.1. Irrigation frequency (IF)

Irrigation frequency is the time it takes a crop to deplete the soil moisture at a given depletion level and can be calculated as follows:

Equation 7

$$IF = \frac{d_{net}}{ET_{crop}}$$

Where:

IF	=	Irrigation frequency (days)
d_{net}	=	Net depth of water application (mm)
ET_{crop}	=	Crop evapotranspiration (mm/day)

It should be mentioned that for design purposes we are particularly interested in the peak daily amount of water used by the crop, which is the worst case scenario. The net peak daily irrigation requirement (IR_n) is determined by subtracting the rainfall (if any) from the peak daily crop water requirements.

3.3.2. Irrigation cycle (IC)

The irrigation cycle is the time it takes to irrigate the entire scheme. If, from an irrigation frequency of 7 days, for example, we take the irrigation cycle to be 5 days, this leaves us 2 days for other works and practices inside and outside the scheme. The greater the difference between the frequency and the cycle, the greater the flexibility to deal with unforeseen situations such as breakdowns. Besides this, it allows for the eventual expansion of the scheme, utilizing the same conveyance and distribution system, once water from the dam or river is found to be in surplus. On the other hand, the greater the difference the more expensive the scheme becomes, as the designer has to go for larger water conveyance and distribution systems. As a rule, the difference between the irrigation frequency and

Example 5

What is the irrigation frequency and what can be the irrigation cycle for Nabusenga scheme?

The irrigation frequency is equal to:

$$IF = \frac{45.5}{6.0} = 7.5 \text{ days}$$

The system should be designed to provide 45.5 mm every 7.5 days. For practical purposes, fractions of days are not used for irrigation frequency purposes. Hence, the irrigation frequency in our example should be 7 days, with a corresponding d_{net} of:

d_{net} = 7 x 6.0 = 42 mm for an IF of 7 days

The d_{gross} at field and overall level will be $\frac{42}{0.50}$ = 84.0 mm and $\frac{42}{0.41}$ = 102.4 mm respectively.

The adjusted allowable depletion for the 7 days (instead of 7.5 days) is equal to:

$$P = \frac{7 \times 6.0}{130 \times 0.7} = 0.46 \text{ or } 46\%$$

Based on the above, the Irrigation Cycle (IC) is fixed at 6 days.

the irrigation cycle should not exceed one day. This is considered as a compromise between convenience and cost.

3.4. System capacity (Q)

System capacity refers to the discharge that has to be abstracted from the headwork during a given period per day and it is used for the design of the headwork and the conveyance system. It is determined by the following equation:

Equation 8

$$Q = \frac{V}{T}$$

Where:

Q = Discharge (m^3/hr or l/sec)

V = Volume of water to be abstracted per day (m^3 or l)

T = Irrigation duration per day (hr or sec)

The volume of water to be abstracted per day is obtained as follows:

Equation 9

$$V = 10 \times A \times d_{gross}$$

Where:

V = Volume of water abstracted per day (m^3)

A = Area irrigated daily (ha)

d_{gross} = Gross depth of application at overall scheme level (mm)

10 = Conversion factor to convert mm to m^3/ha

The area irrigated per day can be calculated as follows:

Equation 10

$$A = \frac{At}{IC}$$

Where:

A = Area irrigated per day (ha)

At = Total area (ha)

IC = Irrigation cycle (days)

A summary of the calculated design parameters for Nabusenga is given in Table 16. The design parameters for Mangui should be calculated in the same way. The result is also summarized in Table 16.

Example 6

What should be the system capacity for Nabusenga scheme, considering an irrigation duration of 10 hours per day?

The area irrigated per day is equal to:

$$A = \frac{15}{6} = 2.5 \text{ ha}$$

The volume of water to be abstracted per day is equal to:

$$V = 10 \times 2.5 \times 102.4 = 2\,560 \text{ m}^3\text{/day}$$

The system capacity, assuming 10 hours of irrigation per day, will be equal to:

$$Q = \frac{2\,560}{10} = 256 \text{ m}^3\text{/hr or } 71.1 \text{ l/sec}$$

If, however, this results in large conveyance dimensions, a night storage reservoir could be introduced so that abstraction from the headworks could be continuous (24 hours/day) at peak demand. In such a case, the conveyance system capacity would be 71.1 x (10/24) = 29.6 l/sec.

Table 16
Summary of the calculated design parameters for Nabusenga and Mangui surface irrigation schemes

	Design parameter	Symbol	Nabusenga	Mangui
1.	Net depth of water application	d_{net}	45.5 mm	30.0 mm
2.	Calculated irrigation frequency	IF	7.5 days	4.8 days
3.	Adjusted irrigation frequency	IF	7 days	5 days
4.	Adjusted net depth of water application	d_{net}	42.0 mm	31.0 mm
5.	Adjusted allowable depletion	P	46%	52%
6.	Proposed irrigation cycle	IC	6 days	4 days
7.	Gross depth of water application (overall level)	d_{gross}	102.4 mm	51.7 mm
8.	Gross depth of water application (field level)	d_{gross}	84.0 mm	51.7 mm
9.	Area to be irrigated per day	A	2.5 ha	0.6 ha
10.	Volume of water to be abstracted per day, assuming an irrigation duration of 10 hours per day	V	2 560 m^3/day	310.2 m^3/day
11.	System capacity, assuming an irrigation duration of 10 hours day	Q	256.0 m^3/hr or 71.1 l/sec	31.02 m^3/hr or 8.62 l/sec
12.	System capacity for 24 hours conveyance and night storage	Q	106.6 m^3/hr or 29.6 l/sec	

By incorporating a night storage reservoir in the design of Nabusenga scheme, the system capacity has been reduced to less than half, thus allowing a smaller size conveyance system to be used. The design of night storage reservoirs is discussed in detail in Chapter 6.

Chapter 4
Layout of a surface irrigation scheme

Design of a surface irrigation system may be required for either a planned new irrigation scheme or an existing irrigation scheme where low performance requires improvement by redesigning the system. In both cases, the data required fall into six categories:

- ❖ The water resources to be used, including source of water, flow rates and water quality
- ❖ The topography of the land surface
- ❖ The physical and chemical characteristics of the soil, including infiltration rates, soil moisture holding capacities, salinity
- ❖ The expected cropping pattern
- ❖ The economic and marketing situation in the area and the availability of services, including the availability of labour, maintenance and replacement services, energy, availability of capital for the work
- ❖ The farming practices of the overall farming enterprise

Each surface irrigation and drainage system layout depends on the local situation. In this chapter, some general rules only will be discussed.

4.1. General layout

The main factors determining a surface irrigation scheme layout are:

- ❖ Topography
- ❖ Farm size and degree of mechanization
- ❖ Possible lengths of furrows and borderstrips or possible basin sizes

Two distinct irrigation layouts can be adopted in the design of surface irrigation systems. The layout to choose is very much dependent on the topography of the land amongst other considerations.

The first layout is adaptable to flat lands with slopes of less than 0.4%. In this layout, main canals/pipelines follow the contour lines as much as possible. Secondary canals/pipelines run perpendicular to the contour lines. If, in the case of canals, the ground slope is steep in relation to the required canal gradient, drop structures are incorporated into the design in order to reduce the water velocity (see Section 6.5). The tertiary canals/pipelines, which get their water from the secondary canals, preferably run more or less parallel to the contour lines while the furrows or borderstrips run in the same direction as the secondary canals or perpendicular to the contour lines. The distance between the field canals depends on the design length of the furrows or borderstrips or on the size of the basins. Where the natural slope is too steep, land levelling should be carried out to reduce the slope. Furrows or borderstrips could run at an angle from the tertiary canal in order to reduce the longitudinal slope of the furrow or borderstrip. Basins would require levelling to make them horizontal. An example of this type of layout is given in Figure 17.

The second layout is adaptable to lands with a gentle to steep topography. In this layout, the tertiary canals/pipelines, which get their water from the secondary ones, are constructed in such a manner that they cross elevation contours almost at right angles. Furrows and borders are then constructed along the contours but slightly running away from them to create some gradient for water flow.

Most land topography allows irrigation layouts with field canals/pipelines irrigating only fields located on one side of the canal/pipeline. Sometimes, however, the topography might allow the field canals/pipelines to effectively irrigate the fields located on both sides of the canal/pipeline. Where this occurs, it is called the herringbone layout (Figure 18). It is most adaptable to even slopes with contour lines running more or less parallel to each other. The tertiary canal/pipeline would then run perpendicular to the contours and irrigate both sides.

The herringbone layout reduces costs for infield developments, as fewer canals/pipelines need to be constructed. However, the layout poses challenges to the designer and the irrigator alike. The available command on the two sides of the canal could be different, making it difficult to apply correct volumes of water to each side. Precautions should be taken to ensure that both sides are adequately under command along the entire length of canal/pipeline.

For both layouts described above the angle between the field canal and the furrow should not be too acute, otherwise a very dense and expensive canal system has to be designed. This also leaves large triangular pieces of land at the head of tertiary canal/pipeline without command.

Figure 17
Typical layout of a surface irrigation scheme on uniform flat topography

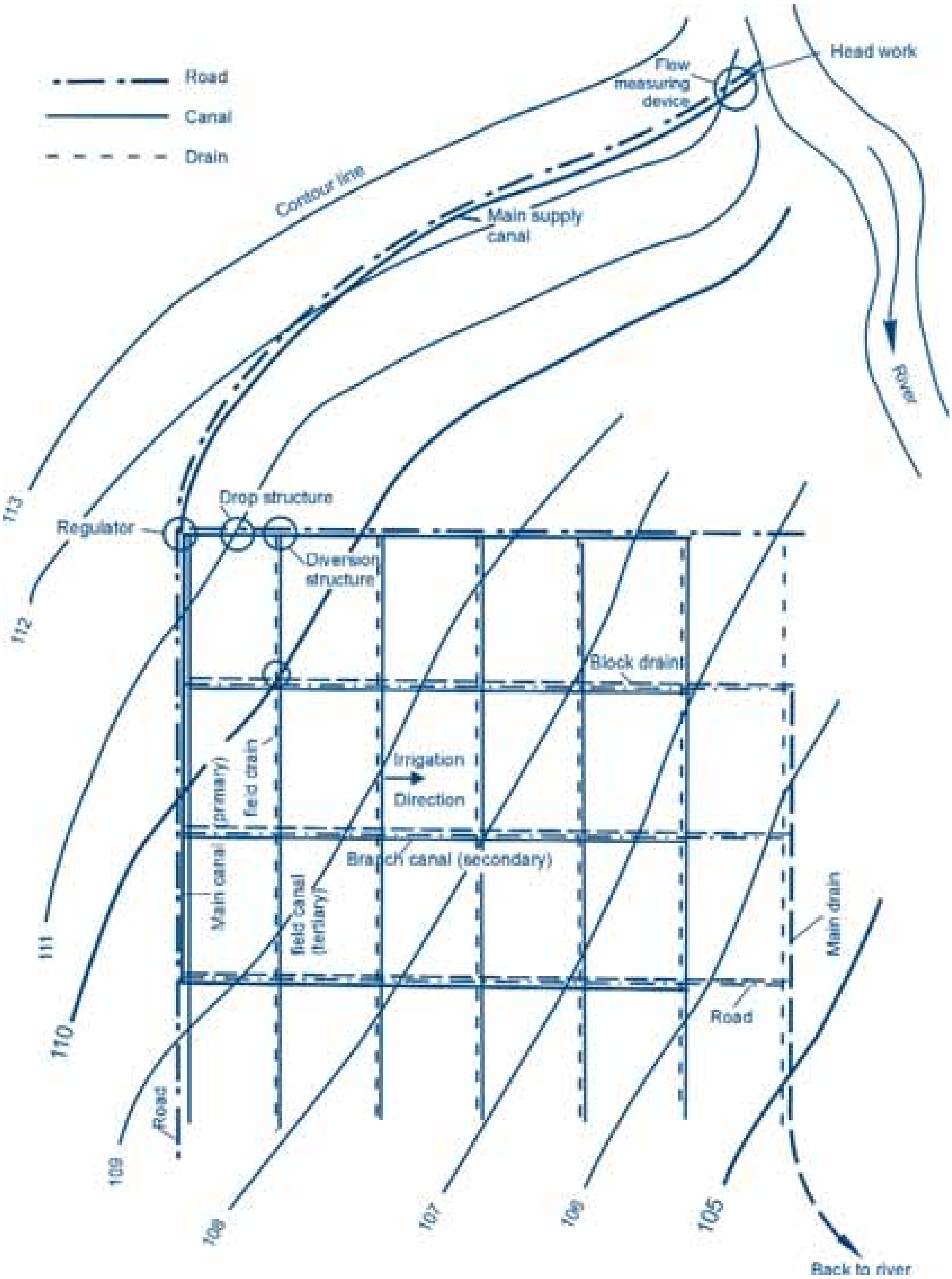
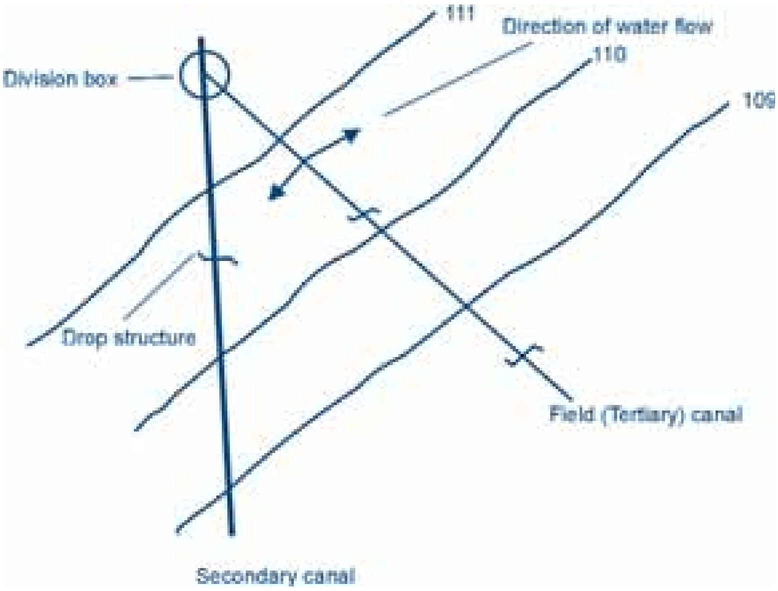


Figure 18
The herringbone irrigation layout



Access to and within the field in the form of roads is important and should be incorporated in the layout. The access road to the scheme is very important for the delivery of inputs as well the marketing of the produce. Roads within the scheme facilitate access for carrying out adequate maintenance works and the transportation of inputs and produce to and from individual farmer plots. It is always desirable to have a perimeter road surrounding the irrigation project as well as a road next to higher order canals and drains. Figures 17 and 19 show the roads network.

4.2. Nabusenga irrigation scheme layout

At Nabusenga, the farmers opted for furrow irrigation. In order to keep the main supply or conveyance system from the water source to the scheme as small as possible, a night storage reservoir (NSR) was incorporated into the layout. The reservoir had to be constructed at the highest point, i.e. southwest of the small stream (Figure 19).

Nabusenga is located on a relatively flat area. Initially the main canal, taking the water from the reservoir, can run parallel to the contours (section *a* in Figure 19). As the area served by the canals is relatively small, no distinction was made between main and secondary canals. The canal starts at the highest point, 92.30 m, and a short section runs in a the northern direction, while another section will run in southwest direction. To avoid this latter section running into the ridge of high ground, it bends westward. The direction of irrigation of section *a* is from south to west. From the canal in section *b* no irrigation takes place, therefore most of it could be in cut. The canal in section *c* will run parallel to contour line 91.50 m.

As the contour lines change direction from contour line 91.00 m downwards, the canal also changes direction to run parallel to that contour line. Water will be siphoned from the canal in section *d*, thus there should be sufficient command. From the end of section *d*, the canal bends in a southwest direction and will run more or less parallel to the river. From here, the highest point of each traverse is located near the river.

Section *f* could irrigate from the canal immediately adjacent to the drain of section *d*, but it can also be irrigated from northwest to southeast direction. This latter option is selected in order to save on canals. The furrow length should be 125 m, so as to use as much land as possible. Section *g* has a slightly steeper slope from northwest to southeast than from northeast to southwest. As the slope is close to 0.5 %, canal *g* will irrigate away from the river from northwest to southeast. The anthill within this plot will be destroyed using a bulldozer. A 90° bend is proposed between sections *i* and *j*. The last three blocks, *h*, *i* and *j,* can be very uniform. The topography is gently sloping and 3 blocks with furrow lengths of 100 m each can be designed from northeast to southwest. Drains are planned parallel to the field canals. The furrow lengths of 100 m fit in very well with the available land.

On the whole furrow lengths were kept in the range of 75-125 m. While the advance and recession test indicated an ideal length of 100 m (see Section 1.3.3), the irregular shape of the land combined with the topography allowed only about 60% of the area to fully comply with the 100 m length of run.

Figure 19
Layout of Nabusenga surface irrigation scheme

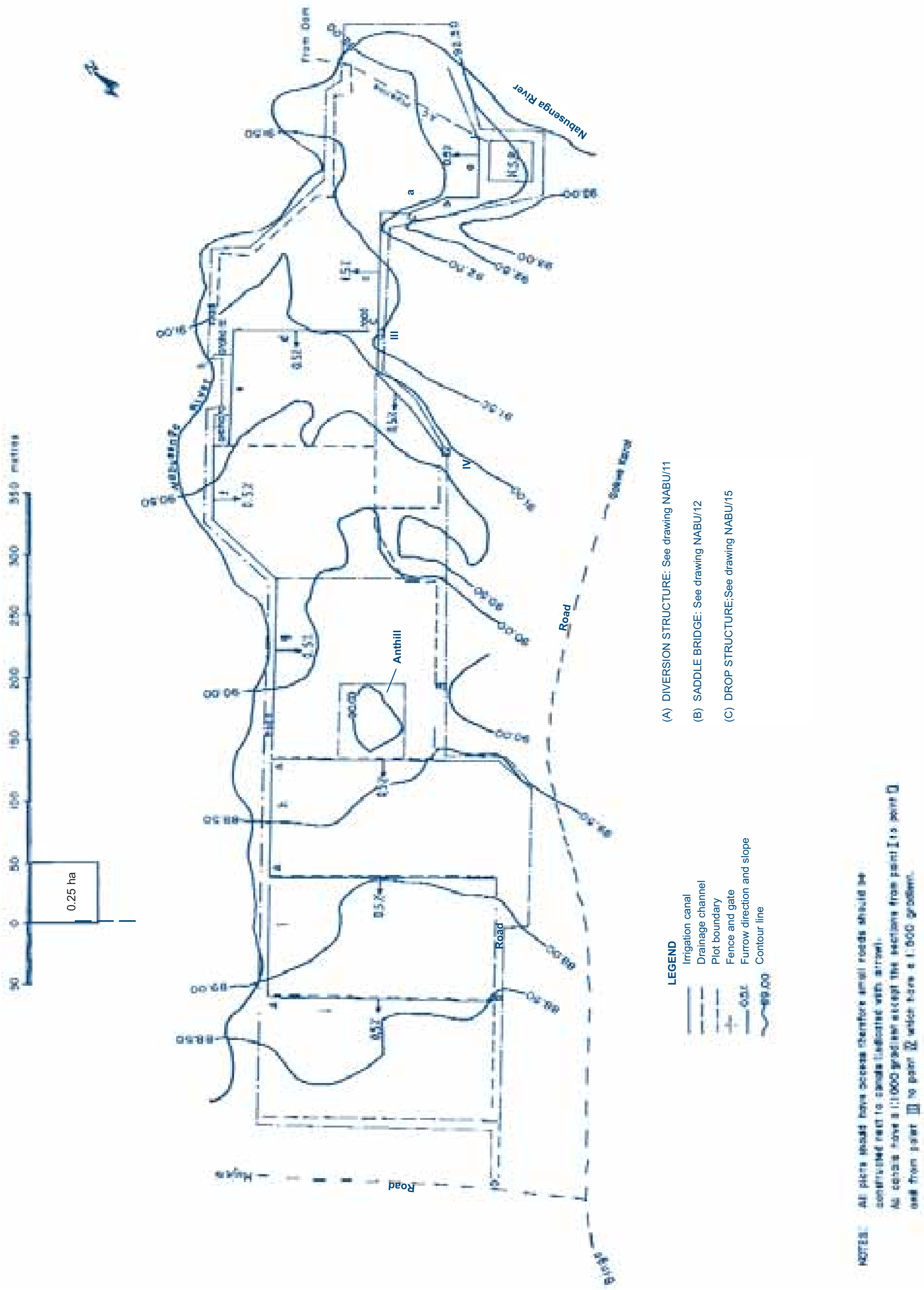
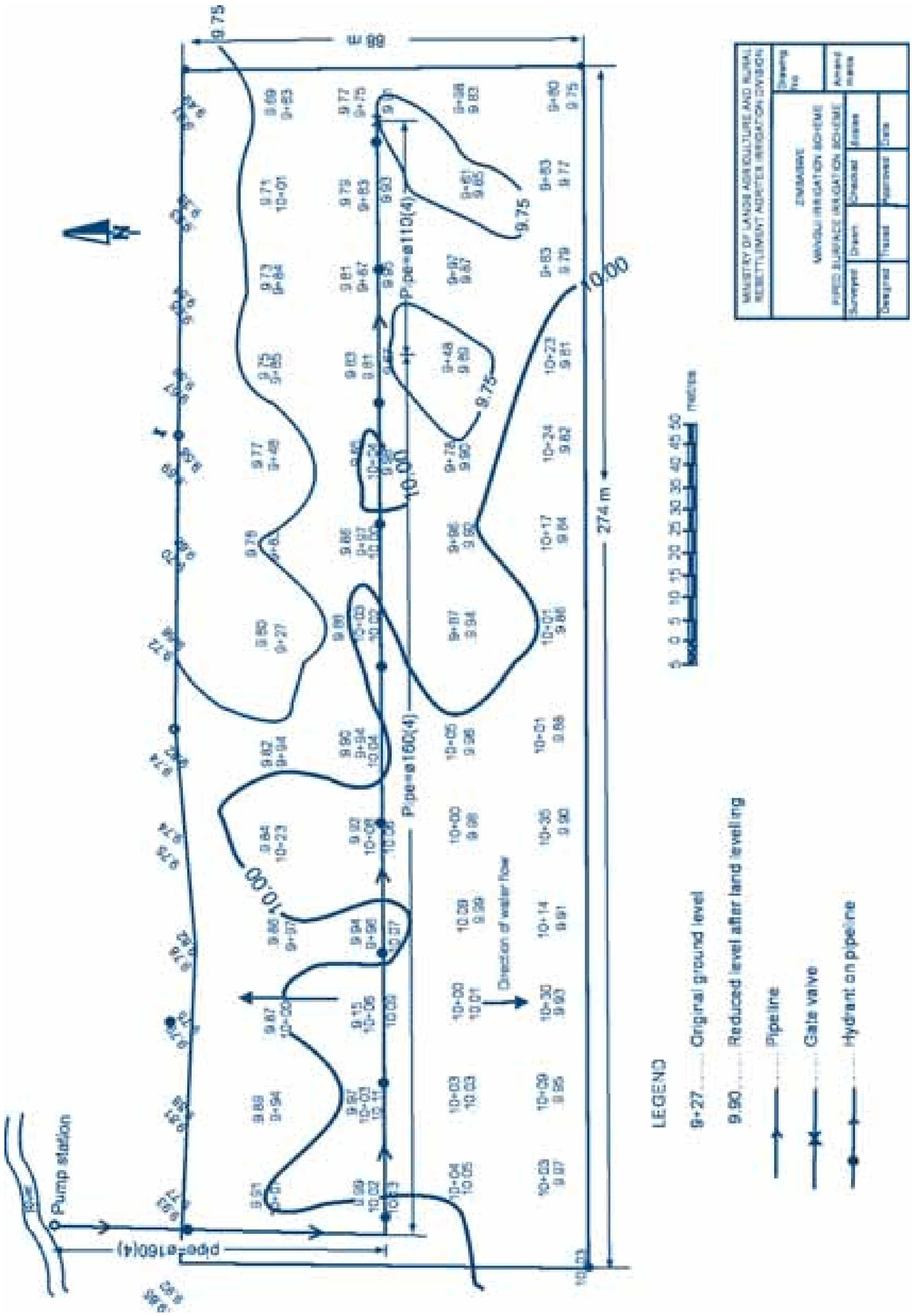


Figure 20
Layout of Mangui piped surface irrigation scheme



4.3. Mangui irrigation scheme layout

Mangui irrigation scheme is located next to a river from where water will be pumped to the scheme (Figure 20). The difference in elevation between the highest point in the field and the water level in the river during the low flow is 5.13 m.

Soil sampling and analysis has shown that the soil is light with available moisture of 80 mm/m by volume. The surveying team did not have access to an infiltrometer, hence no infiltration data were available to the designer. Similarly the design team could not have access to a pump to run an advance and recession test. Therefore the only data available for the designs were the topographic map, the available moisture, climatic data from the nearest meteorological station and the expressed wishes of the farmers to grow specific crops among which maize was prominent.

In order to proceed with the designs, the designers used an infiltration curve from a similar soil (Figure 21), the estimates of furrow length from Table 8 and the CROPWAT software to estimate the crop water requirements. Table 14 and 15 provide the design parameters used. Farmers have requested that the scheme operate for not more than 10 hours per day.

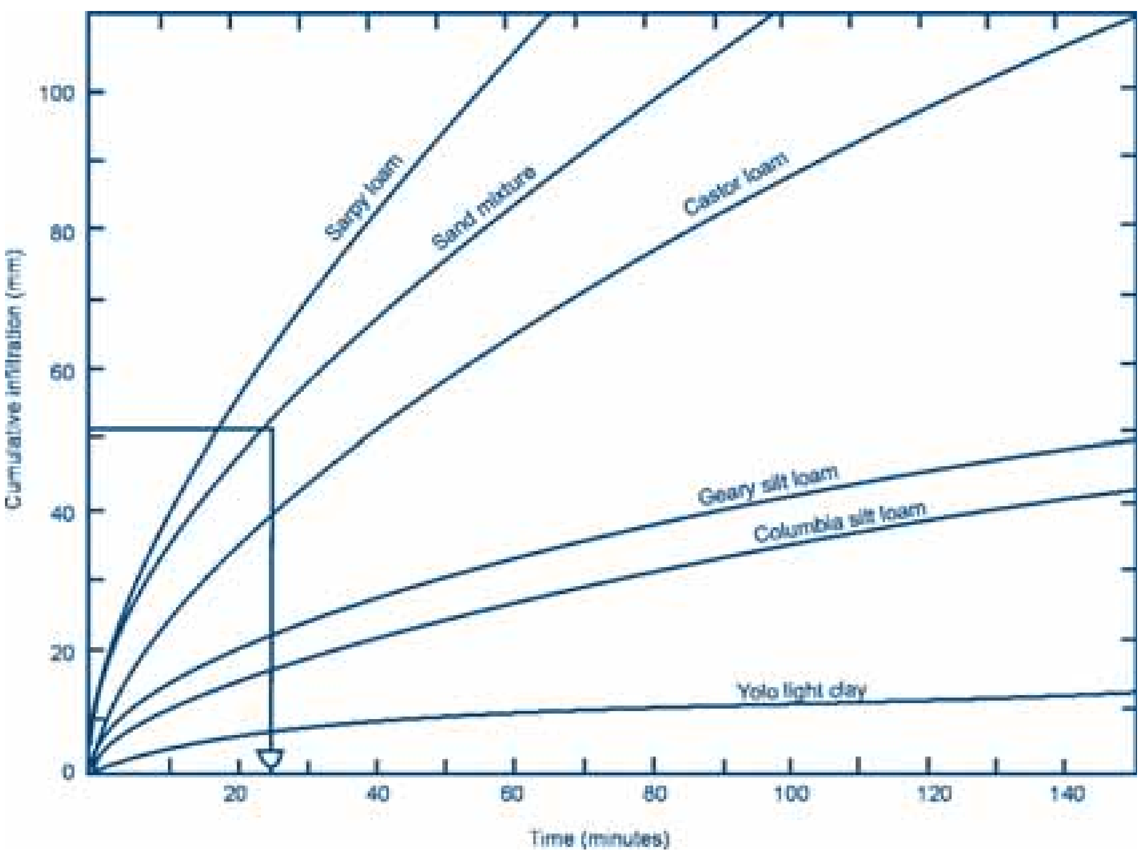
Referring to the cumulative infiltration curve (Figure 21), the contact time required for the application of 51.7 mm of water is estimated to be around 20-25 minutes.

According to Table 8, for light soils the maximum furrow length for a slope of 0.4% should be about 75 m and the maximum stream flow 1.6 l/sec. Looking at the general slope of the land (Figure 20), it appears that it is not possible to accommodate the 75 m length of furrow. It was therefore decided to grade the land and provide a crest in the middle, from east to west, allowing a furrow length of 45 m on each side. The furrow spacing is based on the row spacing for maize, which is 0.9 m.

The irrigation duration for each furrow can be calculated using the following equation:

Figure 21

Cumulative infiltration curves for different soil types (Source: Jensen, 1983)



Equation 11

$$IT = \frac{d_{gross} \times A}{q}$$

Where:

IT = Irrigation duration (seconds)

d_{gross} = Gross depth of irrigation (m)

A = Area covered by one furrow (m^2)

q = Discharge into one furrow (m^3/sec)

Using the above figures gives:

$$IT = \frac{0.0517 \times 0.9 \times 45}{0.0016} = 1\ 309 \text{ seconds}$$

or 22 minutes

This is more or less the same as the contact time according to Figure 21, when considering a sand mixture soil. Applying the one-quarter rule this means that the water should reach the end of the furrow in about 6 minutes.

In order to irrigate the total area of 2.4 ha in 4 days, 0.6 ha per day have to be irrigated, which equals:

$$\frac{6\ 000}{45 \times 0.9} = 148 \text{ furrows}$$

The duration to irrigate 148 furrows = 148 x 22 = 3 256 minutes or 54 hours.

If six furrows are irrigated at the same time, the total duration per day will be 54/6 = 9 hours, which is a slightly less than the 10 hours which is the maximum duration per day that the farmers are willing to irrigate.

In case the flow has to be reduced or cut back once it reaches the end of the furrow (by removing one or more siphons), in order to avoid excessive runoff, the time to finalize the irrigation increases and might even be more than 10 hours, as shown in the calculations below.

When the initial furrow stream of 1.6 l/sec reaches the end of the furrow six minutes after the start, the depth of water will be:

$$d = \frac{(6 \times 60) \times 0.0016}{0.9 \times 45} = 0.0142 \text{ m or } 14.2 \text{ mm}$$

The remaining depth of water to be given is:

51.7 - 14.2 = 37.5 mm

If the flow is reduced to 1.2 l/sec after 6 minutes, the time it takes to apply the remaining depth of 37.5 mm will be:

$$IT = \frac{0.0375 \times 0.9 \times 45}{0.0012} = 1\ 266 \text{ seconds}$$

or 21 minutes

This means that the total duration to irrigate one furrow will be 21 + 6 = 27 minutes instead of 22 minutes.

The duration to irrigate 148 furrows = 148 x 27 = 3 996 minutes or 66 hours.

If six furrows are irrigated at the same time, the total duration per day will be 66/6 = 11 hours, which is more than the 10 hours which is the maximum duration per day that the farmers are willing to irrigated.

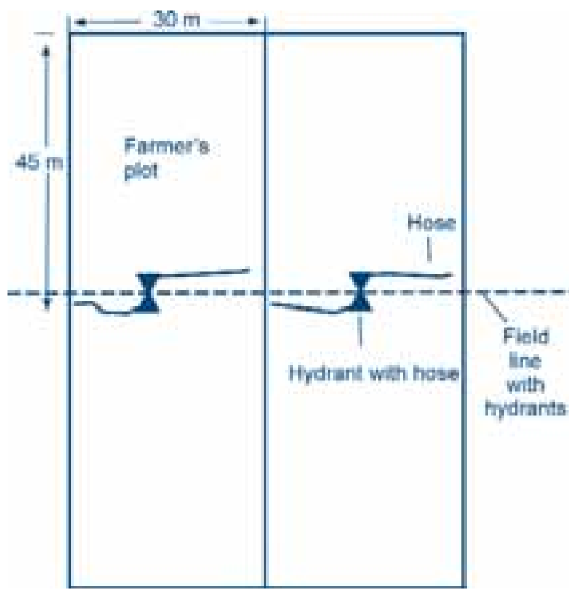
While we retain a maximum flow of 1.6 l/sec per furrow and 6 furrows to be irrigated at the same time (for our design of the piped system of Mangui irrigation scheme in this Module) field tests determining the size of the cutback flow have to be carried out. This will then determine the number of furrows to be irrigated at the same time and thus the system capacity.

When six furrows at a time will be irrigated, as calculated above, the system capacity would then be 6 x 1.6 = 9.6 l/sec, or 34.56 m^3/hr, instead of the originally estimated capacity of 31.02m^3/hr (Table 16).

Looking at Figure 20, a field pipeline runs from west to east in the middle of the field with hydrants installed along this

Figure 22

Plot layout and hydrants



line. Water can be supplied by hoses connected to each hydrant. Along the field line are hydrant risers (bearing the hydrants/gate valves) spaced at 30 m intervals. This line would then be connected to the pumping unit with a supply line. In order to provide some flexibility to the 18 smallholders participating in the scheme, each farmer will be provided with one hydrant and a hose as shown in Figure 22. Each farmer has a plot of 30 m x 45 m. Each farmer will irrigate one furrow at a time, meaning that 6 farmers can irrigate at the same time (see Section 5.2.2).

Each hose will supply the furrows on either side of the hydrant. The distance from the hydrant to the furthest furrow is about 15 m. Therefore, the length of the hose should be about 20 m to allow for loops and avoid sharp bending by the hydrant.

Chapter 5
Design of canals and pipelines

5.1. Design of canals

The canal dimensions and longitudinal slope, whether for irrigation or drainage, can be calculated through trial and error with the Manning formula. This formula is derived from the continuity equation and the equation for unsteady flow. These equations have been simplified by assuming steady uniform flow in the canal (this assumes long canals with constant cross-section and slope).

The Continuity equation is expressed as:

Equation 12

$$Q = A \times V$$

Where:

Q = Discharge (m^3/sec)
A = Wetted cross-sectional area (m^2)
V = Water velocity (m/sec)

The Manning Formula can be expressed as:

Equation 13

$$Q = K_m \times A_s \times R^{2/3} \times S^{1/2}$$

or

$$Q = \frac{1}{n} \times A_s \times R^{2/3} \times S^{1/2}$$

Where:

Q = Discharge (m^3/sec)
K_m = Manning roughness coefficient ($m^{1/3}$/sec)
n = Roughness coefficient; $K_m = 1/n$ or $n = 1/K_m$ (sec/$m^{1/3}$)
A_s = Wetted cross-sectional area (m^2)
P = Wetted perimeter (m)
R = Hydraulic radius (m) ($R=A_s/P$)
S = Canal gradient or longitudinal slope of the canal

A flowchart showing the various steps in applying the Manning Formula is given in Figure 23.

Figure 23
Flowchart for canal design calculations

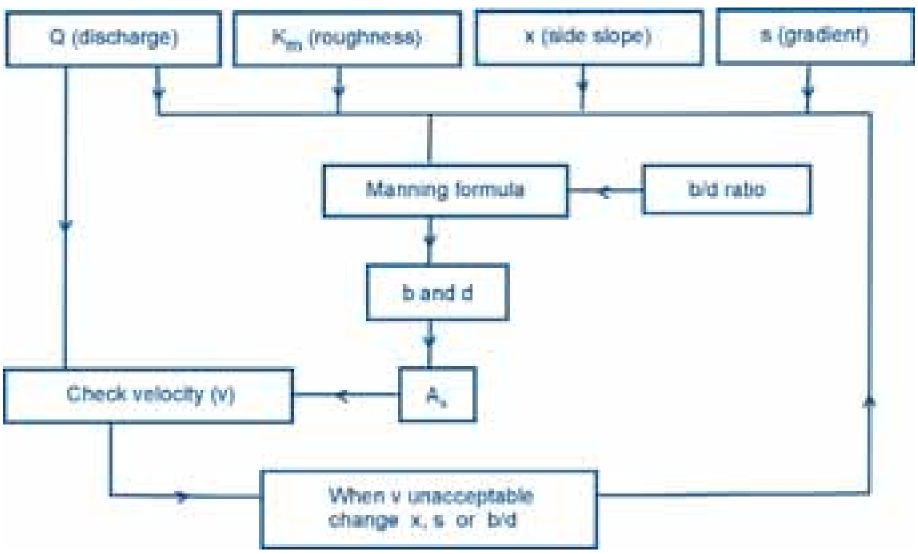
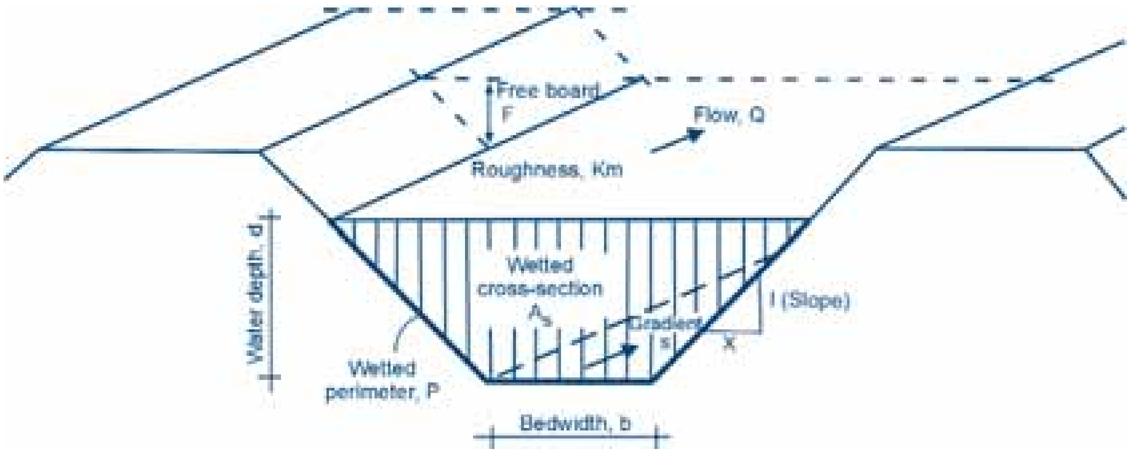


Figure 24
Canal parameters



5.1.1. Calculation of the cross-section, perimeter and hydraulic radius of a canal

Figure 24 shows the different canal parameters. A_s and P, and thus R in the Manning formula, can be expressed in d, b and X, where:

d	=	Water depth (m)
b	=	Bed width (m)
X	=	Side slope = horizontal divided by vertical

For a trapezoidal canal, A_s is the sum of a rectangle and two triangles.

The cross-sectional area of a rectangle is:

Equation 14

$$\text{Area of rectangle} = (b \times d)$$

The cross-sectional area of a triangle is:

Equation 15

$$\text{Area of triangle} = \frac{1}{2}(\text{base} \times \text{height}) = \frac{1}{2}(Xd \times d)$$

Thus, the wetted cross-sectional area A_s of the trapezoidal canal is:

Equation 16

$$\begin{aligned} A_s &= b \times d + 2(1/2 \times Xd \times d) \\ &= b \times d + Xd^2 \\ &= d(b + Xd) \end{aligned}$$

The wetted perimeter is the sum of the bed width b and the two sides from the water level to the bottom. The length of a side, considering the formula $c^2 = a^2 + b^2$, is:

$$\sqrt{d^2 + d^2X^2}$$

Thus the wetted perimeter for the trapezoidal canal section is:

Equation 17

$$P = b + \{2(d^2 + (dX)^2\}^{1/2} = b + 2d(1 + X^2)^{1/2}$$

The hydraulic radius R is:

Equation 18

$$R = \frac{d(b + Xd)}{b + 2d(1 + X^2)^{1/2}}$$

Although the trapezoidal canal shape is very common, other canal shapes, including V-shaped, U-shaped, semi-circular shaped and rectangular shaped canals, can also be designed as shown in Figure 25.

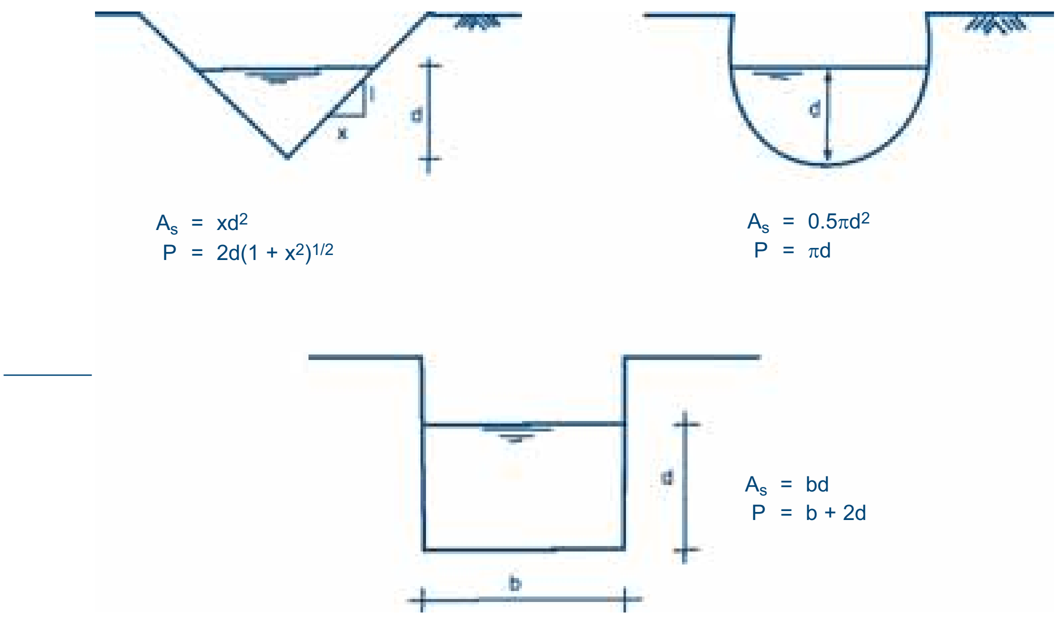
5.1.2. Factors affecting the canal discharge

Canal gradient or longitudinal slope of the canal

The steeper the gradient, the faster the water will flow and the greater the discharge will be. This is substantiated by the Continuity Equation (Equation 12).

Velocity increases with an increase in gradient or longitudinal slope. It therefore follows that a canal with a steeper gradient but with the same cross-section can discharge more water than a canal with a smaller gradient.

Figure 25
Different canal cross-sections



The recommended maximum slope is 1:300 (that is 1 m drop per 300 m canal length), which is equal to 0.33%. Steeper slopes could result in such high velocities that the flow would be super-critical (see Chapter 6). It would then be difficult, for example, to siphon water out of the canal, since an obstruction in a canal where super-critical flow occurs tends to cause a lot of turbulence, which could result in the overtopping of the canal. This is due to the change from the super-critical state to the sub-critical state. The state of flow could be checked using the Froude Number.

The Froude Number (Fr) is given by:

Equation 19

$$Fr = \frac{V}{(g \times I)^{1/2}}$$

Where:

V = Water velocity (m/sec)

g = Gravitational force (9.81m/sec^2)

I = Hydraulic depth of an open canal, defined as the wetted cross-sectional area divided by the width of the free water surface (m)

Fr = 1 for critical flow

Fr > 1 for super-critical flow

Fr < 1 for sub-critical flow

It is important to maintain a Froude number of 1 or less so that flow is at or below the critical level.

Canal roughness

The canal roughness, as depicted by the Manning roughness coefficient, influences the amount of water that passes through a canal. Unlined canals with silt deposits and weed growth and lined canals with a rough finish tend to slow down the water velocity, thus reducing the discharge compared to that of a clean canal with a smooth finish. Canals that slow down the movement of water have a low K_m or a high n (see Equation 13). It should be understood that the higher the roughness coefficient K_m, or the lower n, the higher the ability of the canal to transport water, hence the smaller the required cross-sectional area for a given discharge.

The roughness coefficient depends on:

- The roughness of the canal bed and sides
- The shape of the canal
- Canal irregularity and alignment
- Obstruction in the canal
- Proposed maintenance activities

Typical K_m and n values are given in Table 17.

Manning coefficients K_m often are assumed too high during the design phase compared to what they actually will be during scheme operation due to deterioration of the canals. The result is an increased wetted cross-sectional area of the

Table 17
K_m and n values for different types of canal surface (adapted from: Euroconsult, 1989)

Type of surface	Range of roughness coefficient	
	K_m (= 1/n) in $m^{1/3}$/sec	n (= 1/K_m) in sec/$m^{1/3}$
Pipes, precast and lined canals		
Metal, wood, plastic, cement, precast concrete, asbestos, etc.	65-100	0.010-0.015
Concrete canal and canal structures	65-85	0.012-0.016
Rough concrete lining	40-60	0.017-0.025
Masonry	30-40	0.025-0.035
Corrugated pipe structures	40-45	0.023-0.025
Earthen canals, straight and uniform		
Clean, recently completed	50-65	0.016-0.020
Clean, after weathering	40-55	0.018-0.025
With short grass, few weeds	35-45	0.022-0.027
Earthen canals, winding and sluggish		
No vegetation	35-45	0.023-0.030
Grass, some weeds	30-40	0.025-0.033
Dense weeds or aquatic plants in deep channels	25-35	0.030-0.040
Canals, not maintained, weeds and brush uncut		
Dense weeds, as high as flow depth	8-20	0.050-0.120
Clean bottom, brush on sides	10-25	0.040-0.080

canal during scheme operation with the danger of overtopping the canal banks. This in turn means that the canal discharge has to be reduced to below the design discharge, in order to avoid overtopping. There is therefore a need for regular and proper maintenance of canals.

Canal shape

Canals with the same cross-sectional area, longitudinal slope and roughness, but with different shapes, will carry different discharges because of different wetted perimeters and hydraulic radii (see Equation 13). The most efficient geometry is when the wetted perimeter is minimal for a given discharge. Under these circumstances, the cross-sectional area for a given discharge will also be minimal. The optimum canal shape, hydraulically, also tends to be the cheapest to construct as the amount of surface lining material required will be minimized.

The semi-circle is the canal section that has the lowest wetted perimeter for a given cross sectional area, but semi-circular canals are difficult to construct. The closest canal section to a semi-circle is the trapezoid. This is a quite common cross-section as it is relatively easy to construct. Figure 26 shows rectangular and trapezoidal canals with different hydraulic radii. Canals with narrower beds and higher water depths have a smaller wetted perimeter, and thus a higher discharge, than canals with larger beds and lower water depths, for the same cross-sectional area. This is due to the fact that the hydraulic radius R (= A_s/P) increases if the wetted perimeter decreases, while keeping the wetted cross-sectional area the same (see Equation 13).

Side slope

The side slope X (= horizontal/vertical) should be selected depending on the type of canal, soil type and the expected vegetation cover on the slopes.

Earthen canals

If the side slopes are very steep (low X) there is high risk of banks collapsing, especially after heavy rainfall. Therefore, a compromise has to be reached between loss of land (due to larger width of canal surface) and bank safety. Table 18 gives suggested side slopes for canals in different soil types.

Table 18
Typical canal side slopes

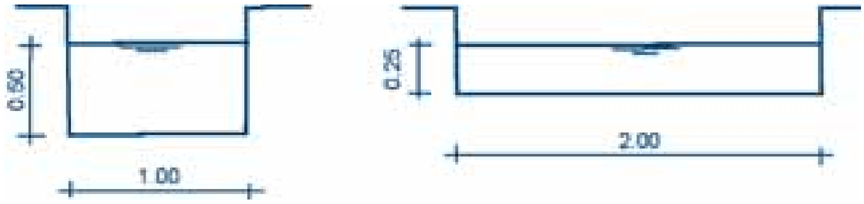
Soil type	Side slope X (=horizontal/vertical)
Stiff clay or earth with concrete lining	1 to 2
Heavy, firm clay or earth for small ditches	1 to 1.5
Earth, with stone lining or earth for large canals	1
Fine clay, clay loam	1.5 to 2
Sandy clay or loose sandy earth	2
Fine sand or sandy loam	2 to 3
Coarse sand	1.5 to 3

Concrete-lined canals

There are no strict rules for the side slopes of concrete-lined canals. A major consideration is ease of construction and the fact that the concrete should stay in place during

Figure 26

Hydraulic parameters for different canal shapes



$A_s = 0.5 \times 1.0 = 0.5\ m^2$
$P = (2 \times 0.5) + 1.0 = 2.0$ m
$R = 0.5 / 2.0 = 0.25$ m

$A_s = 0.25 \times 2.0 = 0.5\ m^2$
$P = (2 \times 0.25) + 2.0 = 2.5$ m
$R = 0.5 / 2.5 = 0.2$ m

$A_s = (0.5 \times 0.75) + (0.75 \times 0.75) = 0.94\ m^2$
$P = 0.5 + 2 \times 0.75 \times (1 + 1^2)^{½} = 2.62$ m
$R = 0.94 / 2.62 = 0.36$ m

$A_s = (1.5 \times 0.475) + (0.475 \times 0.475) = 0.94\ m^2$
$P = 1.50 + 2 \times 0.475 \times (1+1^2)^{½} = 2.84$ m
$R = 0.94 / 2.84 = 0.33$ m

construction, thus the side slope should not be too steep. Side slopes of around 60º should be easy to construct.

Bed width / water depth ratio for trapezoidal canals

The recommended bed width/water depth (b/d) ratios for earthen trapezoidal canals are given in Table 19.

Table 19

Recommended b/d ratios

Water depth	b/d ratio
Small (d < 0.75 m)	1 (clay) - 2 (sand)
Medium (d = 0.75-1.50 m)	2 (clay) - 3 (sand)
Large (d > 1.50 m)	> 3

The bed width should be wide enough to allow easy cleaning. A bed width of 0.20-0.25 m is considered to be the minimum, as this still allows the cleaning of the canal with small tools such as a shovel. Lined trapezoidal canals could have similar b/d ratios as given above.

Maximum water velocities

The maximum permissible non-erosive water velocity in earthen canals should be such that on the one hand the canal bed does not erode and that on the other hand the water flows at a self-cleaning velocity (no deposition). A heavy clay soil will allow higher velocities without eroding than will a light sandy soil. A guide to the permissible velocities for different soils is presented in Table 20. In winding canals, the maximum non-erosive velocities are lower than in straight canals.

Table 20

Maximum water velocity ranges for earthen canals on different types of soil (Source: Peace Corps Information Collection and Exchange, undated)

Soil type	Maximum flow velocity (m/sec)
Sand	0.3 - 0.7
Sandy loam	0.5 - 0.7
Clayish loam	0.6 - 0.9
Clay	0.9 - 1.5
Gravel	0.9 - 1.5
Rock	1.2 - 1.8

Lined canals can manage a range of velocities, as erosion is not an issue. However, for easy management of water, the permissible velocity should be critical or sub-critical.

Freeboard

Freeboard (F) is the vertical distance between the top of the canal bank and the water surface at design discharge. It gives safety against canal overtopping because of waves in canals or accidental raising of the water level, which may be a result of closed gates.

The freeboard can be calculated using Equation 20:

Equation 20

$$F = C \times h^{1/2}$$

Where:

C = 0.8 for discharges of up to 0.5 m^3/sec up to 1.35 for discharges in excess of 80 m^3/sec

h = Water depth (m)

Example 7

What is the water depth for a trapezoidal canal with the following known parameters:

$Q = 0.09\ m^3/sec$	$K_m = 55$ (rough concrete lining)
$S = 0.001$ (0.1%)	$X = 1$ (45°)
$b = d$	$V = < 0.75$ m/sec

The cross-sectional area of a trapezoidal canal is given by Equation 16:

$$A_s = d(b + Xd)$$

Substituting the above given data for b, d, and X gives:

$$A_s = d(d + d) = 2d^2$$

The wetted perimeter is given by Equation 17:

$$P = b + 2d(1 + X^2)^{1/2}$$

Substitution again of the above given data for b, d, and X gives:

$$P = d + 2d(1 + 1^2)^{1/2} = d + 2d(1.414) = d + 2.83d = 3.83d$$

The hydraulic radius is given by Equation 18:

$$R = \frac{A_s}{P} = \frac{2d^2}{3.83d} = 0.52d$$

The Manning formula is given by Equation 13:

$$Q = K_m \times A_s \times R^{2/3} \times S^{1/2}$$

This gives:

$$0.09 = 55 \times 2d^2 \times (0.52d)^{2/3} \times 0.001^{1/2} = 110d^2 \times (0.52d)^{2/3} \times 0.0316 = 1.807d^{2.66}$$

$$d = \sqrt[2.66]{0.09/1.807} \Rightarrow d = 0.30 \text{ m}$$

$$A_s = 2d^2 = 2 \times 0.30^2 = 0.18 \text{ m}^2$$

$$V = \frac{Q}{A_s} = 0.09 / 0.18 = 0.50 \text{ m/sec}$$

This means that the water velocity is less than the maximum allowable velocity given of 0.75 m/sec, which is acceptable. However, the Froude Number should be calculated using Equation 19 to make sure the flow is sub-critical:

$$Fr = \frac{V}{(g \times I)^{1/2}}$$

Where:

$$I = \frac{A_s}{\text{Width of free water surface}} = \frac{A_s}{b + 2d} = \frac{0.18}{(0.30 + 2 \times 0.30)} = 0.20$$

$$\text{Thus, } Fr = \frac{0.50}{(9.81 \times 0.20)^{1/2}} = 0.36 \text{ which is} < 1.$$

This means that the flow is sub-critical.

For lined canals, F ranges from 0.40 m for discharges less than 0.5 m^3/sec up to 1.20 m for discharges of 50 m^3/sec or more. For very small lined canals, with discharges of less than 0.5 m^3/sec, the freeboard depths could be reduced to between 0.05-0.30 m.

5.1.3. Hydraulic design of canal networks using the chart of Manning formula

The hydraulic design of canal networks for irrigation and drainage requires the following steps (Euroconsult, 1989):

1. Design water surface levels in relation to natural ground slope and required head for irrigation of fields or for drainage to outlet, taking into account head losses for turnouts and other structures.

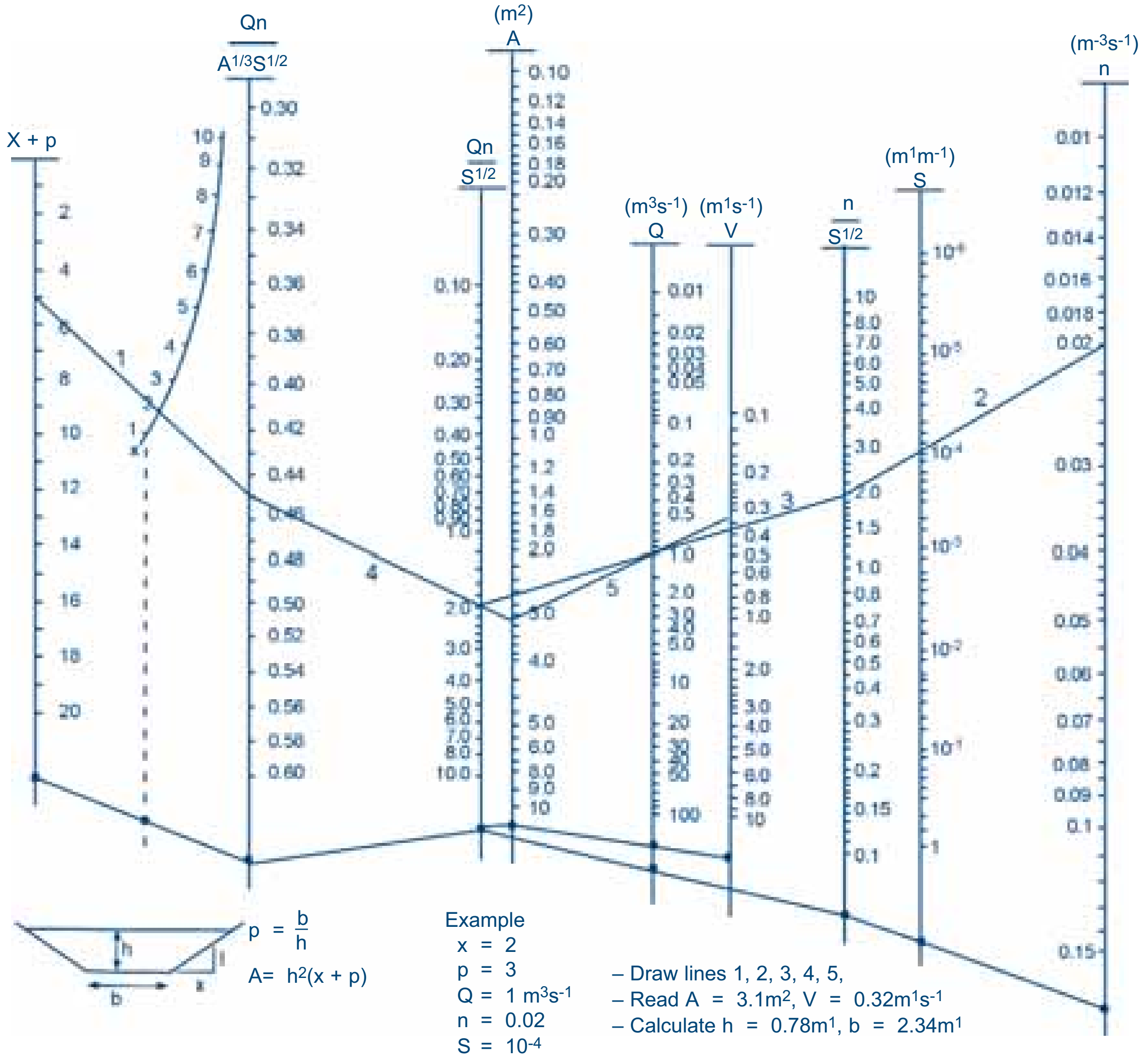
2. Calculate corresponding hydraulic gradients.
3. Divide network into sections of uniform slope (S) and discharge (Q).
4. Determine required design (maximum) discharge per section.
5. Select roughness coefficient (K_m or n)
   – side slopes
   – preferred minimum velocity and permissible maximum velocity
   – bottom width/water depth ratio
6. Calculate hydraulic section dimensions and corresponding velocity, using:
   – nomograph series, if available
   – the nomograph presented in Figure 27 (chart of Manning formula)
   – basic equations and calculator
7. Check calculated velocities against preferred and maximum velocity values; if V is too high, reduce hydraulic gradient and corresponding bottom slope. The gain in head should preferably be used in upstream and downstream canal sections but, if this is impossible, it must be absorbed by drop structures.

The chart presented in Figure 27 can be used to determine the optimum canal parameters for trapezoidal canal sections through trial and error.

5.1.4. Canal section sizes commonly used by Agritex in Zimbabwe

The Irrigation Branch of Agritex in Zimbabwe has adopted a 60° trapezoidal canal. The following standard size sections have been recommended: a flow depth of 0.30 m plus freeboard of 0.05 m, with bed widths of 0.25 m, 0.30 m, 0.375 m and 0.50 m depending upon gradient and capacity

Figure 27
Chart of Manning formula for trapezoidal canal cross-sections



or discharge required. The total depth of 0.35 m (water depth + freeboard) is easily reached by construction gangs while placing concrete. It provides an adequate siphon head and gives efficient flows within range. The narrowest bed width used, 0.25 m, is still easy to clean out with a shovel.

By varying the bed width only and not the depth, transition from one section to another is simplified. This involves no loss of head and also overcomes the need to make an allowance when pegging the canal.

The capacities for the above types of Agritex canal sections have been worked out and are presented in Table 21.

Table 21
Canal capacities for standard Agritex canal sections

Canal gradient & hydraulic data	Canal bottom widths in mm							
	250		300		375		500	
	Velocity (m/sec)	Capacity (l/sec)	Velocity (m/sec)	Capacity (l/sec)	Velocity (m/sec)	Capacity (l/sec)	Velocity (m/sec)	Capacity (l/sec)
1 : 300	0.79	100.0	0.875	124	1.02	168	1.09	205
1 : 500	0.62	78.0	0.675	96	0.78	128	0.85	172
1 : 750	0.50	63.5	0.595	85	0.68	112	0.69	140
1 : 1 000	0.43	54.5	0.475	68	0.55	92	0.60	126
A_S (m^2)	0.127		0.142		0.165		0.202	
P (m)	0.946		0.996		1.071		1.196	
R (m)	0.135		0.143		0.163		0.17	
K_m	55		55		55		55	

Example 8

What is the bed width for a trapezoidal canal with a side slope angle of 60° and a water depth of 0.3 m, assuming K_m = 55 and that the canal has to discharge 78.30 l/sec at a gradient of 0.001 (0.1%) and 0.002 (0.2%) respectively?

In order to calculate X, one has to determine the tangent as follows:

$$\text{Tan } 60° = \frac{1}{X}\text{, therefore } X = \frac{1}{\tan 60°} = \frac{1}{1.73} = 0.58$$

Substituting the value for X and the water depth d = 0.30 m in Equations 16 and 17 respectively gives:

$$A_s = 0.30(b + 0.58 \times 0.30) = 0.30(b + 0.174) = 0.3b + 0.05$$
$$P = b + 2d(1 + X^2)^{1/2} = b + 2(0.30)(1 + 0.58^2)^{1/2} = b + 0.6(1.156) = b + 0.69$$

The hydraulic radius, using Equation 18, is:

$$R = \frac{A_s}{P} = \frac{(0.30b + 0.05)}{(b + 0.69)}$$

Substituting the data in the Manning Formula gives:

$$Q = 55 \times (0.30b + 0.05) \times \left[\frac{(0.30b + 0.05)}{(b + 0.69)}\right]^{2/3} \times 0.001^{1/2}$$

Substituting values of bed widths in the formula by trial and error will result in a bed width that suits the design discharge, fixed at 0.0783.

Try b = 0.20 m. The result of the calculation is a flow Q = 0.049 m^3/sec when the gradient is 0.001. This means that the canal with a bed width b = 0.20 m and a water depth d = 0.30 m will not be able to discharge the design flow of 0.0783 m^3/s.

After a few runs of trial and error, we get Q = 0.0783 m^3/sec, when b =0.35 m, for the 0.001 gradient and 0.24 m for the 0.002 gradient, with water velocities of 0.50 m/s and 0.64 m/s respectively.

The engineer in Zimbabwe designing the canals would simply use Table 21 to choose a canal section with 250 mm bed width at a gradient of 1 : 500 and 350 or 375 mm bed width at a gradient of 1 : 1 000.

Calculation of the Froude Number according to Equation 19 gives a value of 0.26, implying that the flow is sub-critical.

5.1.5. Longitudinal canal sections

The best way to present canal design data for construction is to draw a longitudinal profile of the canal route and to tabulate the data needed for construction. The longitudinal profile shows the chainage or distance along the canal at the horizontal or x-axis and the elevations of the natural ground, the ground after levelling and the canal bed at the vertical or y-axis. The data are tabulated under the graph, showing the elevation of ground and canal bed in figures at each given distance. Water depths could also be shown. The chainage starts from a reference point, which is usually the beginning of the canal. Where possible the survey results of the topographic survey are used. If these are not sufficient, a detailed survey of the proposed alignments should be made. The following are guidelines for the presentation of longitudinal profiles.

- Direction: water flow is always given from left to right.
- Horizontal scale:

  1 : 1 000 for short canals (1 cm = 10 m)

  1 : 5 000 for long canals (1 cm = 50 m)
- Vertical scale:

  1 : 20 for small canals and low gradient (1 cm = 0.2 m)

  1 : 100 for larger canals and higher gradient (1 cm = 1 m)

  (Note: the vertical scale should be chosen in such a way that the water depth is clearly visible)
- The profile should show the ground level, the bed level and eventually the water level at design discharge
- Structures should be marked by a vertical line at the place of the structure, with the structure identification written along the x-axis
- Distance is measured in metres from the canal inlet, with intervals depending on length to be covered (5, 10, 50 m etc.). For very long canals, it can be measured in kilometres. The distance to structures or major change of direction is always measured and added to the tabulated data
- Ground levels are tabulated from survey data
- Bed levels and eventually water levels are tabulated at the end of each reach, which means upstream and downstream of each structure where the water level changes. A single value can be given at structures when there is no fall

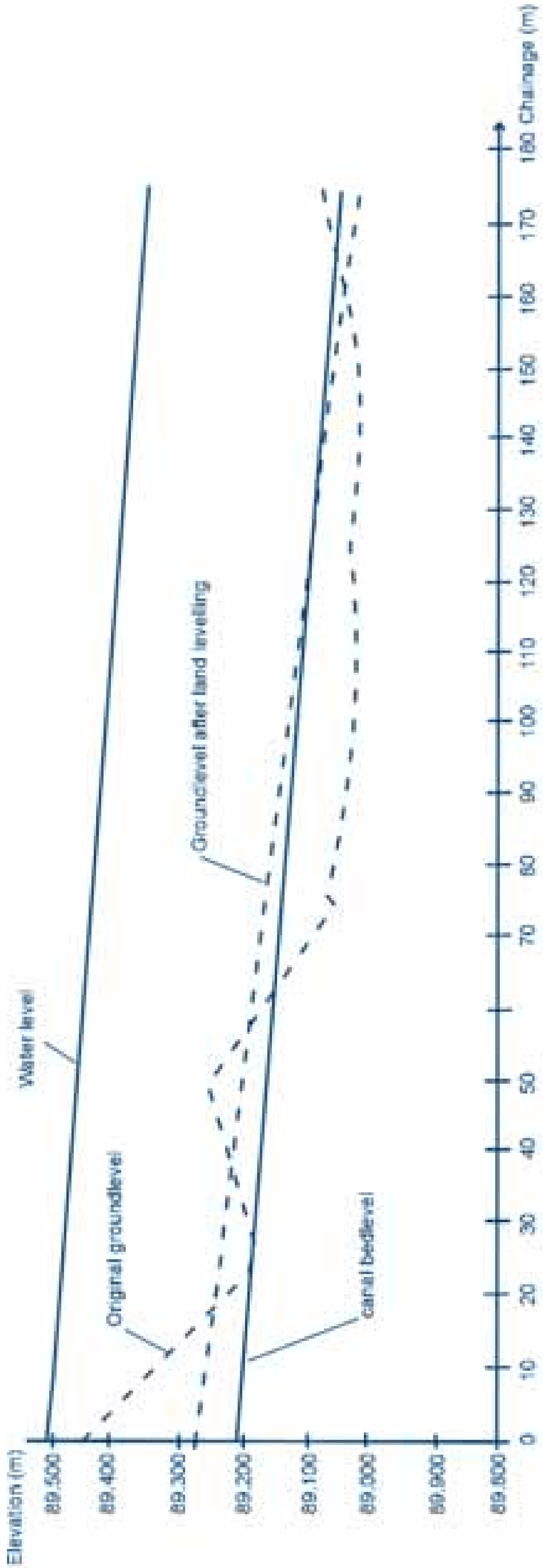
Figures 28, 29 and 20 show longitudinal sections of a field, secondary and conveyance canal respectively.

The field or tertiary canals should have sufficient command over the whole length in order to allow the correct discharge to be supplied to the field. Figure 28 shows an example of a field canal with sufficient command over its full length after land levelling. For these canals the ground elevations after land levelling have to be taken into account in deciding the slope of the canal. As normal practice, the water depth should be more or less 10-15 cm above the levelled ground surface in order to maintain a good siphoning head.

Secondary and main canals can be designed in cut at places where there are no offtakes. The designer should ensure that there is sufficient command at field canal offtakes. Figure 29 illustrates this. Ideally, an offtake should be placed before a drop.

Figures 30 and 31 show examples of longitudinal profiles of a conveyance canal. The starting bed elevation of the conveyance canal should be high enough to give sufficient command to the lower order canals. The conveyance canal itself does not necessarily need to have a water level above ground level since no water will be abstracted from it. It is in fact preferable to design them in cut as much as possible. Where possible, it could run quasi-parallel to the contour line as shown in Figure 19. Drops should be incorporated, when the canal goes in fill, but the command required should be maintained.

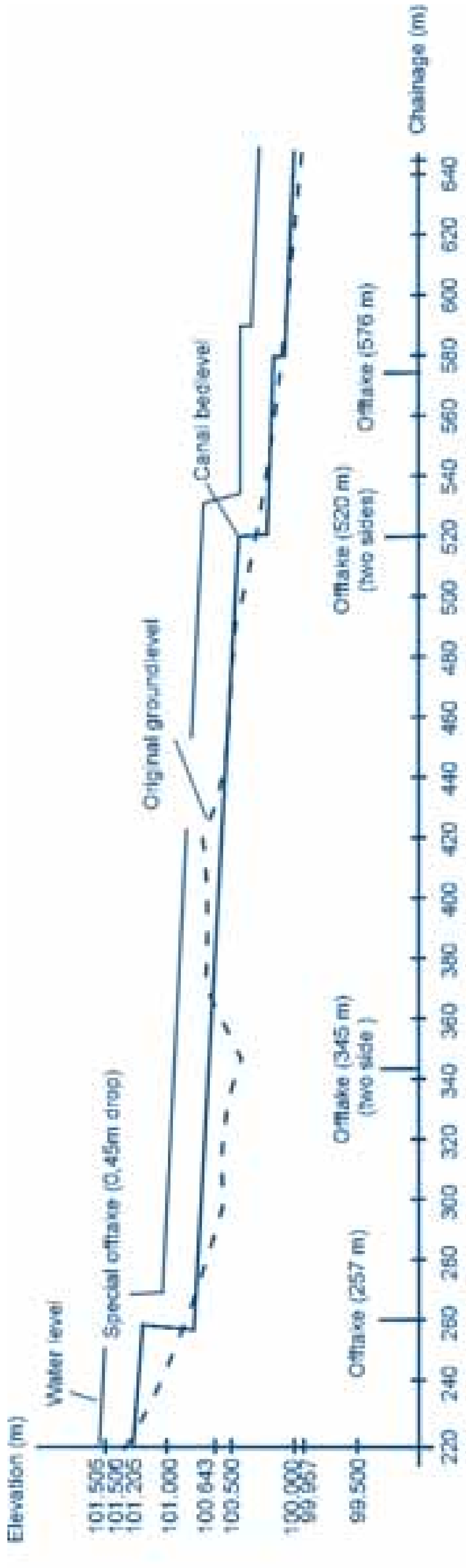
Figure 28
Longitudinal profile of a field or tertiary canal



Chainage (m)	Ground level after levelling (m)	Canal bedlevel (m)
0	89.280	89.220
10	89.271	89.210
20	89.250	89.200
25	89.245	89.195
30	89.240	89.190
40	89.225	89.180
50	89.210	89.170
60	89.194	89.160
70	89.179	89.150
75	89.172	89.145
80	89.164	89.140
90	89.149	89.130
100	89.134	89.120
110	89.118	89.110
120	89.103	89.100
125	89.096	89.095
130	89.088	89.090
140	89.073	89.080
150	89.058	89.070
160	89.042	89.060
170	89.027	89.050
175	89.020	89.145

FIELD CANAL AT CHAINAGE 834 OF SECONDARY CANAL

Figure 29
Longitudinal profile of a secondary or main canal



Chainage (m)	Original groundlevel (m)	Canal bedlevel (m)
245	100.871	101.205
257		101.093 100.643
270	100.621	100.630
295	100.466	100.605
320	100.444	100.580
345	100.614	100.555
370	100.610	100.530
395	100.591	100.505
420	100.641	100.480
445	100.485	100.455
470	100.446	100.430
495	100.370	100.405
520	100.254	100.380 100.130
576	100.034	100.074
580		100.020 100.020
644		99.957

Figure 30
Longitudinal profile of a conveyance canal

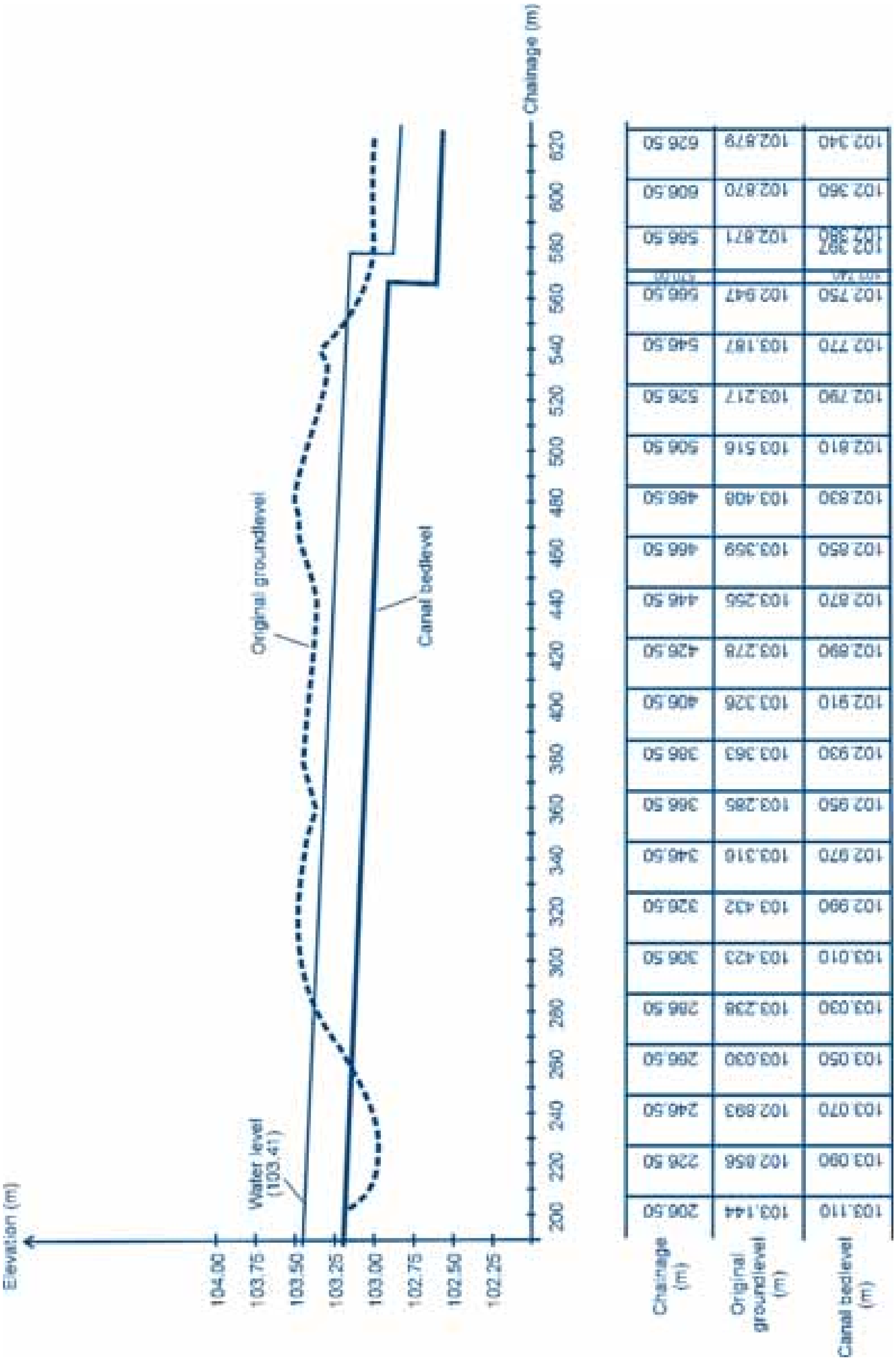


Figure 31

Example of a longitudinal section of a conveyance canal

5.1.6. Field canals for small irrigation schemes

Field canals (tertiary canals and sometimes secondary canals) usually run at an average gradient of 1:500 (0.0020 or 0.2%) to 1:300 (0.0033 or 0.33%). When the existing land slope exceeds the proposed canal gradient, drop structures can be used in order to avoid the canal being suspended too much above the ground level, which would require too much fill.

A common drop in small canals is 0.15 m. Such small drops do not require stilling basins because of their short fall (see Chapter 6). In order to have a minimum of 0.15-0.20 m command, the drop is constructed when the bed level of the canal reaches the ground level after land levelling. A small Cipoletti weir (see Chapter 6) is constructed at every drop in order to allow for support for the check plate.

A problem often encountered is that field canals in irrigation schemes lack command, making siphoning onto adjacent land difficult or even impossible. One main reason for the lack of command is the use of the original ground level to site the drops. Once the canal bed level has reached the original ground level, it is dropped by 0.15 m. However, when land levelling is done afterwards, it might result in fill near the canal, thus reducing or eliminating the command.

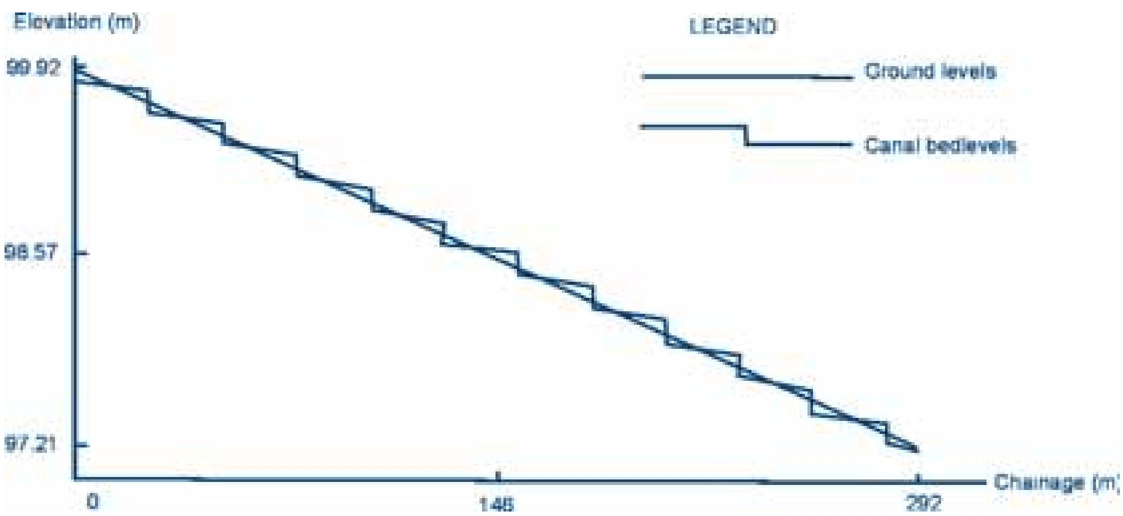
Computer programmes for calculating the location and elevation of the standard drop structures are available nowadays. As an example a short description of the Lonsec programme, which is such kind of programme, is given below. It is written in Quick Basic.

The programme requires the input of:

1. The chainage at the beginning and at the end of the field canal
2. The ground levels after land levelling at these two chainages
3. The canal bed level at the beginning of the canal
4. The canal gradient (it assumes a uniform canal gradient)

Figure 32

Longitudinal canal profile generated by the Lonsec Programme



The output consists of the ground level, the canal bed level immediately before and after a standard drop and the chainage where a drop occurs. Furthermore the ground and canal bed levels are calculated at 10 m intervals, independent of the fact whether there is a drop structure or not. Thus, the output is suitable for use during construction.

The programme can also show and print a visual impression of the longitudinal section of the field canal. Table 22 shows an example of output data, while Figure 32 gives the visual impression, which could for example be included in feasibility reports together with the output tables.

Table 22
Longitudinal profile for field canal – output from the Lonsec computer programme

Chainage (m)	Ground level (m)	Canal level (m)	Canal level after drop (m)
0.0	99.920	99.820	
10.0	99.828	99.787	
20.0	99.737	99.754	
25.6	99.686	99.736	99.586
30.0	99.645	99.572	
40.0	99.554	99.539	
50.0	99.462	99.506	
51.1	99.452	99.502	99.352
60.0	99.371	99.323	
70.0	99.279	99.290	
76.7	99.218	99.268	99.118
80.0	99.188	99.108	
90.0	99.096	99.075	
100.0	99.004	99.042	
102.2	98.984	99.035	98.885
110.0	98.913	98.859	
120.0	98.821	98.826	
127.7	98.751	98.801	98.651
130.0	98.730	98.644	
140.0	98.638	98.611	
150.0	98.547	98.578	
153.3	98.516	98.567	98.417
160.0	98.455	98.395	
170.0	98.364	98.362	
178.8	98.283	98.333	98.183
180.0	98.272	98.179	
190.0	98.180	98.147	
200.0	98.089	98.114	
204.3	98.050	98.100	97.950
210.0	97.997	97.931	
220.0	97.906	97.898	
229.9	97.815	97.866	97.716
230.0	97.814	97.715	
240.0	97.723	97.683	
250.0	97.631	97.650	
255.4	97.582	97.632	97.482
260.0	97.540	97.467	
270.0	97.448	97.434	
280.0	97.357	97.401	
280.9	97.348	97.398	97.248
290.0	97.265	97.219	
292.0	97.248	97.212	

1. The chainage at the beginning of the section = 0.0 m and the chainage at the end of the section = 292.0 m
2. The ground level after levelling at the beginning of the section = 99.92 m and the ground level after levelling at the end of the section = 97.24 m
3. The canal bed level at the beginning of the section = 99.82 m
4. The canal gradient = 0.0033 (1:300)

5.1.7. Seepage losses in earthen canals

Unlined earthen canals are the most common means of conveying irrigation water to irrigated lands. Farmers prefer them because they can be built cheaply and easily and maintained with farm equipment. Unlined canals are also flexible, as it is easy to change their layout, to increase their capacity or even to eliminate or rebuild them the next season. However, unlined canals have many disadvantages that make them less desirable compared to lined canals or underground pipes. These are:

- They usually lose more water due to seepage, leakage and spillage
- Rodents can cause leakage
- Frequent cleaning is needed because of weed growth
- Earth ditches can erode and meander, creating problems in maintaining straight or proper alignments
- Labour costs of maintenance of unlined canals are normally higher than of lined canals and pipelines
- They provide an ideal environment for the vector of bilharzia

When designing earthen canals, it is important to ensure that the slope is such that the bed does not erode and that the water flows at a self-cleaning velocity (see Section 5.1.2). From all standpoints, relatively flat lands on soils with a high percentage of silt and clay are the most suitable for canal construction, because of low infiltration rates.

In earthen canals, seepage occurs through the canal bed and sides. In areas where relatively permeable soils are used to construct canals, high seepage can be expected. The higher the seepage losses in the canals the lower the distribution system (conveyance and field canal) efficiencies, since much less water than that diverted at the headworks reaches the fields.

Seepage is difficult to predict. Two simple ways to estimate seepage losses are:

1. Measurement of inflow into and outflow from the canal at selected points. The difference between the inflow and outflow measurements will not only represent seepage losses, but evaporation losses as well.
2. Measurement of the rate of fall of the water level in a canal stretch that has been closed and where the water is ponding. From these losses the estimated evaporation should be subtracted to get the seepage losses.

Usually, seepage losses are expressed in m^3 of water per m^2 of the wetted surface area of a canal section (P x L) per day. If a field test cannot be carried out, seepage can be estimated from Table 23, which gives average seepage losses for different types of soil.

Table 23
Seepage losses for different soil types

Type of soil	Seepage (m^3 water/ m^2 wetted surface area per day)
Impervious clay loam	0.07 - 0.10
Clay loam, silty soil	0.15 - 0.23
Sandy loam	0.30 - 0.45
Sandy soil	0.45 - 0.55
Sandy soil with gravel	0.55 - 0.75
Pervious gravelly soil	0.75 - 0.90

Seepage could be localized where a portion of highly permeable material has been included in the bank or where compaction has been inadequate during canal construction.

5.1.8. Canal lining

Seepage always occurs, even if the canals are constructed with clay soils. If there is abundant water available that can be diverted under gravity, one might accept the water losses without resorting to lining. In fact, worldwide, unlined canals are the most common as they are the cheapest and easiest type of canal to construct. However, if water has to be used more efficiently, due to its scarcity or if it has to be pumped, it usually becomes economical to line the canals. Another consideration in analyzing the economics is the health-related cost (of medicines and time lost by smallholders due to poor health).

Canal lining is generally done in order to reduce seepage losses and thus increase the irrigation efficiencies. It also substantially reduces drainage problems and canal maintenance as well as water ponding, thus reducing the occurrence of vector-borne diseases. Also, smooth surface linings reduce frictional losses, thereby increasing the carrying capacity of the canals.

Below different lining methods are briefly explained. The actual construction is dealt with in detail in Module 13.

Example 9

An earthen canal with a 1:1000 gradient, constructed in and using sandy loam, is designed to convey 78.3 l/sec for 24 hours per day over a distance L of 2 km. The Manning coefficient for the canal K_m is 30, the side slope X is 1.5 and the b/d is 1.5. What are the seepage losses as a percentage of the daily discharge?

The canal cross sectional area is calculated from the Manning Formula as follows:

$$Q = K_m \times A_s \times R^{2/3} \times S^{1/2}$$

Where:

$A_s = 1.5d^2 + 1.5d^2 = 3d^2$

$P = 5.10d$

Substitution, of A_s and P in the equation gives:

$$0.0783 = 30 \times 3d^2 \times \left[\frac{3d^2}{5.10d}\right]^{2/3} \times (0.001)^{1/2} \Rightarrow d = 0.30 \text{ m}$$

Therefore: $A_s = 0.27 \text{ m}^2$ and $P = 1.53$ m

The total wetted surface area over the 2 km stretch is:

Wetted surface area = P x L = 1.53 m x 2 000 m = 3 060 m^2

The seepage loss through a sandy loam is estimated at 0.40 m^3/m^2 per day (Table 23). Thus, the total estimated seepage loss from the canal is:

Total seepage loss per day = 3 060 x 0.40 = 1 224 m^3/day

The total volume of water supplied per day = 0.0783 x 24 x 60 x 60 = 6 765 m^3

This means that approximately $\frac{1\,224}{6\,765} \times 100 = 18\%$ of the supplied water is lost to seepage.

Material used for lining:

- Clay
- Polyethylene plastic (PE)
- Concrete
- Sand-cement
- Brick
- Asbestos cement (AC)

The selection of a lining method depends mainly on the availability of materials, the availability of equipment, the costs and availability of labour for construction.

Clay

If a sufficient volume of clay soil can be found in the vicinity of the scheme, clay lining might be the cheapest method to use to reduce seepage losses. One has to ensure that the clay is well spread in the canal and well compacted. However, clay lining is susceptible to weed growth and possible soil erosion.

Polyethylene plastic

Polyethylene plastic sheeting can be used for lining canals. The sheets have to be covered with well-compacted soil, since the plastic deteriorates quickly when exposed to light. Furthermore, tools such as shovels and slashers can easily damage it during maintenance works. Weed growth and soil erosion could also cause problems in the canal.

Concrete

The materials required for concrete lining are cement, fine and coarse aggregates. Concrete lining is an expensive but very durable method of lining. When properly constructed and maintained, concrete canals could have a serviceable life of over 40 years. This durability is an important aspect to consider, more so for small-scale self-run schemes in remote areas. Details on the preparation of concrete lining are given in Module 13.

Sand-cement

If coarse aggregates are not available for the preparation of concrete, the method of sand-cement lining could be considered. A strong mixture is either placed in-situ on the canal sides and bed or is precast (thickness 5-7 cm). A mix of 1:4 (cement : river sand) is recommended. More details are given in Module 13.

Brick

If good clayish soils, suitable for producing good quality burnt bricks, are found near the scheme area, brick lining could be considered. The construction however is laborious. Cement is required for mortar and plastering. A disadvantage of this lining method is the large amount of firewood needed to burn the bricks. It could, however, be justified if the scheme area had to be cleared of trees, which could then be used for burning the bricks.

Asbestos cement

Precast asbestos cement flumes can be used as lining materials. The flumes are easy to place and join. A disadvantage is usually the high unit cost and the health risk of working with asbestos.

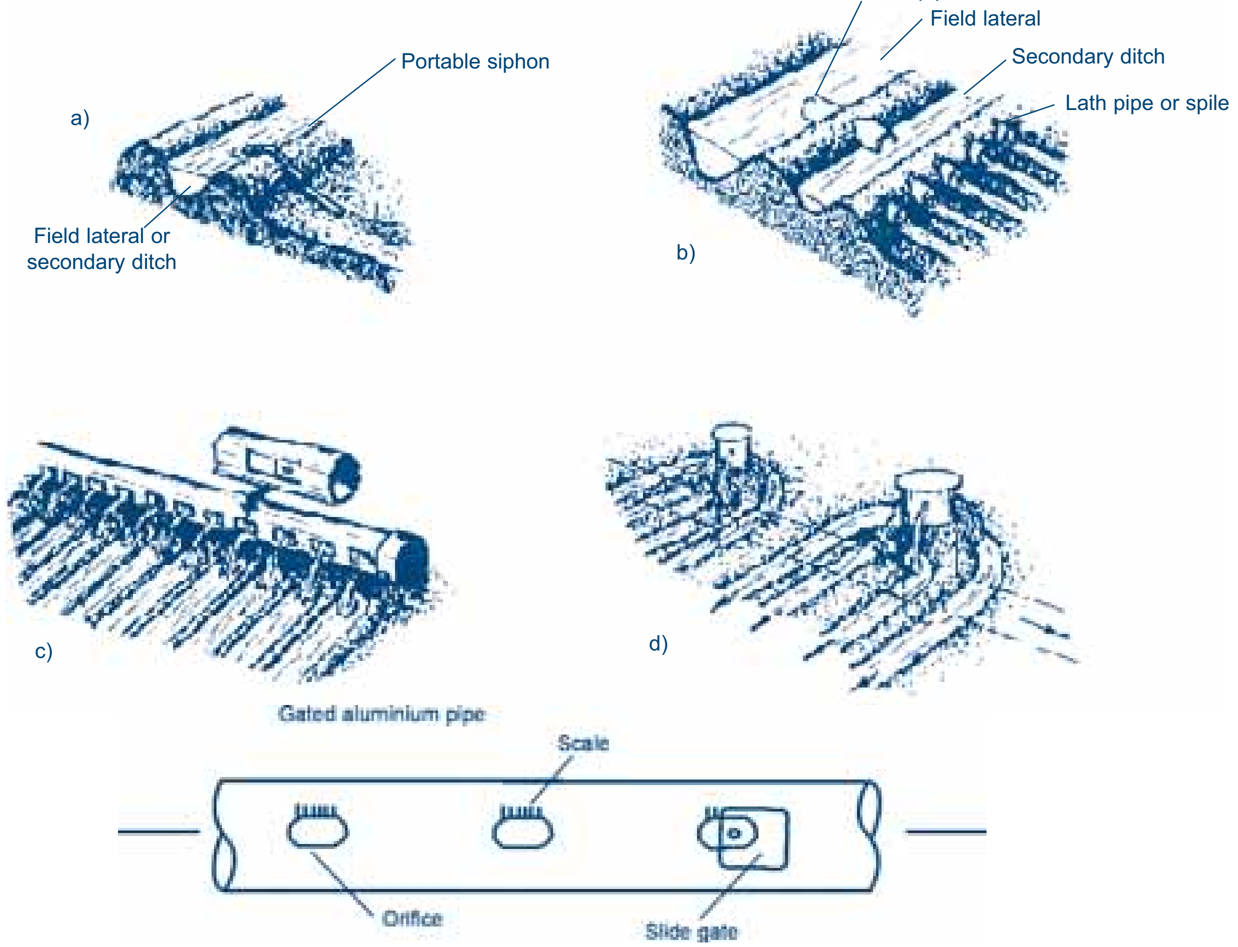
5.2. Design of pipelines

In piped surface irrigation systems, water is transported in closed conduits or pipes in part or all of the distribution system from the headwork up to the field inlet. The pipes can be all buried, with outlets in the form of hydrants protruding above ground level on field pipes. Or only the conveyance and supply lines can be buried with field pipes being portable and laid above ground. In the latter case, the above ground pipes are made of aluminium fitted with adjustable gate openings (Figure 33).

Piped systems for surface irrigation, unlike piped systems for sprinkler irrigation, do not require a lot of head at the hydrant outlet. The head should only be sufficient to push water through the irrigation hose that takes the water from the hydrant to the soil. In view of the low head requirements for the systems, it is possible to employ gravity flow where there is sufficient head to overcome the frictional losses in pipes. In situations where the head is not adequate, small power pumps would be used with low operational costs. Pipes with low-pressure rating are also used for these systems as they operate at reasonably low pressures. At times when the pressure in the system is very low, buried PVC pipes rated at two bar can be used with these systems, if available.

If the water level at the headwork is higher than the water level required at scheme level, the water can be transported through the pipes by gravity. If the water level at the headwork is lower than the water level required at scheme level, then the water needs to be pumped through the pipe

Figure 33
Methods commonly used to introduce water into the field



to arrive at the scheme at the required elevation necessary to be able to irrigate by gravity from the field inlet onwards.

5.2.1. Design of the conveyance pipeline in Nabusenga irrigation scheme

The friction losses of the outlet pipe and the conveyance pipe should not exceed the difference in elevation between the lowest drawdown level in Nabusenga dam and the top of the scheme or the block of fields. To ensure this, there is a need to draw a longitudinal profile of the alignment of the pipeline. The profile will show the elevations of the pipeline corresponding to distances from a reference point (also called chainages) along the pipe alignment. Figure 34 shows the longitudinal section of a pipeline from the Nabusenga dam to the top of the scheme. Figures 35 and 36 can be used to calculate the friction losses in AC and uPVC pipes respectively.

High points along the proposed alignment should be carefully checked in order to ensure that there is enough head available to discharge the required flow over these points. Measures to be taken to ensure this include:

- ❖ Excavation of a deep trench. This may not always be feasible for huge elevation differences due to the nature of the underlying bedrock and the distance over which the digging has to be done
- ❖ Taking a new route altogether for the pipeline
- ❖ Changing the pipe size diameter with the hope that the friction losses would be reduced sufficiently to overcome the problem

The pressure is generally lower at high points along pipelines and air or other gases tend to be released from solution forming an air pocket that interrupts the flow of water. It is imperative that air-release valves be fitted at these points to let air out of the system when it forms. Along our pipeline, an air-release valve would be fitted at chainage 880 m.

5.2.2. Design of the piped system in Mangui irrigation scheme

Based on the layout discussed earlier (see Section 4.3, Figure 20 and Figure 22), each farmer's plot will be equipped with one hydrant and one hose irrigating one furrow at a time. A total of eighteen hydrants (gate valves) have been provided for the system. One option would be allowing six hydrants to operate at a time. Another option would be that all water is delivered to one hydrant and that thus one farmer would irrigate at a time. Such an option, while technically feasible, would increase the cost of the system in addition to requiring more labour per plot to manage the water to the level required for 60% field application efficiency. Also the hose diameter would be too large for the farmer to move around.

Allowable pressure variation and head losses in the hose

Before proceeding with the calculations of the hydraulics, it should be pointed out that the system should be designed for equity in water supply. Therefore, each hose should provide about the same amount of water ± 5%. For this reason, the pressure variation within the system should not exceed 20% of the head losses in the hose.

The Hazen-Williams equation will be used for this purpose.

Equation 21

$$Hf_{100} = \frac{K \times \left(\frac{Q}{C}\right)^{1.852}}{D^{4.87}}$$

Where:

Hf_{100}	=	Friction losses over a 100 m distance (m)
K	=	Constant 1.22×10^{12}, for metric units
Q	=	Flow (l/s)
C	=	Coefficient of retardation based on type of pipe material (C = 140 for plastic)
D	=	Inside diameter (mm)

Table 24 gives C values for different materials.

Table 24

Hazen-Williams C value for different materials

Material	Constant C
uPVC	140 - 150
Asbestos cement (AC)	140
Cast iron (new) (CI)	130
Galvanized steel (new) (GS)	120

Note:When aging, the roughness of cast iron and galvanized steel pipes increases. For example, for a year old cast iron pipe the C might be reduced to 120 and to 100 for a 20-year-old cast iron pipe.

Assuming that a 50 mm inside diameter and 20 m long hose is used, the friction losses for a flow of 1.6 l/s will be as follows:

$$Hf_{100} = \frac{1.22 \times 10^{12} \times \left(\frac{1.6}{140}\right)^{1.852}}{50^{4.87}} = 1.64 \text{ m per 100 m}$$

For the 20 m hose the head losses HL will be:

$$HL = 1.64 \times (20/100) = 0.32 \text{ m}$$

Figure 34
The longitudinal profile of the conveyance pipeline from Nabusenga dam to the night storage reservoir (NSR)

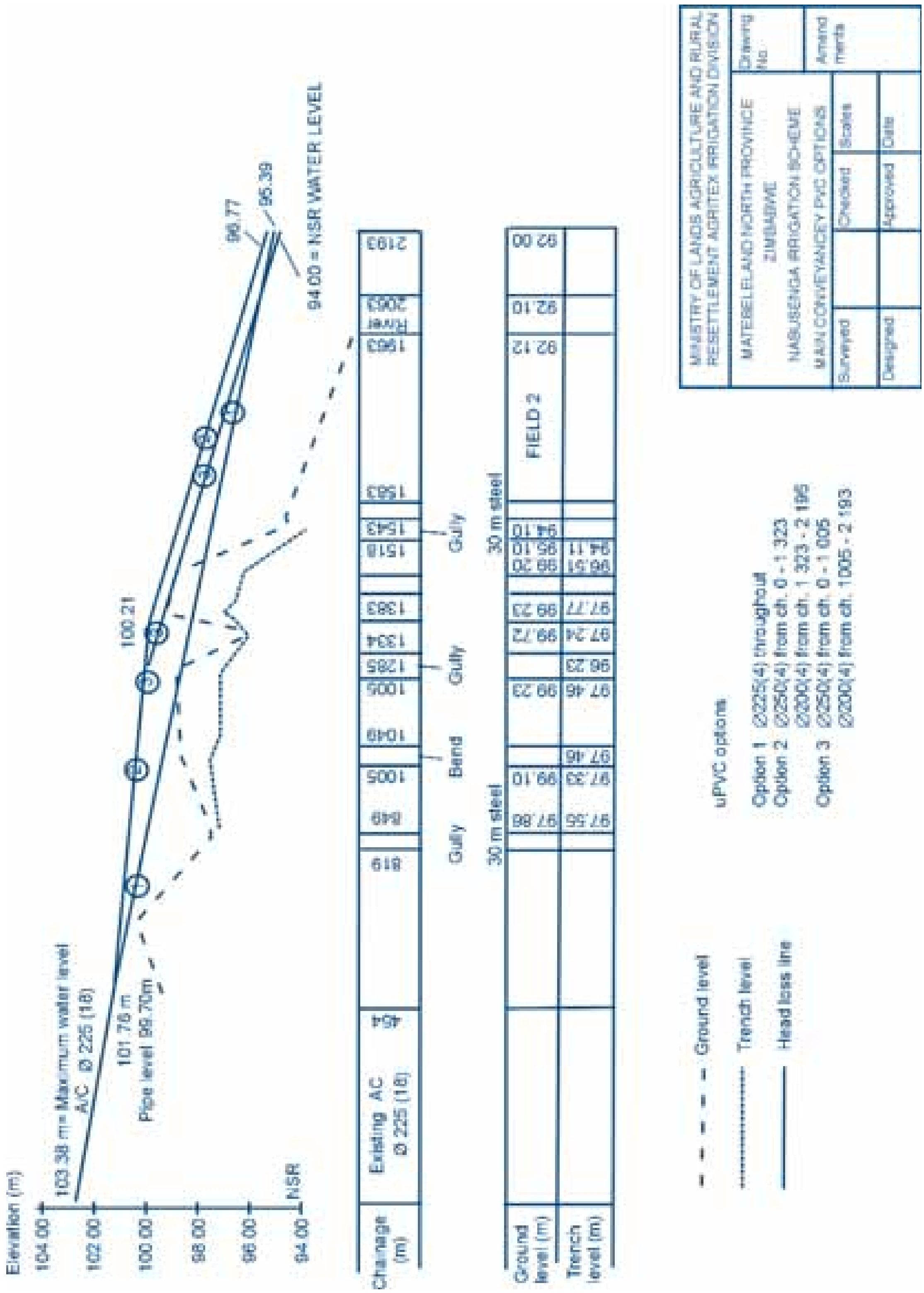


Figure 35
Friction loss chart for AC pipes (Class 18)

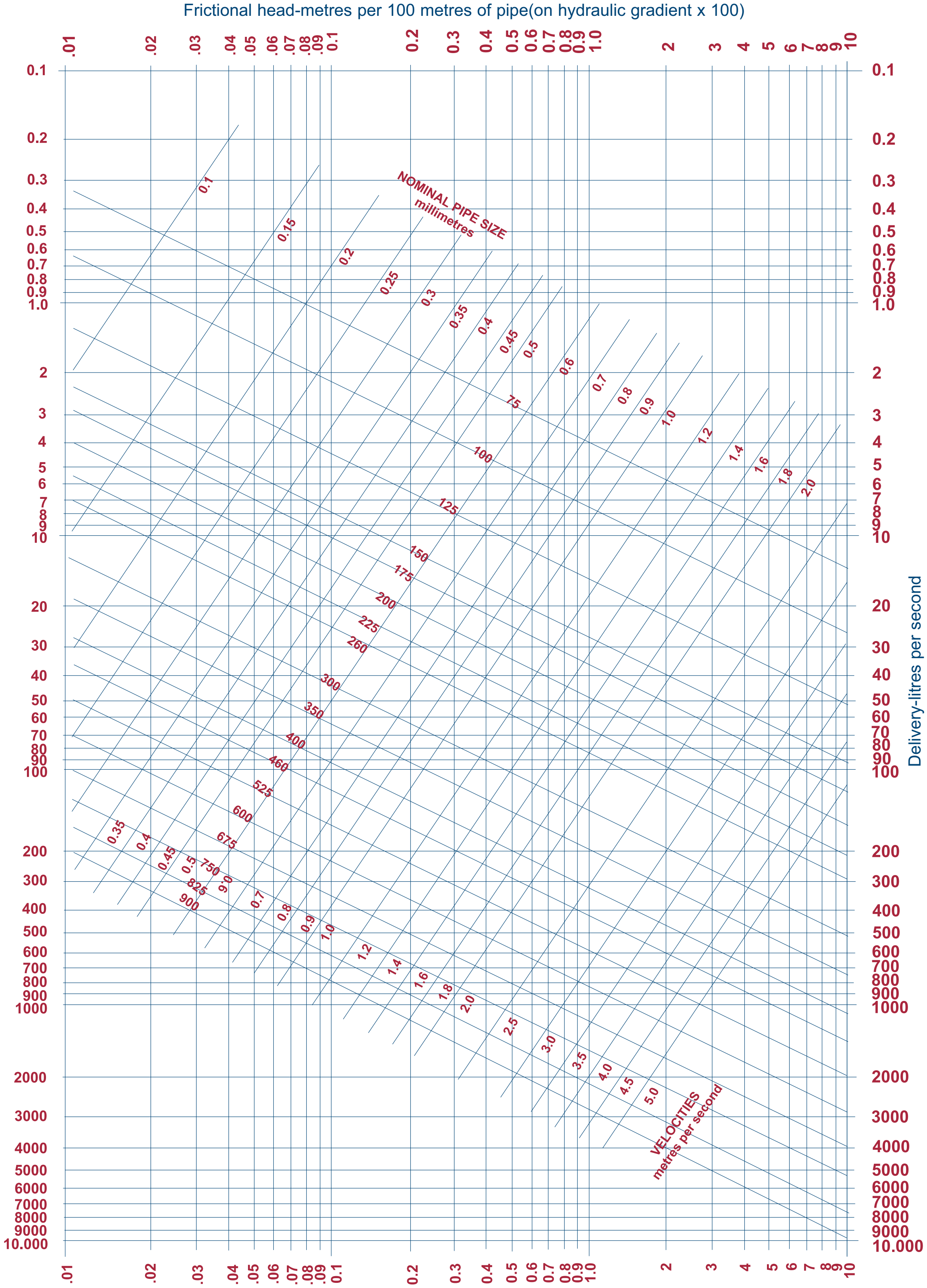
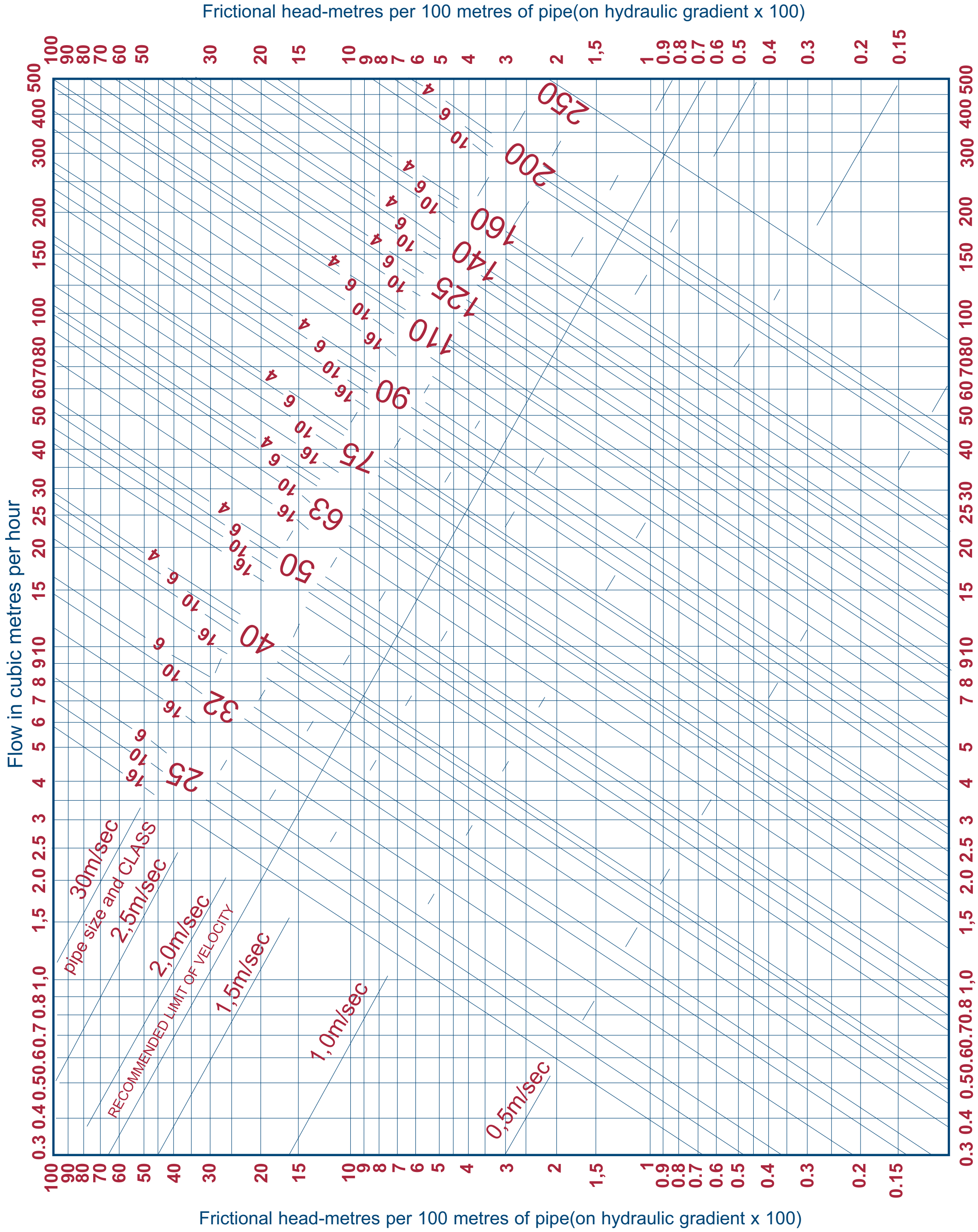


Figure 36

Friction loss chart for uPVC pipes (Source: South African Bureau of Standards, 1976)



Example 10

An existing Class 6 AC pipeline, with a diameter of 225 mm and a length of 464 m, has to be extended by 1739 m (2 193 - 454) of uPVC pipe in order to irrigate an additional area. Figure 34 shows that the minimum water level in the dam is 103.38 m. The outlet level of the existing pipeline is at 99.70 m. The ground is high at certain points along the pipeline. The highest point is at chainage 880 (1334 - 454), where the elevation is 99.72 m. The design maximum water level of the night storage reservoir is 94.0 m. A flow of 32.6 l/sec (117.4 m3/hr) has to be discharged through the pipeline. What is the best pipe to use for conveyance?

Using the friction loss charts (Figure 35 and 36) for AC and uPVC pipes, the friction headlosses per 100 m are drawn up for the different pipe sizes and presented below.

Pipe size (mm)	Friction losses (m per 100 m)
AC 225 (Class 18)	0.29
uPVC 200 (Class 4)	0.42
uPVC 250 (Class 4)	0.15

Thus, the friction losses (HL) in the existing AC pipe, inclusive of 20% extra for losses in fittings are:

HL = (0.29/100) x 464 x 1.20 = 1.62 m

The head loss line will run from 103.38 m, being the minimum water level in the dam, to 101.76 m (103.38 - 1.62) at the end of the AC pipe. The designed maximum water level of the night storage reservoir leaves 7.76 m (101.76 - 94.0) for friction losses within the remaining 1 739 m of pipeline.

In selecting the pipe sizes to be used, it is possible to use different sizes of pipe along the sections of the pipeline. The level of the pipe should be below the head loss line along its length, so that the pipe can pass the design discharge. Therefore, high points should be checked to ensure that the design discharge passes.

If a 200 mm diameter uPVC pipe is selected from that point to the night storage dam it would give a head loss of:

HL = 0.42 x 1.20 = 0.504 m per 100 m, including 20% extra. The high point at chainage 880 m should be checked. At that point the head loss line would be at elevation:

101.76 m – (0.504/100) x (1 739 – 880) = 101.76 – 4.33 = 97.43 m

This is lower than the ground level elevation of 99.72 m at the high point at chainage 880 m. Therefore, the pipe should be laid at a depth below 97.43 m at that point. Figure 34 shows that the trench is dug to elevation 97.24 m, thus the depth is adequate.

Where the night storage reservoir is located, the head loss line would be at elevation:

101.76 – (0.504/100) x 1 739 = 101.76 – 8.76 = 93.0 m.

The head loss between the minimum water level in the dam and chainage 0 is 8.76 m, which is more than the 7.76 m limit. The head loss line of 93.0 m is below the design water level of the night storage reservoir, meaning that there is insufficient head available in order to deliver the discharge required. A different combination of pipes that reduces head losses needs to be selected.

As a second option, a 225 mm diameter AC pipe (same as the existing one) is used from chainage 880 m to 1 739 m. The friction losses for this section would be:

(0.29/100) x 1.2 x (1 739 – 880) = 2.99 m

If, for the remaining 880 m, a 200 mm diameter Class 4 uPVC pipe is used, the friction loss for this section would be:

(0.42/100) x 1.2 x 880 = 4.44 m.

Therefore, the total friction loss of the 1 739 m pipe section is 2.99 + 4.44 = 7.43 m, which is less than the 7.76 m limit. The head loss line at the night storage reservoir is 94.33 m (101.76 m - 7.43 m), giving an excess head of

In order to reduce costs and ease operation the option of a 32 mm inside diameter hose will also be looked at. For this hose the head losses will be:

$$Hf_{100} = \frac{1.22 \times 10^{12} \times \left(\frac{1.6}{140}\right)^{1.852}}{32^{\,4.87}} = 14.4 \text{ m per 100 m}$$

For the 20 m hose the head losses HL will be:

HL = 14.4 x (20/100) = 2.94 m

Hence, the 32 mm inside diameter hose is adopted and the allowable pressure variation would be 20% of the head losses of this hose, which is 2.94 x 0.2 = 0.59 m. This implies that the head losses in the field line, including elevation difference along this line, should not exceed 0.58 m.

Head losses in field pipeline

There are three options in operating the system:

- The last six hydrants (gate valves)* operate at the same time
- The first six hydrants operate at the same time
- The middle six hydrants operate at the same time

The worst case scenario would be when the last six hydrants operate at the same time, hence the adopted calculations. The flow per hydrant will be 34.56/6 = 5.76 m^3/hr. Using Figure 36 the head losses are determined as follows:

Q_1 = 34.56 m^3/hr
D_1 = 160 mm PVC class 4
L_1 = 180 m
HL_1 = 0.19 x 1.8 = 0.34 m

Q_2 = 23.04 m^3/hr (= 34.56 - 5.76 - 5.76 for two hydrants)
D_2 = 110 mm PVC class 4
L_2 = 30 m
HL_2 = 0.58 x 0.3 = 0.17 m

Q_3 = 11.52 m^3/hr (= 23.04 - 5.76 - 5.76)
D_3 = 90 mm PVC class 4
L_3 = 30 m
HL_3 = 0.36 x 0.3 = 0.11 m

HL_{total} = $HL_1 + HL_2 + HL_3$
= 0.34 + 0.17 + 0.11
= 0.62 m

This is above the allowable pressure variation of 0.59 m. The difference in elevation within the hydraulic unit, from the first to the last hydrant, is 0.22 m (= 10.13 - 9.91). However, this is down slope hence the negative difference in elevation, so when added to the total head losses, they drop to 0.40 m (= 0.62 - 0.22) and are thus within the 0.58 limit. This also implies that we can reduce the diameter of part of the 180 m length pipeline from 160 mm to 140 mm and redo the calculations as follows:

Q_1 = 34.56 m^3/hr
D_1 = 160 mm PVC class 4
L_1 = 100 m
HL_1 = 0.19 x 1 = 0.19 m

Q_1 = 34.56 m^3/hr
D_2 = 140 mm PVC class 4
L_2 = 80 m
HL_2 = 0.35 x 0.8 = 0.28 m

Q_2 = 23.04 m^3/hr (= 34.56 - 11.52)
D_3 = 110 mm PVC class 4
L_3 = 30 m
HL_3 = 0.56 x 0.3 = 0.17 m

Q_3 = 11.52 m^3/hr
D_4 = 90 mm PVC class 4
L_4 = 30 m
HL_4 = 0.36 x 0.3 = 0.11 m

HL_{total} = $HL_1 + HL_2 + HL_3$
= 0.19 + 0.28 + 0.17
= 0.75 m

If we include the difference in elevation of -0.22 m the HL_{total} becomes 0.53 m (= 0.75 - 0.22), which is within the allowable pressure variation of 0.59 m.

Head losses in supply pipeline

The head losses in supply pipeline from the pumping station to the first set of hydrants are as follows:

Q_{sp} = 34.56 m^3/hr
D_{sp} = 160 mm PVC class 4
L_{sp} = 90 m
HL_{sp} = 0.19 x 0.9 = 0.17 m

* *A hydrant in this case is a gate valve, fitted on a riser, and there are two of them on each riser. Therefore, six hydrants operating at the same time implies that three hydrant risers are operating at once.*

Head losses in galvanized risers

Using Equation 21 the head losses are as follows for the 1.5 m, 75 mm inside diameter riser, using a C = 80 for old steel pipes:

$$Hf_{100} = \frac{1.22 \times 10^{12} \times \left(\frac{3.2}{80}\right)^{1.852}}{75^{4.87}} \times \frac{1.5}{100}$$

$$= 0.035 \text{ m} = 0.04 \text{ m}$$

Total head requirements

The total head requirements are composed of the suction lift (assumed to be 2 m), the head losses in the supply line, the head losses in the field line, the head losses in the hydrant riser and hose, and miscellaneous losses for fittings, plus the difference in elevation between the water level and the highest point in the field.

They are calculated as follows:

Suction lift	2.00 m	
Supply line	0.17 m	
Field line	0.62 m	
Riser	0.04 m	
Hose	2.90 m	
Miscellaneous 10%	0.57 m	
Difference in elevation	5.13 m	(= 10.13 - 5.00)
Total	11.43 m	

Power requirements

The following equation is used:

Equation 22

$$kW = \frac{Q \times H}{360 \times E_p} \times 1.2$$

Where:

kW	=	Power requirements (kW)
Q	=	Discharge (m^3/hr)
H	=	Head (m)
E_p	=	Pump efficiency (obtained from the pump performance chart)
360	=	Conversion factor for metric untis
1.2	=	20% derating (allowance for losses in transferring the power to the pump)

$$kW = \frac{34.56 \times 11.43}{360 \times 0.5} \times 1.2 = 2.63kW$$

Depending on the availability in the market place, the closest size to 2.7 kW should be selected. However no unit smaller than 2.7 kW should be purchased.

5.2.3. Advantages and disadvantages of piped systems

Following are some advantages of the use of piped systems:

- The cost of medium and small diameter PVC pipes compares very favourably with the cost of constructing smaller canals
- Seepage and evaporation losses are eliminated
- There are no stilling boxes required or other places where stagnant water can collect and become a breeding ground for mosquitoes and snails. Furthermore, there is no weed growth in pipelines
- Pipelines are normally safer than open channels since humans and equipment cannot fall into the water stream
- With only hydrants protruding above ground, it is possible to undertake land levelling and other mechanical cultivation after the scheme has been installed
- The system can be installed faster than canal systems
- Pipelines permit the conveyance of water uphill against the normal slope of the land over certain distances to overcome obstacles
- Very little land is lost at the headlands of each plot as the crops can be planted right up to or even over the pipeline. Also, the use of buried pipes allows the use of most direct routes from the water source to the field
- The farmer has control over the water supply to the plot, and since water can be available "on demand" in case no pumping is required, there is some flexibility in when to irrigate and it is less important to adhere to strict rotation
- The underground pipes form a closed system and as a result the conveyance losses are negligible. There are also no incidences of water poaching as could occur with canal conveyance systems

Amongst the disadvantages, the following can be mentioned:

- The system can be expensive to install, especially when large diameter pipes are to be used and when the trenching requires blasting in some areas
- Some skill is required to fix a hydrant when it gets broken at the bottom. However, these incidences are rare when the hydrants are properly protected