Chapter 7
Land levelling

Proper land levelling is important for efficient surface irrigation. It involves moving soil in order to have level fields for basin irrigation or uniform sloping fields for furrow or borderstrip irrigation.

When levelling or grading land, one should avoid large volumes of cut and fill. Besides being expensive, too much soil movement tends to leave shallow topsoil in areas of cut, which is not ideal for crop production.

A detailed topographic survey, preferably grid, is needed to calculate the most economic land levelling requirements. Based on the spot heights of the grid points and the required gradient of the land, the cut and fill can be calculated. The total volume of cut should preferably exceed the total volume of fill by 10-50% depending on the total volume to be moved and the compressibility of the soil.

The three most widely used methods for calculating the amounts of soil cuts and fills are:

- Profile method
- Contour method
- Plane or centroid method

The plane method is the most popular of the three and will be described more in detail in Section 7.3.

7.1. Profile method

The grid points following the proposed direction of slope are used to represent a strip of land. The ground level elevation points are plotted to show the existing profile. The required gradient is superimposed and the gradient line moved through trial and error until the volume of cut equals the volume of fill. In general, the greater the amount of fill required the greater should be the over-cut in earthwork balances. For the purpose of over-cut the line of equal cut and fill is lowered. After levelling, the work can be checked using a level instrument or profile boards as shown in Figure 88.

7.2. Contour method

The contour method requires an accurate contour map. A new set of contour lines is chosen by visually balancing the areas indicating cut and those indicating fill. Figure 89 shows a layout for the contour method.

The cut and fill areas are measured using a planimeter. Approximate volumes of cut and fill between successive contours are found by multiplying the average of the top and bottom areas by the contour interval. As an example, if the area of cut in zone 1 is 3.75 m^2 and that of cut in zone 2 is 2.25 m^2, the average cut area between contours 98 and 97 m is $(3.75 + 2.25)/2 = 3.00$ m^2. If the distance between the contour lines is 125 m, the volume of cut between these lines is 3.00 m^2 x 125 m = 375 m^3.

All volumes of cut and fill are summed up and checked to ascertain that they balance according to the cut to fill ratio. If this is not correct, the new contours have to be adjusted and the procedure repeated.

Figure 88
The profile method of land levelling: cut and fill and checking gradient levels with profile boards

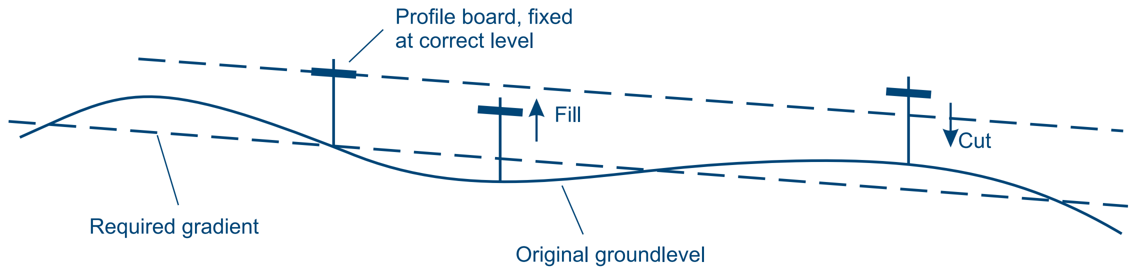
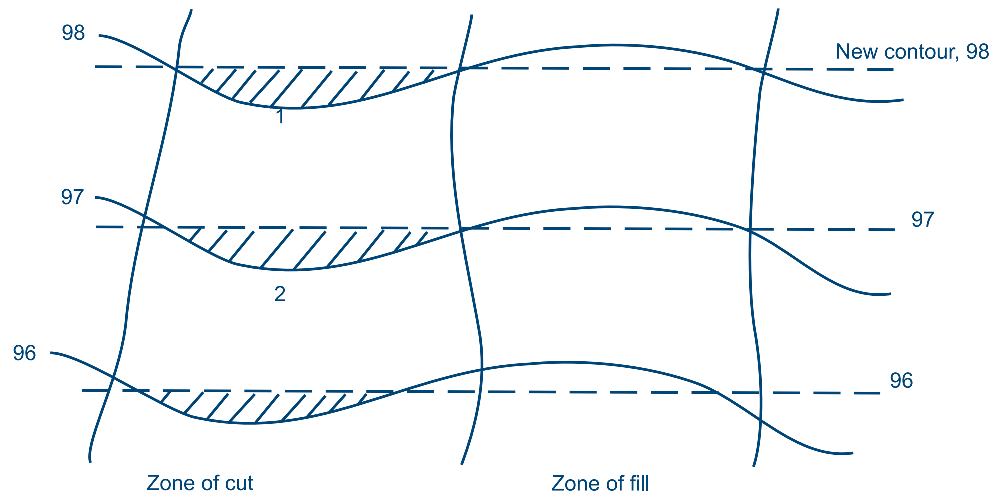


Figure 89
The contour method of land levelling



7.3. Plane method

The plane method is a least-squares fitting of field elevations to a two-dimensional plane with subsequent adjustments for variable cut-fill ratios. The aim is to grade the surface of a field to a uniformly inclined plane. Grid point elevations are used for the calculation. Each grid point is taken to be representative of the square of a grid size of which it is the centre. It is possible to calculate the inclination and direction of the slope for minimum cut and fills, although often a slope suited to the designed irrigation system is selected.

Giving the field a basic X-Y orientation, the plane equation is written as follows:

Equation 56

$$EL(X,Y) = (G_X \times X) + (G_Y \times Y) + C$$

Where:

EL (X,Y)	=	Elevation of the (X,Y) coordinate (m)
G_X and G_Y	=	Regression coefficients
X and Y	=	Distance from origin to grid point (m)
C	=	Elevation of the origin (m)

The calculation of the regression coefficients G_X and G_Y and the elevation of the centroid can be accomplished using a four-step procedure.

Step 1

The initial step is to determine the weighted average elevations of each grid point in the field. The purpose of the weighting is to adjust for any boundary stakes that represent larger or smaller areas than given by the standard grid dimension. The weighting factor is defined as the ratio of actual area represented by a grid point to the standard area. The grid point area is assumed to be the proportional area surrounding the stake or other identification of the grid point elevation.

Figure 90 shows a portion of an irrigation layout with a field irrigation canal planned on the grid line i = 5 and the drainage channel on grid line i = 1. The grid points on the canal and drain alignment and the plot boundaries have to be adjusted as they represent a smaller area than the standard grid dimension of 25 m x 25 m. In the example of Figure 90 the edge points only count for either 25% or 50%, thus the weighting factors are respectively 0.25 and 0.50. The weighting factors, other than those that are 1.00, have been indicated between brackets in Figure 90.

The figures between brackets on the X-axis and the Y-axis represent the distance.

The weighted average elevation has to be determined in both field directions. Using the grid map, the elevations are added by horizontal rows and by vertical columns, taking the weighting factors into account, after which the average of each row and column is calculated.

Figure 90
Grid map showing land elevation and average profile figures

Origin	X	(25m)	(50m)	(75m)	(100m)	(125m)	ELj
	Y	i=1	2	3	4	5	
	j=1 (25m)	88.837 (0.25)	89.159 (0.50)	89.057 (0.50)	89.098 (0.50)	89.478 (0.25)	89.118 (2)
	2 (50m)	88.520 (0.50)	89.017	89.108	88.976	89.181 (0.50)	88.988 (4)
	3 (75m)	88.731 (0.50)	88.814	89.043	89.027	89.264 (0.50)	88.970 (4)
	4 (100m)	88.983 (0.50)	88.908	88.775	88.722	89.066 (0.50)	88.857 (4)
	5 (125m)	88.654 (0.50)	88.802	88.905	88.846	89.026 (0.50)	88.848 (4)
	6 (150m)	88.623 (0.50)	88.768	88.957	88.864	89.039 (0.50)	88.855 (4)
	7 (175m)	88.591 (0.50)	88.697	88.918	88.981	89.024 (0.50)	88.851 (4)
	8 (200m)	88.450 (0.25)	88.668 (0.50)	88.900 (0.50)	88.940 (0.50)	89.081 (0.25)	88.818 (2)
	ELi	88.678 (3.5)	88.846 (7)	88.955 (7)	88.919 (7)	89.126 (3.5)	

Weighting factor (between brackets)

Drain

Irrigation canal

Plot boundary

The average elevation of column i (ELi in Figure 90) is calculated by:

Equation 57

$$EL_i = \frac{\sum_{j=1}^{N} \Theta_{ij} \times EL_{ij}}{\sum_{j=1}^{N} \Theta_{ij}}$$

Where:

EL_{ij} = Elevation of the (i,j) coordinate, found from field measurements (m)

Θ_{ij} = Weighing factor of the (i,j) coordinate, which is the ratio of actual area represented by grid point (i,j) to the standard grid area

Similarly, the average elevation of row j (El_j) is expressed by:

Equation 58

$$EL_j = \frac{\sum_{i=1}^{M} \Theta_{ij} \times EL_{ij}}{\sum_{i=1}^{M} \Theta_{ij}}$$

For example, the average elevation of row EL_1 (j=1) is:

$$\frac{(0.25 \times 88.837) + (0.5 \times 89.159) + (0.5 \times 89.057) + (0.5 \times 89.098) + (0.25 \times 89.478)}{0.25 + 0.50 + 0.50 + 0.50 + 0.25} = 89.118 \text{ m}$$

Step 2

Locate and calculate the elevation of the centroid of the field with respect to the grid system. Usually, an origin is located one grid spacing in each direction away from the first grid position. The origin could, however, be related to any corner of the field. The final results will be the same, irrespective of the origin location. The distance from the origin to the centroid in the i direction is found by:

Equation 59

$$X_{cen} = \frac{\sum_{i=1}^{M} \Theta_i \times X_i}{\sum_{i=1}^{M} \Theta_i}$$

Where:

X_{cen} = Distance from origin to centroid (m)

X_i = Distance in x direction from origin to i-th grid position (m)

Θ_i = $\sum_{j=1}^{N} \Theta_{ij}$

Similarly, the distance from the origin to the centroid in the j direction is:

Equation 60

$$Y_{cen} = \frac{\sum_{j=1}^{N} \Theta_j \times Y_j}{\sum_{j=1}^{N} \Theta_j}$$

The elevation of the centroid is the average of the average row or the average column elevations and is calculated as follows:

Equation 61

$$EL_{cen} = \frac{\sum_{i=1}^{M} \Theta_i \times EL_{average,\, i}}{\sum_{i=1}^{M} \Theta_i}$$

Where:

EL_{cen} = Elevation of the centroid (m)

$EL_{average,\, i}$ = Average elevation of column i (m)

In Figure 90, EL_{cen} is:

$$\frac{(3.5 \times 88.678) + (7 \times 88.846) + (7 \times 88.919) + (3.5 \times 89.126)}{3.5 + 7 + 7 + 7 + 3.5}$$

$= 88.905$ m

Step 3

Calculate the best fitting straight line through the average row and column elevations using the least squares method. This is called linear regression, which is a statistical method to calculate a straight line that best fits a set of two or more data pairs. Thus, using this method the calculated slope line fits the average profile best. These slopes, G_X and G_Y, can be calculated with the following formulae:

Equation 62

$$G_x = \frac{\sum_{i=1}^{M} X_i \times EL_{average,i} - \frac{\left\langle \left[\sum_{i=1}^{M} X_i\right] \times \left[\sum_{i=1}^{M} EL_{average,i}\right] \right\rangle}{M}}{\sum_{i=1}^{M} X_i^2 - \frac{\left[\sum_{i=1}^{M} X_i\right]^2}{M}}$$

Where:

G_X = Slope in the x direction

X_i = Distance of average grid point elevation $EL_{average}$ from the origin (m)

$EL_{average,i}$ = Average elevation of column i (m)

M = Number of grid points in the X-direction

The formula for the calculation of G_Y is:

Equation 63

$$G_Y = \frac{\sum_{j=1}^{N} Y_j \times EL_{average,j} - \frac{\left\langle \left[\sum_{j=1}^{N} Y_j\right] \times \left[\sum_{j=1}^{N} EL_{average,j}\right] \right\rangle}{N}}{\sum_{j=1}^{N} Y_j^2 - \frac{\left[\sum_{j=1}^{N} Y_j\right]^2}{N}}$$

G_X and G_Y can be calculated with a normal standard calculator, although this is a very laborious method. A programmable calculator, or one with linear regression functions, could be used. Also, a number of land levelling programmes have been written for use by computer. Examples are given in Section 7.5.

Figure 91 gives a graphical impression of the lines of best fit.

Step 4

The final step involves defining the best-fit plane (Equation 56) and requires the determination of C, which is the elevation of the origin. As the lines of best fit go through the centroid, the elevation of that point can be used to calculate C as follows:

$$C = EL_{centroid} - (G_X \times X_{cen}) - (G_Y \times Y_{cen})$$

In the above example:

$$C = 88.905 - (0.0039 \times 75) - (-0.0015 \times 112.50)$$
$$= 88.781 \text{ m}$$

Example 37

For the example of figure 90 the value for G_X can be calculated as follows:

We substitute M = 5 in the following equations:

$$\sum_{i=1}^{5} X_i \times EL_{average,i} = (25 \times 88.678) + (50 \times 88.846) + (75 \times 88.955) + (100 \times 88.919) + (125 \times 89.126) = 33\,363.525 \text{ m}^2$$

$$\sum_{i=1}^{5} X_i = 25 + 50 + 75 + 100 + 125 = 375 \text{ m}$$

$$\sum_{i=1}^{5} EL_{average,i} = 88.678 + 88.846 + 88.919 + 89.126 = 444.524 \text{ m}$$

$$\sum_{i=1}^{5} (X_i)^2 = (25^2 + 50^2 + 75^2 + 100^2 + 125^2 = 34\,375 \text{ m}^2$$

$$\left[\sum_{i=1}^{5} X_i\right]^2 = (25 + 50 + 75 + 100 + 125)^2 = 140\,625 \text{ m}^2$$

Substitution of the above data in the Equation 62 gives:

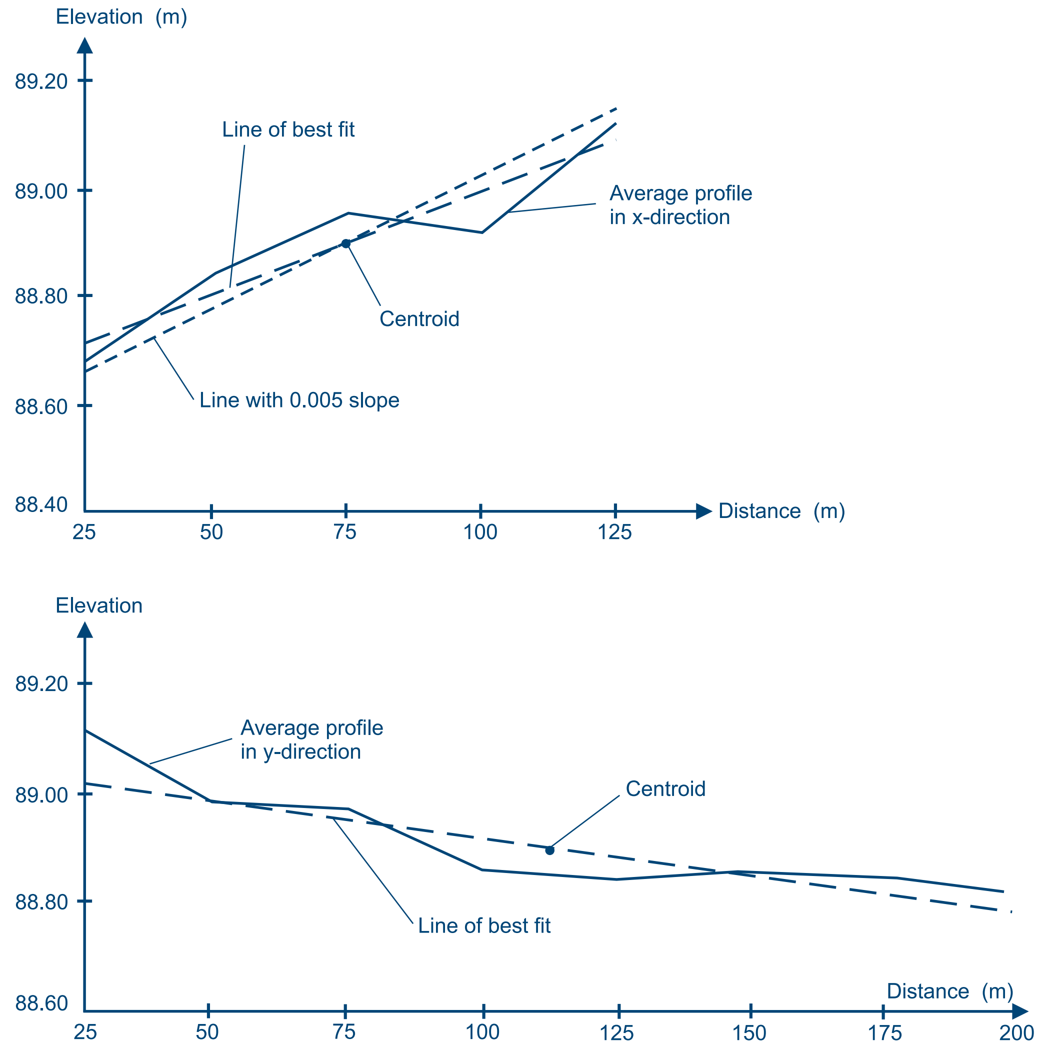
$$G_X = \frac{33\,363.525 - \frac{(375 \times 444.524)}{5}}{34\,375 - \frac{140\,625}{5}} = \frac{24.225}{6\,250} = 0.0039$$

This means that the line of best fit will rise from the origin at 0.39 cm per metre distance (0.39 m/100 m).

A similar calculation for G_Y would give a value -0.0015. This means that the line of best fit would drop from the origin (because of the minus sign) at 0.15 cm per metre distance.

It should be noted that if the origin had been selected at the bottom right side of the field, the G_X would have a negative sign and the G_Y a positive one. The values would, however, remain the same.

Figure 91
Average profile and lines of best fit



Thus the equation for computing the elevation at any grid point will be (Equation 56):

EL(X,Y) = (0.0039 x X) - (0.0015 x Y) + 88.781

The value of each grid point elevation can now be calculated by substituting the distances of each point from the origin. As an example, the elevation at the point with (X,Y) = (25,25) coordinate is:

EL(25,25) = (0.0039 x 25) - (0.0015 x 25) + 88.781
= 88.841 m

Table 33 gives the results of all calculations. The differences in elevation (3rd row in Table 33) are the necessary cuts, where the calculated EL is lower than surveyed grid point elevation, or fills, where the calculated EL is higher than surveyed grid point elevation.

The volumes of cut and fill can be calculated by multiplying the depth of cut or fill at each grid point with the grid area, in this case an area of 625 m^2 (= 25 m x 25 m) per grid point, except for points with a weighing factor smaller than 1. The cut and fill volumes of our example of Table 33 are 764 m^3 and 757 m^3 respectively. The fourth row (adjusted cut or fill) will be discussed later.

If the slopes G_X and/or G_Y of the lines of best fit are too steep or too flat to suit the irrigation method, they can be changed. The slopes should still pass through the centroid, which means that the volume of earth to be moved will normally increase. The adjusted slopes are entered in the equation to calculate C. If, for example, the slope in the X-direction is changed to 0.005, the C-value becomes 88.698 m. Thus the equation for computing the elevation at any grid point becomes:

EL(X,Y) = (0.005 x X) - (0.0015 x Y) + 88.698

Table 33
Land levelling results

Surveyed ground level	88.837	89.159	89.057	89.098	89.478
Elevation after levelling	88.841	88.939	89.036	89.134	89.231
Cut or Fill	+0.004	-0.220	-0.021	+0.036	-0.470
Adjusted cut or Fill	-0.004	-0.228	-0.029	+0.028	-0.255
X : Y	25 : 25	50 : 25	75 : 25	100 : 25	125 : 25
Surveyed ground level	88.520	89.017	89.108	88.976	89.181
Elevation after levelling	88.804	88.901	88.999	89.096	89.194
Cut or Fill	+0.284	-0.116	-0.109	+0.120	+0.013
Adjusted cut or Fill	+0.276	-0.124	-0.117	+0.112	+0.005
X : Y	25 : 50	50 : 50	75 : 50	100 : 50	125 : 50
Surveyed ground level	88.731	88.814	89.043	89.027	89.264
Elevation after levelling	88.766	88.864	88.961	89.059	89.156
Cut or Fill	+0.035	+0.050	-0.082	+0.032	-0.108
Adjusted cut or Fill	+0.027	+0.042	-0.090	+0.024	-0.116
X : Y	25 : 75	50 : 75	75 : 75	100 : 75	125 : 75
Surveyed ground level	88.983	88.908	88.775	88.722	89.066
Elevation after levelling	88.729	88.826	88.924	89.021	89.119
Cut or Fill	-0.254	-0.082	+0.149	+0.299	+0.053
Adjusted cut or Fill	-0.262	-0.090	+0.141	+0.291	+0.045
X : Y	25 : 100	50 : 100	75 : 100	100 : 100	125 : 100
Surveyed ground level	88.654	88.802	88.905	88.846	89.026
Elevation after levelling	88.691	88.789	88.886	88.984	89.081
Cut or Fill	+0.037	-0.013	-0.019	+0.138	+0.055
Adjusted cut or Fill	+0.029	-0.021	-0.027	+0.130	+0.047
X : Y	25 : 125	50 : 125	75 : 125	100 : 125	125 : 125
Surveyed ground level	88.623	88.768	88.957	88.864	89.039
Elevation after levelling	88.654	88.751	88.849	88.946	89.044
Cut or Fill	+0.031	-0.017	-0.108	+0.082	+0.005
Adjusted cut or Fill	+0.023	-0.025	-0.116	+0.074	-0.003
X : Y	25 : 150	50 : 150	75 : 150	100 : 150	125 : 150
Surveyed ground level	88.591	88.697	88.918	88.981	89.024
Elevation after levelling	88.616	88.714	88.811	88.909	89.006
Cut or Fill	+0.025	+0.017	-0.107	-0.072	-0.018
Adjusted cut or Fill	+0.017	+0.009	-0.115	-0.080	-0.026
X : Y	25 : 175	50 : 175	75 : 175	100 : 175	125 : 175
Surveyed ground level	88.450	88.668	88.900	88.940	89.081
Elevation after levelling	88.579	88.676	88.774	88.871	88.969
Cut or Fill	+0.129	+0.008	-0.126	-0.069	-0.112
Adjusted cut or Fill	+0.121	+0.000	-0.134	-0.077	-0.120
X : Y	25 : 200	50 : 200	75 : 200	100 : 200	125 : 200

If the same calculations on volumes of cut and fill are done again using the above equation, they result in a total volume of cut of 822 m^3 and a total volume of fill of 829 m^3.

If the change in slope would give unsatisfactory results, such as an excessive cut, it could be more beneficial to irrigate at an angle to the canal.

This method of calculating the cut and fill volumes assumes that the elevation of a grid point is representative for a full grid area. This assumption is, of course, not always true. A more accurate, but also more laborious, method to calculate the cut and fill volumes is the Four-Corners method. This method takes the depth of cut or fill at each corner of a square into account. For boundaries, where complete grid spacings are not present, the procedure is to assume that the elevations of the field boundaries are the same as those of the nearest grid point, while the actual edge area is taken into account.

Equation 64

$$V_c = \frac{L^2 \times C^2}{4 \times (C + F)}$$

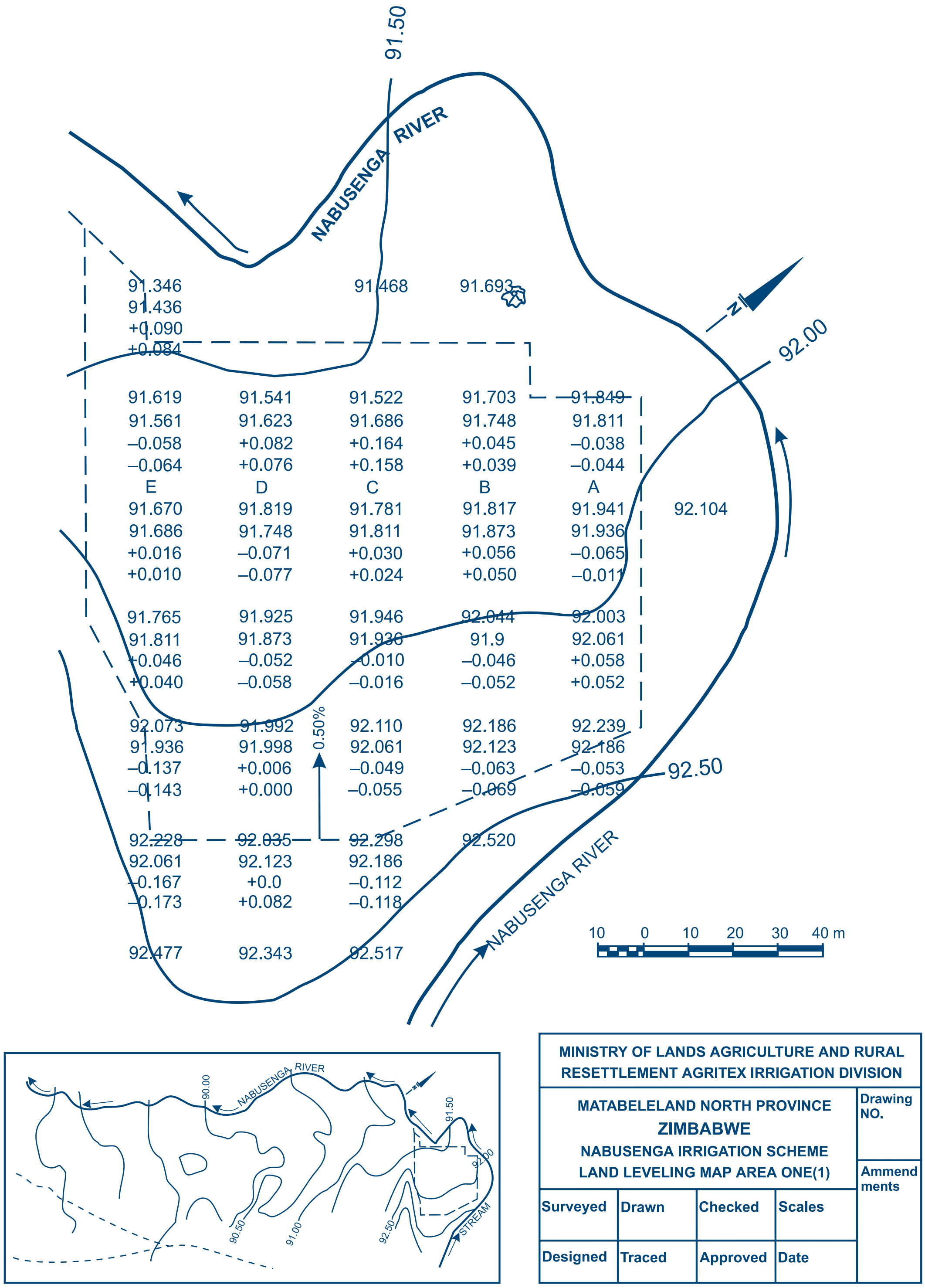
Equation 65

$$V_f = \frac{L^2 \times F^2}{4 \times (C + F)}$$

Where:

V_c = Volume of cut (m^3)
V_f = Volume of fill (m^3)
L = Grid spacing (m)
C = Sum of cut depth at grid points (m)
F = Sum of fill depth at grid points (m)

Figure 92
Part of the completed land levelling map for Nabusenga, assuming $G_X = 0.005$



As the calculations are very elaborate, they should preferably be carried out with a programmable calculator or a computer.

Figure 92 shows part of the completed land levelling map for Nabusenga surface irrigation scheme, assuming G_X = 0.005.

More often than not, one tends to get a variety of slopes within a scheme or a block of fields or even a field. To level it as an entity will result in a lot of compromises as far as the depths of cuts and fills are concerned. To avoid this, the scheme or block of fields or fields can be divided into sections. A section could be taken as a piece of land with a uniform slope and can be treated as an area commanded by a field canal or pipeline. The sections are levelled separately with different parameters being used.

7.4. The cut : fill ratio

As explained above, the volume of cut (V_c) should exceed the volume of fill (V_f) since the disturbance of the soil reduces its density. The ratio is called the cut : fill ratio (R) and should be in the range of 1.1 to 1.5, depending on soil type and its condition. Selecting a cut : fill ratio remains a matter of judgement and is therefore subjective.

As an example, if the volume of cut should exceed the volume of fill by 20%, the cut : fill ratio is 1.20. The depth required in order to lower the surface plane to achieve a cut : fill ratio of 1.20 can be estimated with the following formula:

Equation 66

$$d = \frac{(R \times V_f) - V_c}{\sum_{i=1}^{I}(A_i \times (1 + R))}$$

Where:

- d = Depth by which the surface plane has to be lowered (m)
- R = Cut : fill ratio
- V_f = Volume of fill (m^3)
- V_c = Volume of cut (m^3)
- A_i = Total grid area which requires cut (m^2)

Following the example of Table 33, where 10 full grid areas, 7 half grid areas and 2 quarter grid areas have cuts (negative values in 3rd row):

$$d = \frac{(120 \times 757) - 764}{\big((10 + (0.5 \times 7) + (0.25 \times 2)) \times 25 \times 25\big) \times 2.20}$$

$$= 0.0075 \text{ m}$$

Thus, in order to achieve a cut : fill ratio of approximately 1.20, the plane has to be lowered by 7.5 mm (the 4th row in Table 33). This results in a final cut volume of 836 m^3 and a final fill volume of 689 m^3.

7.5. Use of computers

As already indicated previously, a number of programmes have been written to calculate the land levelling requirements by computer. One such programme, written by E.C. Olsen of Utah State University, is called LEVEL 4EM.EXE. It calculates land-grading requirements based on the least squares analysis for both rectangular and irregularly shaped fields. The inputs required are given in Table 34 below.

Some results of the use of computer for land levelling calculations for different cut : fill ratios are given in Tables 35, 36 and 37.

Table 34
Input and output data types for computer land levelling programme LEVEL 4EM.EXE

INPUTS	OUTPUTS
The minimum and maximum acceptable cut : fill ratios	Elevations after grading
The units, either metric or imperial	Grade in horizontal direction
Number of grid points in horizontal direction	Grade in vertical direction
Grid distance in horizontal direction	Cut or fill required
Grid distance in vertical direction	Centroid elevation
Weighing factors other than 1	Cut : fill ratio
Number of grid points in vertical direction	Area levelled
Elevations of all grid points	Volume of excavation

Table 35

Land levelling calculations with line of best fit and cut:fill ratio of 1.01

Location N	M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	88.84	88.84	C 0.00
1	2	89.16	88.94	C 0.22
1	3	89.06	89.04	C 0.02
1	4	89.10	89.14	F 0.04
1	5	89.48	89.24	C 0.23
2	1	88.52	88.80	F 0.28
2	2	89.02	88.90	C 0.12
2	3	89.11	89.00	C 0.11
2	4	88.98	89.10	F 0.13
2	5	89.18	89.21	F 0.02
3	1	88.73	88.76	F 0.03
3	2	88.81	88.86	F 0.05
3	3	89.04	88.96	C 0.08
3	4	89.03	89.06	F 0.04
3	5	89.26	89.17	C 0.10
4	1	88.98	88.72	C 0.26
4	2	88.91	88.82	C 0.09
4	3	88.78	88.92	F 0.15
4	4	88.72	89.03	F 0.30
4	5	89.07	89.13	F 0.06
5	1	88.65	88.68	F 0.03
5	2	88.80	88.78	C 0.02
5	3	88.90	88.89	C 0.02
5	4	88.85	88.99	F 0.14
5	5	89.03	89.09	F 0.06
6	1	88.62	88.64	F 0.02
6	2	88.77	88.75	C 0.02
6	3	88.96	88.85	C 0.11
6	4	88.86	88.95	F 0.09
6	5	89.04	89.05	F 0.01
7	1	88.59	88.60	F 0.01
7	2	88.70	88.71	F 0.01
7	3	88.92	88.81	C 0.11
7	4	88.98	88.91	C 0.07
7	5	89.02	89.01	C 0.01
8	1	88.45	88.57	F 0.12
8	2	88.67	88.67	F 0.00
8	3	88.90	88.77	C 0.13
8	4	88.94	88.87	C 0.07
8	5	89.08	88.97	C 0.11

Final grade in M direction : 0.41 m/100 m
Final grade in N direction : -0.15 m/100 m
Final centroid elevation : 88.905 m
Final ratio of cuts/fill : 1.01
Area levelled : 1.750 ha
Final volume of excavation : 768.301 m^3

Table 36

Land levelling calculations with 0.5% gradient in the X direction and cut:fill ratio of 1.01

Location N	M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	88.84	88.79	C 0.05
1	2	89.16	88.91	C 0.25
1	3	89.06	89.04	C 0.02
1	4	89.10	89.16	F 0.06
1	5	89.48	89.29	C 0.19
2	1	88.52	88.75	F 0.23
2	2	89.02	88.87	C 0.14
2	3	89.11	89.00	C 0.11
2	4	88.98	89.12	F 0.15
2	5	89.18	89.25	F 0.07
3	1	88.73	88.71	F 0.02
3	2	88.81	88.84	F 0.02
3	3	89.04	88.96	C 0.08
3	4	89.03	89.09	F 0.06
3	5	89.26	89.21	C 0.05
4	1	88.98	88.67	C 0.31
4	2	88.91	88.80	C 0.11
4	3	88.78	88.92	F 0.15
4	4	88.72	89.05	F 0.33
4	5	89.07	89.17	F 0.11
5	1	88.65	88.64	F 0.02
5	2	88.80	88.76	C 0.04
5	3	88.90	88.89	C 0.02
5	4	88.85	89.01	F 0.17
5	5	89.03	89.14	F 0.11
6	1	88.62	88.60	F 0.02
6	2	88.77	88.72	C 0.04
6	3	88.96	88.85	C 0.11
6	4	88.86	88.97	F 0.11
6	5	89.04	89.10	F 0.06
7	1	88.59	88.56	F 0.03
7	2	88.70	88.69	F 0.01
7	3	88.92	88.81	C 0.11
7	4	88.98	88.94	C 0.04
7	5	89.02	89.06	C 0.04
8	1	88.45	88.52	F 0.07
8	2	88.67	88.65	F 0.02
8	3	88.90	88.77	C 0.13
8	4	88.94	88.90	C 0.04
8	5	89.08	89.02	C 0.06

Final grade in M direction : 0.50 m/100 m
Final grade in N direction : -0.15 m/100 m
Final centroid elevation : 88.905 m
Final ratio of cuts/fill : 1.01
Area levelled : 1.750 ha
Final volume of excavation : 841.959 m^3

Table 37

Land levelling calculations with line of best fit and cut:fill ratio of 1.21

Location N	M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	88.84	88.83	C 0.01
1	2	89.16	88.93	C 0.23
1	3	89.06	89.03	C 0.03
1	4	89.10	89.13	F 0.04
1	5	89.48	89.24	C 0.24
2	1	88.52	88.79	F 0.27
2	2	89.02	88.89	C 0.13
2	3	89.11	88.99	C 0.11
2	4	88.98	89.10	F 0.12
2	5	89.18	89.20	F 0.02
3	1	88.73	88.75	F 0.02
3	2	88.81	88.85	F 0.04
3	3	89.04	88.95	C 0.09
3	4	89.03	89.06	F 0.03
3	5	89.26	89.16	C 0.11
4	1	88.98	88.71	C 0.27
4	2	88.91	88.81	C 0.09
4	3	88.78	88.92	F 0.14
4	4	88.72	89.02	F 0.30
4	5	89.07	89.12	F 0.05
5	1	88.65	88.67	F 0.02
5	2	88.80	88.78	C 0.03
5	3	88.90	88.88	C 0.03
5	4	88.85	88.98	F 0.13
5	5	89.03	89.08	F 0.06
6	1	88.62	88.64	F 0.01
6	2	88.77	88.74	C 0.03
6	3	88.96	88.84	C 0.12
6	4	88.86	88.94	F 0.08
6	5	89.04	89.04	F 0.00
7	1	88.59	88.60	F 0.01
7	2	88.70	88.70	F 0.00
7	3	88.92	88.80	C 0.12
7	4	88.98	88.90	C 0.08
7	5	89.02	89.00	C 0.02
8	1	88.45	88.56	F 0.11
8	2	88.67	88.66	F 0.01
8	3	88.90	88.76	C 0.14
8	4	88.94	88.86	C 0.08
8	5	89.08	89.97	C 0.11

Final grade in M direction : 0.41 m/100 m
Final grade in N direction : -0.15 m/100 m
Final centroid elevation : 88.897 m
Final ratio of cuts/fill : 1.21
Area levelled : 1.750 ha
Final volume of excavation : 841.988 m³

The computer programme has also been used to calculate the land levelling requirements for the gross area of Mangui piped surface irrigation scheme and the results are shown in Table 38 and in Figure 20. The slope along the pipeline has been maintained as fairly level, while the slope perpendicular to the pipeline, which is the furrow slope, has been fixed at 0.4%.

Table 38a

Computer printout of land levelling data for the area south of the main pipeline in Mangui piped surface irrigation scheme

Location N	M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	9.70	10.15	F 0.45
1	2	10.02	10.13	F 0.11
1	3	10.03	10.11	F 0.08
1	4	10.06	10.09	F 0.03
1	5	9.96	10.07	F 0.11
1	6	10.08	10.06	C 0.02
1	7	9.94	10.04	F 0.10
1	8	10.03	10.02	C 0.01
1	9	9.97	10.00	F 0.03
1	10	10.04	9.98	C 0.06
1	11	9.81	9.97	F 0.16
1	12	9.87	9.95	F 0.08
1	13	9.83	9.93	F 0.10
1	14	9.75	9.91	F 0.16
2	1	9.99	10.07	F 0.08
2	2	10.04	10.05	F 0.01
2	3	10.03	10.03	F 0.00
2	4	10.00	10.01	F 0.01
2	5	10.09	9.99	C 0.10
2	6	10.00	9.98	C 0.02
2	7	10.05	9.96	C 0.09
2	8	9.87	9.94	F 0.07
2	9	9.96	9.92	C 0.04
2	10	9.78	9.90	F 0.13
2	11	9.48	9.89	F 0.41
2	12	9.97	9.87	C 0.10
2	13	9.61	9.85	F 0.24
2	14	9.98	9.83	C 0.15
3	1	10.06	9.99	C 0.07
3	2	10.03	9.97	C 0.06
3	3	10.09	9.95	C 0.14
3	4	10.30	9.93	C 0.37
3	5	10.14	9.91	C 0.23
3	6	10.35	9.90	C 0.45
3	7	10.01	9.88	C 0.13
3	8	10.01	9.86	C 0.15
3	9	10.17	9.84	C 0.33
3	10	10.24	9.82	C 0.41
3	11	10.23	9.81	C 0.42
3	12	9.83	9.79	C 0.04
3	13	9.83	9.77	C 0.06
3	14	9.80	9.75	C 0.05

Final grade in M direction = -0.09 m/100 m
Final grade in N direction = -0.40 m/100 m
Final centroid elevation = 9.950m
Final ration of cut/fills = 1.48
Area levelled = 1.680 ha
Final volume of excavation = 1 396.401 m³

Table 38b

Computer printout of land levelling data for the area north of the main pipeline in Mangui piped surface irrigation scheme

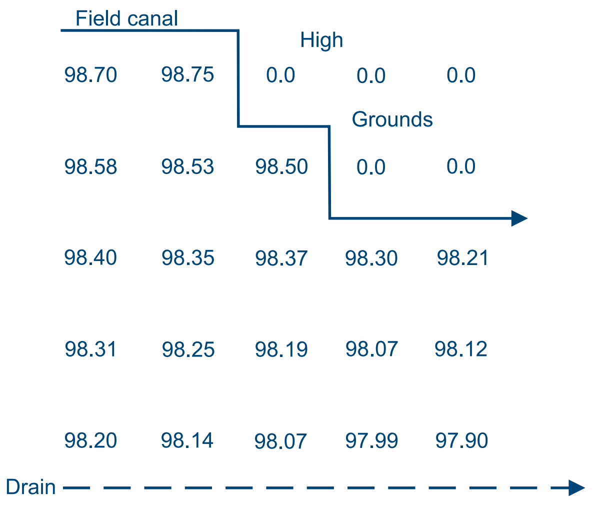
Location N	M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	9.70	10.01	F 0.31
1	2	10.02	9.99	C 0.03
1	3	10.03	9.97	C 0.06
1	4	10.06	9.95	C 0.11
1	5	9.96	9.94	C 0.02
1	6	10.08	9.92	C 0.16
1	7	9.94	9.90	C 0.04
1	8	10.03	9.88	C 0.15
1	9	9.97	9.86	C 0.11
1	10	10.04	9.85	C 0.19
1	11	9.81	9.83	F 0.02
1	12	9.87	9.81	C 0.06
1	13	9.83	9.79	C 0.04
1	14	9.75	9.77	F 0.02
2	1	9.75	9.93	F 0.18
2	2	10.07	9.91	C 0.16
2	3	9.94	9.89	C 0.05
2	4	10.00	9.87	C 0.13
2	5	9.97	9.86	C 0.11
2	6	10.23	9.84	C 0.39
2	7	9.94	9.82	C 0.12
2	8	9.27	9.80	F 0.53
2	9	9.83	9.78	C 0.05
2	10	9.48	9.77	F 0.29
2	11	9.85	9.75	C 0.10
2	12	9.84	9.73	C 0.11
2	13	10.01	9.71	C 0.30
2	14	9.83	9.69	C 0.14
3	1	9.92	9.85	C 0.07
3	2	9.77	9.83	F 0.06
3	3	9.88	9.81	C 0.07
3	4	9.75	9.79	F 0.04
3	5	9.82	9.78	C 0.04
3	6	9.74	9.76	F 0.02
3	7	9.82	9.74	C 0.08
3	8	9.68	9.72	F 0.04
3	9	9.68	9.70	F 0.02
3	10	9.58	9.69	F 0.11
3	11	9.56	9.67	F 0.11
3	12	9.51	9.65	F 0.14
3	13	9.39	9.63	F 0.24
3	14	9.49	9.61	F 0.12

Final grade in M direction = -0.09 m/100 m
Final grade in N direction = -0.40 m/100 m
Final centroid elevation = 9.810m
Final ration of cut/fills = 1.30
Area levelled = 1.680 ha
Final volume of excavation = 1 163.194 m^3

For irregularly shaped fields, zero elevations are given to grid points that fall outside the field boundary as shown in Figure 93. These points will not be included in the calculation.

Figure 93

Irregular shaped field (elevations 0.0 are located outside the field)



Chapter 8
Design of the drainage system

Good water management of an irrigation scheme not only requires proper water application but also a proper drainage system. Agricultural drainage can be defined as the removal of excess surface water and/or the lowering of the groundwater table to below the root zone in order to improve plant growth. The common sources of the excess water that has to be drained are precipitation, over-irrigation and the extra water needed for the flushing away of salts from the root zone. Furthermore, an irrigation scheme should be adequately protected from drainage water coming from adjacent areas.

Drainage is needed in order to:

- Maintain the soil structure
- Maintain aeration of the root-zone, since most agricultural crops require a well aerated root-zone free of saturation by water; a notable exception is rice
- Assure accessibility to the fields for cultivation and harvesting purposes
- Drain away accumulated salts from the root zone

A drainage system can be surface, sub-surface or a combination of the two.

8.1. Factors affecting drainage

8.1.1. Climate

An irrigation scheme in an arid climate requires a different drainage system than one in a humid climate. An arid climate is characterized by high-intensity, short-duration rainfall and by high evaporation throughout the year. The main aim of drainage in this case is to dispose of excess surface runoff, resulting from the high-intensity precipitation, and to control the water table so as to prevent the accumulation of salts in the root zone, resulting from high evapotranspiration. A surface drainage system is most appropriate in this case.

In a humid climate, that is a climate with high rainfall during most of the year, the removal of excess surface and subsurface water originating from rainfall is the principal purpose of drainage. Both surface and subsurface drains are common in humid areas.

8.1.2. Soil type and profile

The rate at which water moves through the soil determines the ease of drainage. Therefore, the physical properties of the soil have to be examined for the design of a subsurface drainage system. Sandy soils are easier to drain than heavy clay soils.

Capillary rise is the upward movement of water from the water table. It is inversely proportional to the soil pore diameter. The capillary rise in a clay soil is thus higher than in a sandy soil.

In soils with a layered profile drainage problems may arise, when an impermeable clay layer exists near the surface for example.

8.1.3. Water quantity

The quantity of water flowing through the soil can be calculated by means of Darcy's law:

Equation 67

$$Q = k \times A \times i$$

Q = Flow quantity (m^3/sec)
k = Hydraulic conductivity (m/sec)
A = Cross-sectional area of the soil through which the water moves (m^2)
i = Hydraulic gradient

The hydraulic conductivity, or the soils' ability to transmit water, is an important factor in drainage flow. Procedures for field measurements of hydraulic conductivity are discussed below.

8.1.4. Irrigation practice

The irrigation practice has a bearing on the amount of water applied to the soil and the rate at which it is removed. For example, poor water management practices result in excess water being applied to the soil, just as heavy mechanical traffic results in a soil with poor drainage properties due to compaction.

8.2. Determining hydraulic conductivity

Hydraulic conductivity is very variable, depending on the actual soil conditions. In clear sands it can range from 1-1 000 m/day, while in clays it can range from 0.001-1 m/day. Several methods for field measurement of hydraulic conductivity have been established. One of the best-known field methods for use when a high water table is present is Hooghoudt's single soil auger hole method (Figure 94).

A vertical auger hole is drilled to the water table and then drilled a further 1-1.5 m depth or until an impermeable layer or a layer with a very low permeability is reached. The water level in the hole is lowered by pumping or by using buckets. The rate of recharge of the water table is then timed.

For the calculation of the hydraulic conductivity the following formula has been established:

Equation 68

$$k = \frac{3\,600 \times a^2}{(d + 10a) \times \left[2 - \frac{y}{d}\right] + y} \times \frac{\Delta H}{\Delta t}$$

Where:

- k = Hydraulic conductivity (m/day)
- a = Radius of the auger hole (m)
- d = Depth of the auger hole below the static ground water table (m)
- ΔH = Rise in groundwater table over a time (t) (cm)
- Δt = Time of measurement of rise in groundwater table (sec)
- y = Average distance from the static groundwater table to the groundwater table during the measurement: $y = 0.5 \times (y_1 + y_2)$ (m)

Note that this is an empirical formula and the units should be as explained above.

Example 38

An auger hole with a radius of 4 cm is dug to a depth of 1.26 m below the static groundwater table. The rise of the groundwater table, measured over 50 seconds, is 5.6 cm. The distance from the static groundwater table to the groundwater table is 0.312 m at the start of the measurement and 0.256 m at the end of the measurement. What is the hydraulic conductivity?

a = 0.04 m

d = 1.26 m

ΔH = 5.6 cm

Δt = 50 sec

y = 0.5 x (0.312 + 0.256) = 0.284 m

Substituting the above data in Equation 68 gives:

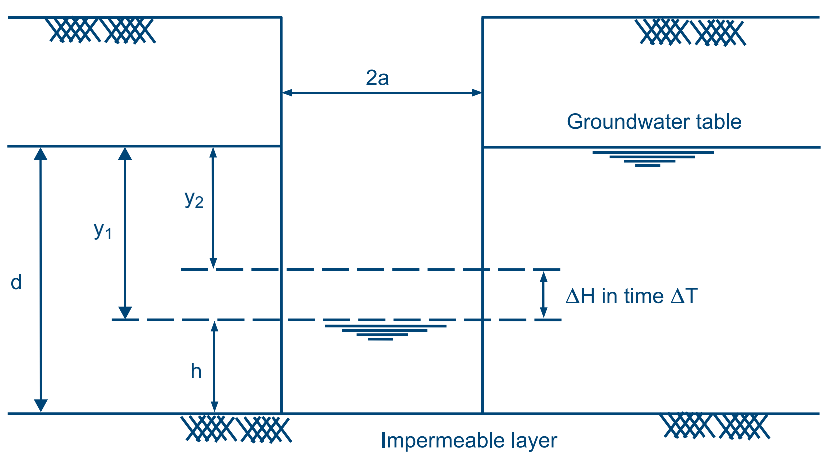
$$k = \frac{3\,600 \times 0.04^2}{(1.26 + 10 \times 0.04) + \left[2 - \frac{0.0284}{1.26}\right] \times 0.284} \times \frac{5.6}{50}$$

$$\Rightarrow k = 0.77 \text{ m/day}$$

If the water table is at great depth, the inverted auger hole method can be used. The hole is filled with water and the rate of fall of the water level is measured. Refilling has to continue until a steady rate of fall is measured. This figure is used for determination of k, which can be found from graphs.

Figure 94

Parameters for determining hydraulic conductivity using the auger hole method



8.3. Surface drainage

When irrigation or rainfall water cannot fully infiltrate into the soil over a certain period of time or cannot move freely over the soil surface to an outlet, ponding or waterlogging occurs. Grading or smoothening the land surface so as to remove low-lying areas in which water can settle can partly solve this problem. The excess water can be discharged through an open surface drain system. Examples of a layout of a drainage system are given in Figure 17 and 19, the latter representing Nabusenga surface irrigation scheme.

The drainage water can flow directly over the fields into the field drains. Drains of less than 0.50 m deep can be V-shaped. In order to prevent erosion of the banks, field drains often have flat side slopes, which in turn allow the passage of equipment. The side slopes could be 1:3 or flatter. Larger field drains and most higher orders drains usually have a trapezoidal cross-section as shown in Figure 95.

The water level in the drain at design capacity should ideally allow free drainage of water from the fields. The design of drain dimensions should be based on a peak discharge. It is, of course, impractical to attempt to provide drainage for the maximum rainfall that would likely occur within the lifetime of a scheme. It is also not necessary for the drains to instantly clear the peak runoff from the selected rainfall because almost all plants can tolerate some degree of waterlogging for a short period. Therefore, drains must be designed to remove the total volume of runoff within a certain period. If, for example, 12 mm of water (= 120 m^3/ha) is to be drained in 24 hours, the design steady drainage flow of approximately 1.4 1/sec per ha (= (120 x 10^3)/(24 x 60 x 60)) should be employed in the design of the drain.

If rainfall data are available, the design drainage flow, also called the drainage coefficient, can be calculated more precisely for a particular area. The following method is usually followed for flat lands. The starting point is a rainfall-duration curve, an example of which is shown in Figure 96. This curve is made up of data that are generally available from meteorological stations. The curve connects, for a certain frequency or return period, the rainfall with the period of successive days in which that rain is falling. Often a return period of 5 years is assumed in the calculation. It describes the rainfall which falls in X successive days as being exceeded once every 5 years. For design purposes involving agricultural surface drainage systems X is often chosen to be 5 days. Thus from Figure 96 it follows that the rainfall falling in 5 days is 85 mm. This equals a drainage flow (coefficient) of 1.97 l/sec per ha (= (85 x 10 x 10^3)/(5 x 24 x 60 x 60)).

Figure 95
Cross-sections of drains

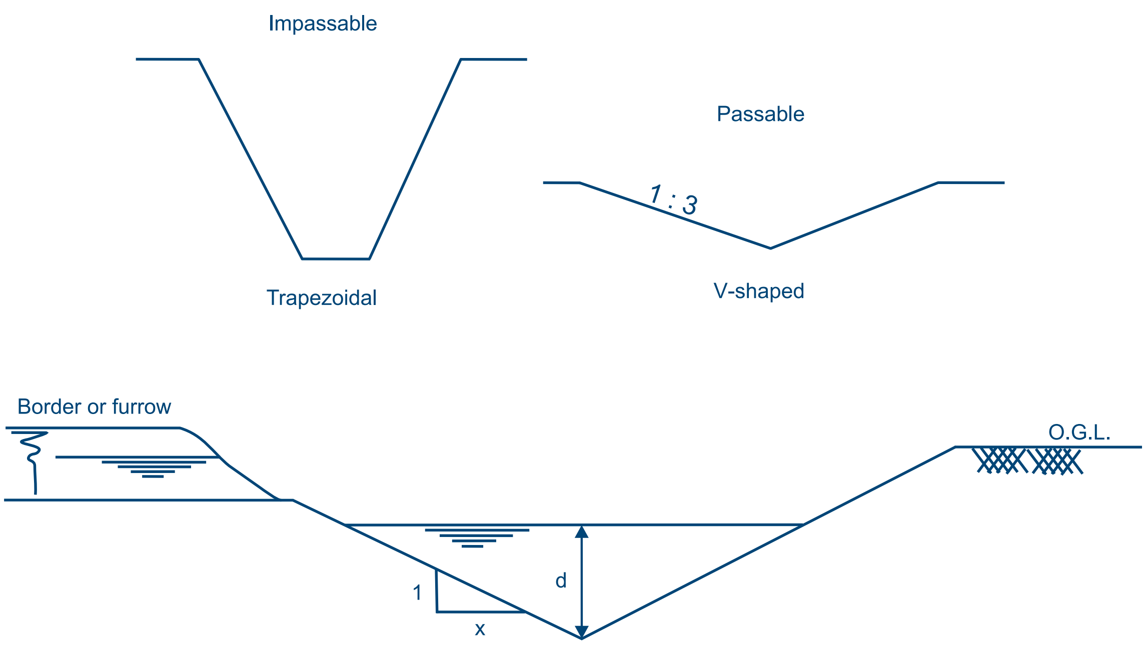
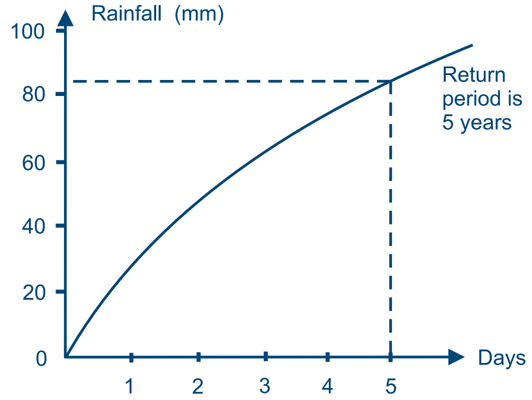


Figure 96
Rainfall-duration curve



The design discharge can be calculated, using the following equation:

Equation 69

$$Q = \frac{q \times A}{1\,000}$$

Where:

Q = Design discharge (m^3/sec)
q = Drainage flow (coefficient) (l/sec per ha)
A = Drainage area (ha)

It would seem contradictory to take 5 days rainfall, when the short duration storms are usually much more intensive. However, this high intensity rainfall usually falls on a restricted area, while the 5 days rainfall is assumed to fall on the whole drainage area under consideration. It appears from practice that a drain designed for a 5 days rainfall is, in general, also suited to cope with the discharge from a short duration storm.

Having said this, the above scenario is not necessarily true in small irrigation schemes, especially on sloping lands (with slopes exceeding 0.5%), which may cover an area that could entirely be affected by an intense short duration rainfall. The design discharge could then be calculated with empirical formulas, two of the most common ones being:

- ❖ The rational formula
- ❖ The curve number method

The rational formula is the easier of the two and generally gives satisfactory results. It is also widely used and will be the one explained below. The formula reads:

Equation 70

$$Q = \frac{C \times I \times A}{360}$$

Where:

Q = Design discharge (m^3/sec)
C = Runoff coefficient
I = Mean rainfall intensity over a period equal to the time of concentration (mm/hr−)
A = Drainage area (ha)

The time of concentration is defined as the time interval between the beginning of the rain and the moment when the whole area above the point of the outlet contributes to the runoff. The time of concentration can be estimated the following formula:

Equation 71

$$T_c = 0.0195 \times K^{0.77}$$

Where:

T_c = Time of concentration (minutes)
K = $\frac{L}{\sqrt{S}}$ and S = $\frac{H}{L}$
L = Maximum length of drain (m)
H = Difference in elevation over drain length (m)

The runoff coefficient represents the ratio of runoff volume to rainfall volume. Its value is directly dependent on the infiltration characteristics of the soil and on the retention characteristics of the land. The values are presented in Table 39.

Table 39
Values for runoff coefficient C in Equation 70

	Slope (%)	Sandy loam	Clay silty loam	Clay
Forest	0-5	0.10	0.30	0.40
	5-10	0.25	0.35	0.50
	10-30	0.30	0.50	0.60
Pastures	0-5	0.10	0.30	0.40
	5-10	0.15	0.35	0.55
	10-30	0.20	0.40	0.60
Arable land	0-5	0.30	0.50	0.60
	5-10	0.40	0.60	0.70
	10-30	0.50	0.70	0.80

Example 39

An irrigation scheme of 100 ha with sandy loam soils and a general slope of less than 5% has a main drain of 2.5 km long with a difference in elevation of 10 m. What is the time of concentration?

$$S = \frac{H}{L} = \frac{10}{2\,500} = 0.004 \text{ or } 0.4\%$$

$$K = \frac{L}{\sqrt{S}} = \frac{2\,500}{\sqrt{0.004}} = 39\,528$$

Substituting this value of K into Equation 71 gives:

$$T_c = 0.0195 \times 39\,528^{0.77} = 68 \text{ minutes}$$

The rainfall intensity can be obtained from a rainfall-duration curve, such as shown in Figure 96. For short duration rainfall, it is necessary to make a detailed rainfall-duration curve for the first few hours of the rainfall.

Example 40

In Example 39, the 68 minutes rainfall with a return period of 5 years is estimated at 8.5 mm. What is the design discharge of the drain?

The mean hourly rainfall intensity is (60/68) x 8.5 = 7.5 mm/hour.

The runoff coefficient for sandy loam arable land with a slope of less than 5% is 0.30 (Table 39).

Thus the design discharge for the scheme is:

$$Q = \frac{C \times I \times A}{360} = \frac{0.30 \times 7.5 \times 100}{360}$$

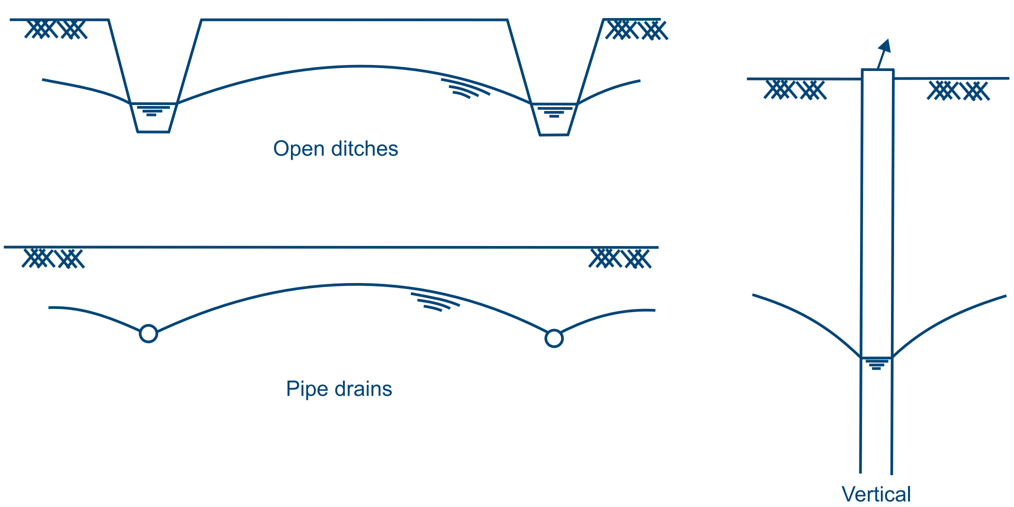
$$\Rightarrow Q = 0.625 \text{ m}^3\text{/sec or } 6.25 \text{ litres/sec per ha}$$

Once the design discharge has been calculated, the dimensions of the drains can be determined using the Manning Formula (Equation 13). It should be noted that higher order canal design should not only depend on the design discharge, but also on the need to collect water from all lower order drains. Therefore, the outlets of the minor drains should preferably be above the design water level of the collecting channel.

8.4. Subsurface drainage

Subsurface drainage is used to control the level of groundwater so that air remains in the root zone. The natural water table can be so high that without a drainage system it would be impossible to grow crops. After establishing the irrigation system the groundwater table might rise into the root zone because of percolation of water to the groundwater table. These situations may require a subsurface drainage system.

Figure 97
Subsurface drainage systems at field level



A subsurface drainage system at field level can consist of any of the systems shown in Figure 97:

- Horizontal drainage by open ditches (deep and narrowly-spaced open trenches) or by pipe drains
- Vertical drainage by tubewells

8.4.1. Horizontal subsurface drainage

Open drains can only be justified to control groundwater if the permeability of the soil is very high and the ditches can consequently be spaced widely enough. Otherwise, the loss in area is too high and proper farming is difficult because of the resulting small plots, especially where mechanized equipment has to be used.

Instead of open drains, water table control is usually done using field pipe drains. The pipes are installed underground (thus there is no loss of cultivable land) to collect and carry away excess groundwater. This water could be discharged through higher order pipes to the outlet of the area but, very often, open ditches act as transport channels.

The materials used for pipe drains are:

- Clay pipes (water enters mainly through joints)
- Concrete pipes (water enters mainly through joints)
- Plastic pipes (uPVC, PE, water enters through slots)

Plastic pipes are the most preferred choice nowadays, because of lower transport costs and ease of installation, although this usually involves special machinery

The principal design parameters for both open trenches and pipe drains are spacing and depth, which are both shown in Figure 98 and explained below Equation 72.

The most commonly used equation for the design of a subsurface drainage system is the Hooghoudt Equation:

Equation 72

$$S^2 = \frac{(4 \times k_1 \times h^2) + (8 \times k_2 \times d \times h)}{q}$$

Where:

S = Drain spacing (m)

k_1 = Hydraulic conductivity of soil above drain level (m/day)

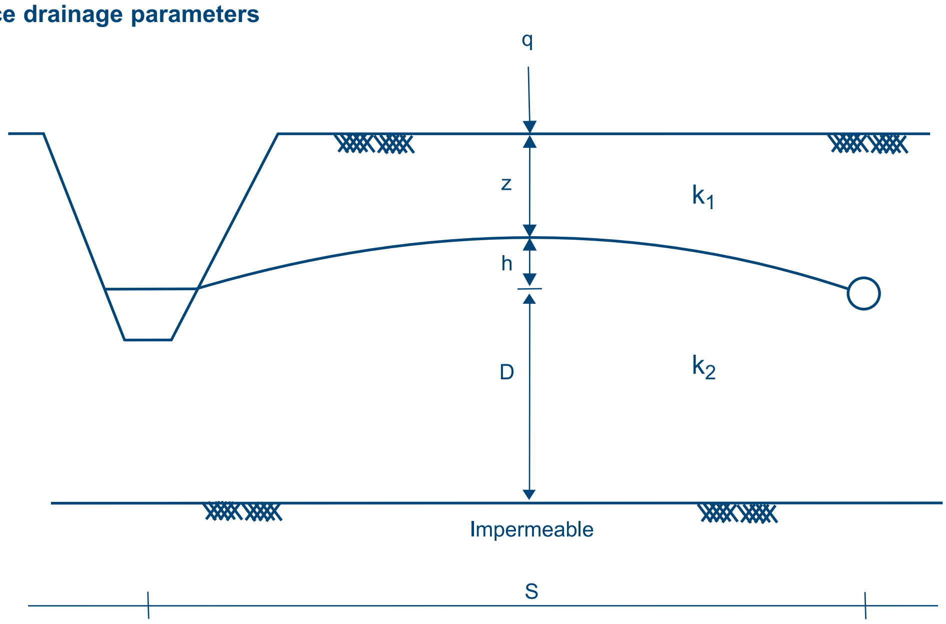
k_2 = Hydraulic conductivity of soil below drain level (m/day)

h = Hydraulic head of maximum groundwater table elevation above drainage level (m) (Figure 98)

q = Discharge requirement expressed in depth of water removal (m/day)

d = Equivalent depth of substratum below drainage level (m) (from Figure 99)

Figure 98
Subsurface drainage parameters



It should be noted that the Hooghoudt Equation is a steady state one, which assumes a constant groundwater table with supply equal to discharge. In reality, the head losses due to horizontal and radial flow to the pipe should be considered, which would result in complex equations. To simplify the equation, a reduced depth (d) was introduced to treat the horizontal/radial flow to drains as being equivalent to flow to a ditch with the impermeable base at a reduced depth, equivalent to d. The equivalent flow is essentially horizontal and can be described using the Hooghoudt formula. The average thickness (D) of the equivalent horizontal flow zone can be estimated as:

$$D = d + \frac{h}{2}$$

Nomographs have been prepared to determine the equivalent depth more accurately (Figure 99).

Nomographs have also been developed to determine the drain spacing (Figure 100). From example 41:

$$\frac{(4 \text{ x } k_1 \text{ x } h^2)}{q} = \frac{(4 \text{ x } 0.80 \text{ x } 0.6^{2)}}{0.002} = 576$$

and

$$\frac{(8 \text{ x } k_2 \text{ x } h)}{q} = \frac{(8 \text{ x } 0.80 \text{ x } 0.6)}{0.002} = 1\,920$$

Drawing a line from 576 on the right y-axis to 1 920 on the left y-axis in Figure 100, gives an S of about 90 m at the point where D = 5 m. Note that results obtained from the nomographs could differ slightly from the ones calculated with trial and error as above, because of reading inaccuracies.

Example 41

A drain pipe of 10 cm diameter should be placed at a depth of 1.80 m below the ground surface. Irrigation water is applied once every 7 days. The irrigation water losses, recharging the already high groundwater table, amount to 14 mm per 7 days and have to be drained away. An average water table depth, z of 1.20 m below the ground surface ,has to be maintained. k_1 and k_2 are both 0.8 m/day (uniform soil). The depth to the impermeable layer D is 5 m. What should be the drain spacing?

q = 14/7 = 2 mm/day or 0.002 m/day and h = 1.80 – 1.20 = 0.60 m

The calculation of the equivalent depth of the substratum d is done through trial and error. Initially the drain spacing has to be assumed (Figure 99). After determining d, the assumed S should be checked with the calculated S from the Hooghoudt Equation.

Lets assume S = 90 m. The wetted perimeter of the drain pipe, u, is 0.32 m (= 2 x π x r = 2 x 3.14 x 0.05). Thus D/u = 5/0.32 = 15.6.

From Figure 99 it follows that d = 3.65 m. This has been determined as follows:

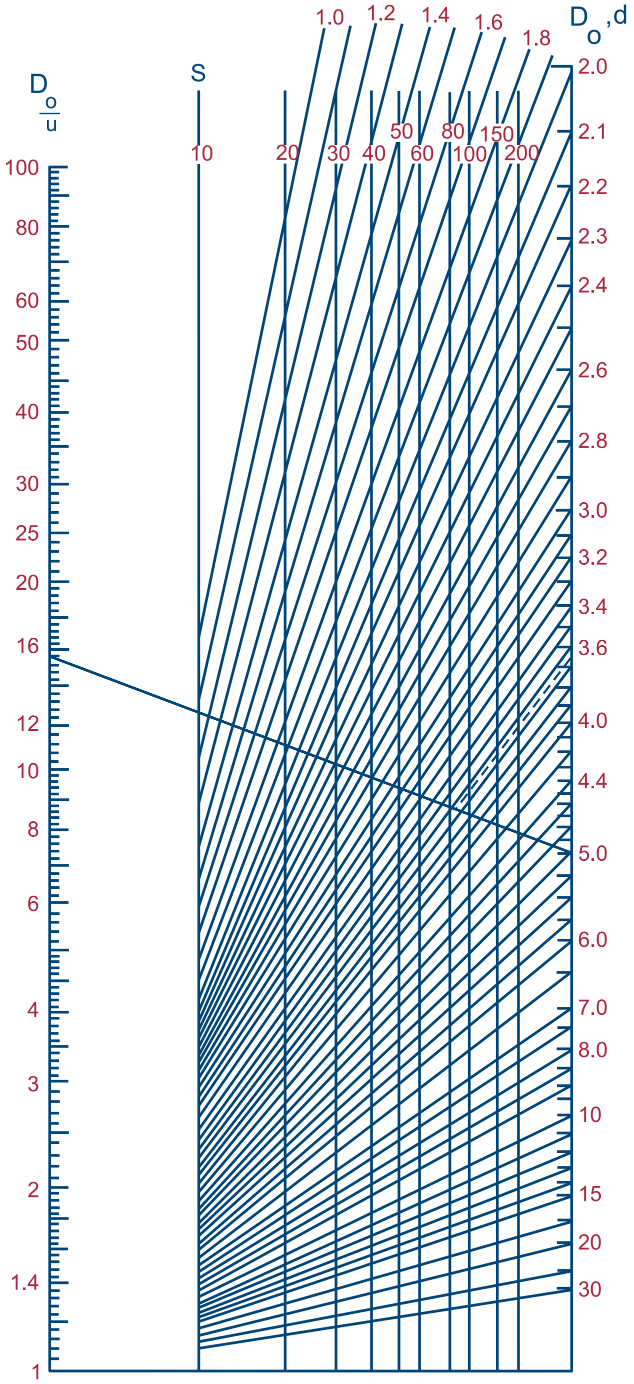
- Draw a line from D = 5 on the right y-axis to D/u = 15.6 on the left y-axis
- Determine the intersection point of the above line with the S = 90 line
- Draw a line from this point to the right y-axis, as shown by the dotted line
- The point where it reaches the right y-axis gives the d value

Substitution of all known parameters in the Hooghoudt Formula (Equation 72) gives:

$$S^2 = \frac{(4 \text{ x } 0.8 \text{ x } 0.6^{2)} + (8 \text{ x } 0.8 \text{ x } 3.65 \text{ x } 0.6)}{0.002} = 7\,584 \text{ m}^2$$

Thus S = 87 m, which means that the assumed drain spacing of 90 m is quite acceptable.

Figure 99
Nomograph for the determination of equivalent sub-stratum depths



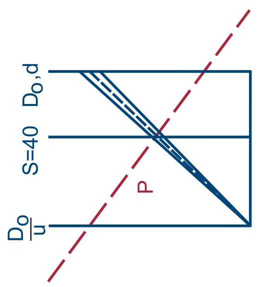
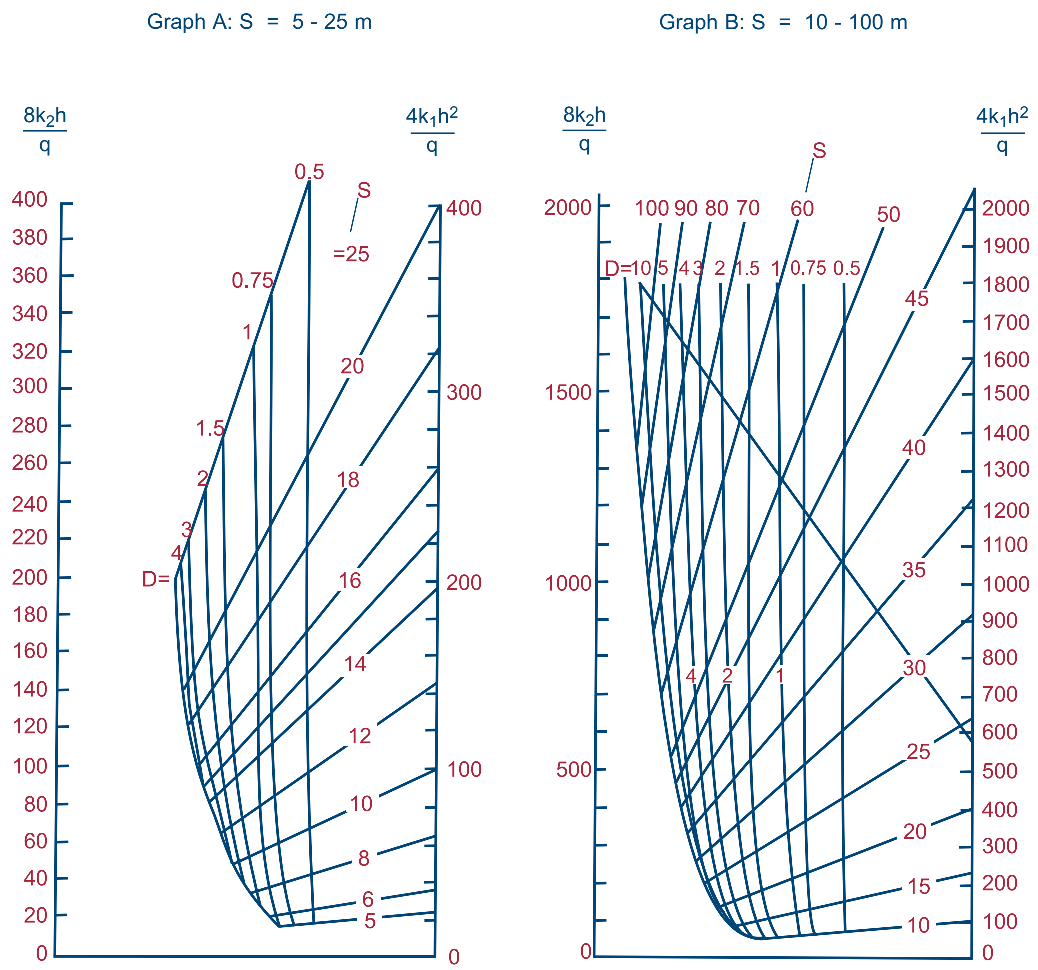


Figure 100
Nomograph for the solution of the Hooghoudt drain spacing formula



For pipe drains with r_o = 0.04 - 0.10 m, u = 0.30 m

8.4.2. Vertical subsurface drainage

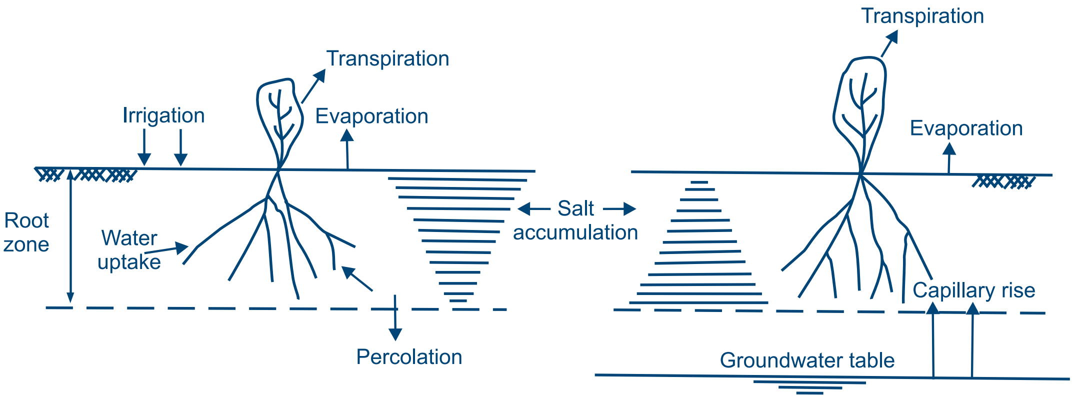
Where soils are of high permeability and are underlain by highly permeable sand and gravel at shallow depth, it may be possible to control the water table by tubewells located in a broad grid, for example at one well for every 2-4 km^2. Tubewells minimize the cost of and disturbance caused by field ditches and pipe drains and they require a more sparse drainage disposal network. If the groundwater is of good quality, it could be re-used for irrigation.

8.5. Salt problems

Salt problems in the root-zone occur mainly in arid countries. Drainage systems installed for the purpose of salinity control aim at removing salts from the soils so that a salinity level that would be harmful to plants is not exceeded. Irrigation water always contains salts, but to varying degrees. When the water is applied to the soil surface, some of it evaporates or is taken up by the plants, leaving salts behind in the root zone. If the groundwater table is too high, there will be a continuous capillary rise into the root zone and if the groundwater is salty, a high concentration of salts will accumulate in the root-zone. Figure 101 demonstrates this phenomenon.

Leaching is the procedure whereby salt is flushed away from the root-zone by applying excess water, sufficient in quantity to reduce the salt concentration in the soil to a desired level. Generally, about 10-30% more irrigation water than is needed by the crops should be applied to the soil for this purpose. This excess water has to be drained away by the subsurface drainage system.

Figure 101
Salt accumulation in the root zone and the accompanying capillary rise



Chapter 9
Bill of quantities

During the design stage, detailed technical drawings have to be made. These drawings are not only needed during the implementation stage, but they are also needed for the calculation of the bill of quantities and costs. An implementation programme or time schedule should be prepared as well, to give an estimate of the labour and equipment requirements. Details on the preparation of the implementation schedule are shown in Module 13. This chapter provides examples of the calculation of bill of quantities for a concrete-lined canal, a saddle bridge and a diversion structure at Nabusenga irrigation scheme and for a piped system at Mangui irrigation scheme. At the end, the overall bill of quantities for each scheme is given (excluding the headworks, conveyance system and night storage reservoirs).

9.1. Bill of quantities for Nabusenga irrigation scheme

9.1.1. The construction of a concrete-lined canal

From the Nabusenga design prepared, it can be seen that a total of 980 m of concrete-lined trapezoidal canal with a bed width of 350 mm has to be constructed. The cross-section for this canal is given in Figure 102.

Materials for the preparation of concrete

The volume of concrete required per metre of canal length is the sum of the volumes represented by the bottom, the two slanting sides and the two lips at the top. The dimensions can be measured from a design drawing of the canal cross-section. In our example of the 350 mm bottom width canal section, the volumes of the different sections are calculated as shown below:

- The area of the concrete for a side of the canal is (length L x thickness) and is calculated as follows:

  sin 60° = (0.05 + 0.30 + 0.05)/L

  → L 0.40/0.866 = 0.46 m

  (length x thickness) = 0.46 x 0.05 = 0.023 m

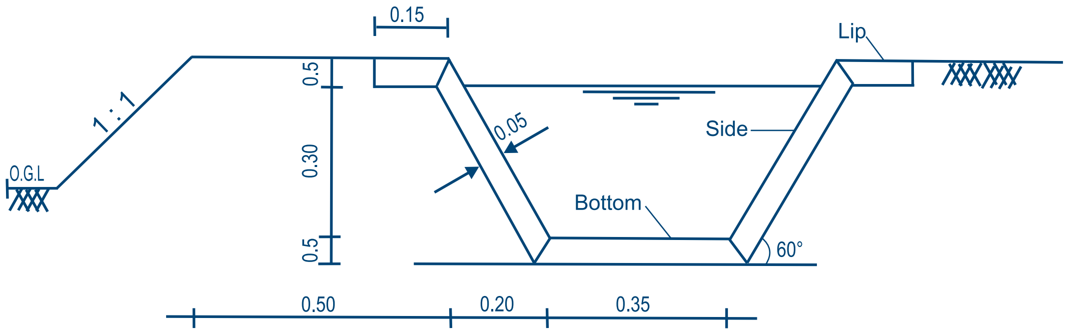
  → for two sides, the area is 2 x 0.023 = 0.046 m^2

- The width of the bottom is 0.35 m. However, this includes part of the slanting side (about 0.015 m at each side), when drawing the slanting sides down diagonally till the lower side of the bottom. Therefore, the length of the bottom to be used in the calculation is (0.35 - 0.03) = 0.32 m, giving a concrete area of (0.32 x 0.05) = 0.016 m^2

- The length of the lip is 0.15 m. However, this also includes part (about 0.05 m) of the slanting side. Therefore, the length of the lip to be used in the calculation is (0.15 - 0.05) = 0.10 m, giving a concrete area of 0.10 x 0.05 = 0.005 m^2 for one lip or 2 x 0.005 = 0.010 m^2 for both lips.

Thus, the concrete volume required for one metre length of canal is 0.016 + 0.046 + 0.010 = 0.072 m^3. Since the canal length is 980 m, the concrete volume required is

Figure 102
Cross-section of a concrete-lined canal at Nabusenga



(980 x 0.072) = 70.56 m^3. It is advisable to add 10% to the volume to cater for waste and uneven concrete thickness in excess of the 5 cm, thus the volume of concrete will be 1.10 x 70.56 = 77.6 m^3. This is the figure that will appear in the bill of quantities.

Table 40 gives the volume of concrete required for a number of trapezoidal cross-sections, similar to the one in Figure 102, whereby only the bed width changes.

Table 40

Concrete volume for different trapezoidal canal cross-sections

Bed width (mm)	Concrete volume (m^3 per100 m)
250	6.70
300	6.95
350	7.20
400	7.45
450	7.70
500	7.95

Different structures require different types of concrete grades, as discussed in Module 13. For concrete canals, a good concrete mix is 1:2:3, by volume batching. The materials required for such a mix per m^3 of concrete are calculated as follows:

Measuring by volume is based on loose volume. It can be assumed that a 50 kg bag of cement is equivalent to 40 litres of loose volume and that the yield of the mix is 60% of the loose volume of cement, fine aggregate (sand) and coarse aggregate (stone). This means that about 1.68 m^3 or 1 680 litres of cement, sand and stone required for the preparation of 1 m^3 of concrete. For a mixture of 1:2:3, this means that the loose volume is: 40 x 1 (cement) + 40 x 2 (sand) + 40 x 3 (stone) = 240 litres. Thus the yield is 0.6 x 240 = 144 litres. This gives the following quantities:

Cement: 1000/144 = 6.94 = 7 bags of 50 kg each

Sand: (7 x 40 x 2) = 560 litres or 0.56 m^3

Stones: (7 x 40 x 3) = 840 litres or 0.84 m^3

Thus for 980 m of canal, requiring 77.6 m^3 of concrete, the material requirements are:

Cement: 77.6 x 7 = 543 bags

Sand: 77.6 x 0.56 = 44 m^3

Stones: 77.6 x 0.84 = 65 m^3

Transport of materials

The materials for the construction of concrete (cement, sand and stone) are bulky and are therefore very expensive to transport to construction sites. To save on costs, cheap forms of transport should be sought. For example, if the site is close to a railway line, it is advisable to use this kind of transport, as in most countries it is cheaper than transport by road. One can also decide to combine the two modes of transport: rail can take the materials to some point and then the remaining distance can be covered by road. Where transport by road is used, it may be wise to go for big tonnage trucks, if possible, as these tend to be cheaper than smaller trucks because of reduced number of truck loads needed to deliver a given quantity of construction materials.

For the Nabusenga scheme, cement and coarse aggregate have to be transported from factories to the project site, while good quality sand is available from local rivers. In this example, let us assume that the cement (packed in 50 kg bags) and coarse aggregate would be transported by rail from the factory to nearest railway siding in the project vicinity, a distance of 240 km. The transport from the factory to that point is charged per ton. The weight of 1 m^3 of coarse aggregate (crushed stone) is approximately 1 600 kg. Thus in our example the total tonnage for the cement and coarse aggregate is:

(543 bags of cement x 50 kg per bag) +
(65 m^3 of stones x 1.600 kg/m^3)
= 131 150 kg ≈ 131 tons.

From the siding, the materials are transported by road to Nabusenga over a distance of 240 km. A 15 or a 30 ton truck can be hired for this purpose. The hire price can be charged either per ton per loaded km or include a charge for the empty return trip. In our case, the charge will be per loaded km.

Assuming the use of a 30 ton truck, the number of trips required for the transport of cement and coarse aggregate would be (131 tons/30 tons per trip) = 4.4. If this is to be the total load to be transported for the scheme, 5 trips have to be made. However, as cement and coarse aggregate are also needed for other works, such as structures, a non-integer figure can be used for this particular item in the bill of quantities and cost estimates.

Fine aggregate is usually collected from nearby rivers and sometimes at no cost, except for transport costs. This depends on the area or country in question. In this example, let us assume that large deposits of river sand are found within a distance of 20 km from Nabusenga. Due to the rough terrain conditions, a small 7 ton lorry would preferably be used, as very large lorries would have problems

Table 41

Summary of the bill of quantities for the construction of the 980 m long lined canal at Nabusenga

Item	Quantity	Unit	Unit cost	Total cost
Material:				
– Cement	543	bag		
– Coarse aggregate (stones)	65	m^3		
– Fine aggregate (sand)	44	m^3		
Transport:				
– Rail (cement and stones)	131	ton		
– Road (cement and stones)	4.4 x 240	trips x km		
– Road (sand)	11 x 20	trips x km		
Labour:				
– Skilled	80	person-day		
– Unskilled	640	person-day		
Equipment:				
– Concrete mixer	16 x 1	days x no.		
– Motorized bowser	16 x 20	days x km/day		
– Tractor + trailer	16 x 5	days x hr/day		
– 7 ton lorry	16 x 50	days x km/day		
SUB-TOTAL (including 10% contingencies)				

negotiating the bad roads. For this item, one has to know the running cost for a 7 ton lorry per loaded km or the hire price. Assuming damp sand will be collected from the river, the weight per m^3 will be approximately 1 700 kg (dry, loose sand weighs approximately 1 400 kg/m^3 and wet sand approximately 1 800 kg/m^3). Thus, 44m^3 of sand weighs an estimated (44 x 1700)/1 000 = 75 tons. This would require approximately (75/7) = 11 trips, using a 7 ton lorry.

Labour

A time schedule for the construction has to be drawn up, as discussed in Module 13. In general terms, a gang of 5 skilled and 40 unskilled workers should be able to complete 70 m of 350 mm bed-width canal per day. Thus, the 980 m length of canal could be completed in (980/70) = 14 days. It again is advisable to add 10% to the days for unforeseen circumstances to the labour requirements, which means that the work could be completed in (1.10 x 14) = approximately 16 days. The total labour requirement becomes:

Skilled: 5 persons x 16 days = 80 person-days

Unskilled: 40 persons x 16 days = 640 person-days

For the calculation of the cost, one has to know the rates for skilled band and unskilled labour per person day, which differs from one country to the other.

Equipment

The following equipment will be required during the construction period of 16 days, the rates of which should be obtained from those who hire out construction equipment:

- ❖ 1 concrete mixer (at a cost per day or per month)
- ❖ 1 motorized water bowser (at a cost per km, assuming the water is 10 km (one way) from site)
- ❖ 1 tractor (at a cost per hour) + 1 trailer (at a cost per day), assuming that the running hours per day are five
- ❖ 1 lorry of 7 tons (at a cost per km, assuming that the lorry runs 50 km per day for jobs like collecting materials, diesel, etc.)

The summary of the bill of quantities for lining the 980 m long canal are summarized in Table 42.

From Table 41, the cost per metre of the construction of a 350 mm canal in Nabusenga can be determined.

The bill of quantities and cost estimates for the other canal cross-section sizes can be determined in a similar way. The same gang of 5 skilled and 40 unskilled workers would construct about 100 m of 250 mm bed width canal and 50 m of 500 mm bed width canal per day.

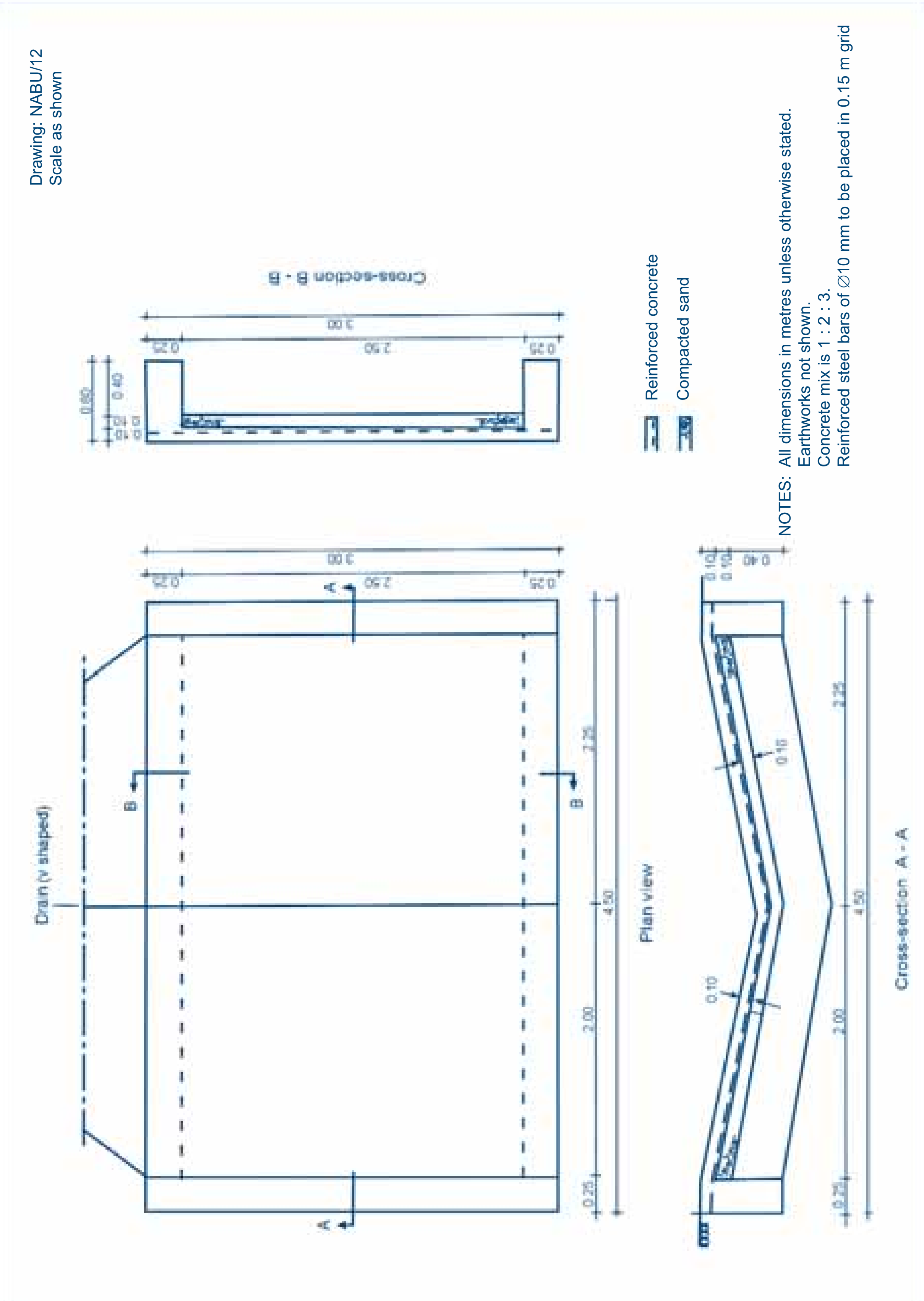
9.1.2. The construction of a saddle bridge

Figure 103 shows a typical design of a saddle bridge or drain-road crossing.

Materials for the preparation of concrete

The volume of concrete required for the structure is the sum of the volume of the slab and the toe around the structure. The dimensions can be measured from the design drawing. The slab volume (minus the area covered by the toe) is (length x width x thickness) = (4 x 2.5 x 0.10) = 1.0 m^3. The toe volume is (length x height x thickness) = [(2.5 x 2) + (4.5 x 2)] x 0.60 x 0.25) = 2.1 m^3.

Figure 103
Saddle bridge for Nabusenga



Thus the total concrete volume, inclusive of 10% contingencies, is 1.1 x (1.0 + 2.1) = 3.4 m^3.

The concrete mix will again be 1:2:3, thus the material requirements are:

Cement: 3.4 x 7 = 24 bags

Sand: 3.4 x 0.56 m^3 = 1.91 m^3

Stones: 3.4 x 0.84 = 2.86 m^3

Reinforcement steel

Plain steel bars of 10 mm diameter will be placed in the floor at a grid spacing of 15 cm. At the ends there should be a concrete cover of approximately 7.5 cm. In the direction of the width of the structure there should be (3.0/0.15) = 20 steel bars of 4.35 m each. In the direction of the length of the structure there should be (4.50/0.15) = 30 steel bars of 2.85 m each. Thus the total length of steel required, inclusive of 10% contingencies, is: {(20 x 4.35) + (30 x 2.85)} x 1.10 = 190 m

Transport of materials

The same procedure, as followed for the transportation of concrete canal lining materials, will apply for the saddle bridge:

Transport of cement and coarse aggregate by rail:
(24 bags x 50 kg per bag) + (2.86 m^3 x 1.600 kg/m^3)
= 5 776 kg » 5.8 tons

Transport of cement and coarse aggregate by road:
5.8 tons/30 tons per trip = 0.2 trips

Transport of fine aggregate from river:
1.91 m^3 x 1.700 kg/m^3 = 3 247 kg
= 3.25 tons/7 tons per trip = 0.47 trips.

Labour

It can be assumed that the saddle bridge can be completed in 2 days with a gang of 2 skilled workers and 4 unskilled workers. Thus the labour requirements are:

Skilled: 2 persons x 2 days = 4 person-days

Unskilled: 4 persons x 2 days = 8 person-days

The wages would be similar to those applicable for the construction of the canal.

Equipment

The equipment required for the construction is:

- 1 concrete mixer
- 1 motorized water bowser
- 1 tractor and trailer

This equipment will be required during the entire two days of construction. The charges would be the same as those for canal construction.

Table 42 is a bill of quantities for the saddle bridge.

Table 42
Summary of the bill of quantities for the construction of a saddle bridge

Item	Quantity	Unit	Unit cost	Total cost
Material:				
– Cement	24	bag		
– Coarse aggregate (stones)	2.86	m^3		
– Fine aggregate (sand)	1.91	m^3		
– Reinforcement steel bars	190	m		
Transport:				
– Rail (cement and stones)	5.8	ton		
– Road (cement and stones)	0.2 x 240	trips x km		
– Road (sand)	0.47 x 20	trips x km		
Labour:				
– Skilled	4	person-day		
– Unskilled	8	person-day		
Equipment:				
– Concrete mixer	2 x 1	days x no.		
– Motorized bowser	2 x 20	days x km/day		
– Tractor + trailer	2 x 2	days x hr/day		
SUB-TOTAL (including 10% contingencies)				

9.1.3. The construction of a diversion structure

A standard diversion structure could be constructed in one day. The calculation of the bill of quantities is similar to the one for the saddle bridge.

Materials for the preparation of concrete

The floor is made of reinforced concrete. The concrete mix is again 1:2:3. The concrete volume, including 10% contingencies, is (length x width x thickness) = (2.25 x 1.45 x 0.10) x 1.10 = 0.36 m^3

The walls are built up with concrete blocks. Assuming a mortar mix of 1:4, the material requirements per m^3 of mortar are 8 bags of cement and 1.28 m^3 of fine aggregate. The volume of mortar for the walls is (height x thickness x length). The openings for the canal and sluice gates should be excluded. Thus the volume, including 10% contingencies, is (0.25 x 0.50 x 5.15) x 1.10 = 0.71 m^3

The material requirements for the floor and the walls are:

Cement: (0.36 x 7) + (0.71 x 8) = 9 bags

Sand: (0.36 x 0.56) + (0.71 x 1.28) = 1.11 m^3

Stones: (0.36 x 0.84) = 0.30 m^3

Reinforcement steel and gates

The grid of steel bars is again 15 cm, thus the length of steel bars (assuming 7.5 cm concrete cover), including 10% contingencies, will be [(1.50/ 0.15) x 2.10 + (2.25/0.15) x 1.30] x 1.10 = 45 m.

The structure has two sliding gates to control the water distribution.

Transport of materials

Following the same procedure as in the previous two examples, the transport requirements are as follows:

Transport of cement and coarse aggregate by rail:
(9 bags x 50 kg/bag) +(0.30 m^3 x 1 600 kg/m^3)
= 930 kg = 0.93 tons

Transport of cement and coarse aggregate by road:
0.93 tons/30 tons per trip = 0.03 trips

Transport of fine aggregate from river:
1.11m^3 x 1 700 kg/m^3 = 1 887 kg
= 1.89 tons/7 tons per trip = 0.27 trips.

Labour

As indicated earlier, a gang of 2 skilled workers and 4 unskilled workers could complete a diversion structure in one day. Thus the labour requirements are:

Skilled: 2 persons x 1 day = 2 person-days

Unskilled: 4 persons x 1 day = 4 person-days

Equipment

The same equipment as required for the construction of the saddle bridge is also required for the construction of the diversion structure. Therefore, one concrete mixer, one water bowser and one tractor and trailer are needed for the construction of the diversion structure.

Table 43 is a bill of quantities for the diversion structure.

Table 43
Summary of the bill of quantities for the construction of a diversion structure

Item	Quantity	Unit	Unit cost	Total cost
Material:				
– Cement	9	bag		
– Coarse aggregate (stones)	0.30	m^3		
– Fine aggregate (sand)	1.11	m^3		
– Reinforcement steel bars	45	m		
– Sliding bar	2	no.		
Transport:				
– Rail (cement and stones)	0.93	ton		
– Road (cement and stones)	0.03 x 240	trips x km		
– Road (sand)	0.27 x 20	trips x km		
Labour:				
– Skilled	2	person-day		
– Unskilled	4	person-day		
Equipment:				
– Concrete mixer	1 x 1	days x no.		
– Motorized bowser	1 x 20	days x km/day		
– Tractor + trailer	1 x 1	days x hr/day		
SUB-TOTAL (including 10% contingencies)				

9.1.4. The overall bill of quantities for Nabusenga irrigation scheme

The bill of quantities and costs are usually summarized in a table, that shows the material, labour, transport and equipment requirements, as well as the costs for the specific job. Table 44 shows the bill of quantities for the construction of Nabusenga, downstream of the night storage reservoir. The material requirements could be summarized in a separate table to facilitate procurement (Table 45).

Table 44

Bill of quantities for Nabusenga scheme, downstream of the night storage reservoir

	Item		Quantity	Unit	Unit cost	Total cost
1.	**Templates and formers**					
	1.1.	1 former and 3 screeding frames for 250 mm width canal section	3	set		
	1.2.	1 former and 3 screeding frames for 350 mm width canal section	3	set		
2.	**250 mm bottom width canal section (1 325 m)**					
	2.1.	Cement	683	bag		
	2.2.	Coarse aggregate	82	m^3		
	2.3.	Fine aggregate	55	m^3		
	2.4.	Labour skilled	75	person-day		
	2.5.	Labour unskilled	600	person-day		
	2.6.	Equipment	–	lump		
	2.7.	Transport	–	lump		
3.	**350 mm bottom width canal section (980 m)**					
	3.1.	Cement	543	bag		
	3.2.	Coarse aggregate	65	m^3		
	3.3.	Fine aggregate	44	m^3		
	3.4.	Labour skilled	80	person-day		
	3.5.	Labour unskilled	640	person-day		
	3.6.	Equipment	–	lump		
	3.7.	Transport	–	lump		
4.	**Drainage channel**		1 400	m		
5.	**Road**					
	5.1.	Perimeter road, 5 m wide	1 600	m		
	5.2.	Field road, 2.5 m wide	650	m		
6.	**Land levelling**		15	ha		
7.	**Measuring device (2 pieces)**					
	7.1.	Steel bar 10 mm	40	m		
	7.2.	Cement	16	bag		
	7.3.	Coarse aggregate	1.6	m^3		
	7.4.	Fine aggregate	1.0	m^3		
	7.5.	Labour skilled	4	person-day		
	7.6.	Labour unskilled	8	person-day		
	7.7.	Equipment	–	lump		
	7.8.	Transport	–	lump		
8.	**Diversion structure (5 pieces)**					
	8.1.	Steel bar 10 mm	225	m		
	8.2.	Cement	45	bag		
	8.3.	Coarse aggregate	1.5	m^3		
	8.4.	Fine aggregate	5.6	m^3		
	8.5.	Labour skilled	10	person-day		
	8.6.	Labour unskilled	20	person-day		
	8.7.	Equipment	–	lump		
	8.8.	Sliding gate	10	each		
	8.9.	Transport	–	lump		
9.	**Canal-road crossing (1 piece)**					
	9.1.	Steel bar 10 mm	344	m		
	9.2.	Cement	30	bag		
	9.3.	Coarse aggregate	3.2	m^3		
	9.4.	Fine aggregate	2.1	m^3		
	9.5.	Labour skilled	6	person-day		
	9.6.	Labour unskilled	18	person-day		
	9.7.	Equipment	–	lump		
	9.8.	Transport	–	lump		

Item		Quantity	Unit	Unit cost	Total cost
10.	**Drain-road crossing/saddle bridge (3 pieces)**				
	10.1. Steel bar 10 mm	570	m		
	10.2. Cement	72	bag		
	10.3. Coarse aggregate	8.6	m^3		
	10.4. Fine aggregate	5.7	m^3		
	10.5. Labour skilled	12	person-day		
	10.6. Labour unskilled	24	person-day		
	10.7. Equipment	–	lump		
	10.8. Transport	–	lump		
11.	**Tail-end structure (5 pieces)**				
	11.1. Steel bar 10 mm	100	m		
	11.2. Cement	30	bag		
	11.3. Coarse aggregate	0.8	m^3		
	11.4. Fine aggregate	3.5	m^3		
	11.5. Labour skilled	10	person-day		
	11.6. Labour unskilled	20	person-day		
	11.7. Equipment	–	lump		
	11.8. Transport	–	lump		
12.	**Drop structures**	–	lump		
13.	**Check plates**	20	each		
14.	**Siphons**	250	m		
15.	**Fencing**				
	15.1. Anchor	48			
	15.2. Barbed wire, 4 lines	2 500			
	15.3. Corner post	23			
	15.4. Dropper	340			
	15.5. Gate, large 4.25 m	3			
	15.6. Labour skilled	20			
	15.7. Labour unskilled	200			
	15.8. Pignetting (4 ft, 3 inch)	2 500			
	15.9. Standard	170			
	15.10. Straining post	1			
	15.11. Transport (7 ton lorry 510 km)	–			
	15.12. Tying wire	3			
16.	**Miscellaneous**				
	16.1. Grain bags	200	each		
	16.2. Labour skilled[1]	945	person-day		
	16.3. Labour unskilled[2]	270	person-day		
	16.4. Materials and equipment (wheelbarrow, trowels, shovels, clothing)	–	lump		
	16.5. Preparatory work (site establishment)[3]	–	lump		
TOTAL (including 10% contingencies)					

Notes:

1. *It is assumed that 15 extra skilled workers are on site for 3 months. These include drivers, surveyors and a storekeeper.*
2. *Unskilled labour is required for setting out the irrigation works and finishing/cleaning up after construction is finished.*
3. *Site establishment on this project mainly consists of setting up tents. The water supply and other site requirements already exist at the project site.*

Table 45
Summary of material requirements for Nabusenga (including 10% contingencies)

Description	Quantity	Unit
Steel bar 10 mm	1 175	m
Cement	1 088	bag
Check plate	20	each
Coarse aggregate	125.3	m^3
Fine aggregate	90.3	m^3
Sliding gate	10	each
Siphon (38 mm diameter)	250	m
Fencing:		
– Anchor	48	each
– Barbed wire	13	roll
– Corner post	23	each
– Dropper	340	each
– Gate	3	each
– Pignetting	50	roll
– Standard	170	each
– Straining post	1	each
– Tying wire	3	roll
Former and screeding frames:		
– 250 mm width	3	set
– 350 mm width	3	set
Grain bag	200	each

9.2. Bill of quantities for Mangui irrigation scheme

In Table 46 only the bill of quantities for the pipes and fittings and pumping plant at Mangui scheme are given. All the other requirements (labour, transport, fencing, roads, structures, equipment, etc.) are calculated in a similar way as was done for Nabsenga scheme.

Table 46

Bill of quantities for pipes and fittings and pumping plant at Mangui scheme

Item			Quantity	Unit	Unit cost	Total cost
1.	**Piping**					
	1.1.	PVC pipe, 160 mm, class 4	198	m		
	1.2.	PVC pipe, 140 mm, class 4	90	m		
	1.3.	PVC pipe, 110 mm, class 4	36	m		
	1.3.	PVC pipe, 90 mm, class 4	36	m		
2.	**Fittings on pipelines**					
	2.1.	BP 160 mm 90°	1	no.		
	2.2.	RBP 160 mm to 140 mm	1	no.		
	2.3.	RBP 140 mm to 110 mm	1	no.		
	2.4.	RBP 110 mm to 90 mm	1	no.		
	2.5.	TCP plus TRBP 90 mm	1	no.		
	2.6.	Cast iron gate valve, 6 inch	1	no.		
	2.7.	Cast iron gate valve, 4 inch	1	no.		
	2.8.	TCP with TRBP 160 mm	2	no.		
	2.9.	Bolts and nuts to secure CI gate valves	lump	lump		
3.	**Hydrant assemblies**					
	3.1.	Saddle 160 mm with 3 inch BSP socket	4	no.		
	3.2.	Saddle 140 mm with 3 inch BSP socket	3	no.		
	3.3.	Saddle 110 mm with 3 inch BSP socket	1	no.		
	3.4.	Saddle 90 mm with 3 inch BSP socket	1	no.		
	3.5.	GI pipe 3 inch x 1.5 m long, male threaded on both ends	9	no.		
	3.6.	GI 3 inch equal Tee, female threaded on three ends	9	no.		
	3.7.	3 inch x 2 inch reducing bush, male threaded	18	no.		
	3.8.	Brass gate valve 2 inch	18	no.		
	3.9.	Reinforced plastic hose, 32 mm x 20 m long, 4 bar pressure	18	no.		
	3.10.	Hose clips, 32 mm	18	no.		
	3.11.	Hose adapters, 32 mm	18	no.		
4.	**Pumping plant**					
	Pumping (unit) plant capable of delivering 34.56 m^3/hr against a head of 11.5 m, with the highest possible efficiency. Pump to be directly coupled to a diesel engine of appropriate horse power rating or electric motor of acceptable kilowatt power rating. Pumping unit to be complete with suction and delivery pipes, valves, strainer, non-return and air release valves, pressure gauge.					
		SUB-TOTAL				
		Contingencies 10%				
		TOTAL				

Chapter 10

Operation and maintenance of surface irrigation systems

10.1. Operation of the irrigation system

10.1.1. Water delivery to the canals

There are three methods for delivering water to canals:

- Continuous delivery
- Rotational delivery
- Delivery on demand

Continuous water delivery

Each field canal or pipeline receives its calculated share of the total water supply as an uninterrupted flow. The share is based on the irrigated area covered by each canal or pipeline. Water is always available, although it may not always be necessary to use it. This method is easy and convenient to operate, but has a disadvantage in its tendency to waste water. The method is rarely used in small irrigation schemes.

Rotational water delivery

Water is moved from one field canal or pipeline or from a group of field canals or pipelines to the next. Each user receives a fixed volume of water at defined intervals of time. This is a quite common method of water delivery.

Water delivery on demand

The required quantity of water is delivered to the field when requested by the user. This on-demand method requires complex irrigation infrastructure and organization, especially when it has to be applied to small farmer-operated schemes where the number of irrigators is large and plot holdings are small.

10.1.2. Water delivery to the fields

The water, delivered in an open canal or pipeline, can be supplied onto the fields in different ways, which are briefly explained below.

Bank breaching

Bank breaching involves opening a cut in the bank of a field canal to discharge water onto the field. Although this method is practiced widely, it is not recommended, as the canal banks become weak because of frequent destruction and refill. It also becomes difficult to control the flow properly. Figure 104 shows how bank breaching is done.

Permanent outlet structures

Small structures, installed in the bank of a field canal are used to release water from the field canal onto the fields.

Figure 104

Field canal bank breaching in order to allow the water to flow from the canal onto the field

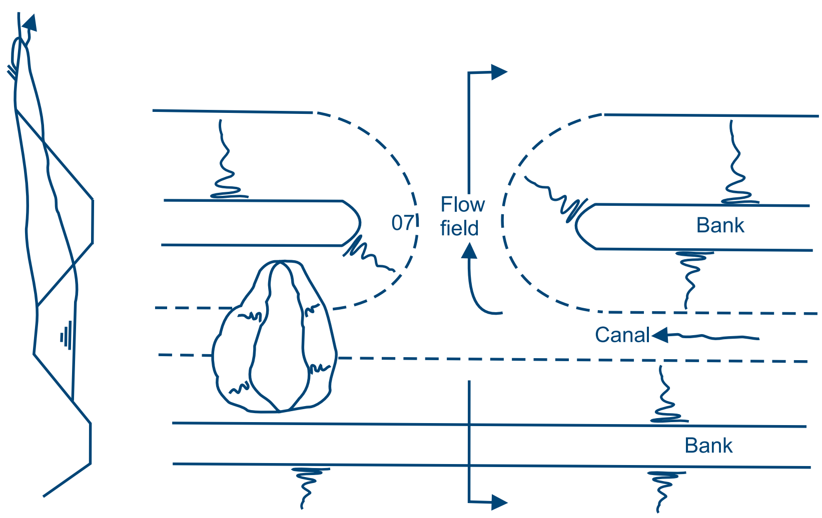


Figure 105 shows a permanent outlet structure.

The structures can be made of timber with wooden stop logs or of concrete with steel gates. This method is especially used for borderstrip and basin irrigation. It usually gives good water control to the fields. The disadvantage is that the structures are fixed, thereby reducing the flexibility of water distribution. Table 48 gives approximate discharges of small wooden field outlets like those shown in Figure 105 (FAO, 1975a).

Table 48
Discharge of permanent wooden field outlet structures

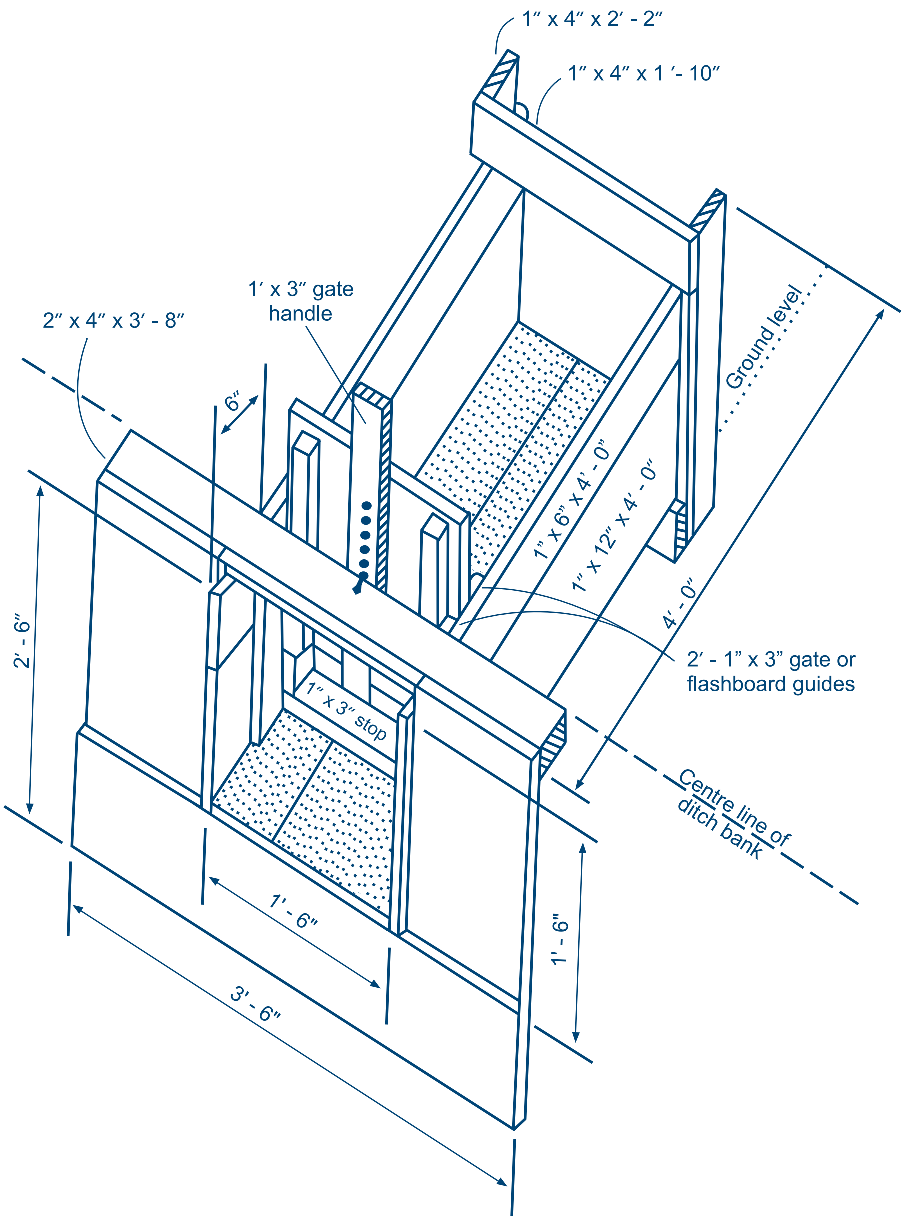
Depth of water over the sill at the intake (cm)	Discharge per 10cm width of the sill (l/sec)
10	6
15	11
20	17
25	22

Spiles

Spiles are short lengths of pipes made from rigid plastic, concrete, steel, bamboo or other material and buried in the canal bank as shown in Figure 106.

The discharge depends on the pipe diameter and the head of water available. A plug is used to close the spile on the inlet side. Since spiles are permanently installed, they have

Figure 105
Permanent outlet structure used to supply water from the canal onto the field (Source: FAO, 1975a)



the same disadvantage as the permanent outlet structures. The approximate discharge can be calculated using Equation 34 (see Section 6.1.3):

$$Q = C \times A \times \sqrt{2gh}$$

Where:

- Q = discharge through the spile (m^3/sec)
- C = discharge coefficient
- A = cross sectional area of outlet (m^2)
- g = gravitational force (9.81 m/sec^2)
- h = head of water, measured from the centre of the spile (m)

Table 49 gives approximate flows through small spiles.

Table 49
Rates of discharge through spiles (l/sec)

Diameter of pipe (cm)	Pressure head (cm) 5	10	25	20	25
20	18.7	26.4	32.3	37.3	41.7
25	29.2	41.3	50.5	58.3	65.2
30	42.0	59.4	72.8	84.0	93.9
35	57.2	80.9	99.0	114.4	127.8

For piped systems, the openings at hydrants act in the same manner as spiles and the discharge at the hydrant opening is calculated using Equation 34 (see Section 6.1.3).

Siphons

Siphons are short lengths of pipe usually made of plastic, rubber hose, or aluminium and are used to convey water from open channels to the field. They are portable and easy to install and to remove without disturbing the canal bank. The discharge of water onto the irrigated area varies according to the number of siphons in the furrow, border strip or basin.

In order to use a siphon, it is put with one end in the water and then filled with water (through suction by hand) to take out the air. It is then laid over the canal bank while a hand placed over the end of the pipe prevents air re-entering the pipe. This process is called priming.

The discharge through the siphon depends on its diameter, its length and the difference in level, h, between the water level in the canal and the water level on the adjacent field (or the centre of the pipe outlet if the pipe is not submerged in water (see also Section 1.3.3). Figure 107 shows a siphon in operation. Since the pipe is usually short, the influence of its frictional losses on the flow is negligible.

The water level in the canal should always be above the level of the siphon outlet. A proper siphon command (h) should be between 10 cm and 30 cm. The discharge through the siphon can be calculated using Equation 34. The C value is approximately 0.55.

Figure 106
An example of a spile used to supply water from the canal onto the field (Source: FAO, 1975a)

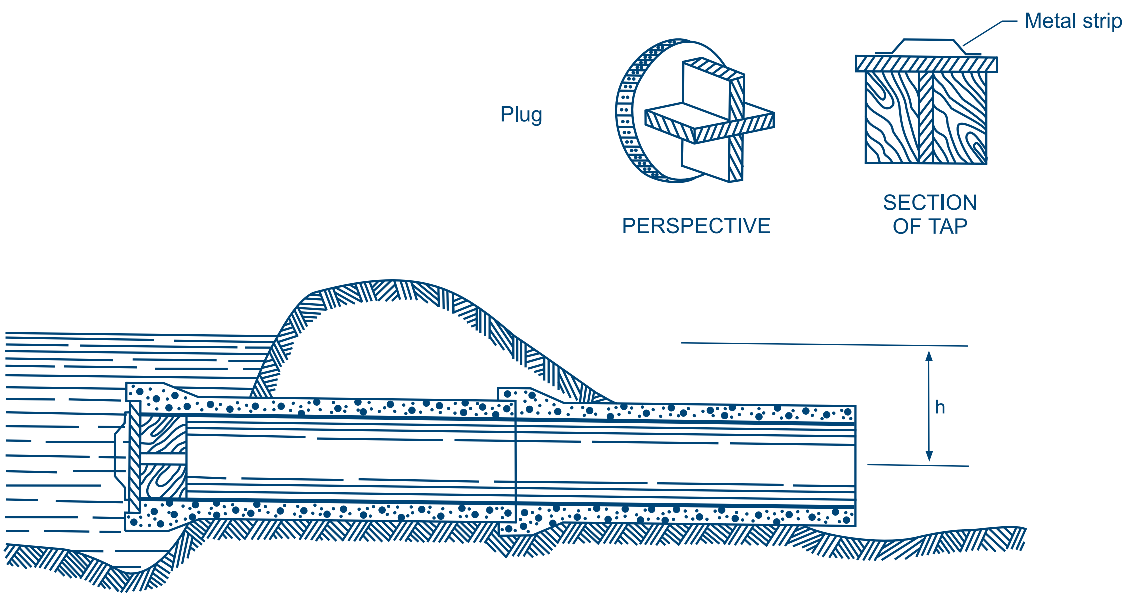
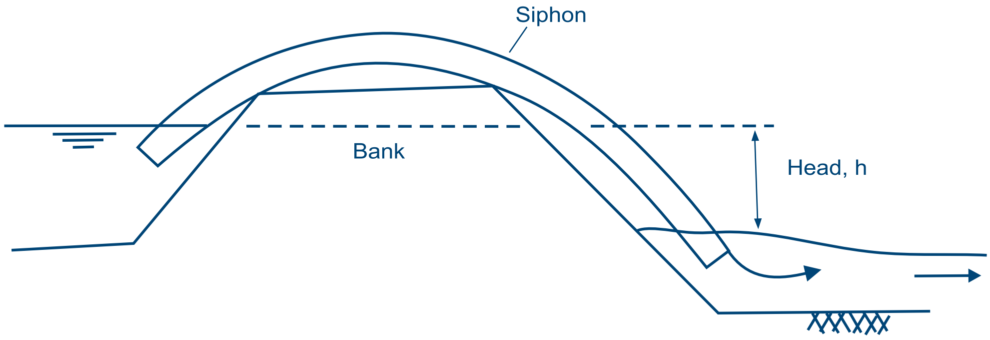


Figure 107
A siphon supplying water from a canal onto the field



Example 42

The flow through siphon is Q = 4.55 l/sec. The head of water h = 0.18 m. What should be the diameter of siphon?

Substituting the above data in the Equation 34 gives:

$$0.00455 = 0.55 \times \frac{1}{4} \times \pi \times d^2 \times (2 \times 9.81 \times 0.18)^{1/2}$$

Solving this equation results in a required siphon diameter of 7.5 cm

Example 43

A field canal carries a flow of 78.3 l/sec to irrigate a field using furrow irrigation. Each furrow requires a flow of approximately 3.31 l/sec. What should be the number of siphons that can be used to irrigate a furrow and what is the total number of siphons for the discharge of 78.3 l/sec?

From Table 50 it follows that for each furrow for example two siphons with a diameter of 6 cm each can be used, if the available head is 5 cm, or one siphon with a diameter of 6 cm can be used, if the command is 20 cm. In order to utilize the total discharge of 78.3 l/sec, 24 (=78.3/3.31) furrows can be irrigated at the same time. In case the command is 5 cm only, this means that the total number of siphons for the 24 furrows is equal to 24 x 2 = 48 siphons.

Table 50 gives rates of discharge of 2 m to 3 m long siphons for different diameter, d, and head, h (see also Section 1.3.3 for smaller sizes). From the table it can be concluded that the discharge changes when the head changes. It is therefore important to maintain a constant head in the canal.

Using more than one siphon gives the opportunity to remove one (cut back the flow), once the water reaches the end of the furrow (see Section 4.3). According to the quarter contact time rule, water should reach the end of the furrow in about 1/4 of the contact time. In order to reduce runoff losses after the water has reached the end of the furrow, the flow should be reduced, ideally such that the inflow equals the actual infiltration. This reduction is easier when there is initially more than one siphon.

10.1.3. Operational success determinants

Proper operation of irrigation schemes requires attention to the following points:

- The water distribution should be in line with the design and crop water requirements
- There should be equitable water distribution among farmers
- Advice on proper water management in order to minimize water losses should be given

Table 50
Discharge for siphons for different head and pipe diameter (l/sec)

Pipe ϕ (cm)	Head (cm)						
	5	7.5	10	12.5	15	17.5	20
4	0.75	0.91	1.06	1.18	1.29	1.40	1.49
5	1.17	1.43	1.65	1.85	2.02	2.18	2.33
6	1.68	20.6	2.38	2.66	2.91	3.14	3.36
7	2.29	2.80	3.24	3.62	3.96	4.28	4.58
8	2.99	3.66	4.23	4.72	5.18	5.59	5.98
9	3.78	4.63	5.35	5.98	6.55	7.07	7.56
10	4.67	5.72	6.60	7.38	8.09	8.73	9.34

Water distribution and application

As discussed in the previous sections, there are three methods of distributing water: continuous flow, rotational water supply and on-demand water delivery. The best method to adopt depends entirely on the situation at hand.

As a rule, rotational water supply is used for smallholders because of its simplicity. However, fixed rotation does not correspond to the different water requirements of the crops at different stages of growth. Thus, farmers are obliged to apply the same frequency and to some extent the amount of water, irrespective of water demand by the crops. This results in reduced yields and water wastage. To improve the rotational distribution, blocking has been introduced in Southern Africa. The total scheme is divided into four blocks, one for each major crop. Each farmer would then be allocated a plot within each block and a rotation of water supply is used among the four blocks. This improves the potential for applying a rotational irrigation schedule and improves the equitable distribution of water among users.

Equitable water distribution among farmers

Ideally, irrigators should get their fair share of irrigation water. However, this is often not the case. The most common problems are unauthorized water abstraction and lack of sufficient water for tail-end users. In the latter case, farmers at the head of the irrigation system receive and tend to use more water than they need, while those at the tail-end receive less than they need. In order to solve these problems, good cooperation and trust among the irrigators is important. If all the water can be diverted into one or a few canals at a time, there is less chance of illegal water abstraction. Once the water is diverted into a few canals, tail-end problems could be further reduced by allowing farmers to irrigate in groups, starting from the bottom end of the canals, going upwards. The incorporation of stiffer penalties in the farmers' bylaws and their enforcement also helps to reduce the problems.

Advice to farmers on proper water management

In many new irrigation projects, the farmers involved do not have experience with irrigation. They need agronomic advice as well as assistance in water management. With regards to water management, the farmers should be assisted in determining parameters like contact time, advance and recession and the number of siphons to use in each furrow, border or basin. Similarly, they should be trained in operating structures such as measuring devices and night storage reservoirs.

10.2. Maintenance of the irrigation system

There are three main types of maintenance namely:

1. Special maintenance
2. Deferred maintenance
3. Routine or normal maintenance

10.2.1. Special maintenance

Special maintenance includes work that is done to repair the irrigation system in response to unforeseen damages, such as those caused by floods or earthquakes. In this case no specific preventative measures would have been taken to circumvent the damage.

10.2.2. Deferred maintenance

Deferred maintenance or rehabilitation includes any work that is done on the irrigation infrastructure in order to restore the capacity of the system. In this case, the system is allowed to deteriorate to a certain level, beyond which it would not operate well, before it is restored to its design operational level. Sometimes, deferred maintenance and rehabilitation are differentiated on the basis of the source of funds. The funds for deferred maintenance come from the operation and maintenance budget, while that of rehabilitation comes as an investment funded by loans or national development budgets.

10.2.3. Routine maintenance

This includes all the work that is done in order to keep the irrigation system operating satisfactorily. It is normally done annually.

During the construction of the irrigation scheme, the future irrigators should provide labour for construction activities. Besides the advantage of promoting scheme ownership by farmers, farmer involvement in construction work teaches them several aspects of repair and maintenance.

Once the scheme is operational, the irrigation committee should mobilize the farmers for repair and maintenance activities. The works to be included in a maintenance programme are discussed below.

Headworks

The main problems with the headworks are leakages. Regular desilting is also necessary.

Night storage reservoirs

Night storage reservoirs should not stay dry for a long time as this allows cracks in the clay in the core and bed to develop. It is necessary, however, to empty the reservoirs from time to time in order to clear them of weeds. Weeds, besides harbouring snails, tend to reduce the capacity of night storage reservoirs. It is also recommended to allow the water level in the reservoirs to fluctuate to control snails.

Canal system

The main problems are with unlined canals siltation, weed growth, bank breaching, erosion caused by rainfall or burrowing by animals. Lined canals have problems of damaged joints, siltation, cracked sections or erosion of canal banks. Weed growth can also be a problem in lined canals, especially if silt is allowed to accumulate. As soon as these problems are noticed, they should be rectified. Regular desilting and weed removal is required. Both can be done by hand. Table 51 gives a simplified typical weed management programme for some schemes in some hot areas of Zimbabwe. This should be used as a guide only, since management depends on the climate of a particular area.

Drains

The most common problem with drains is weed growth. Weeds should be frequently removed so as to maintain the design capacity of the drains. Table 51 gives the guidelines.

Roads

Roads need refilling of potholes and gullies that may develop.

Embankments

The common problems of embankments are erosion, leakages and weed growth. Refill and soil compaction should be done when repairing embankments. Weeds should be slashed.

Land levelling

After the initial land levelling during project construction, it is necessary to periodically level the fields in order to maintain the desired field slope. This can be done by machinery or manually. If levelling is done manually, it is still recommended that after every two to four seasons farmers use machinery, such as a land plane.

Structures

The common maintenance problems are with structures siltation, leakages caused by cracking and weed growth. They should be maintained accordingly.

Gates

Gates can have problems of rusting or sticking over time and leaking. They should be painted to prevent rusting. Any movable parts should be greased or oiled to prevent sticking. Replacing warn-out water seals, if there are any, can minimize leaking.

10.3. Operation and maintenance responsibilities

The operation and maintenance of smallholder irrigation schemes can be the responsibility of either the government, the irrigation agency, individual farmers or groups of farmers. It can also be a joint responsibility between groups of farmers and the government, depending on the size of the scheme. In large schemes or government-run schemes, the irrigation agency and the farmers often share the responsibility of operating and maintaining the irrigation infrastructure. In such cases, the operation and maintenance of the water delivery and storage system is normally the responsibility of the agency, while the farmers are responsible for maintaining field level infrastructure such as canals and small hydraulic structures. The dividing line, however, is not very clear. Therefore, the agency and the farmers need to agree on their responsibilities and write them down in bylaws. Where irrigation projects are operated and maintained by farmers, as is the case for small community schemes, the farmers themselves bear all responsibilities for operation and maintenance. But even in this case, rules and regulation should be written down in bylaws.

Table 51
Weed management and effectiveness

Canal/drain	Maintenance	Effectiveness
Concrete-lined field canal	Hoeing within canal Slashing/hoeing sides 2-3 times per year	up to 4 weeks slashing 4 weeks; hoeing 6-8 weeks
Concrete-lined main canal	Slashing canal shoulders 3 times per year	up to 4 weeks
Night storage reservoirs	Desilting every 5 years	every 5 years
Infield drains	Slashing within drain	up to 4 weeks in wet season
Main drains	Slashing 2 times per year hoeing and reprofiling once per year	up to 3-4 months in dry season up to 6 months; up to one year

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