SAFR/AGLW/DOC/007

Irrigation Manual

Planning, Development Monitoring and Evaluation of Irrigated Agriculture with Farmer Participation

Developed by

Andreas P. SAVVA
Karen FRENKEN

Volume II
Module 7

Food and Agriculture Organization of the United Nations (FAO)
Sub-Regional Office for East and Southern Africa (SAFR)
Harare, 2002

The views expressed in this paper are those of the authors and do not necessarily reflect the views of the Food and Agriculture Organization of the United Nations

ISBN 0-7974-2315-X

Design and Layout: Fontline Electronic Publishing, Harare, Zimbabwe
Printed by: Préci-ex, Les Pailles, Mauritius

Foreword

The first edition of the Irrigation Manual was published in 1990 in two volumes by the "Smallholder Irrigation" Project (UNDP/FAO/AGRITEX/ZIM/85/004). The authors of this first edition were FAO Staff on the project[1]. This edition of one hundred copies ran out within two years from publishing.

Although the manual was written with Zimbabwe in mind, it soon became popular in several countries of the sub-region. In view of the high demand, it was decided to proceed with a second edition. The experience gained from using the first edition of the manual as the basic reference for the AGRITEX[2] training programme of irrigation practitioners and the University of Zimbabwe, was incorporated in the second edition which was published in 1994, in one volume by the "Technical Assistance to AGRITEX" project (UNDP/FAO/AGRITEX/ZIM/91/005). This second edition was published under the same authors as the first edition, with the assistance of a review committee from AGRITEX[3]. The two hundred copies of this edition also ran out within two years of publishing.

In 1995, the FAO Sub-regional Office for East and Southern Africa (SAFR) was established in Harare, Zimbabwe, in order to provide easy access to technical assistance and know-how for the countries of the sub-region[4]. In view of the high demand for support in the field of smallholder irrigation by the countries of the sub-region, this office was strengthened with four water resources management officers and a number of on-going programmes have been developed to provide this support. One of these programmes is the publishing of a new regional edition of the irrigation manual in support of the on-going national training programmes within several countries in the sub-region and to provide the basic reference for another important programme, which is the sub-regional training on planning and design of smallholder irrigation schemes.

This third edition aspires to further strengthen the engineering, agronomic and economic aspects of the manual and to introduce new modules related to social, health and environmental aspects of irrigation development. The emphasis is directed towards the engineering, agronomic and economic aspects of smallholder irrigation, in view of the limited practical references in this area. This manual, being directed to the irrigation practitioner, does not provide an in-depth analysis of the social, health and environmental aspects in irrigation development. It only attempts to introduce the irrigation practitioner to these areas, providing a bridge between the various disciplines involved in irrigation development.

The initiatives and efforts of the Water Resources Management Team of SAFR in publishing this Manual are considered as a valuable contribution to the dissemination of knowledge and training of irrigation practitioners in the sub-region. The material covered by this manual is expected to support both national and sub-regional training programmes in the planning, design, construction, operation and maintenance and on-farm water management of irrigation schemes. This will support the implementation of FAO's mandate to increase food production through water control, intensification and diversification, which are the basic components of the Special Programme for Food Security (SPFS).

The manual is the result of several years of field work and training irrigation engineers in the sub-region. The approaches have been field tested and withstood the test of time.

---

[1] *A.P. Savva, Chief Technical Advisor; J. Stoutjesdijk, Irrigation Engineer; P.M.A. Regnier, Irrigation Engineer; S.V. Hindkjaer, Economist.*

[2] *Agritex: Department of Agricultural Technical and Extension Services, Ministry of Lands and Agriculture, Zimbabwe.*

[3] *Review committee: E. Chidenga, Acting Chief Irrigation Officer; P. Chipadza, Senior Irrigation Specialist; A. Dube, Senior Irrigation Specialist; L. Forichi, Irrigation Specialist; L. Madhiri, Acting Principal Irrigation Officer; S. Madyiwa, Irrigation Specialist; P. Malusalila, Chief Crop Production; R. Mariga, Assistant Secretary, Economic and Markets Branch; D. Tawonezvi, Agricultural Economist.*

[4] *The following 21 countries are part of the FAO-SAFR region: Angola, Botswana, Burundi, Comoros, Eritrea, Ethiopia, Kenya, Lesotho, Madagascar, Malawi, Mauritius, Mozambique, Namibia, Rwanda, Seychelles, South Africa, Swaziland, Tanzania, Uganda, Zambia, Zimbabwe.*

For ease of reference to the various topics covered by this Manual, the material has been divided into 14 modules, covering the following:

Module 1: Irrigation development: a multifaceted process

Module 2: Natural resources assessment

Module 3: Agronomic aspects of irrigated crop production

Module 4: Crop water requirements and irrigation scheduling

Module 5: Irrigation pumping plant

Module 6: Guidelines for the preparation of technical drawings

Module 7: Surface irrigation systems: planning, design, operation and maintenance

Module 8: Sprinkler irrigation systems: planning, design, operation and maintenance

Module 9: Localized irrigation systems: planning, design, operation and maintenance

Module 10: Irrigation equipment for pressurized systems

Module 11: Financial and economic appraisal of irrigation projects

Module 12: Guidelines for the preparation of tender documents

Module 13: Construction of irrigation schemes

Module 14: Monitoring the technical and financial performance of an irrigation scheme

To those who have been waiting for so long for a practical irrigation engineering manual: here it is. I am sure that it will have a lot to offer to both the new and experienced irrigation engineers.

Victoria Sekitoleko
FAO Sub-Regional Representative
for East and Southern Africa

Irrigation Manual
Module 7

Surface Irrigation Systems

Planning, Design, Operation and Maintenance

Developed by

Andreas P. SAVVA
and
Karen FRENKEN

Water Resources Development and Management Officers
FAO Sub-Regional Office for East and Southern Africa

In collaboration with

Simon MADYIWA, Irrigation Engineer Consultant
Patrick CHIGURA, Irrigation Engineer Consultant
Lee TIRIVAMWE, National Irrigation Engineer, Zimbabwe
Victor MTHAMO, Irrigation Engineer Consultant

Harare, 2002

Acknowledgements

The preparation of the third edition of the Irrigation Manual is an initiative of FAO's Sub-Regional Office for East and Southern Africa (SAFR).

The whole project was managed and coordinated by Andreas P. Savva and Karen Frenken, Water Resources Development and Management Officers at FAO-SAFR, who are considered as the main authors. Karen Frenken is also the main technical editor.

The inputs by Simon Madyiwa, Patrick Chigura, Lee Tirivamwe and Victor Mthamo to this Module 7 are appreciated. The preparation of several drawings by Solomon Maina for this Module is acknowledged.

Special appreciation is extended to Chris Pappas for his substantial contribution to the layout of the irrigation manual.

Contents

Foreword	iii
Acknowledgements	vi
List of figures	x
List of tables	xiii
List of abbreviations	xv
Unit conversion table	xvii
1. INTRODUCTION TO SURFACE IRRIGATION	1
1.1. Components of a surface irrigation system	1
1.1.1. The water source	1
1.1.2. The intake facilities	1
1.1.3. The conveyance system	1
1.1.4. The water storage facilities	1
1.1.5. The field canal and/or pipe system	1
1.1.6. The infield water use system	3
1.1.7. The drainage system	3
1.1.8. Accessibility infrastructure	3
1.2. The four phases of surface irrigation	3
1.2.1. The advance phase	3
1.2.2. The storage or ponding phase	3
1.2.3. The depletion phase	4
1.2.4. The recession phase	4
1.3. Infiltration and contact time	4
1.3.1. Estimation of the infiltration rate using the infiltrometer method	5
1.3.2. Estimation of the infiltration rate using the actual furrow method	7
1.3.3. Determination of optimum stream size and furrow length	9
1.3.4. Determination of optimum stream size and borderstrip length	10
2. CRITERIA FOR THE SELECTION OF THE SURFACE IRRIGATION METHOD	13
2.1. Furrow irrigation	13
2.1.1. Furrow shape	13
2.1.2. Furrow spacing	15
2.1.3. Furrow length	15
2.2. Borderstrip irrigation	17
2.2.1. Borderstrip width	18
2.2.2. Longitudinal slope of the borderstrip	18
2.2.3. Borderstrip length	18
2.2.4. Guidelines for the determination of borderstrip width and length	19
2.3. Basin irrigation	20
2.3.1. Basin size	20
2.4. Efficiencies of surface irrigation systems and of the different surface irrigation methods	22
2.4.1. The different types of efficiencies in an irrigation system	22
2.4.2. Efficiencies of the different surface irrigation methods	23
2.5. Criteria for the selection of the surface irrigation method	24
2.5.1. Soil type	24
2.5.2. Type of crop	24
2.5.3. Required depth of irrigation application	24
2.5.4. Land slope	24

2.5.5. Field shape 24
2.5.6. Labour availability 24

3. DESIGN PARAMETERS FOR THE INFIELD WORKS 25
3.1. Crop and irrigation water requirements 25
3.2. Net and gross depth of water application 25
3.2.1. Net depth of water application 25
3.2.2. Gross depth of water application 26
3.3. Irrigation frequency and irrigation cycle 26
3.3.1. Irrigation frequency 26
3.3.2. Irrigation cycle 26
3.4. System capacity 27

4. LAYOUT OF A SURFACE IRRIGATION SCHEME 29
4.1. General layout 29
4.2. Nabusenga irrigation scheme layout 31
4.3. Mangui irrigation scheme layout 34

5. DESIGN OF CANALS AND PIPELINES 37
5.1. Design of canals 37
5.1.1. Calculation of the cross-section, perimeter and hydraulic radius of a canal 38
5.1.2. Factors affecting the canal discharge 38
5.1.3. Hydraulic design of canal networks using the chart of Manning formula 42
5.1.4. Canal section sizes used by Agritex in Zimbabwe 43
5.1.5. Longitudinal canal sections 45
5.1.6. Field canals for small irrigation schemes 49
5.1.7. Seepage losses in earthen canals 51
5.1.8. Canal lining 51
5.2. Design of pipelines 53
5.2.1. Design of the conveyance pipeline in Nabusenga irrigation scheme 54
5.2.2. Design of the piped system in Mangui irrigation scheme 54
5.2.3. Advantages and disadvantages of piped systems 60

6. HYDRAULIC STRUCTURES 61
6.1. Headworks for river water offtake 61
6.1.1. Headwork for direct river offtake 62
6.1.2. River offtake using a weir 63
6.1.3. River offtake using a dam 70
6.1.4. Scour gates for sedimentation control 71
6.2. Night storage reservoirs 73
6.2.1. Types of reservoirs 74
6.2.2. Reservoir components 75
6.3. Head regulators 77
6.4. Cross regulators 80
6.5. Drop structures and tail-end structures 80
6.5.1. Vertical drop structure 83
6.5.2. Chutes 85
6.5.3. Tail-end structures 86
6.6. Discharge measurement in canals 86
6.6.1. Discharge measurement equations 86
6.6.2. Weirs 89
6.6.3. Flumes 96

6.6.4. Orifices 107
6.6.5. Current meter 108
6.7. Discharge measurement in pipelines 110
6.7.1. Differential pressure flow meters 110
6.7.2. Rotating mechanical flow meters 110

7. LAND LEVELLING 111
7.1. Profile method 111
7.2. Contour method 111
7.3. Plane method 112
7.4. The cut : fill ratio 119
7.5. Use of computers 119

8. DESIGN OF THE DRAINAGE SYSTEM 123
8.1. Factors affecting drainage 123
8.1.1. Climate 123
8.1.2. Soil type and profile 123
8.1.3. Water quality 123
8.1.4. Irrigation practice 123
8.2. Determining hydraulic conductivity 124
8.3. Surface drainage 125
8.4. Subsurface drainage 127
8.4.1. Horizontal subsurface drainage 128
8.4.2. Vertical subsurface drainage 131
8.5. Salt problems 131

9. BILL OF QUANTITIES 133
9.1. Bill of quantities for Nabusenga irrigation scheme 133
9.1.1. The construction of a concrete-lined canal 133
9.1.2. The construction of a saddle bridge 135
9.1.3. The construction of a diversion structure 138
9.1.4. The overall bill of quantities for Nabusenga irrigation scheme 139
9.2. Bill of quantities for Mangui irrigation scheme 141

10. OPERATION AND MAINTENANCE OF SURFACE IRRIGATION SYSTEMS 143
10.1. Operation of the irrigation system 143
10.1.1. Water delivery to the canals 143
10.1.2. Water delivery to the fields 143
10.1.3. Operational success determinants 146
10.2. Maintenance of the irrigation system 147
10.2.1. Special maintenance 147
10.2.2. Deferred maintenance 147
10.2.3. Routine maintenance 147
10.3. Operation and maintenance responsibilities 148

REFERENCES 149

List of figures

1.	Typical components of a surface irrigation system	2
2.	Definition sketch showing the surface irrigation phases	3
3.	Basic infiltration rate and cumulative infiltration curves	4
4.	Cylinder infiltrometers	5
5.	Analysis of the data of an infiltration test using an infiltrometer on a clay loam soil	6
6.	Analysis of the data of an infiltration test using actual furrows on a clay loam soil	8
7.	Time-advance graph for various stream flows in a furrow	9
8.	Determining the head	10
9.	Advance and recession of water on a borderstrip	10
10.	Advance and recession curves for different borderstrip length needing different total volumes of water to be applied	12
11.	An example of a furrow irrigation system using siphons	13
12.	Furrow shape depending on soil type	14
13.	Soil moisture distribution on various soil types as a determinant of furrow spacing	14
14.	Example of a borderstrip irrigation system	17
15.	Cross-section of a borderstrip	18
16.	Layout of basin irrigation	21
17.	Typical layout of a surface irrigation scheme on uniform flat topography	30
18.	The herringbone irrigation layout	31
19.	Layout of Nabusenga surface irrigation scheme	32
20.	Layout of Mangui piped surface irrigation scheme	33
21.	Cumulative depth of irrigation versus time for different types of soil	34
22.	Plot layout and hydrants	36
23.	Flowchart for canal design calculations	37
24.	Canal parameters	38
25.	Different canal cross-sections	39
26.	Hydraulic parameters for different canal shapes	41
27.	Chart of Manning formula for trapezoidal canal cross-sections	43
28.	Longitudinal profile of a field or tertiary canal	46
29.	Longitudinal profile of a secondary or main canal	47
30.	Longitudinal profile of a conveyance canal	48
31.	Example of a longitudinal profile of a conveyance canal	49
32.	Longitudinal canal profile generated by the Lonsec Programme	49
33.	Methods commonly used to introduce water into the field	53
34.	The longitudinal profile of the conveyance pipeline from Nabusenga dam to the night storage reservoir	55
35.	Friction loss chart for AC pipes (Class 18)	56
36.	Friction loss chart for uPVC pipes	57
37.	Schemes irrigated from different water sources	61
38.	Headwork with offtake structure only	62
39.	Offtake possibilities in straight reach of river	62

40. Possible arrangements for offtakes based on site conditions 62
41. An example of an intake arrangement of a headwork 63
42. An example of a diversion structure 64
43. C_1 coefficient for different types of weirs in relation to submergence, based on crest shape 64
44. C_2 coefficient for different types of weirs in relation to crest shape 65
45. Types of weirs 65
46. Gabion weir 66
47. Typical parameters used in the design of a stilling basin 67
48. Schematic view of a weir and apron 68
49. Masonry weir and apron 70
50. Dam cross-section at Nabusenga 70
51. Gravity offtake with diversion dam 72
52. Scour sluice 72
53. Design of a typical earthen night storage reservoir 75
54. Courses in brick wall of a reservoir 76
55. A simple in-situ concrete proportional flow division structure 77
56. Precast concrete block division box 78
57. Timber division structures 79
58. Duckbill weir photograph 80
59. Duckbill weir design 81
60. Diagonal weir 82
61. Some drop structures used in open canals 83
62. Standard drop structure without stilling basin 84
63. A vertical drop structure 85
64. A chute structure 86
65. Static and velocity heads 87
66. Variation of specific energy with depth of flow for different canal shapes 88
67. Hydraulic jump over a concrete apron 89
68. The form of a hydraulic jump postulated in the momentum theory 89
69. Parameters of a sharp-crested weir 90
70. Trapezoidal (Cipoletti) weir 92
71. V-notch weirs 93
72. Broad-crested weir 94
73. Romijn broad-crested weir, hydraulic dimensions of weir abutments 95
74. Romijn broad-crested weir, sliding blades and movable weir crest 95
75. Approach velocity coefficient, C_v, as a function of the total head over the movable weir, H_{crt} 96
76. Parshall flume 97
77. Discharge correction factors for Parshall flumes with different throat widths 100-2
78. Head loss through Parshall flumes 103
79. Trapezoidal flume 105
80. Cut-throat flume 106
81. Cut-throat flume coefficients 106
82. Examples of orifices 107

83. Free discharging flow through an orifice 107
84. Sluice gate under submerged conditions 108
85. Ott C31 propeller instrument 109
86. Depth-velocity integration method 109
87. Venturi flow meter 110
88. The profile method of land levelling: cut and fill and checking gradient levels with profile boards 111
89. The contour method of land levelling 112
90. Grid map showing land elevation and average profile figures 113
91. Average profile and lines of best fit 116
92. Part of the completed land levelling map for Nabusenga, assuming $G_X = 0.005$ 118
93. Irregular shaped field (elevations 0.0 are located outside the field) 122
94. Parameters for determining hydraulic conductivity 124
95. Cross-sections of drains 125
96. Rainfall-duration curve 126
97. Subsurface drainage systems at field level 127
98. Subsurface drainage parameters 128
99. Nomograph for the determination of equivalent sub-stratum depths 130
100. Nomograph for the solution of the Hooghoudt drain spacing formula 131
101. Salt accumulation in the root zone and the accompanying capillary rise 132
102. Cross-section of a concrete lined canal at Nabusenga 133
103. Saddle bridge for Nabusenga 136
104. Field canal bank breaching in order to allow the water to flow from the canal onto the field 143
105. Permanent outlet structure used to supply water from the canal onto the field 144
106. An example of a spile used to supply water from the canal onto the field 145
107. A siphon supplying water from a canal onto a field 146

List of tables

1. Typical infiltration rates for different soils 4
2. Infiltration rate data from an infiltrometer test 6
3. Infiltration rate measurement in a 100 m long furrow 8
4. Discharge for siphons, depending on pipe diameter and head 10
5. Guidelines to determine when to stop the water supply onto a borderstrip 10
6. Measurement of water advance and recession distance and time on a borderstrip 11
7. Furrow lengths in metres as related to soil type, slope, stream size and irrigation depth 16
8. Practical values of maximum furrow lengths in metres depending on soil type, slope, stream size and irrigation depth for small-scale irrigation 17
9. Typical borderstrip dimensions in metres as related to soil type, slope, irrigation depth and stream size 19
10. Suggested maximum borderstrip widths and lengths for smallholder irrigation schemes 20
11. Criteria for basin size determination 20
12. Basin area in m^2 for different stream sizes and soil types 21
13. Approximate values for the maximum basin width 22
14. Selection of an irrigation method based on soil type and net irrigation depth 24
15. Design parameters for Nabusenga and Mangui surface irrigation schemes 25
16. Summary of the calculated design parameters for Nabusenga and Mangui surface irrigation schemes 28
17. K_m and n values for different types of canal surface 40
18. Typical canal side slopes 40
19. Recommended b/d ratios 41
20. Maximum water velocity ranges for earthen canals on different types of soil 41
21. Canal capacities for standard Agritex canal sections 44
22. Longitudinal profile for field canal - output from the Lonsec computer programme 50
23. Seepage losses for different soil types 51
24. Hazen-Williams C value for different materials 54
25. Weighted-creep ratios for weirs depending on soil type 68
26. Reinforcement requirements in a clay brick wall of a reservoir 76
27. Cross-sectional areas of reinforcement steel rods 77
28. Discharge Q (m^3/sec) for contracted rectangular weir, depending on h and b 91
29. Discharge Q (m^3/sec) for Cipoletti weir, depending on h and b 92
30. Discharge Q (m^3/sec x 10) for a 90° V-notch weir, depending on h 94
31. Standard dimensions of Parshall flumes 98
32. Discharge characteristics of Parshall flumes 99
33. Land levelling results 117
34. Input and output data types for computer land levelling programme LEVEL 4EM.EXE 119
35. Land levelling calculations with line of best fit and cut:fill ratio of 1.01 120
36. Land levelling calculations with 0.5% gradient in the X direction and cut:fill ratio of 1.01 120
37. Land levelling calculations with line of best fit and cut:fill ratio of 1.21 121
38. Computer printout of land levelling data for Mangui piped surface irrigation scheme 121-2
39. Values for runoff coefficient C in Equation 70 126

40. Concrete volume for different trapezoidal canal cross-sections 134
41. Summary of the bill of quantities for the construction of the 980 m long lined canal at Nabusenga 135
42. Summary of the bill of quantities for the construction of a saddle bridge 137
43. Summary of the bill of quantities for the construction of a diversion structure 138
44. Bill of quantities for Nabusenga scheme, downstream of the night storage reservoir 139
45. Summary of material requirements for Nabusenga (including 10% contingencies) 141
46. Bill of quantities for pipes and fittings and pumping plant at Mangui scheme 142
47. Discharge of permanent wooden field outlet structures 144
48. Rates of discharge through spiles (l/sec) 145
49. Discharge of siphons for different head and pipe diameter (l/sec) 146
50. Weed management and effectiveness 148

List of abbreviations

A	Area
AC	Asbestos Cement
ASAE	American Society of Agricultural Engineers
C	Cut
CI	Cast Iron
γ	Density of water
D or d	Diameter
d	Water depth
d_{gross}	Gross depth of water application
d_{net}	Net depth of water application
E	Efficiency
EL	Elevation
F	Freeboard
F	Fill
FC	Field Capacity
Fr	Froude Number
g	Acceleration due to gravity
G	Regression coefficient
GS	Galvanized Steel
h	water depth
H	Head
Hf_{100}	Friction losses per 100 m of pipe
HL	Head Loss
IC	Irrigation Cycle
IF	Irrigation Frequency
IT	Irrigation Time
K_m	Manning roughness coefficient
kPa	Kilopascal
kW	kilowatt
L	Length
n	Roughness coefficient ($=1/K_m$)
NSR	Night Storage Reservoir
P	Allowable moisture depletion
P	Wetted Perimeter
P	Pressure
PWP	Permanent Wilting Point
q	Discharge into one furrow or discharge per m width
Q	Discharge
R	Hydraulic radius

R	Cut : Fill ratio
RZD	Effective Root Zone Depth
S	Slope or gradient
T	Irrigation time
TDH	Total Dynamic Head
uPVC	unplasticized Polyvinyl Chloride
V	Volume
V or v	Water velocity
z	Elevation

Unit conversion table

Length

1 inch (in)	0.0254 m
1 foot (ft)	0.3048 m
1 yard (yd)	0.9144 m
1 mile	1609.344 m
1 metre (m)	39.37 inches (in)
1 metre (m)	3.28 feet (ft)
1 metre (m)	1.094 yards (yd)
1 kilometre (km)	0.62 miles

Area

1 square inch (in^2)	6.4516×10^{-2} m^2
1 square foot (ft^2)	0.0929 m^2
1 square yard (yd^2)	0.8361 m^2
1 acre	4046.86 m^2
1 acre	0.4046 ha
1 square centimetre (cm^2)	0.155 square inches (in^2)
1 square metre (m^2)	10.76 square feet (ft^2)
1 square metre (m^2)	1.196 square yard (yd^2)
1 square metre (m^2)	0.00024 acres
1 hectare (ha)	2.47 acres

Volume

1 cubic inch (in^3)	1.6387×10^{-5} m^3
1 cubic foot (ft^3)	0.0283 m^3
1 cubic yard (yd^3)	0.7646 m^3
1 cubic centimetre (cm^3)	0.061 cubic inches (in^3)
1 cubic metre (m^3)	35.315 cubic feet (ft^3)
1 cubic metre (m^3)	1.308 cubic yards (yd^3)

Capacity

1. imperial gallon	0.0045 m^3
1. US gallon	0.0037 m^3
1. imperial barrel	0.1639 m^3
1. US. barrel	0.1190 m^3
1 pint	0.5681 l
1 US gallon (dry)	0.0044 m^3
1 litre (l)	0.22 imp. gallon
1 litre (l)	0.264 U.S. gallon
1 litre (l)	0.0061 imperial barrel
1 hectolitre (hl)	100 litres = 0.61 imperial barrel = 0.84 US barrel
1 litre (l)	1.760 pints
1 cubic metre of water (m^3)	1000 l = 227 U.S. gallon (dry)
1 imperial barrel	164 litres

Mass

1 ounce	28.3286 g
1 pound	0.4535 kg
1 long ton	1016.05 kg
1 short ton	907.185 kg
1 gram (g)	0.0353 ounces (oz)
1 kilogram (kg)	1000 g = 2.20462 pounds
1 ton	1000 kg = 0.984 long ton = 1.102 short ton

Pressure

1 pound force/in^2	6894.76 N/m^2
1 pound force/in^2	51.7 mm Hg
1 Pascal (PA)	1 N/m^2 = 0.000145 pound force /in^2
1 atmosphere	760 mm Hg = 14.7 pound force/in^2 (lbf/in^2)
1 atmosphere	1 bar
1 bar	10 metres
1 bar	100 kpa

Energy

1 B.t.u.	1055.966 J
1 foot pound-force	1.3559 J
1 B.t.u.	0.25188 Kcalorie
1 B.t.u.	0.0002930 KWh
1 Joule (J)	0.000947 B.t.u.
1 Joule (J)	0.7375 foot pound-force (ft.lbf)
1 kilocalorie (Kcal)	4185.5 J = 3.97 B.t.u.
1 kilowatte-hour (kWh)	3600000 J = 3412 B.t.u.

Power

1 Joule/sec	0.7376 foot pound/sec
1 foot pound/sec	1.3557 watt
1 cheval-vapor	0.9861 hp
1 Kcal/h	0.001162 kW
1 watt (W)	1 Joule/sec = 0.7376 foot pound/sec (ft lbf/s)
1 horsepower (hp)	745.7 watt 550 ft lbf/s
1 horsepower (hp)	1.014 cheval-vapor (ch)
1 kilowatt (kW)	860 Kcal/h = 1.34 horsepower

Temperature

^{0}C (Celsius or centigrade-degree)	$^0C = 5/9 \times (^0F - 32)$
^{0}F (Fahrenheit degree)	$^0F = 1.8 \times {}^0C + {}^0F$
K (Kelvin)	$K = {}^0C + 273.15$

Chapter 1
Introduction to surface irrigation

Surface irrigation is the oldest and most common method of applying water to crops. It involves moving water over the soil in order to wet it completely or partially. The water flows over or ponds on the soil surface and gradually infiltrates to the desired depth. Surface irrigation methods are best suited to soils with low to moderate infiltration capacities and to lands with relatively uniform terrain with slopes less than 2-3% (FAO, 1974).

1.1. Components of a surface irrigation system

Figure 1 presents the components of a surface irrigation system and possible structures, which are described in Chapter 6. The water delivery system, shown in Figure 1, includes the conveyance system and the field canal system described below. The water use system refers to the infield water use system, showing one field in the block. The tail water ditch and the water removal system are part of the drainage system.

1.1.1. The water source

The source of water can be surface water or groundwater. Water can be abstracted from a river, lake, reservoir, borehole, well, spring, etc.

1.1.2. The intake facilities

The intake is the point where the water enters into the conveyance system of the irrigation scheme. Water may reach this point by gravity or through pumping. Intake facilities are dealt with during the design of headworks in Chapter 6. Pumping units are discussed in detail in Module 5.

1.1.3. The conveyance system

Water can be conveyed from the headworks to the inlet of a night storage reservoir or a block of fields either by gravity, through open canals or pipes, or through pumping into pipelines. The method of conveyance depends mostly on the terrain (topography and soil type) and on the difference in elevation between the intake at the headworks and the irrigation scheme. In order to be able to command the intended area, the conveyance system should discharge its water at the highest point of the scheme. The water level in the conveyance canal itself does not need to be above ground level all along the canal, but its starting bed level should be such that there is sufficient command for the lower order canals. Where possible, it could run quasi-parallel to the contour line. Design aspects of canals and pipelines are discussed in Chapter 5.

Although an open conveyance canal may be cheaper per unit length than a pipeline, the latter would need to be selected when:

- ❖ The water source is at lower elevation than the irrigation area, and thus pumping is required
- ❖ The topography of the land is very uneven, such that constructing an open canal could either be more expensive or even impossible (for example when crossing rivers and gullies)

A piped conveyance system also eliminates water losses through evaporation and seepage. An added advantage is that it does not provide the environment for water-borne disease vectors along the conveyance.

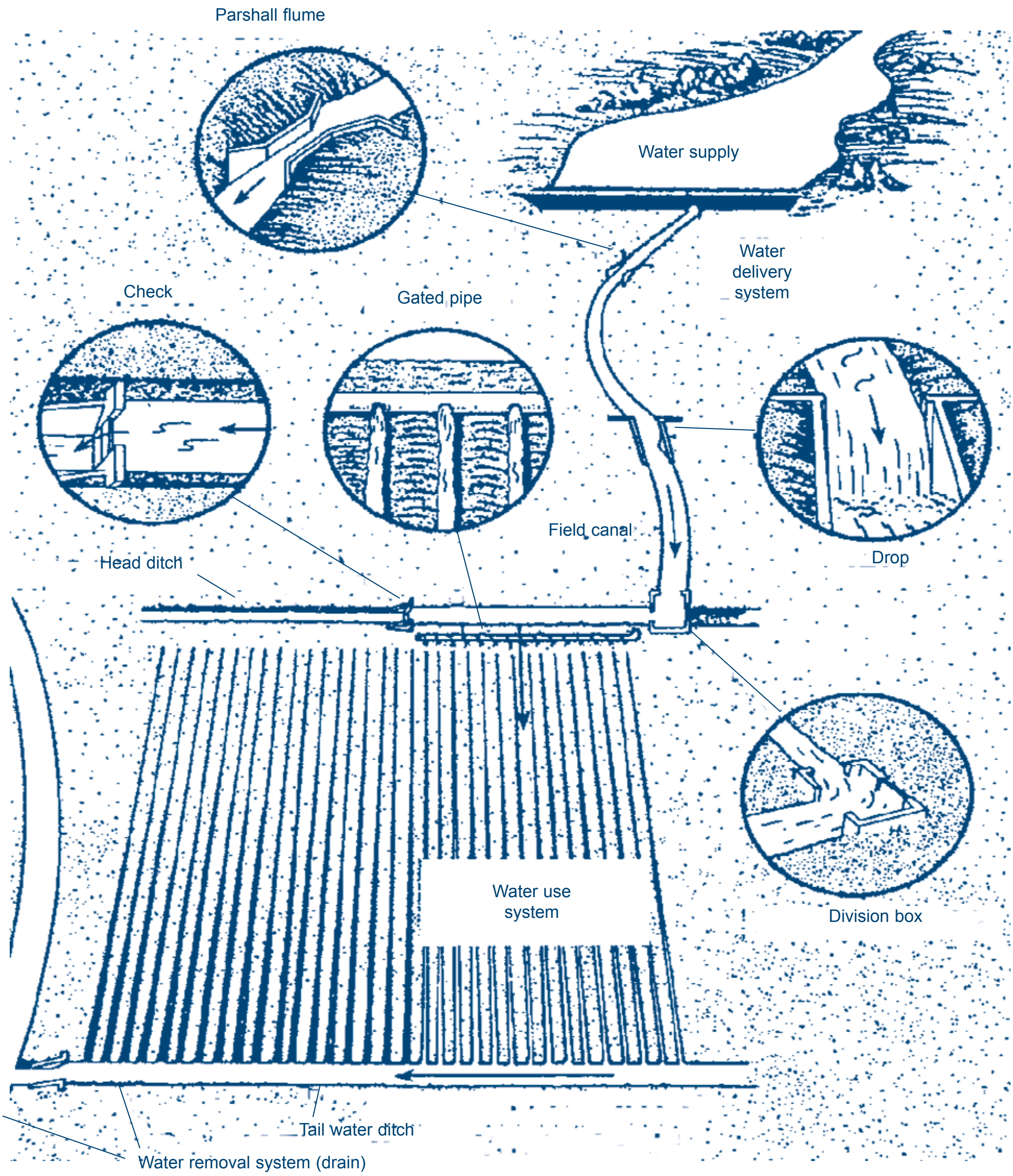
1.1.4. The water storage facilities

Night storage reservoirs (NSR) could be built if the irrigation scheme is large enough to warrant such structures. They store water during times when there is abstraction from the water source, but no irrigation. In Southern Africa it is common practice to have continuous flow in the conveyance system combined with a NSR located at the highest point of a block or the scheme. Irrigation would then be practiced during daytime using the combined flow from the conveyance system and the NSR. Depending on the size of the scheme one could construct either one reservoir located at the highest part of the scheme or a number of reservoirs, each located at the entrance of a block of fields. The conveyance system ends at the point where the water enters the reservoir.

1.1.5. The field canal and/or pipe system

Canals or pipelines are needed to carry the water from the conveyance canal or the NSR to a block of fields. They are called the main canal or pipeline. Secondary canals or pipelines supply water from the main canal or pipeline to the tertiary or field canals or pipelines, which

Figure 1
Typical components of a surface irrigation system (Source: Walker and Skogerboe, 1987)



are located next to the field. Sometimes no distinction is made between main and secondary and the canal or pipe system from the reservoir to the tertiary canal is called main canal or pipeline. The tertiary canals or the pipelines with hydrants are used to supply water to the furrows or borderstrips or basins. Where canals are used to deliver irrigation water, they should be constructed above ground level, as the water level in canals should be above field level for siphoning to take place. At times, water from the field canal is siphoned to a field earthen ditch from where the furrows, borderstrips and basins are supplied. When a piped distribution system is used, the gated pipe is connected to the hydrant and water is provided to the field from the gates of the gated pipe. Alternatively, a hose is connected to the hydrant to supply water to the field.

1.1.6. The infield water use system

This refers mainly to the method of water application to the field, which can be furrow, borderstrip or basin irrigation. These methods are described in detail in Chapter 2. It is important to note that the method of conveyance and distribution up to field level is independent of the selected infield irrigation method.

In irrigation system design, the starting point is the infield water use system as this provides information on the surface irrigation method to use, the amount of water to be applied to the field and how often it has to be applied. With this information, we can then work backwards or upstream to designing the field canal, distribution, storage, conveyance system and ultimately the intake facilities, and we can work forwards or downstream to determine the capacity of the drainage facilities.

1.1.7. The drainage system

This is the system that removes excess water from the irrigated lands. The water level in the drains should be below the field level and hence field drains should be constructed at the lower end of each field. These field or tertiary drains would then be connected to secondary drains and then the main drain, from where excess water is removed from the irrigation scheme.

1.1.8. Accessibility infrastructure

The scheme is to be made accessible through the construction of main roads leading to the scheme, and farm roads within the scheme.

1.2. The four phases of surface irrigation

When water is applied to the soil surface by any of the three surface irrigation methods (furrow, borderstrip or basin), it will infiltrate into the soil to the required depth in order to bring the soil back to field capacity. Using the borderstrip and basin irrigation method, the entire soil surface is wetted and the water movement through the soil is predominantly vertical. Using the furrow irrigation method, part of the soil surface is wetted and the water movement through the soil is both vertical and lateral.

The surface irrigation event is composed of four phases, as illustrated in Figure 2 and explained below.

1.2.1. The advance phase

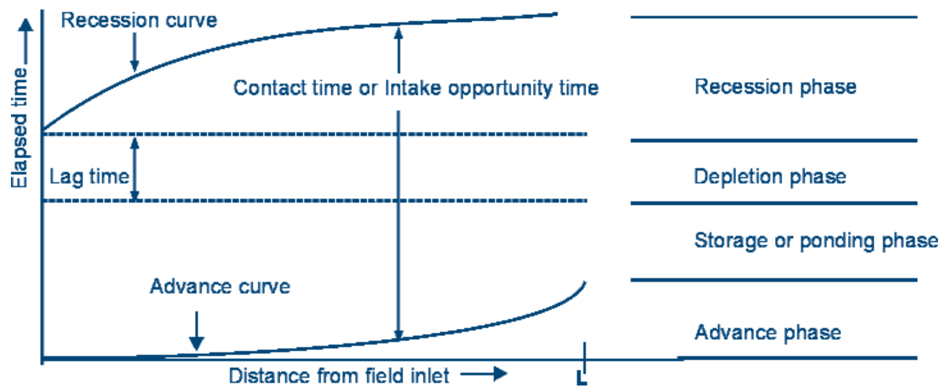
The advance phase begins when water is applied onto the field at the upstream end and ends when it reaches the downstream end of the field. The stream size applied at the head of the furrow, borderstrip and basin should be greater than the soil infiltration rate. This means that part of the water advances over the soil surface to the end of the field and part of the water infiltrates into the soil. The time between the start of irrigation and water advancement to the end of the field is called the *advance phase*. The advance curve in Figure 2 is the line showing the relationship between the elapsed time (on y-axis) and the advance distance (on x-axis).

1.2.2. The storage or ponding phase

When the water arrives at the tail end and the water supply at the head is continued, water floods the whole field. Some water continues infiltrating into the soil, some water ponds on the field and some excess water is collected as runoff. The time elapsed between the arrival of the water at the tail end and the stopping of the inflow at the top end is called the *storage phase* or *ponding phase*. This phase ends when the inflow at the head of the field is stopped.

Figure 2

Definition sketch showing the surface irrigation phases (Source: Basset *et al.*, 1980)



1.2.3. The depletion phase

After stopping the inflow at the head end, water may continue to pond on the soil surface for a while. Some water still infiltrates the soil, with the excess being collected as runoff. At a certain moment water will start receding from the head end. The time between the stop of the inflow at the head end and the appearance of the first bare soil that was under water is called the lag time or *depletion phase*.

1.2.4. The recession phase

After water starts receding from the head end, it continues to the tail end. The time when water starts to disappear at the head end until it eventually recedes from the whole field is called the *recession phase*.

The time-difference between the recession and advance curve is called the *contact time* or the *intake opportunity time*. This is the time in hours or minutes that any particular point in the field is in contact with water. Thus, by increasing or decreasing the contact time, one can, within limits, regulate the depth of water applied.

The following three basic principles are fundamental for surface irrigation, though the possibility of applying them depends a lot on the soil type:

i) The depth of infiltration varies in relation to contact time

ii) The contact time can be increased by using flatter slopes, increasing the length of run or reducing the stream flow; any one or a combination of these factors may be used

iii) The contact time can be decreased by steepening the slope, shortening the length of run or increasing the stream flow

1.3. Infiltration and contact time

Infiltration, which is the movement of water into the soil, is an important factor affecting surface irrigation in that it determines the time the soil should be in contact with water (the intake opportunity time or the contact time). It also determines the rate at which water has to be applied to the fields, thereby controlling the advance rate of the overland flow and avoiding excessive deep percolation or excessive runoff. The infiltration or intake rate is defined as the rate at which water enters into the soil, usually expressed in mm/hr.

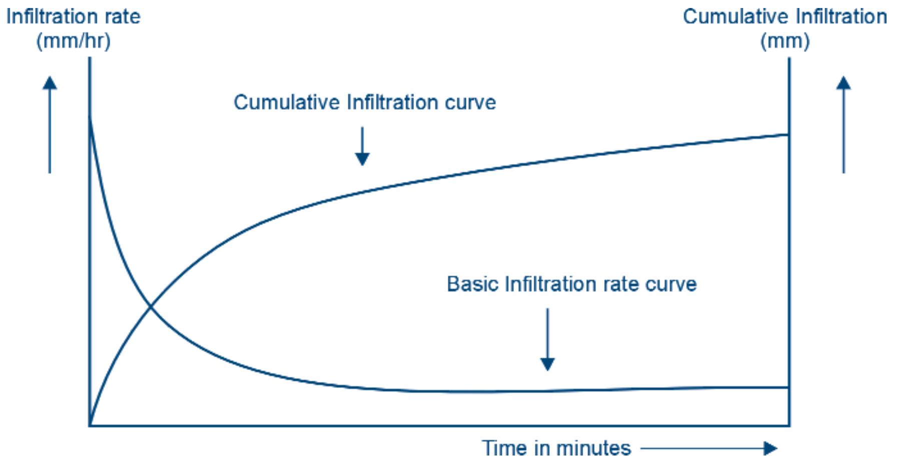
No matter where water infiltrates rapidly when it first arrives, after which it slows down until it reaches a steady state. This steady state is referred to as the basic infiltration rate, which is close to the value of the saturated hydraulic conductivity. When the basic infiltration rate is reached, the cumulative infiltration curve becomes a straight line and the basic infiltration rate curve becomes a horizontal line. This phenomenon is shown using a graph in Figure 3.

The infiltration rates of soils are influenced, among others, by the soil texture. Heavy soils have low infiltration rates by virtue of their small pore sizes, while light soils have high infiltration rates because of larger pore sizes. Some typical infiltration rates for different soil types are given in Table 1.

Table 1
Typical infiltration rates for different soils

Soil Type	Infiltration rate mm/hr
Sand	> 30
Sandy Loam	30-20
Silty Loam	20-10
Clay Loam	10-5
Clay	< 5

Figure 3
Basic infiltration rate and cumulative infiltration curves



The infiltration rate is a difficult parameter to define accurately, but it has to be determined in order to describe the hydraulics of the surface irrigation event. When planning a furrow irrigation scheme, one can determine the infiltration rate by two methods: the infiltrometer method and the actual furrow method. The former method can also be used to determine the infiltration rate for borderstrip and basin irrigation schemes.

1.3.1. Estimation of the infiltration rate using the infiltrometer method

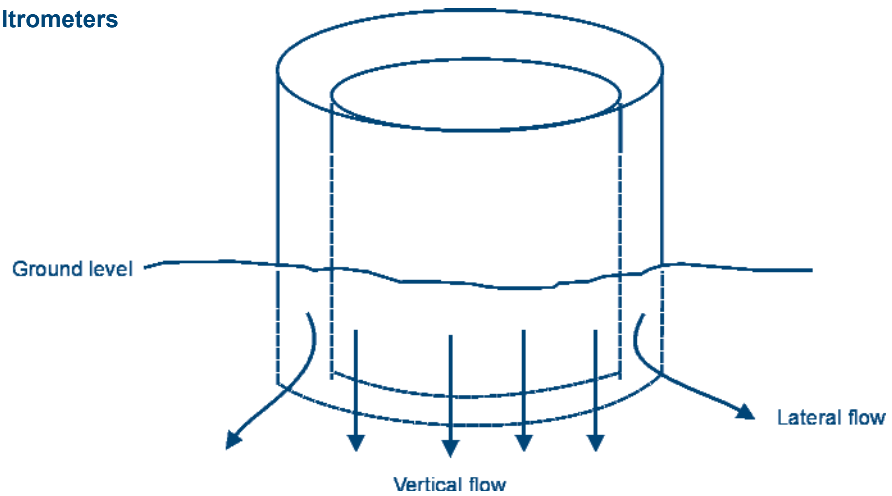
With this method, infiltration is measured by observing the fall of water within the inner cylinder of two concentric cylinders, with a usual diameter of 0.4 and 0.5 m and a height of about 0.4 m, driven vertically into the soil surface layer as illustrated in Figure 4. The outer ring acts as a buffer preventing lateral seepage of water from the inner one. This allows infiltration measurement from the inner ring to be representative of infiltration from the actual irrigation of a large area.

The procedure for installing the infiltrometer and for taking measurements is as follows:

- Select possible locations for three to four infiltrometers spread over the irrigation scheme and examine the sites carefully for signs of unusual surface disturbance, animal burrows, stones and so on, as they may affect the test results
- Drive the cylinder into the soil to a depth of approximately 15 cm by placing a driving plate over the cylinder, or placing heavy timber on top, and using a driving hammer. Rotate the timber every few pushes or move the hammer equally over the surface in order to obtain a uniform and vertical penetration
- Fix a gauge (almost any type) to the inner wall of the inner cylinder so that the changes in water level can be measured
- Fill the outer ring with water to a depth approximately the same as will be used in the inner ring and also quickly add water to the inner cylinder till it reaches 10 cm or 100 mm on the gauge
- Record the clock time immediately when the test begins and note the water level on the measuring rod
- The initial infiltration will be high and therefore regular readings at short intervals should be made in the beginning, for example every minute, after which they can increase to 1, 2, 5, 10, 20, 30 and 45 minutes, for example. The observation frequencies should be adjusted to infiltration rates
- After a certain period infiltration becomes more or less constant (horizontal line in Figure 3). Then the basic infiltration rate is reached. After reading equal water lowering at equal intervals for about 1 or 2 hours, the test can stop.
- The infiltration during any time period can be calculated by subtracting the water level measurement before filling at the end of the period from the one after filling at the beginning of that same period. For example, the infiltration between 09.35 hr and 09.45 hr in Table 2 is 100 - 93 = 7 mm, which is 0.7 mm/min or 0.7 x 60 = 42 mm/hr
- After the tests the cylinders should be washed before they become encrusted. This makes them easy to drive into the soil, with minimal soil disturbance, next time they are to be used

If the actual moisture conditions in the soil at the start of the infiltration test are low, it will take longer to reach the basic

Figure 4
Cylinder infiltrometers



infiltration rate compared to the same soil wherein the moisture is only slightly depleted. Preferably, tests should be carried out at the expected depletion level during irrigation.

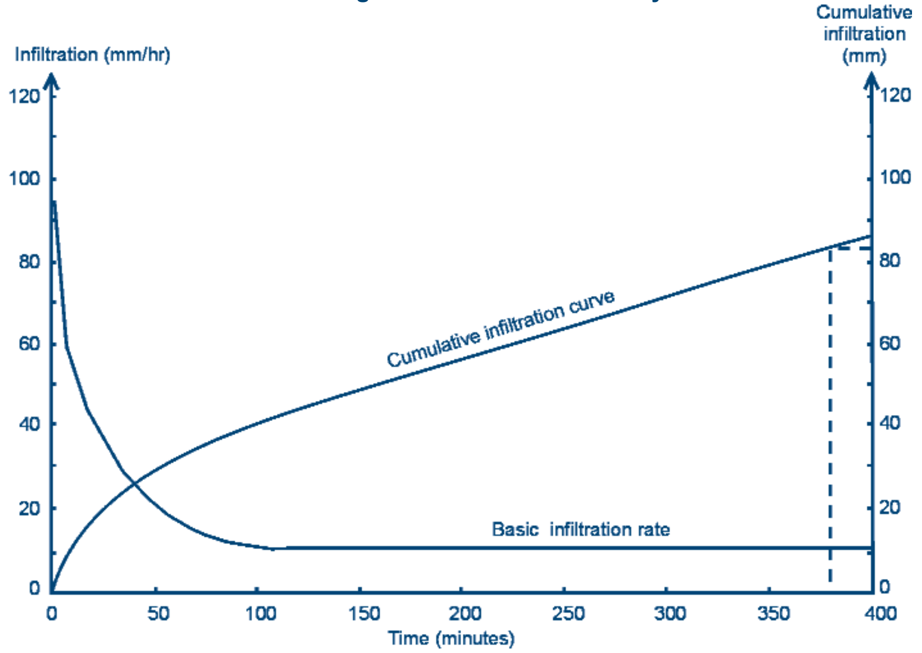
Results of an infiltrometer test on a clay loam soil are given in Table 2.

Table 2
Infiltration rate data from an infiltrometer test

Site location: Nabusenga **Soil type: Clay Loam** **Test date: 5 October 1990**

Watch reading (hr:min)	Time interval (min)	Cumulative time (min)	Water level reading before filling (mm)	Water level reading after filling (mm)	Infiltration (mm)	Infiltration rate (mm/min)	Infiltration rate (mm/hr)	Cumulative infiltration (mm)
	Start = 0							
09:25		0		100				Start = 0
	2				3.0	1.50	90.0	
09:27		2	97.0	100				3.0
	3				3.5	1.17	70.0	
09:30		5	96.5	101				6.5
	5				4.5	0.90	54.0	
09:35		10	96.5	100				11.0
	10				7.0	0.70	42.0	
09:45		20	93.0	99				18.0
	10				5.5	0.55	33.0	
09:55		30	93.5	100				23.5
	20				7.0	0.35	21.0	
10:15		50	93.0	100				30.5
	20				5.0	0.25	15.0	
10:35		70	95.0	100				35.5
	40				7.0	0.175	10.5	
11:15		110	93.0	101				42.5
	40				7.0	0.175	10.5	
11:55		150	94.0	100				49.5
	40				7.0	0.175	10.5	
12:35		190	93.0					56.5

Figure 5
Analysis of the data of an infiltration test using an infiltrometer on a clay loam soil



Example 1

The net peak crop water requirement for a scheme is 6.0 mm/day. The available moisture for the clay loam is 130 mm/m and depletion is allowed up to around 46%. The root zone depth is 0.70 m. After how many days should irrigation take place to replenish the soil moisture?

The moisture available to the crop in the root zone is 130 x 0.70 x 0.46 = 42 mm. The peak water requirement being 6.0 mm/day, after 42/6 = 7 days irrigation should take place to replenish the 42 mm soil moisture. This is equal to the net irrigation requirement.

Example 2

Assuming a field application efficiency of 50% in the previous example, what is the time required to replenish the 42 mm soil moisture?

The net irrigation requirement being 42 mm and considering a field application efficiency of 50%, the gross irrigation requirement is 42/0.50 = 84 mm. From the cumulative curve in Figure 5 it can be seen that the time required to replenish this depth of water is approximately 384 minutes.

If the field application efficiency increased to 65%, due to improved water management, the gross irrigation requirement would be 42/0.65 = 64.6 mm and the time required to replenish this depth would be reduced to 260 minutes.

It can be seen from Table 2, that the steady state has been reached somewhere between 70 and 110 minutes after the start of the test. From that moment on, the basic infiltration rate curve (Figure 3) will be a horizontal line and the cumulative infiltration curve will be a straight line. The results of the above test are graphically presented in Figure 5.

Examples 1 and 2 demonstrate the use of the intake rate in estimating the time required to replenish the soil moisture during irrigation.

The time required in Example 2 to replenish the required depth of water is called the contact time, which is the time the water should be in contact with the soil in order to have the correct depth of water replenished in the soil.

1.3.2. Estimation of the infiltration rate using the actual furrow method

With furrows, the infiltration rate and cumulative infiltration curve can also be determined as follows. Three adjacent furrows of a specific length, for example 30 m or 100 m, are wetted at the same time. Two measuring devices, such as for example portable Parshall flumes, are placed at the beginning and end of the middle furrow respectively, and the inflow at the top end and outflow at the tail end are measured simultaneously. The outer furrows function in the same way as the outer ring of the infiltrometer by preventing excessive lateral flow from the middle furrow. The infiltration in l/min (volume/time) can be converted into an infiltration in mm/hr (depth/time) by dividing it by the area covered by furrow. Table 3 and Figure 6 show the result for the same clay loam soil.

In this example, the furrow test starts at 14.00 hr with a continuous uniform flow of 98 l/min (1.63 l/sec) being discharged into the furrow. At the first recording of the inflow at the top, there is no outflow at the bottom end of the furrow. In this example, the outflow starts at 14.05 hr. From a recording of zero outflow there is a sudden outflow of 17 l/min (0.28 l/sec).

While the inflow remains constant, the outflow increases with time until the basic infiltration rate is reached at 15.10 hr. The infiltration is the difference between the inflow and the average of the outflow during a given time period. For example, between 14.20 hr and 14.30 hr, the inflow is 98 l/min and the average outflow is 56.7 l/min (={46 + 67.4}/2). Thus, the average infiltration rate over this period is 98 - 56.7 = 41.3 l/min. The average infiltration per period is calculated by multiplying the infiltration per minute by the time period. The sum of the infiltrations gives the cumulative infiltration.

The contact time can be determined from the cumulative infiltration curve, as shown in Figure 6. Considering the same data as used in Example 1 and 2, with a gross irrigation requirement of 84 mm, and considering a furrow length of 100 m and a furrow spacing of 0.75 m, the volume of water required per furrow is:

$$V = 0.084 \times 100 \times 0.75 = 6.300 \text{ m}^3 = 6\,300 \text{ litres}$$

Figure 6 again gives a contact time of 384 minutes (x-axis) for this volume of 6 300 litres on the right-hand y-axis (cumulative infiltration).

Table 3

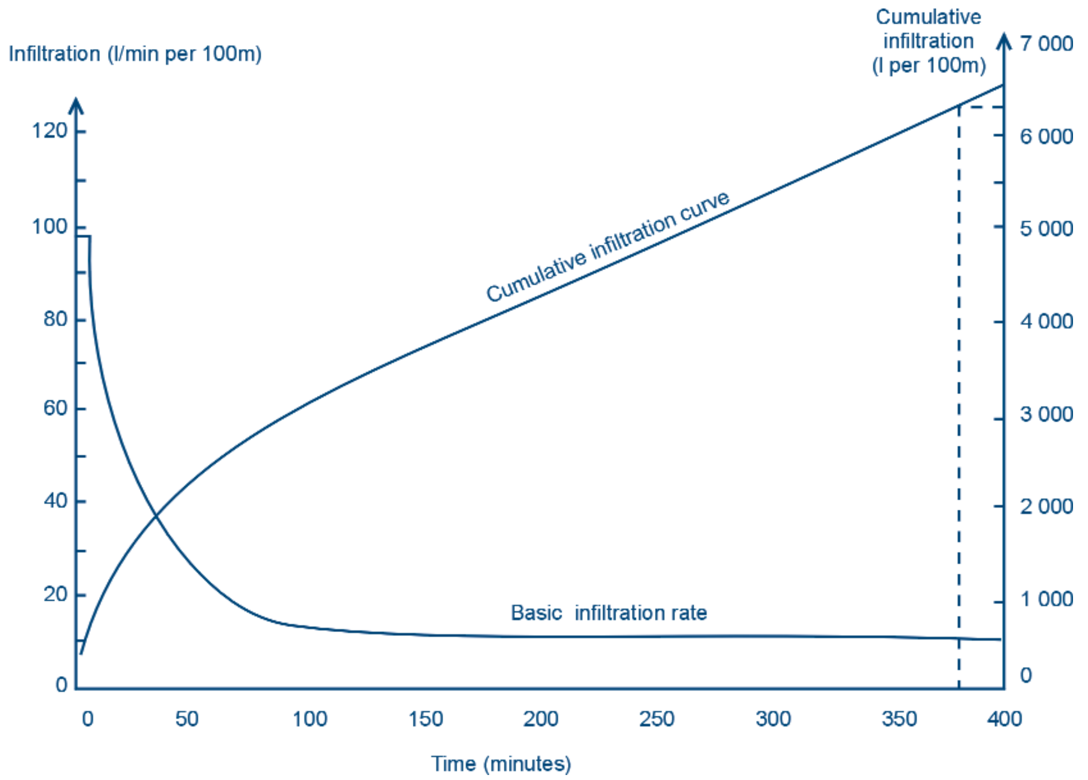
Infiltration rate measurement in a 100 m long furrow

Site location: Nabusenga | **Soil type: Clay Loam** | **Test date: 5 October 1990**

Watch reading (hr:min)	Time interval (min)	Cumulative time (min)	Water measurements Inflow (l/min)	Water measurements Outflow (l/min)	Intake rate l/min over 100 m	Intake (l/100 m)	Cumulative intake (l/100 m)
	Start = 0						
14:00		0	98.0	0			0
	5				98.00	490	
14:05		5	98.0	0 to 17			490
	5				67.00	335	
14:10		10	98.0	45.00			825
	10				52.50	525	
14:20		20	98.0	46.00			1 350
	10				41.30	413	
14:30		30	98.0	67.40			1 763
	20				26.25	525	
14:50		50	98.0	76.10			2 288
	20				17.51	375	
15:10		70	98.0	84.88			2 638
	40				13.12	525	
15:50		110	98.0	84.88			3 163
	40				13.12	525	
16:30		150	98.0	84.88			3 688
	40				13.12	525	
17:10		190	98.0	84.88			4 213

Figure 6

Analysis of the data of an infiltration test using actual furrows on a clay loam soil



1.3.3. Determination of optimum stream size and furrow length

In order to wet the root zone as uniformly as possible and to have minimum percolation losses at the top end of the field and minimum runoff at the bottom end of the field an appropriate stream size has to be chosen. As explained earlier, water flows from the top end of the field to the bottom end. This is called the advance stream (Figure 2). When water supply stops, the water moves away from the top of the field, which is called the recession of the waterfront. Usually the advance is slower than the recession because water infiltrates quicker in dry soil. Therefore, the top end of the field usually receives more water than the bottom end of the field and water will be lost through deep percolation. If the stream size is too small, it will take a long time before the water reaches the end of the field, therefore deep percolation will be high if the bottom end is also to receive enough water. On the other hand, if the stream size is too large, the waterfront will reach the bottom fast and runoff losses will occur, unless the stream size is reduced. Therefore the appropriate combination of stream flow size and length of borderstrip or furrow has to be selected.

As a rule of thumb, one can say that the stream size must be large enough to reach the end of the furrow in approximately one quarter of the contact time. This is called the one-quarter rule.

The optimum furrow length, or the optimum non-erosive stream flow in existing schemes with known furrow length, could be determined in the field with a test whereby the advance of selected stream sizes is measured in furrows. The results are plotted in a time-advance graph (Figure 7).

Following the above example, where the conditions would be such that a furrow length of 100 m would be preferable, the contact time is 384 minutes. Using the one-quarter rule of thumb, the water should reach the end of the furrow in 96 minutes (= 1/4 x 384). Different flows are brought onto the land and, using the plotted time-advance graph in Figure 7, the optimum furrow length would be 60 m for a flow of 0.5 l/sec and 100 m for a flow of 0.7 l/sec. If 0.7 l/sec is a non-erosive flow, which depends on the soil type and the actual state of the soil (which has to be checked in the field), the 100 m long furrow could be selected, which allows a more cost-effective layout. If the field shape allows it, a furrow should be as long as possible, in order to minimize the number of field canals that have to supply water to the field. This has a direct bearing on cost, since the cost increases with the number of canals to be constructed.

If the flow is not reduced once the water reaches the end of the furrow, a large runoff will occur. Therefore, the flow is usually reduced once or twice during an irrigation, such that runoff remains small. However, if the flow becomes too small, deep percolation losses at the top of the field might increase. The flow could be reduced by taking out siphons from the furrow. Field tests are usually carried out in order to make recommendations to farmers.

The discharge through siphons depends on the diameter of the siphon and the head. For drowned or submerged discharge, the head is the difference between the water level in the canal and the water level in the field (Figure 8a). For free discharge, the head is the difference between the water level in the canal from where the siphon takes the water and the outlet from the siphon (Figure 8b). Discharge can be altered by a change in pipe diameter or a change in the head (Table 4).

Figure 7

Time-advance graph for various stream flow sizes in a furrow

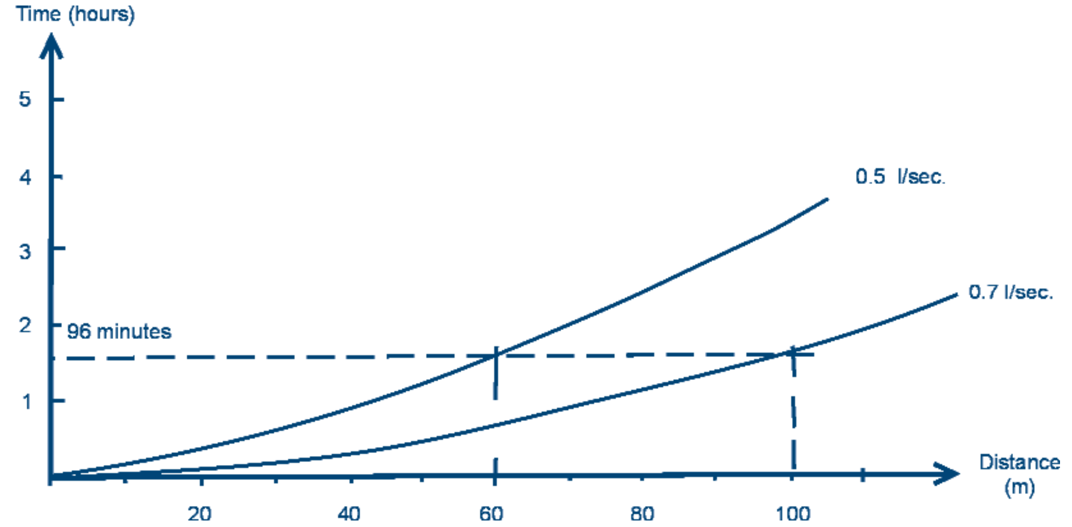


Figure 8

Determining the head (Source: FAO, 1988)

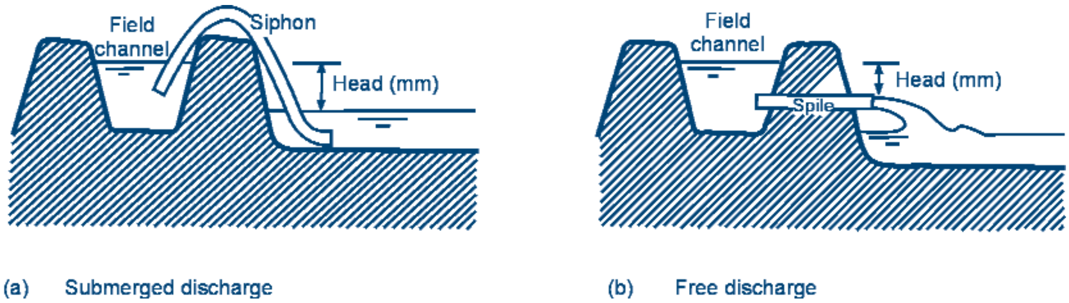


Table 4

Discharge for siphons, depending on pipe diameter and head (l/sec)

Pipe diameter (cm)	Head (cm)			
	5	10	15	20
2	0.19	0.26	0.32	0.73
3	0.42	0.59	0.73	0.84
4	0.75	1.06	1.29	1.49
5	1.17	1.65	2.02	2.33

1.3.4. Determination of optimum stream size and borderstrip length

As in the case of furrow irrigation, it is important to use the right stream size for the soil and land slope and to stop the flow at the right time so that just enough water infiltrates into the soil to satisfy the required irrigation depth. If the flow is stopped too soon, there may not be enough water on the borderstrip to achieve the required irrigation depth at the bottom end of the borderstrip. If the water is left running for too long, there may be large runoff losses. As a rule of thumb, the water supply is stopped when the waterfront reaches between 2/3 and 3/4 of the borderstrip length. On clayish soil, the inflow is usually stopped earlier than on loamy soils, while on sandy soils the water could almost cover the whole borderstrip length before the flow is stopped. New irrigators can rely on the general guidelines given in Table 5 to decide when to stop the flow. The actual field cut-off times should then be decided through field experience.

Table 5

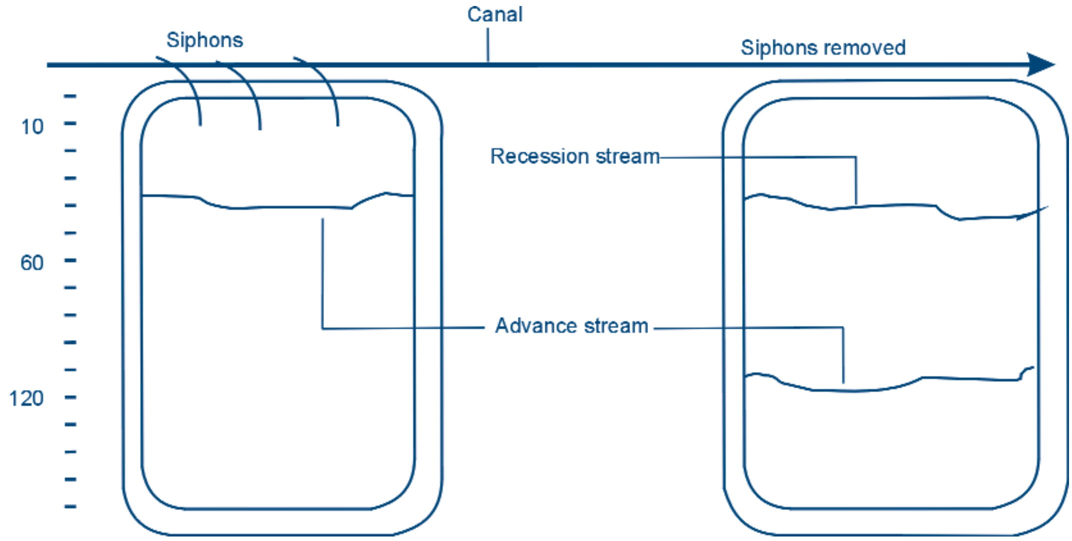
Guidelines to determine when to stop the water supply onto a borderstrip

Soil Type	Stop the flow when advance reaches the following portion of borderstrip
Clay	Two thirds of total length
Loam	Three quarters of total length
Sand	Almost end of borderstrip

Where possible, it is recommended to carry out field tests to determine the best borderstrip length. To do this, a borderstrip is marked with pegs at 10 m interval along its

Figure 9

Advance and recession of water on a borderstrip



length. A selected discharge is then brought onto the strip and the advance of the water is measured, which is the time that it takes for the water to pass through pre-determined distances along the borderstrip length (Figure 9). When the desired volume of water has been delivered to the borderstrip, the flow of water from the canal onto the borderstrip is stopped. As explained above, usually this is done before the water has reached the end of the border. From that moment on, time is taken when the end of the water flow passes through the pre-determined distances. This is called the recession of the water. For a fixed irrigation depth, the total volume desired depends on the size of the borderstrip and thus is larger for a longer borderstrip in cases where the width is the same.

Having measured the time and the distance of the advance and the recession of the water, the advance and recession curves can be drawn. If testing, for example, two different borderstrip lengths, the total volumes of water to be applied are different, leading to two different recession curves, since the inflow is stopped later when the borderstrip is longer. Table 6 shows the data for the advance stream, which is the same for both recessions and the data for recession 1 and recession 2.

The advance curve and the two recession curves have been presented as graphs in Figure 10.

Table 6 shows the data for the advance stream, which is similar for both recession streams. The same table also shows the results from the two separately conducted recession tests. From these data it can be seen that the supply of test 2 was stopped earlier (at 404 minutes) when compared to test 1 (at 464 minutes). The volume brought onto the land during test 1 had completely infiltrated in the soil at 744 minutes, covering the first 130 m of the border. In test 2 all water disappeared after 624 minutes and the required depth had been applied to the first 100 m.

Considering Example 2, where a contact time of 384 minutes was calculated, the ideal intake opportunity time curve was drawn in Figure 10 to be parallel to the advance curve derived from the data of Table 6. The same figure presents the two recession curves derived from the data of Table 6. The closer a recession curve is to the ideal intake opportunity time curve the more efficient the water application. Looking at the two recession curves, it is clear that recession curve 2 is situated closer to the ideal intake opportunity curve than recession curve 1. Recession curve 1 shows that over-irrigation took place, especially over the first 100 m reducing later on for the remaining 30 m. While some deep percolation is also indicated by the position of recession curve 2, this is much less than that demonstrated by recession curve 1. Hence the designs should be based on 100 m length of border.

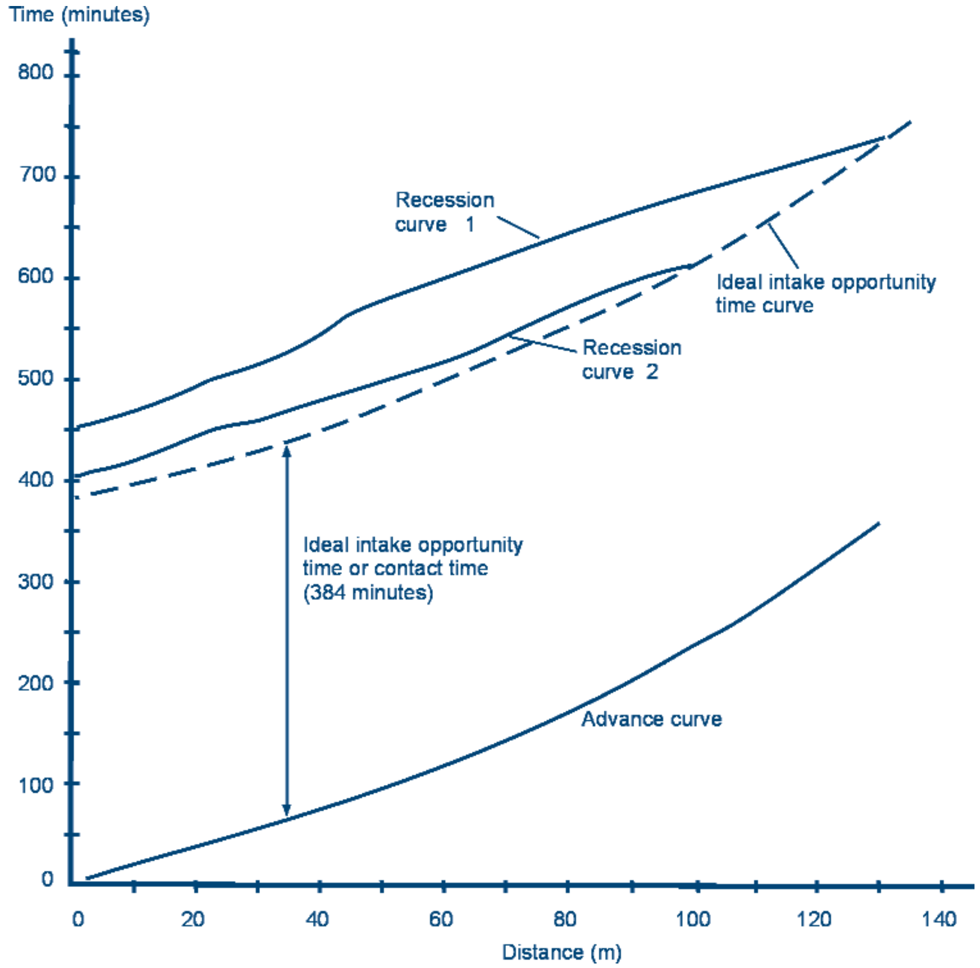
Table 6

Measurement of water advance and recession distance and time on a borderstrip

Advance (m)	Time (min)	Recession 1 (m)	Time (min)	Recession 2 (m)	Time (min)
0	0	0	464	0	404
10	20	10	476	10	422
20	38	20	494	20	444
30	57	30	524	30	464
40	80	40	560	40	480
50	100	50	586	50	502
60	120	60	608	60	524
70	143	70	629	70	554
80	170	80	649	80	582
90	205	90	671	90	604
100	240	100	692	100	624
110	278	110	710	110	
120	315	120	726	120	
130	350	130	744	130	

Figure 10

Advance and recession curves for different borderstrips lengths, needing different total volumes of water to be applied



Chapter 2
Criteria for the selection of the surface irrigation method

Surface irrigation methods refer to the technique of water application over the soil surface in order to wet it, either partially or completely. They do not bear any reference to the conveyance and field canal or distribution system. Three surface irrigation methods can be distinguished:

- ❖ Furrow irrigation
- ❖ Borderstrip irrigation
- ❖ Basin irrigation

Good surface irrigation practice calls for efficient water management to be achieved through:

- ❖ Distributing the water evenly in the soil
- ❖ Providing adequate water to the crops (not too much, not too little)
- ❖ Avoiding water wastage, soil erosion and salinity

2.1. Furrow irrigation

A furrow irrigation system consists of furrows and ridges. The water is applied by means of small channels or furrows, which follow a uniform longitudinal slope. The method is best suited to row crops such as maize, potatoes, onions, tomatoes, etc.

Water can be diverted from the field canal or the tertiary canal into furrows by means of siphons placed over the side of the ditch or canal bank and be allowed to flow downstream along the furrow (Figure 11). The water level in the canal must be raised to a sufficient height above the level of the furrows by using a piece of wood, check plates, or canvas filled with sand. This creates a head difference between the water level in the field ditch and the furrow, which is necessary for the water flow. Water can also be diverted into furrows through gated pipes or hoses connected to a hydrant fitted on buried pipes.

Figure 11
An example of a furrow irrigation system using siphons (Source: Kay, 1986)

The water is gradually absorbed into the soil and spreads laterally to wet the area between the furrows. The amount of water that infiltrates the soil at any point along the furrow depends on the soil type and the period during which the water is in contact with the soil at that particular point. This is known as the contact time or the intake opportunity time (see Chapter 1). With furrow irrigation, water is mainly lost by deep percolation at the head end of furrow and runoff at the tail end. Furrows can be used on most soil types, although coarse sands are not recommended since percolation losses, especially at the top end, would be high because of high infiltration rates. Soils that crust easily are especially suited for furrow irrigation, since the water does not flow over the ridge, which means that the soil in which the plants grow remains friable.

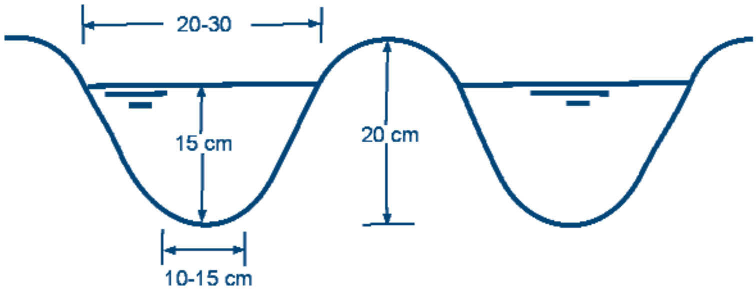
Furrow design is an iterative process that should consider the shape of the furrow, the spacing between furrows, with the furrow length determined, amongst other factors, by the stream size to apply and its application time, the soil type and the slope.

As mentioned by Michael (1994), rational procedures for predicting the water front advance and tail water recession in furrows, which are applicable to field designs, have not been developed. Various workers have proposed a number of quasi-rational procedures with varying degrees of adaptability. In the absence of more precise information on predicting the water advance and recession in furrows, general principles regarding stream size, furrow length and furrow slope to obtain efficient irrigation are followed in field design.

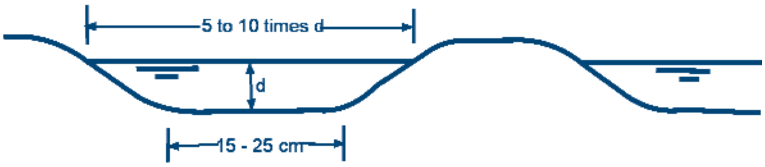
2.1.1 Furrow shape

The furrows are generally V-shaped or U-shaped in cross-section and are 15-30 cm deep and 25-40 cm wide at the top. The shape of the furrow depends on the soil type and

Figure 12
Furrow shape depending on soil type

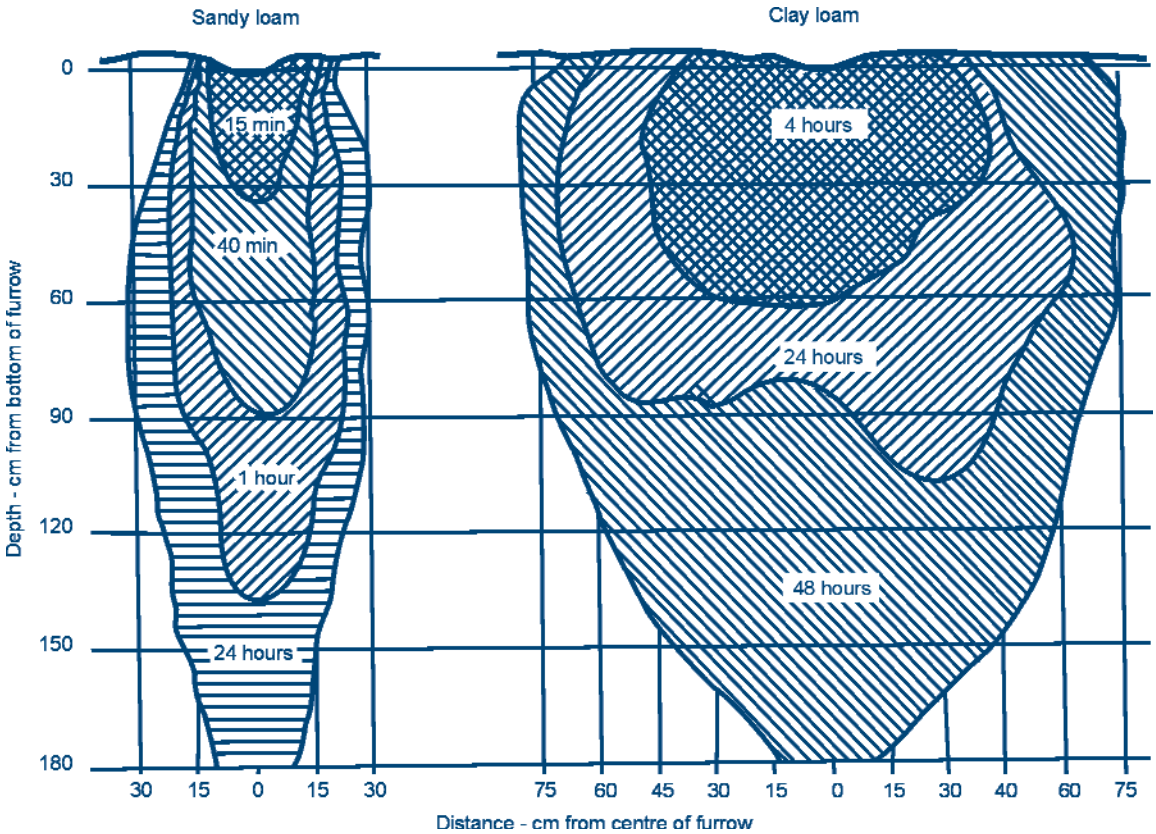


a. Deep, narrow furrow on sandy soil



b. Wide, shallow furrow on clay soil

Figure 13
Soil moisture distribution on various soil types as a determinant of furrow spacing (Source: FAO, 1985)



the stream size. Soils with low infiltration rates have usually shallow wide parabolic or U-shaped furrows to reduce water velocity and to obtain a large wetted perimeter to encourage infiltration. Sandy soils, on the other hand, require more or less V-shaped furrows to reduce the wetted perimeter through which water infiltrates (Figure 12).

Shallow furrows are suited to fields that are graded to uniform slope. The furrows should have uniform depth and shape along the whole length to prevent overtopping. The furrow should be large enough to carry the stream flow and, in general, the larger the stream size the larger the furrow must be to carry the desired flow.

Shallow-rooted crops require shallow furrows. Young crops with shallow rooting depth should be wetted with shallow furrows such that mainly the ridge is wetted. The furrow can be deepened with increased crop growth.

2.1.2. Furrow spacing

The spacing between furrows depends on the water movement in the soil, which is texture related, on the crop agronomic requirements as well as on the type of equipment used in the construction of furrows. In practice a compromise often has to be reached between these factors.

When water is applied to a furrow, it moves vertically under the influence of gravity and laterally by capillarity. Clay soils have more lateral movement of water than sandy soils because of their small pores, which favour capillary action (Figure 13). In this regard, larger spacing can be used in heavier soils than in light soils.

In general, a spacing of 0.3 m and 0.6 m has been proposed for coarse soils and fine soils respectively. For heavy clay soils up to 1.2 m has been recommended.

It should also be realized that each crop has its own optimum spacing and the ridges should be spaced according to the agronomic recommendations. In addition, the equipment available on the farm determines the furrow spacing, as this is adjustable only within limits. However, in all instances the furrow spacing adopted should ensure a lateral spread of water between adjacent furrows that will adequately wet the entire root zone of the plants.

When water and labour are scarce, or when large areas must be irrigated quickly, alternate furrows may be irrigated. In this case, the lateral spread of water is only partial, and crop yields may be reduced. An economic analysis should be carried out to determine whether the advantages of this practice outweigh any possible limitations on yield.

2.1.3. Furrow length

Under mechanized agriculture, furrows should preferably be as long as possible in order to reduce labour requirements and system costs. However, they also should be short enough to retain a reasonable application efficiency and uniformity. Application efficiency and uniformity normally increase as the furrow length decreases. Thus, when labour is not a constraint or inexpensive and/or the water supply is limited, short furrows may be most suitable. This does increase the number of field canals and overall cost of the system. For proper design of the furrow length, the following factors have to be taken into account:

- Soil type
- Stream size
- Irrigation depth
- Slope
- Field size and shape
- Cultivation practices

Soil type

Furrow lengths should be shorter in sandy soils than in clayish soils, since water infiltrates faster into light soils than into heavy soils. If furrow lengths are too long in sandy soils, too much water will be lost as deep percolation at the top end of the furrow. On heavier soils, water infiltrates slowly and therefore furrows should be longer to allow sufficient time for the water to infiltrate the soil to the desired depth. In Eastern and Southern Africa designers are often confronted with the fact that smallholders occupy land with high soil variability, both vertically and horizontally, necessitating field tests to establish the appropriate furrow length.

Stream size

On similar soils, and of the same slope and irrigation depth, furrows can be longer when a larger stream size is used for irrigation. This is because water will be advancing rapidly down the furrow. However, the stream size should not exceed the maximum non-erosive stream size determined in field trials. The following equation provides guidance in selecting stream sizes for field trials.

Equation 1

$$Q_{max} = \frac{K}{S_o}$$

Where:

Q_{max}	=	Maximum non-erosive stream size (l/min)
S_o	=	Furrow slope in the direction of flow (%)
K	=	Unit constant (= 40)

Furrow stream sizes are sometimes selected on the basis of the one-quarter rule. This rule states that the time required for water to advance through a furrow till the end should be one quarter of the total irrigation time (contact time). However, it should be noted that only field experience will show when to actually close the inflow, because of all the different factors involved (see Section 1.3.3).

Irrigation depth

A larger irrigation depth requires more contact time for water to infiltrate to the desired depth than a shallow irrigation depth, as explained in Chapter 1. The irrigation depth can be increased by making the furrow longer in order to allow more time for the water to reach the end of furrow, which increases the contact time. Care should be taken, however, to avoid too high percolation losses at the top end.

Slope

Furrows should be put on proper gradients that allow water to flow along them and at the same time allow some water to infiltrate into the soil. Furrows put on steeper slopes can be longer because water moves more rapidly. However, with slopes steeper than 0.5% (0.5 m drop per 100 m length), the stream sizes should normally be reduced to avoid erosion, thus shorter furrows have to be used. Under smallholder conditions the maximum slope of 0.5% should not be exceeded (James, 1988).

Field size and shape

Field size and shape provides challenges to designers. For the system to operate at the level of efficiency earmarked by the designer, the field shape of each farmer's plot should be regular and the length of run uniform for all farmers. This would facilitate uniform water delivery throughout the field and scheme. It is therefore advisable that for new developments this principle be adhered to and discussed in detail with the farmers during the planning consultations. Unfortunately, in practice in most cases no or insufficient effort is made to discuss this matter with smallholders.

Where an area had been cultivated by farmers under dryland conditions prior to the installation of the irrigation scheme, farmers are often even more reluctant to change the shape and borders of their individual fields. As a result the original, irregular shapes of the fields are maintained and variable lengths of run are used. This results in a complex operation of the system and water shortages, and at the same time water is wasted, leading to low irrigation efficiencies and higher operation costs. For the same reason the number of field canals increases, resulting in high development costs. This shows the importance of involving farmers from the planning stage onwards, so that they themselves also will become convinced of the advantages and necessity of regular shaped fields of equal size and thus are willing to change the original borders of their individual fields.

Cultivation practices

When cultivation practices are mechanized, furrows should be made as long as possible to facilitate working with machinery. Short furrows also require a lot of labour, as the flow must be changed frequently from one furrow to the next.

Guidelines for the determination of furrow lengths

Table 7 summarizes the main factors affecting the furrow length and the suggested practical allowable furrow lengths according to Kay (1986). The data given in this table are appropriate for large-scale and fully mechanized conditions.

Table 7

Furrow lengths in metres as related to soil type, slope, stream size and irrigation depth (Source: Kay, 1986)

Soil type		Clay		Loam			Sand		
		Average irrigation depth (mm)							
Furrow slope %`	Maximum stream size (l/sec)	75	150	50	100	150	50	75	100
0.05	3.0	300	400	120	270	400	60	90	150
0.10	3.0	340	440	180	340	440	90	120	190
0.20	2.5	370	470	220	370	470	120	190	250
0.30	2.0	400	500	280	400	500	150	220	280
0.50	1.2	400	500	280	370	470	120	190	250
1.00	0.6	280	400	250	300	370	90	150	220
1.50	0.5	250	340	220	280	340	80	120	190
2.00	0.3	220	270	180	250	300	60	90	150

Table 8

Practical values of maximum furrow lengths in metres depending on soil type, slope, stream size and irrigation depth for small-scale irrigation (Source: FAO, 1988)

	Soil type	Clay		Loam		Sand	
		Net irrigation requirements (mm)					
Furrow slope %`	Maximum stream size per furrow (l/sec)	50	75	50	75	50	75
0.0	3.0	100	150	60	90	30	45
0.1	3.0	120	170	90	125	45	60
0.2	2.5	130	180	110	150	60	95
0.3	2.0	150	200	130	170	75	110
0.5	1.2	150	200	130	170	75	110

Table 8 provides more realistic data for smallholder irrigation.

The soil variability in most smallholders' schemes, combined with the small size of holdings, makes the scheme more manageable when shorter furrows are used. The figures in both tables should only be used as a guide in situations where it is not possible to carry out field tests. As much as practically possible, the furrow lengths should be determined in the field based on the tests described in Chapter 1.

2.2. Borderstrip irrigation

Borderstrips are strips of land with a downward slope but are as horizontal as possible in cross-section (Figures 14 and 15). A horizontal cross-section facilitates an even rate of water advance down the longitudinal slope. Borderstrips can vary from 3-30 m in width and from 60-800 m in length. They are separated by parallel dykes or border ridges (levées).

Normally water is let onto the borderstrip from the canal through intakes, which can be constructed with gates on the wall of the canal or, when unlined canals are used, by temporarily making an opening in the canal wall. The latter is not recommended since it weakens the walls of the canal, leading to easy breakage. Other means used for the same purpose is the insertion of short PVC pipes into the canal through the wall. The short pipes are usually equipped with an end cup, which is removed when irrigation is practiced. Some farmers use cloth or plastic sheet to close and open the pipe. The most appropriate method of supplying water from the canal to the field, however, is the use of siphons.

Figure 14

Example of a borderstrip irrigation system (Source: Kay, 1986)

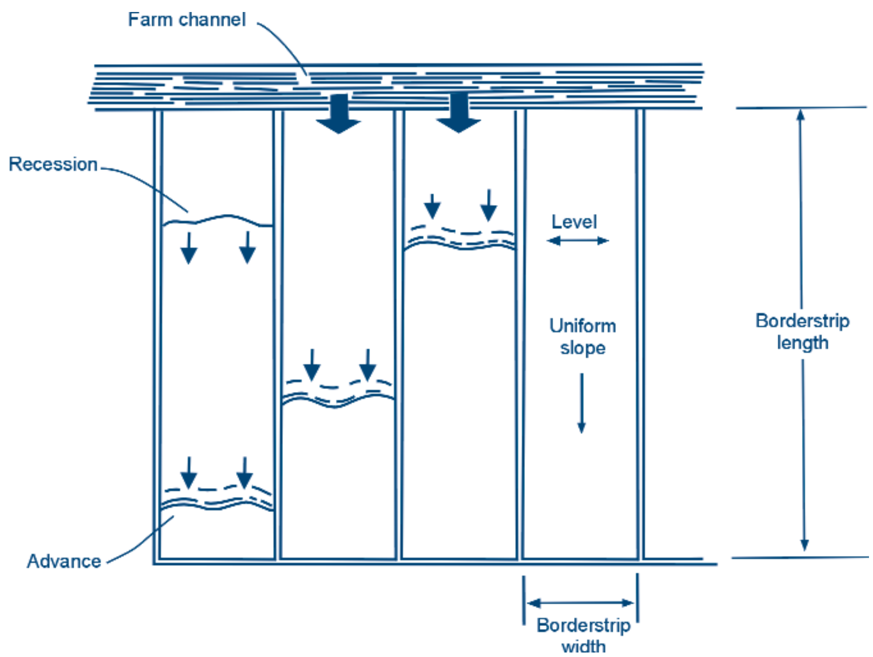
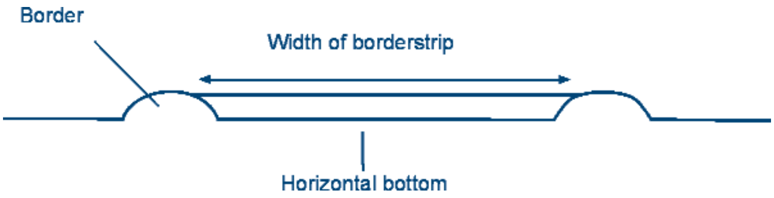


Figure 15
Cross-section of a borderstrip



Leakage should be expected from all other techniques and, as a consequence, waterlogging at the beginning of the borderstrips. The use of siphons instead of one water inlet also has the advantage that they can be spread over the width of the borderstrip, thus facilitating an equal spread of the water across the borderstrip width and then down the slope towards the lower end. To ensure a proper lateral spread, it is recommended that, longitudinally, the borderstrips be horizontal for the first 10 m or so, with a uniform downward slope thereafter. This irrigation method is particularly suitable for pasture and close-growing crops like wheat.

2.2.1. Borderstrip width

The borderstrip width depends on the topography of the field, which determines the possible width that can be obtained while keeping a horizontal cross-section without requiring too much soil movement, and on the stream size. The stream size also restricts strip width, as it should be sufficient to allow complete lateral spreading throughout the borderstrip width and length. The strip width also depends on the cultivation practices, mechanized or non-mechanized for example. Borderstrips should not be wider than 9 m on 1% cross-slopes (James, 1988).

2.2.2. Longitudinal slope of the borderstrip

The slope should, wherever possible, be adapted to the natural topography in order to reduce the need for land grading, which may lead to the removal of too much topsoil. On the one hand, slopes on sandy soils should be greater than on heavy soils to avoid deep percolation losses at the top of the field. Furthermore, the maximum slope depends on the risk of soil erosion, which is greater in sandy soils than in clayish soils. Crop cover can be a factor that counts in determining the borderstrip slope. For example, where the borderstrip will be covered by a permanent crop, such as pasture, slopes can be up to 7% for clay soils with stable aggregates (James 1988).

Borderstrips should have a minimum slope of 0.05-0.1% to allow water to flow downstream over it. The maximum slope on sandy soils ranges from 0.3-1% in humid areas and 1-2% in arid areas on bare soils and soils with good crop cover respectively. For clay soils the slope ranges from 0.5-2% in humid areas and 2-5% in arid areas on bare soils and soils with crop cover respectively (see also Module 1).

2.2.3. Borderstrip length

To determine the borderstrip length, the following factors need to be considered:

- Soil type
- Stream size
- Irrigation depth
- Land slope
- Field size and shape
- Cultivation practices

Soil type

The borderstrip length can be longer on heavy soil than on light soil because water infiltrates heavy soil more slowly. If a border is made too long on a light soil, too much water will infiltrate the soil at the top part, leading to too much water loss due to deep percolation.

Stream size

When a larger stream size is available, borderstrips can be longer on the same soils, because water will spread more rapidly across the soil surface. As a rule of thumb, the stream size must be large enough to adequately spread water across the width of border. However, it should not exceed the maximum non-erosive stream size. The design of the stream size must also result in rates of advance and recession that are essentially equal (see Section 1.3.4).

Irrigation depth

A larger irrigation depth requires more contact time for water to infiltrate to the desired depth than does a shallow irrigation depth, as explained in Chapter 1. The irrigation depth can be increased by making the borderstrip longer in order to allow more time for the water to reach the end of borderstrip, which increases the contact time. However, soil type is a limiting factor.

Slope

Borderstrips should be put on proper gradients that allow water to flow downstream over the surface yet at the same time to allow some water to infiltrate the soil. The borderstrip can be longer on steeper slopes, since water moves more rapidly. However, precautions should be taken against erosion.

Field size and shape

Existing field size and shape are often practical limits to the size of borders. However, the same remarks as mentioned under furrow irrigation (see Section 2.1.3) are valid for borderstrip irrigation.

Cultivation practices

Because borders are normally long so as to achieve good water distribution, they are very suitable for mechanized farming. The width should preferably be a multiple of the farm machinery used.

2.2.4. Guidelines for the determination of borderstrip width and length

Table 9 provides some typical borderstrip widths and lengths for various soil types, slopes, irrigation depths and flows. It should be noted that in practice borderstrip lengths are often shorter because of poor levelling. Sometimes they have to be reduced after construction, if the irrigation efficiencies turn out to be too low. It should be noted that the figures for width and length in Table 9 apply to highly-mechanized agriculture on properly levelled lands under good water management.

The figures in Table 9 may not apply to small-scale irrigators on small landholdings. Table 10 provides some guidelines for determining borderstrip dimensions felt suitable for smallholder irrigation in communal areas, where farmers are responsible for the operation and maintenance of these schemes.

Table 9

Typical borderstrip dimensions in metres as related to soil type, slope, irrigation depth and stream size (Source: Withers and Vipond, 1974)

Soil Type	Slope (%)	Depth applied (mm)	Flow (l/sec)	Strip width (m)	Strip length (m)
Coarse	0.25	50	240	15	150
		100	210	15	250
		150	180	15	400
	1.00	50	80	12	100
		100	70	12	150
		150	70	12	250
	2.00	50	35	10	60
		100	30	10	100
		150	30	10	200
Medium	0.25	50	210	15	250
		100	180	15	400
		150	100	15	400
	1.00	50	70	12	150
		100	70	12	300
		150	70	12	400
	2.00	50	30	10	100
		100	30	10	200
		150	30	10	300
Fine	0.25	50	120	15	400
		100	70	15	400
		150	40	15	400
	1.00	50	70	12	400
		100	35	12	400
		150	20	12	400
	2.00	50	30	10	320
		100	30	10	400
		150	20	10	400

Table 10
Suggested maximum borderstrip widths and lengths for smallholder irrigation schemes

Soil type	Borderstrip slope (%)	Unit flow per metre width* (l/sec)	Borderstrip width (m)	Borderstrip length (m)
Sand (Infiltration rate greater than 25 mm/h)	0.2-0.4 0.4-0.6 0.6-1.0	10-15 8-10 5-8	12-30 9-12 6-9	60-90 80-90 75
Loam (Infiltration rate of 10 to 25 mm/h)	0.2-0.4 0.4-0.6 0.6-1.0	5-7 4.6 2-4	12-30 9-12 6	90-250 90-180 90
Clay (Infiltration rate less than 10 mm/h)	0.2-0.4 0.4-0.6 0.6-1.0	3-4 2-3 1-2	12-30 6-12 6	180-300 90-180 90

* *The flow is given per metre width of the border. The total flow into a border is equal to the unit flow multiplied by the border width (in metres)*

However, in reality it seems that the widths of the borderstrips in smallholder schemes are still less than the figures given in Table 10. As an example, a typical borderstrip in a smallholder irrigation scheme in Zimbabwe can have a width varying between 2-4 m, which is even less than half of the smallest width given in Table 10.

As much as practically possible, borderstrip lengths should be determined in the field based on the tests described in Chapter 1. However, the method is best suited to projects at the planning stage. For projects that have passed through this stage, and where the length and slope of the borderstrip have been fixed by field shape and land topography during the planning stage rather than by testing, a high uniformity of water distribution can only be achieved by adjusting the stream size and the time to stop inflow, on the condition that the stream size is non-erosive. Such an arrangement, however, makes the operation of the scheme very complex for management by smallholders with limited or no past experience with surface irrigation.

2.3. Basin irrigation

A basin is a horizontal area of land surrounded by earthen bunds and totally flooded during irrigation. Basin irrigation is the most common type of surface irrigation. It is particularly used in rice cultivation, where the fields are submerged, but it is equally suitable for other crops like cereals, fruit trees and pastures – as long as waterlogging conditions do not last for too long. Ideally, the waterlogging should not last longer than 24-48 hours. It is also used for the leaching of salts by deep percolation in the reclamation of saline soils. A basin irrigation system layout is illustrated in Figure 16.

Flooding should be done using a large stream size that advances quickly in order for water to spread rapidly over the basin. The advance time should not exceed a quarter of the contact time, so as to reduce difference in contact time on the different sections of the basin. It may be used on a wide variety of soil textures, though fine-textured soils are preferred. As the area near the water inlet is always longer in contact with the water, there will be some percolation losses, assuming the entire root zone depth is filled at the bottom of the field. Coarse sands are not generally recommended for basin irrigation as high percolation losses are expected at the areas close to water intake.

2.3.1. Basin size

The size of basin is critical in the design of this irrigation method and, as for furrow and borderstrip irrigation, depends on the following factors:

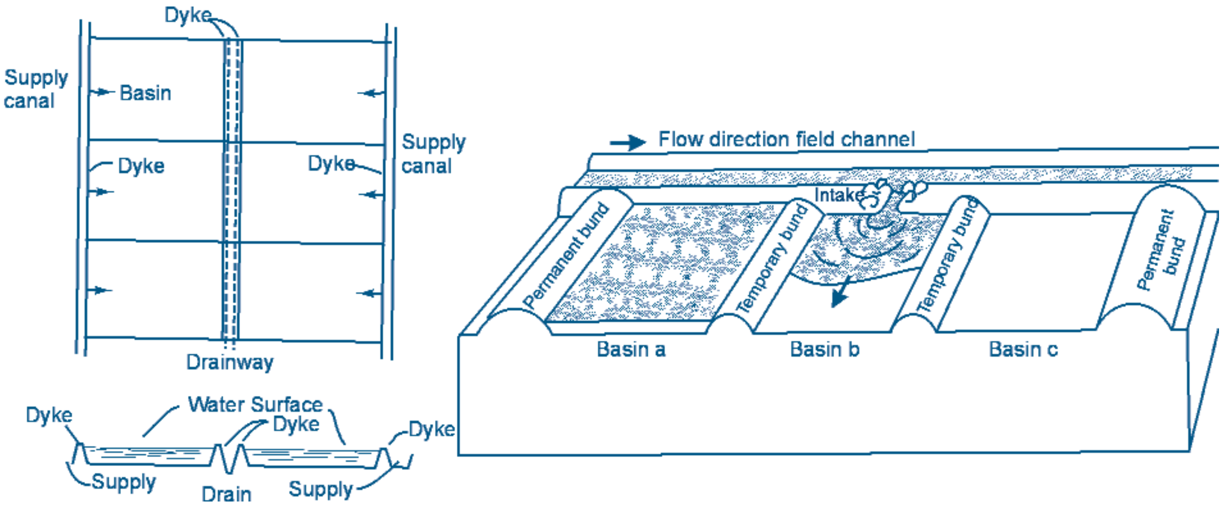
- ❖ Soil type
- ❖ Stream size
- ❖ Irrigation depth
- ❖ Field size and shape
- ❖ Land slope
- ❖ Farming practices

Table 11 shows in summary the general criteria for selecting a basin size, all of which will be discussed more in detail below.

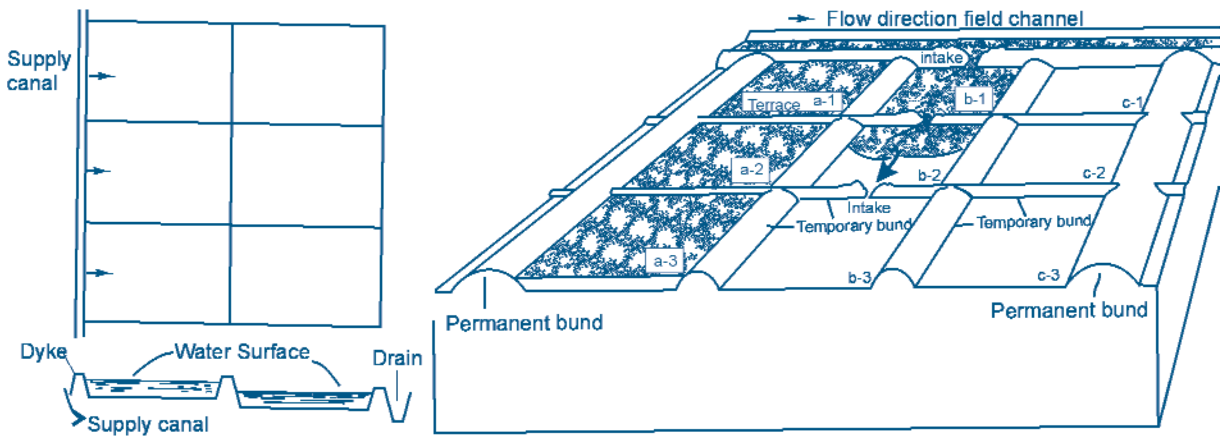
Table 11
Criteria for basin size determination

Criteria	Basin size small	Basin size large
Soil type	Sandy	Clay
Stream size	Small	Large
Irrigation depth	Small	Large
Land slope	Steep	Gentle or flat
Field preparation	Hand or animal traction	Mechanized

Figure 16
Layout of basin irrigation (Source: FAO, 1985)



Direct method of water supply to the basins with a drainway midway between supply canals. “Basin a” is irrigated, then “Basin b”, and so on.



Cascade method of water supply to the basins with a tier arrangement. Ideal on terraced land, where water is supplied to the highest terrace, and then allowed to flow to a lower terrance and so on.

Soil type

Water infiltrates heavier soils more slowly than lighter soils. This means that there is more time for water to spread over the soil surface on heavy soils than on light soils before infiltrating. Therefore basins can be larger on heavier soils than on lighter soils.

Stream size

Basin size can increase with larger stream size, because water will spread more rapidly over the basin. Table 12 below gives some guidelines on basin sizes in relation to stream size and soil type.

Table 12
Basin area in m^2 for different stream sizes and soil types (Withers and Vipond, 1974)

Stream size (l/sec)	Sand	Sandy loam	Clay loam	Clay
5	35	100	200	350
10	65	200	400	650
15	100	300	600	1 000
30	200	600	1 200	2 000
60	400	1 200	2 400	4 000
90	600	1 800	3 600	6 000

Irrigation depth

A deeper irrigation depth requires more contact time for water to infiltrate to the desired depth than does a shallow irrigation depth, as explained in Chapter 1. The irrigation depth can be increased by increasing the size of the basin in order to allow more time for the water to reach the end of basin. This increases the contact time. However, soil type is a limiting factor. If the soil is too light then too much water will infiltrate and be lost by deep percolation next to the water inlet and too little will infiltrate furthest away from the water inlet, hence the unsuitability of basins for very light soils. It would be better to wet the whole basin as soon as possible and to leave the water ponding until the desired volume of water has been applied.

Land slope

The soil surface within each basin should be horizontal. Basins can be as large as the stream size and soil type can allow on level land. On steep slopes, the removal of the topsoil and the associated land levelling costs may be limiting factors for the basin size. Typically, terrace width varies from about 2 m for 4% land slopes up to 150 m for 0.1 % land slopes. Table 13 below provides some guidelines on the possible width of a basin, in relation to the land slope.

Table 13
Approximate values for the maximum basin width (m)

Slope (%)	Maximum width (m)	
	Average	Range
0.2	45	35-55
0.3	37	30-45
0.4	32	25-40
0.5	28	20-35
0.6	25	20-30
0.8	22	15-30
1.0	20	15-25
1.2	17	10-20
1.5	13	10-20
2.0	10	5-15
3.0	7	5-10
4.0	5	3-8

Field size and shape

Basins are best adapted to regular field shapes (square or rectangular). Irregular field shapes necessitate adapting basins. This may lead to basin sizes being different from what would be recommended based on other factors such as soil types, etc. Although a regular shape is favourable, basins can be shaped to follow contours. These are contour basins or terraces, which are seen mainly on steep slopes used for rice.

Cultivation practices

Mechanized farming requires relatively large basins in order to allow machines to turn round easily and to have long runs without too many turns. In this case, the dimensions of basins should be a multiple of the machine width. On small farms, in general 1-2 ha or less in developing countries, farming operations are done by hand or animal power. Small basins are used on such lands. The basins are often levelled by hand.

The types of crops grown may also influence the size of basins. For example, small basins could be used per single tree for orchards.

2.4. Efficiencies of surface irrigation systems and of the different surface irrigation methods

Surface irrigation schemes are designed and operated to satisfy the irrigation water requirements of each field while controlling deep percolation, runoff, evaporation and operational losses. The performance of the system is determined by the efficiency with which water is conveyed to the scheme from the headworks, distributed within the scheme and applied to the field, and by the adequacy and uniformity of application in each field (see also Module 1).

2.4.1. The different types of efficiencies in an irrigation scheme

Conveyance efficiency (E_c)

Conveyance efficiency is the ratio of water received at the inlet to a block of fields or a night storage reservoir to the water released from the project headworks. Factors affecting this efficiency include canal lining, evaporation of water from the canal, technical and managerial facilities of water control, etc. Conveyance efficiency is higher when water is conveyed in a closed conduit than when it is conveyed in an open one, since water in the latter is very much exposed to evaporation as well as to ‘poaching’ by people and to livestock watering.

Field canal efficiency (E_b)

This is the ratio of water received at the field inlet to the water received at the inlet to a block of fields or a night storage reservoir. Among other factors, this efficiency is affected by the types of lining in respect to seepage losses, by the length of canals and by water management. Piped systems have higher field canal efficiencies than do open canal systems for reasons explained earlier.

Field application efficiency (E_a)

This is the ratio of water directly available to the crop to water received at the field inlet. It is affected, for example, by the rate of supply, infiltration rate of soil, storage capacity of the root zone, land levelling, etc. For furrow and borderstrip irrigation, water is mostly lost through deep percolation at the head end and through runoff at the tail end, while for basin irrigation it is mostly through deep percolation and evaporation, since the basin is closed.

Distribution system efficiency (E_d)

Conveyance efficiency E_c and field canal efficiency E_b are sometimes combined and called distribution system efficiency E_d, expressed as:

Equation 2

$$E_d = E_c \times E_f$$

Farm irrigation efficiency (E_f)

Field canal efficiency E_b and field application efficiency E_a are sometimes combined and called farm irrigation efficiency E_f, expressed as:

Equation 3

$$E_f = E_b \times E_a$$

Overall irrigation efficiency (E_p)

The overall or project irrigation efficiency of an irrigation scheme is the ratio of water made available to the crop to that released at the headwork. It is the product of three efficiencies, namely conveyance efficiency (E_c), field canal efficiency (E_b) and field application efficiency (E_a), and is expressed as:

Equation 4

$$E_p = E_c \times E_b \times E_a \quad \text{or:}$$
$$E_p = E_d \times E_a \quad \text{or:}$$
$$E_p = E_c \times E_f$$

The field application efficiency (E_a) is the one that contributes most to the overall irrigation efficiency and is quite specific to the irrigation method, as discussed below. Any efforts that are made to improve on this efficiency will impact heavily on the overall efficiency.

2.4.2. Efficiencies of the different surface irrigation methods

Furrow irrigation could reach a field application efficiency of 65% when it is properly designed, constructed and managed. The value ranges from 50-70%. Losses will occur through deep percolation at the top end of the field and runoff at the bottom end. Properly designed and managed borderstrips can reach a field application efficiency of up to 75%, although a more common figure is 60%. With basin irrigation it is possible to achieve field application efficiencies of 80% on properly designed and managed basins, although a more common figure used for planning varies between 60-65%. For more details on efficiencies the reader is referred to Module 1.

Example 3

At Nabusenga surface irrigation schemes, it is assumed that E_c and E_b both are 90% for concrete-lined canals continuous flow and that E_a is 50%. What is the overall irrigation efficiency?

$E_p = E_c \times E_b \times E_a = 0.90 \times 0.90 \times 0.50 = 0.41$ or 41%

In order to show the importance of contribution of E_a to the overall irrigation efficiency while keeping the same E_c and E_b but increasing the field application efficiency E_a from 50% to 70%, an overall irrigation efficiency of 0.57 or 57% (0.90 x 0.90 x 0.70) can be achieved instead of 41%.

There are common problems that can reduce the field application efficiency of the three surface irrigation methods to a considerable extent. The most common ones are discussed below:

- Poor land levelling can lead to waterlogging in some places and inadequate water application in others. If the cross slope is not horizontal for borderstrip irrigation, water will flow to the lowest side causing over-irrigation in that area.
- Different soil types along the furrows, borderstrips and basins result in different infiltration rates.
- Too small an advance stream results in too long an advance time, leading to over-irrigation at the top end of the borderstrip and furrow. A small stream size diverted into a basin will take too long to cover the entire basin area, resulting in a contact time that is very different at the various sections of the basin.
- Too large a stream size will result in water flowing too fast down the borderstrip and furrow leading to a cut-off taking place before the root zone has been filled with water. If the flow is allowed to continue under these conditions there will be excessive runoff at the end. A large stream size, on the other hand, can be desirable for basins as this reduces the difference in contact time on the various sections of the basin.

- The use of irregular-shaped plots with variable lengths of run complicates the operation of the system, resulting in poor efficiencies.

As a rule of thumb, overall irrigation efficiencies used in Zimbabwe for design purposes are in the range of 40-50% when using concrete-lined canals and 55-65% for piped systems.

2.5. Criteria for the selection of the surface irrigation method

Though it is not possible to give specific guidelines as to which surface irrigation method to select under a given set of conditions, each option usually has advantages and disadvantages. The selection of a surface irrigation method depends mainly on soil type, crops to be irrigated, the irrigation depth, land slope, field shape, labour availability and water source. Table 14 gives some guidelines on which method would be most appropriate depending on soil type, crop rooting depth and net depth of application.

2.5.1. Soil type

All three surface irrigation methods prefer heavy soils, which have lower infiltration rates. A light soil with high infiltration rates favours deep percolation losses at the top of the fields, resulting in low field application efficiency.

2.5.2. Type of crop

Furrow irrigation is particularly suitable for irrigating row crops such as maize and vegetables. Furrows are also more suitable for shallow-rooted crops. Borderstrip irrigation can also be used for row crops or for close-growing crops that do not favour water ponding for long durations, such as wheat and alfalfa. Any crop, whether row or close-growing, that can stand a very wet soil for up to 24 hours is best grown in basins.

2.5.3. Required depth of irrigation application

If the application depth is small, furrow irrigation is the most appropriate method of irrigation. Field experience has shown that large application depths can be applied most efficiently with basin irrigation.

In general, the gross irrigation depth is much larger than the net irrigation depth for all three surface irrigation methods, due to the lower irrigation efficiencies of surface irrigation compared to pressurized systems (see Module 1). However, of the three surface irrigation methods, basin irrigation can have higher irrigation efficiency and use less water for the same crop on the same soil than the other methods as water is confined within bunds (see Section 2.4.2).

2.5.4. Land slope

In general terms, all surface irrigation methods favour flat land as steep slopes would necessitate excessive land levelling in order to avoid erosion, which is expensive and can lead to the removal of top soil.

Flat land with a slope of 0.1% or less is best suited for basin irrigation (which needs a zero slope) since it requires minimum land levelling. Borderstrip irrigation may be used on steeper land, even up to 5%, depending upon other limiting factors such as soil type. One has to be cautious with furrow irrigation on such steep slopes. This is because the flow is confined to a small channel (the furrow), which could result in erosion.

2.5.5. Field shape

In general, furrow irrigation requires regular field shapes, allowing the use of the same stream size for the same furrow lengths. However, for all three types regularly shaped fields are preferable, as explained in Sections 2.1, 2.2 and 2.3.

2.5.6. Labour availability

Basin irrigation requires less labour than the other two methods and might have to be considered if there is a critical labour shortage.

Table 14

Selection of an irrigation method based on soil type and net irrigation depth (Source: Jensen, 1983)

Soil type	Rooting depth of crop	Net irrigation depth per application (mm)	Surface irrigation method
Sand	Shallow	20-30	Short furrows
	Medium	30-40	Medium furrows, short borders
	Deep	40-50	Long furrows, medium borders, small basins
Loam	Shallow	30-40	Medium furrows, short borders
	Medium	40-50	Long furrows, medium borders, small basins
	Deep	50-60	Long borders, medium basins
Clay	Shallow	40-50	Long furrows, medium borders, small basins
	Medium	50-60	Long borders, medium basins
	Deep	60-70	Large basins

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

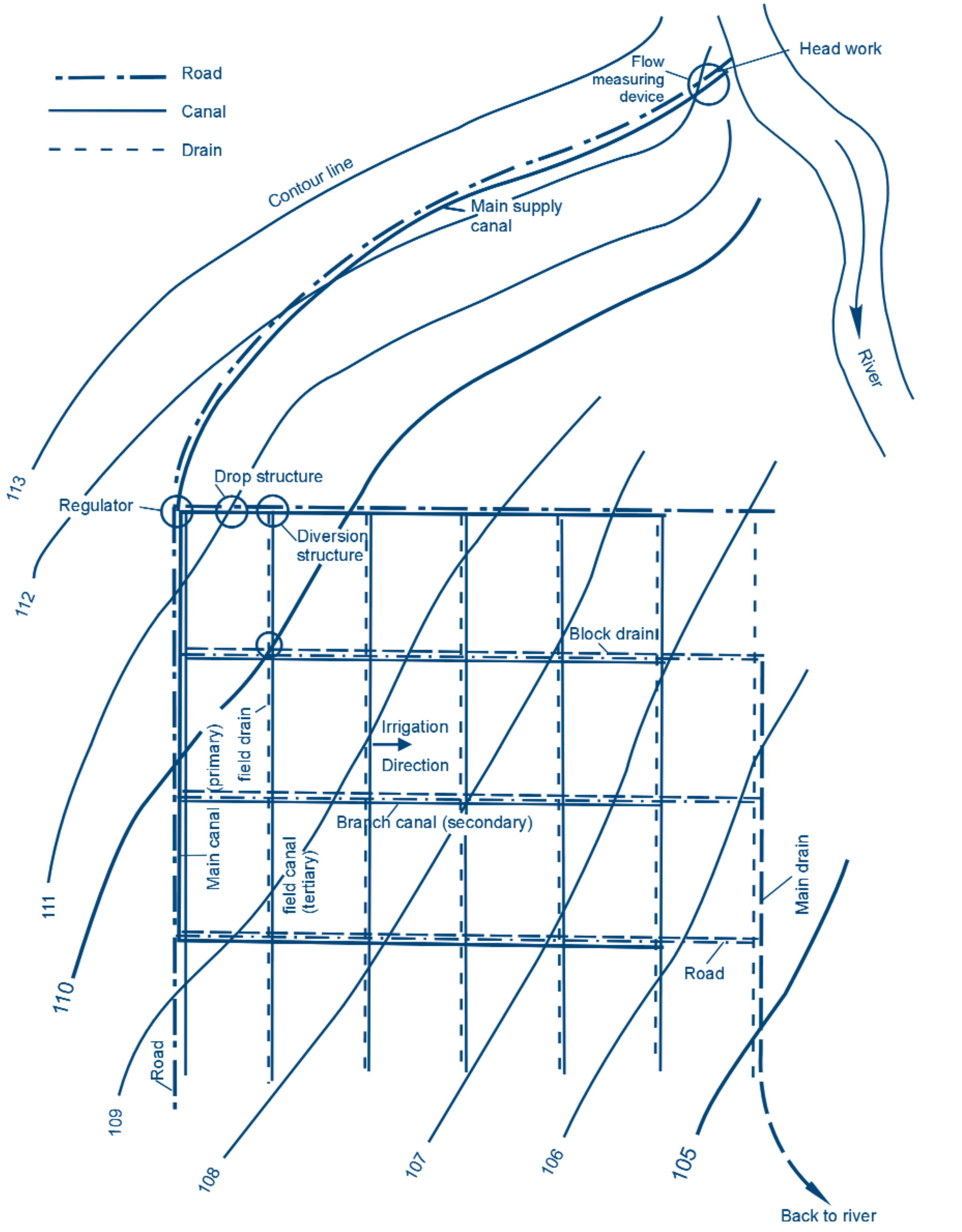
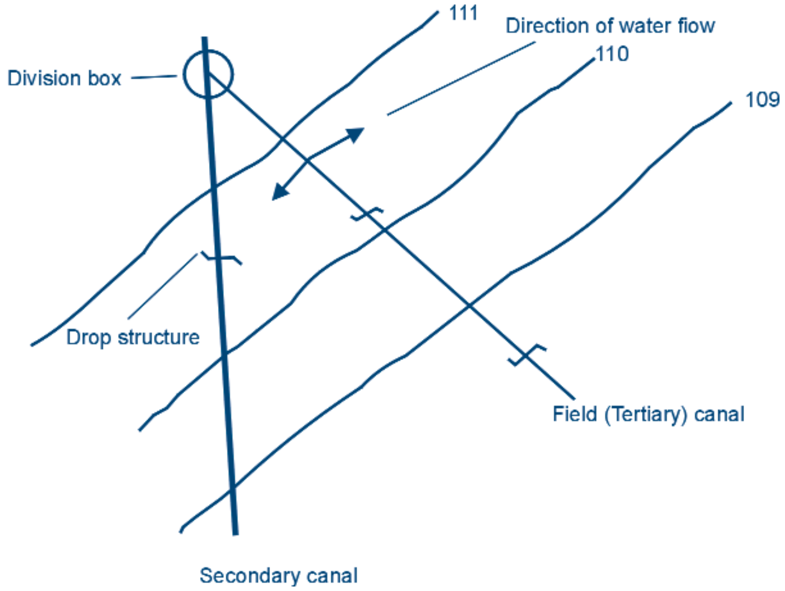


Figure 18
The herringbone irrigation layout



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

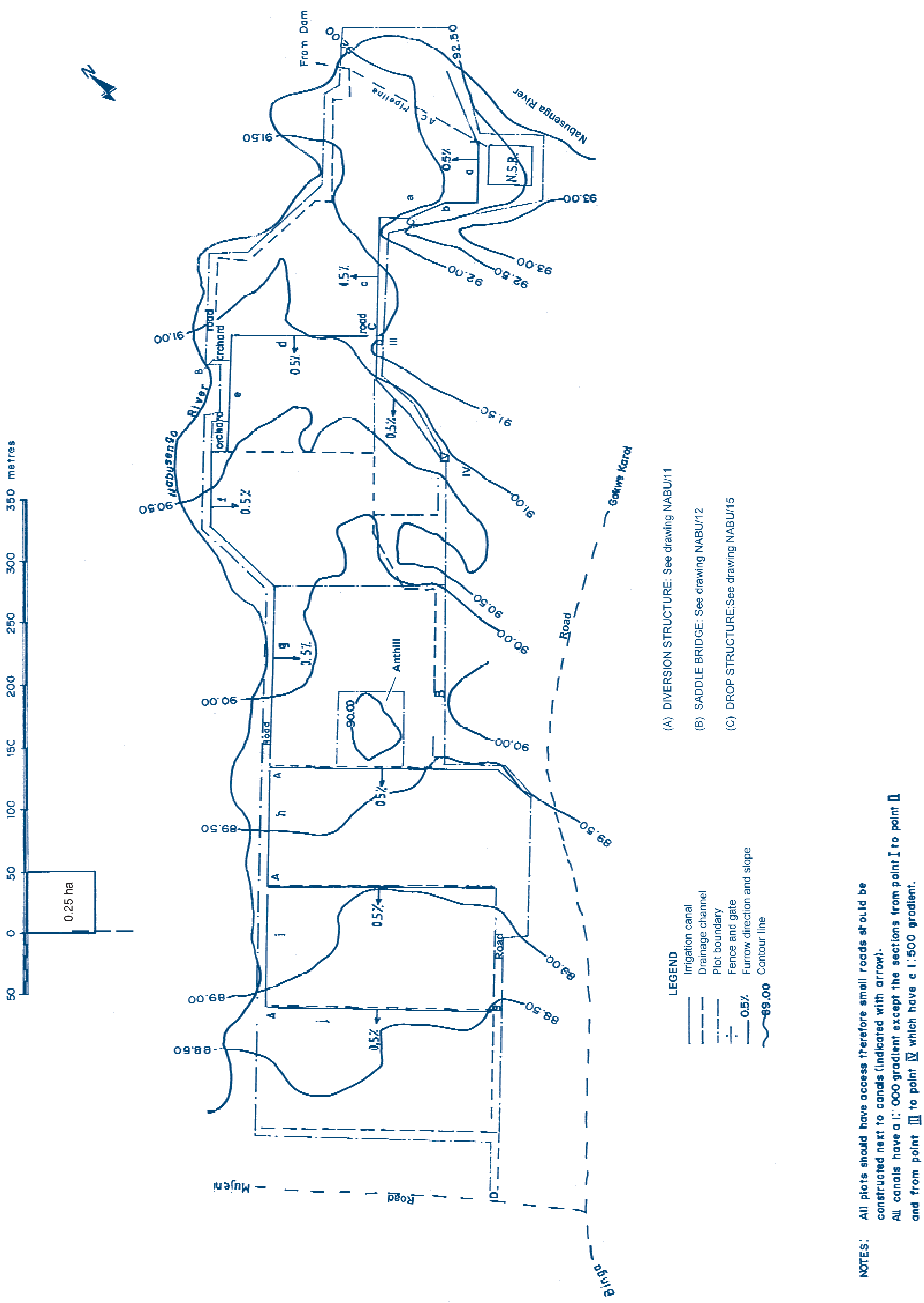
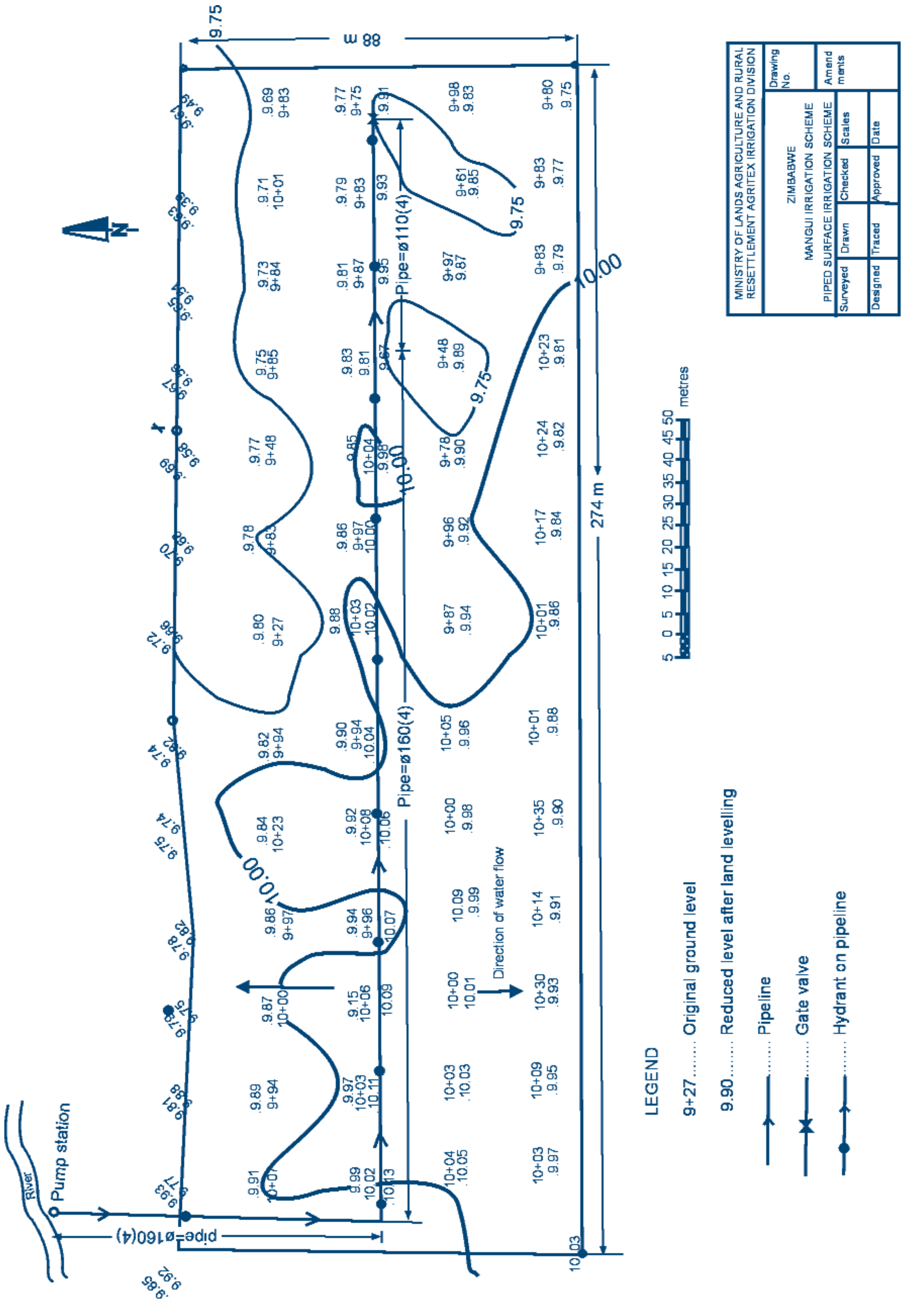


Figure 20
Layout of Mangui piped surface irrigation scheme



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.

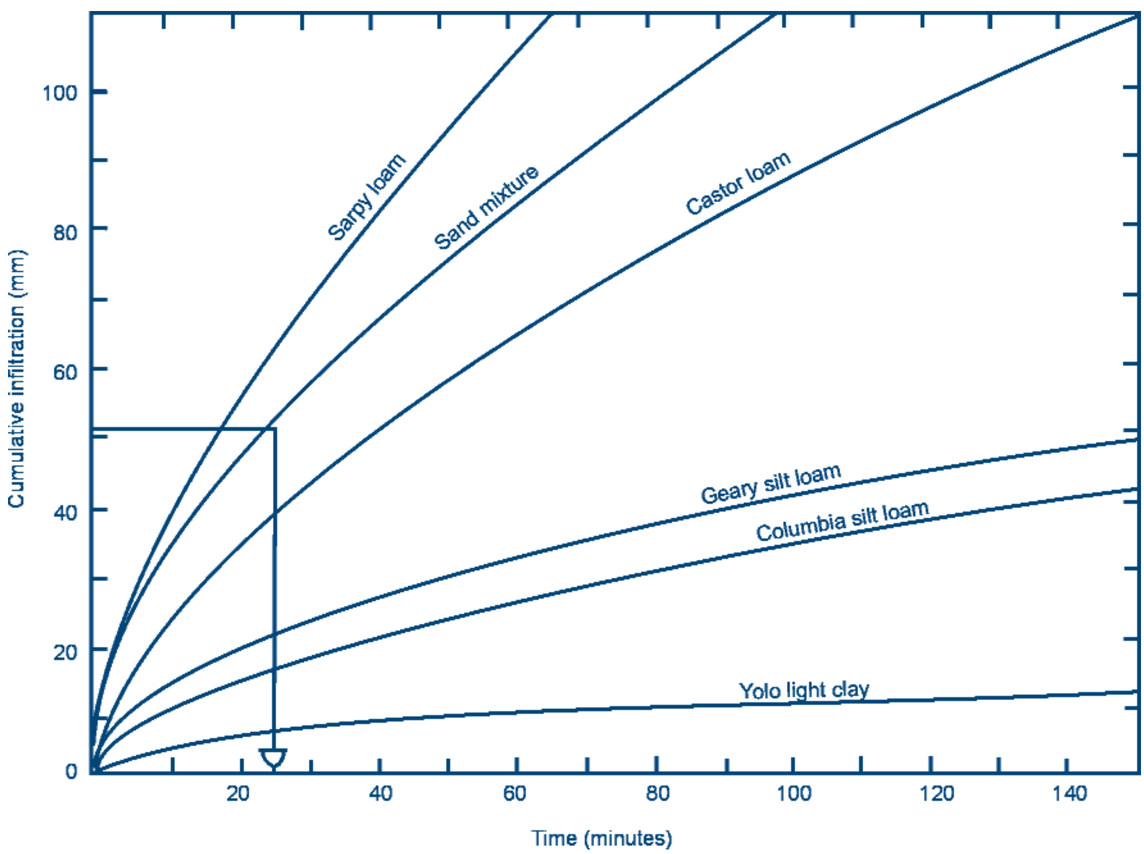
Referring to the cumulative infiltration curve (Figure 21), the contact time required for the application of 51.7 mm of water is estimated to be around 20-25 minutes.

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.

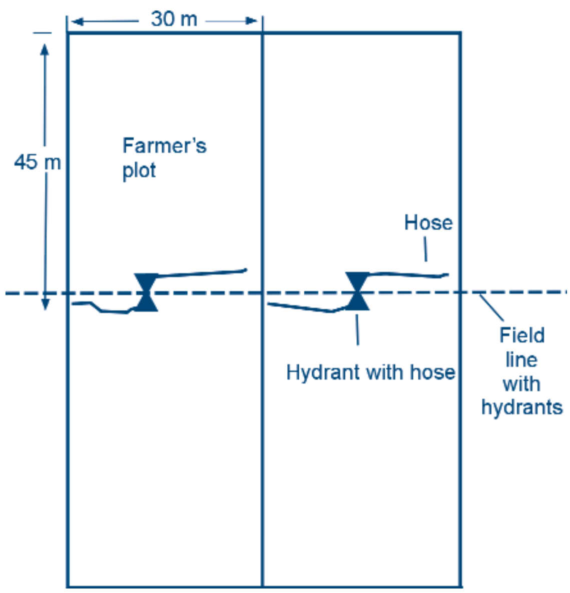
If six furrows are irrigated at the same time, the total duration per day will be 66/6 = 11 hours, which is more than the 10 hours which is the maximum duration per day that the farmers are willing to irrigated.

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

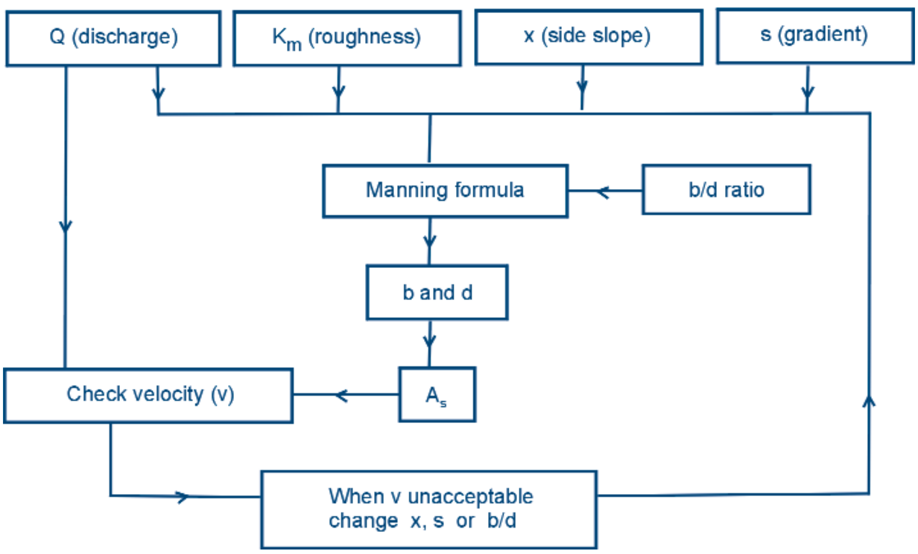
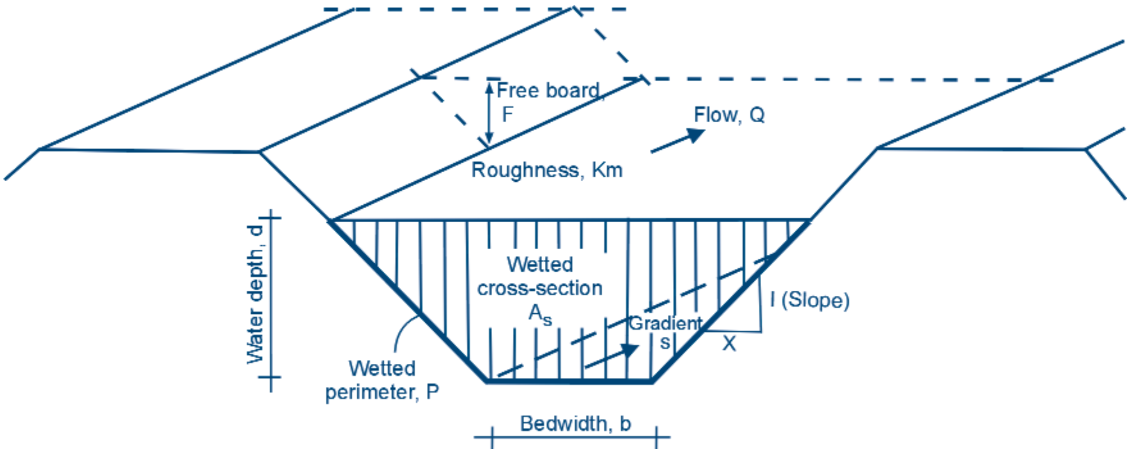


Figure 24
Canal parameters



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.

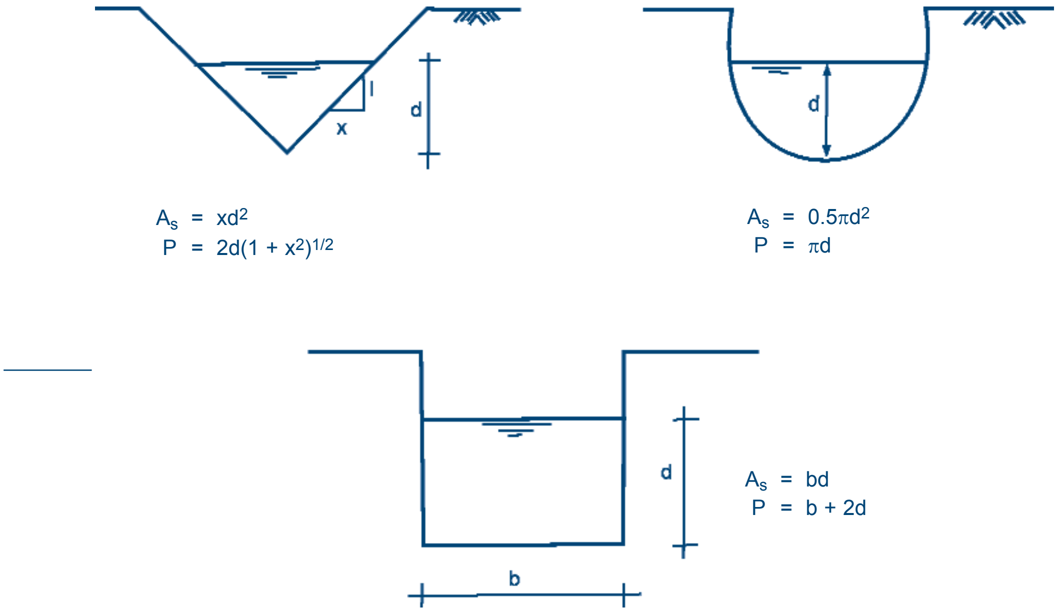
5.1.2. Factors affecting the canal discharge

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



$A_s = xd^2$
$P = 2d(1 + x^2)^{1/2}$

$A_s = 0.5\pi d^2$
$P = \pi d$

$A_s = bd$
$P = b + 2d$

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

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

Where:

V = Water velocity (m/sec)
g = Gravitational force (9.81m/sec^2)
l = 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

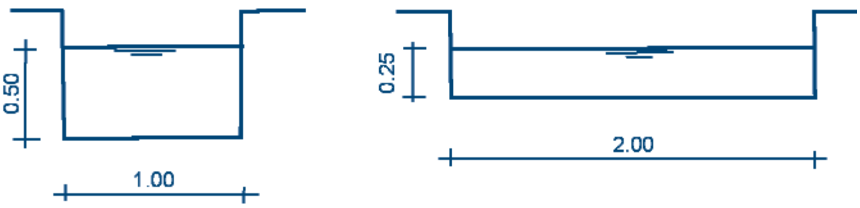
Typical canal side slopes

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 x 1.0 = 0.5 m^2
P = (2 x 0.5) + 1.0 = 2.0 m
R = 0.5 / 2.0 = 0.25 m

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

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

A_s = (1.5 x 0.475) + (0.475 x 0.475) = 0.94 m^2
P = 1.50 + 2 x 0.475 x $(1+1^2)^{½}$ = 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:

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

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

$$A_s = 2d^2 = 2 \times 0.30^2 = 0.18\ m^2$$

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

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.

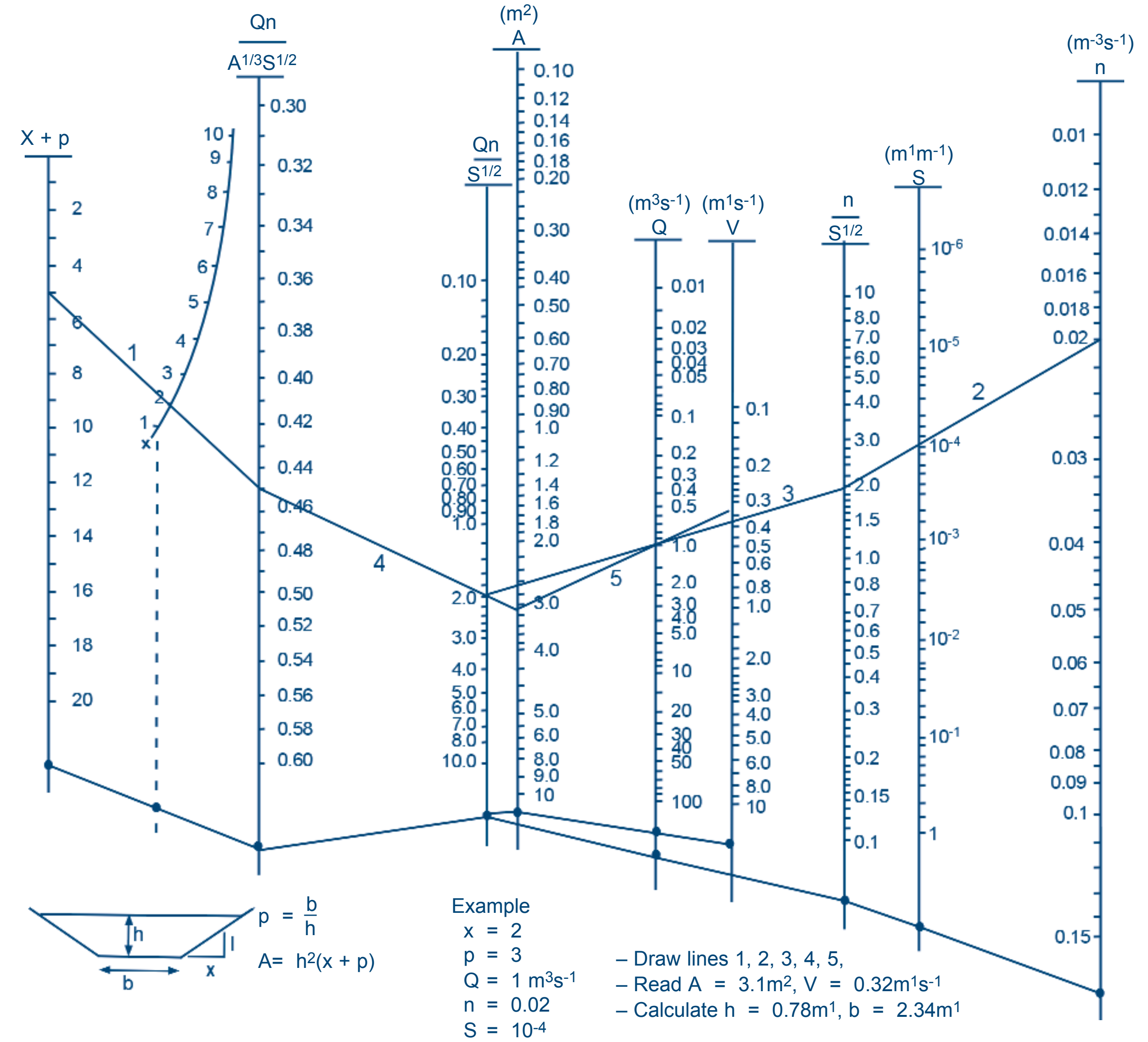
2. Calculate corresponding hydraulic gradients.
3. Divide network into sections of uniform slope (S) and discharge (Q).
4. Determine required design (maximum) discharge per section.
5. Select roughness coefficient (K_m or n)
   – side slopes
   – preferred minimum velocity and permissible maximum velocity
   – 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

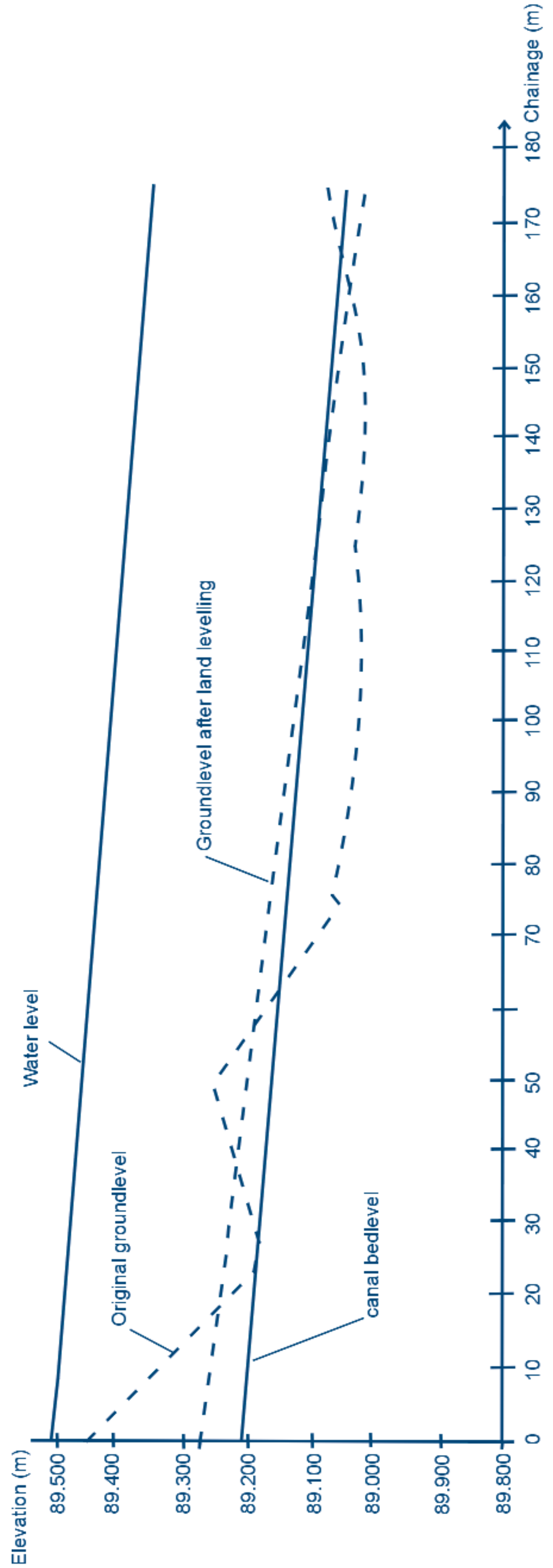
Figures 28, 29 and 20 show longitudinal sections of a field, secondary and conveyance canal respectively.

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.

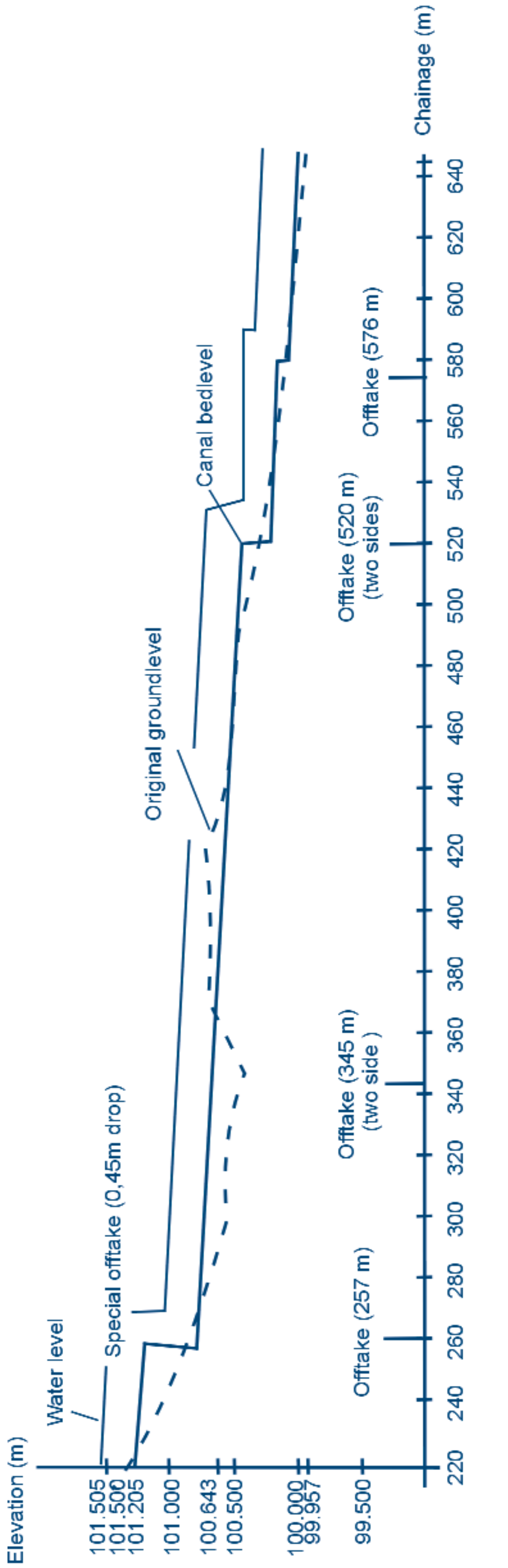
Figure 28
Longitudinal profile of a field or tertiary canal



FIELD CANAL AT CHAINAGE 834 OF SECONDARY CANAL

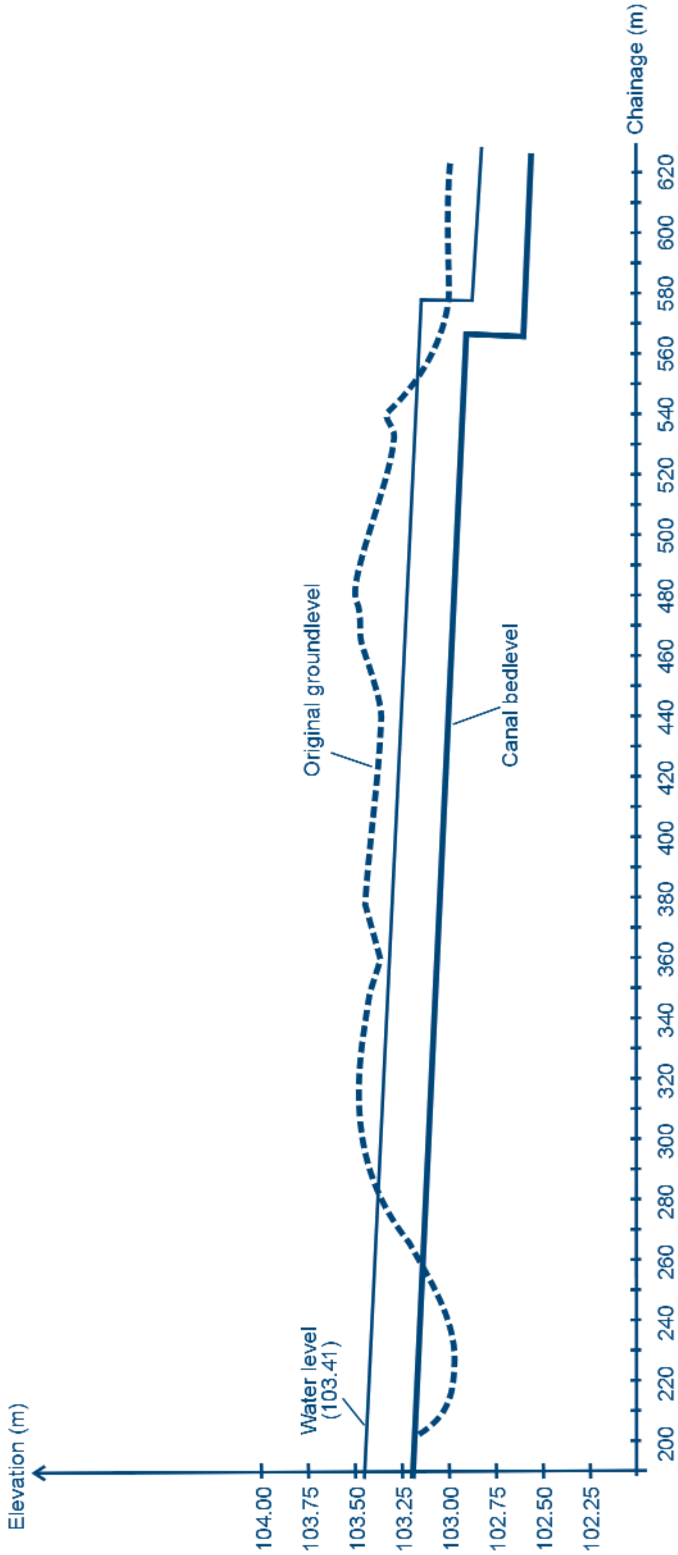
Chainage (m)	Ground level after levelling (m)	Canal bedlevel (m)
0	89.280	89.220
10	89.271	89.210
20	89.250	89.200
25	89.248	89.195
30	89.240	89.190
40	89.225	89.180
50	89.210	89.170
60	89.194	89.160
70	89.179	89.150
75	89.172	89.145
80	89.164	89.140
90	89.149	89.130
100	89.134	89.120
110	89.118	89.110
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

Figure 29
Longitudinal profile of a secondary or main canal



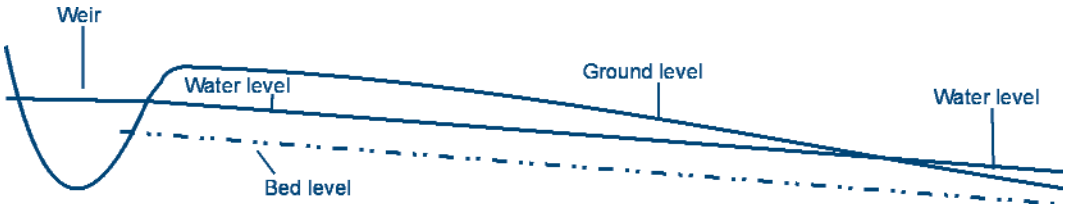
Chainage (m)	Original groundlevel (m)	Canal bedlevel (m)
245	100.871	101.205
257		101.093
270	100.621	100.643 100.630
295	100.468	100.605
320	100.444	100.580
345	100.814	100.555
370	100.610	100.530
395	100.591	100.505
420	100.641	100.480
445	100.485	100.455
470	100.445	100.430
495	100.370	100.405
520	100.254	100.380 100.130
576	100.034	100.074
580		100.070 100.020
644		99.957

Figure 30
Longitudinal profile of a conveyance canal



Chainage (m)	Original groundlevel (m)	Canal bedlevel (m)
206.50	103.144	103.110
226.50	102.856	103.090
246.50	102.893	103.070
266.50	103.030	103.050
286.50	103.238	103.030
306.50	103.423	103.010
326.50	103.432	102.990
346.50	103.316	102.970
366.50	103.285	102.950
386.50	103.363	102.930
406.50	103.326	102.910
426.50	103.278	102.890
446.50	103.255	102.870
466.50	103.359	102.850
486.50	103.408	102.830
506.50	103.516	102.810
526.50	103.217	102.790
546.50	103.187	102.770
566.50	102.947	102.750
570.00		102.740
586.50	102.871	102.397 102.380
606.50	102.870	102.360
626.50	102.879	102.340

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.

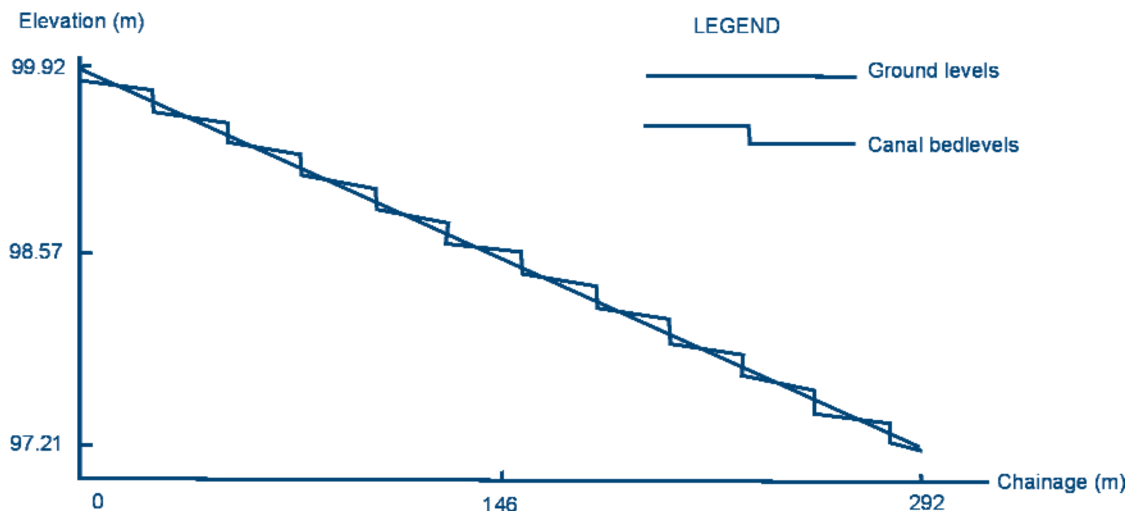
A problem often encountered is that field canals in irrigation schemes lack command, making siphoning onto adjacent land difficult or even impossible. One main reason for the lack of command is the use of the original ground level to site the drops. Once the canal bed level has reached the original ground 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.

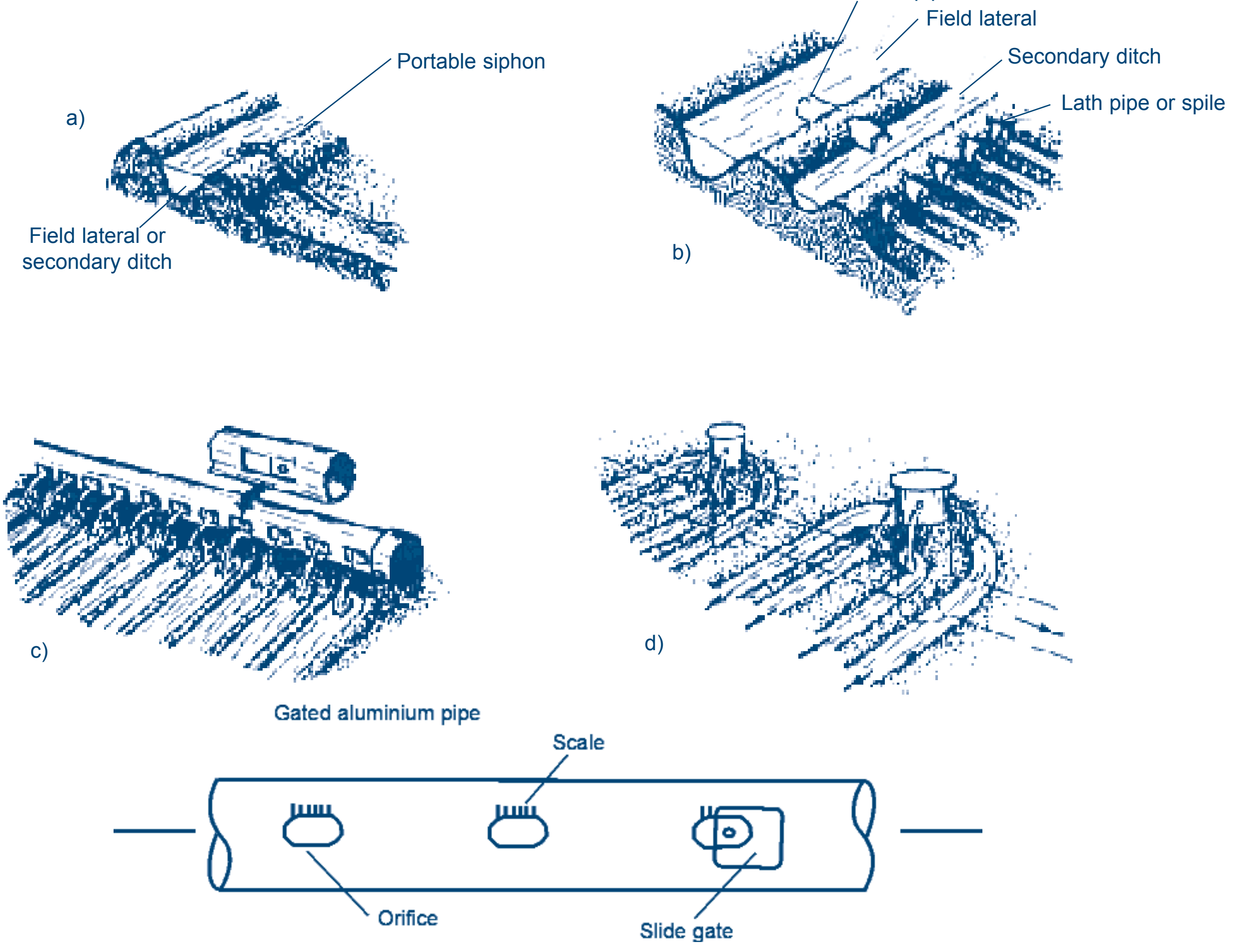
5.2. Design of pipelines

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 x 1 0^{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 x (20/100) = 0.32 m

Figure 34

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

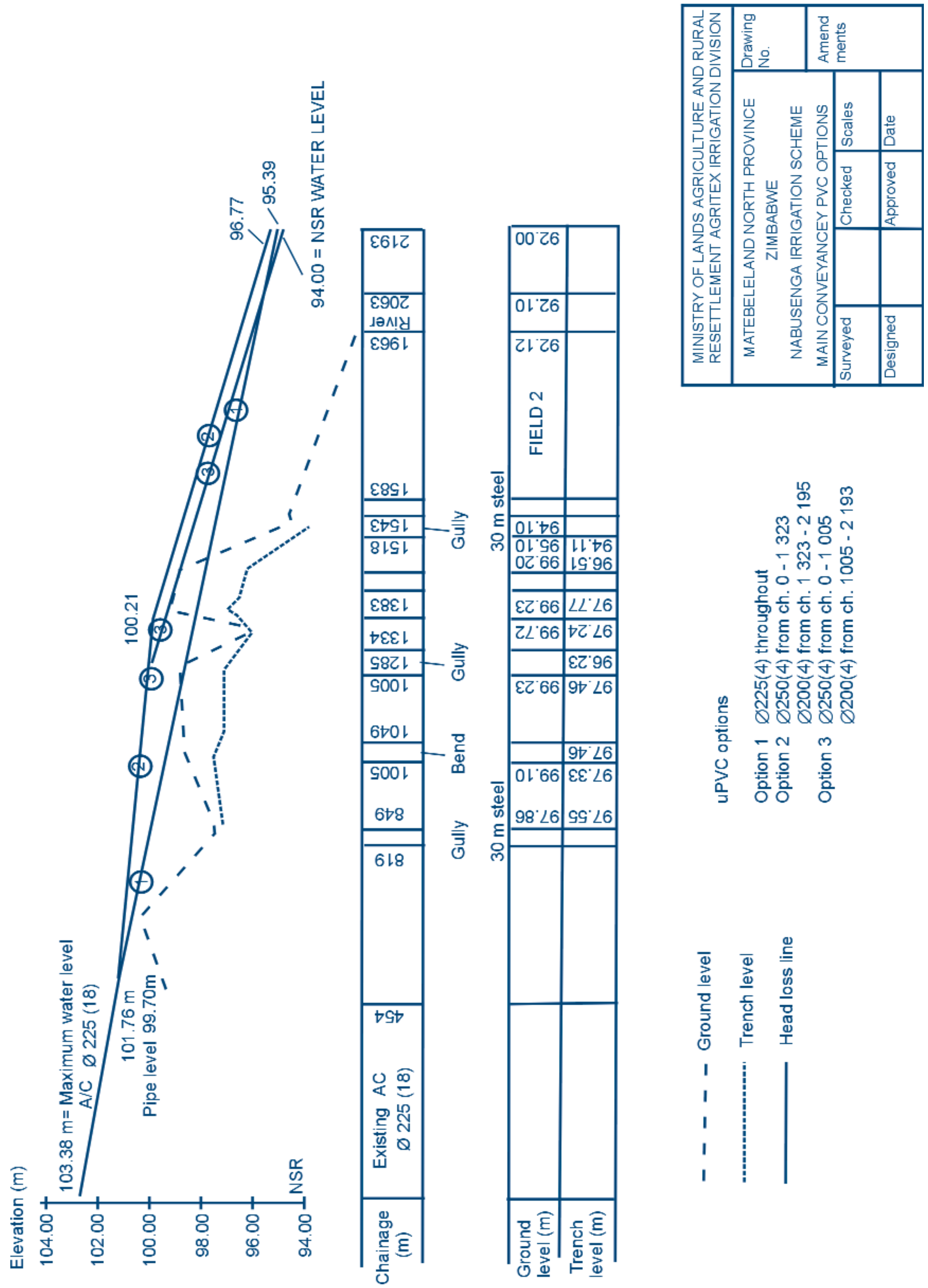


Figure 35
Friction loss chart for AC pipes (Class 18)

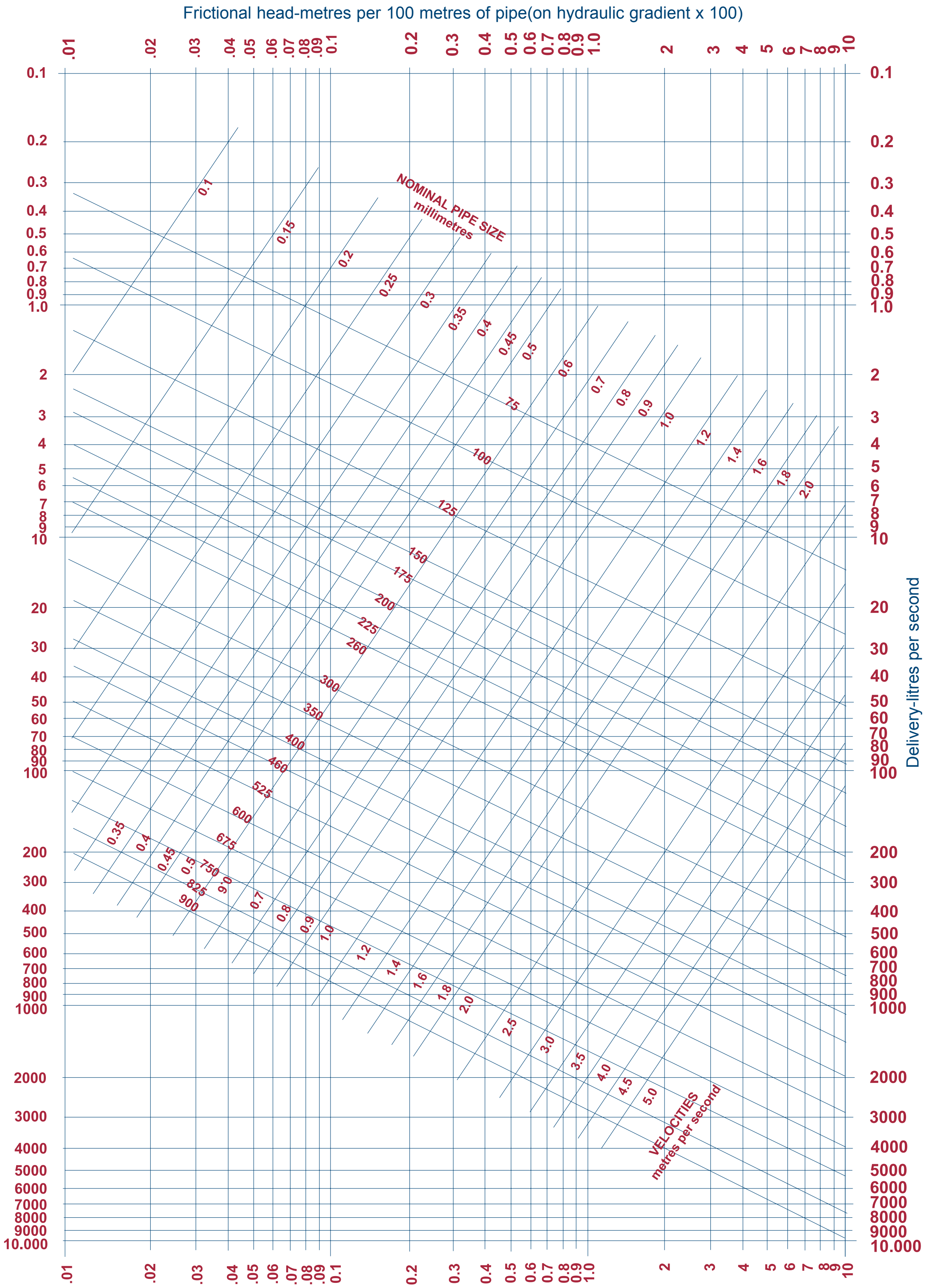
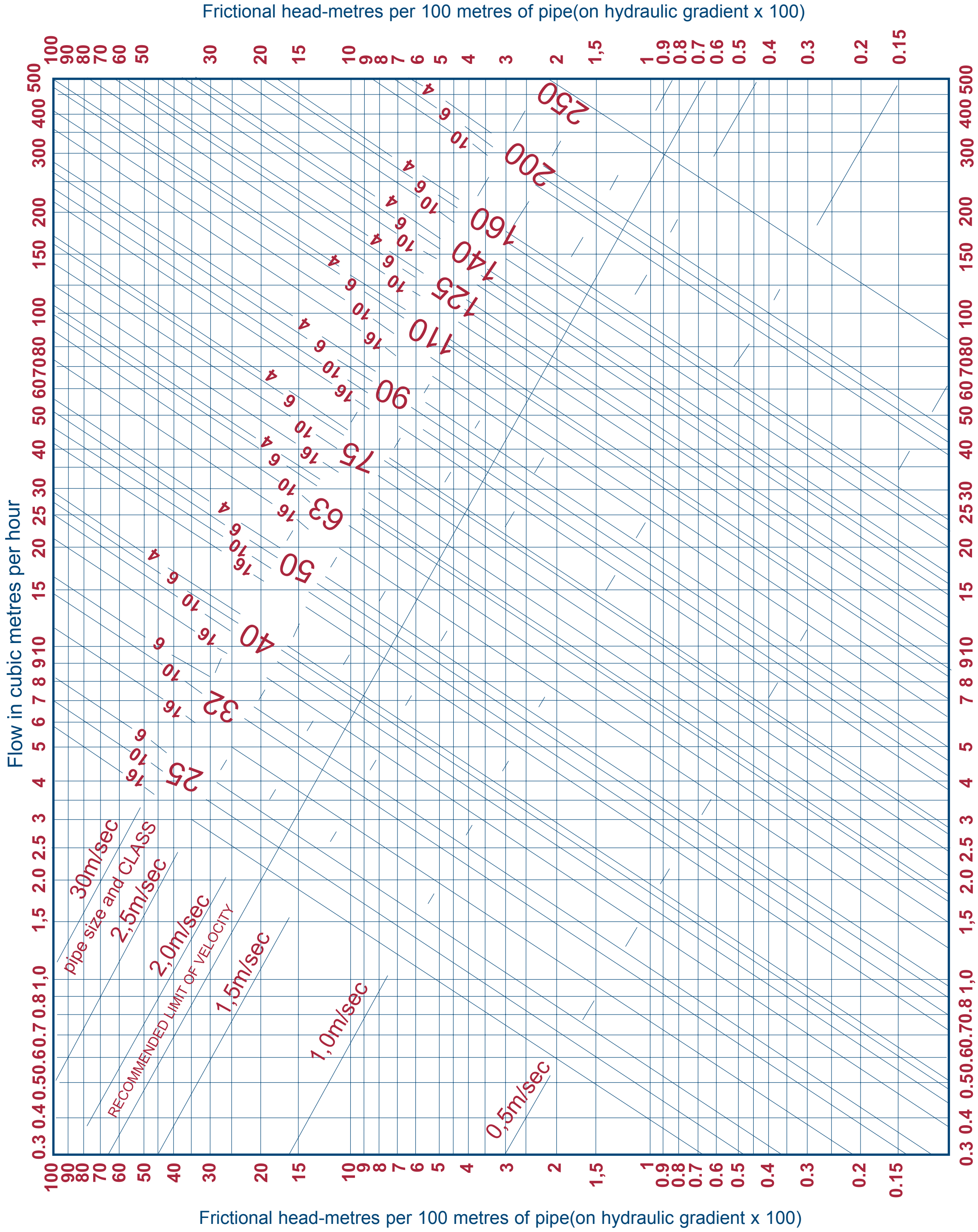


Figure 36

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:

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

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

Chapter 6
Hydraulic structures

Hydraulic structures are installed in open canal irrigation networks to:

- Control and measure discharge
- Control water levels for command requirements
- Dissipate unwanted energy
- Deliver the right volume of water to meet crop water requirements
- Incorporate recycled tail water, if available

The most common structures are:

a. Headworks for river water offtake

b. Night storage reservoirs

c. Head regulators

d. Cross regulators

e. Drop structures

f. Tail-end structures

g. Canal outlets

h. Discharge measurement structures

i. Crossings, like bridges, culverts, inverted siphons

Depending on the size and complexity of the irrigation scheme, some or all of the above-mentioned structures could be incorporated in the design.

6.1. Headworks for river water offtake

Abstraction and/or diversion of water from its source to the scheme is often difficult and can be quite costly, depending on its complexity. Figure 37 presents a sketch of schemes irrigated from different water sources.

Figure 37
Schemes irrigated from different sources (Source: FAO, 1992)

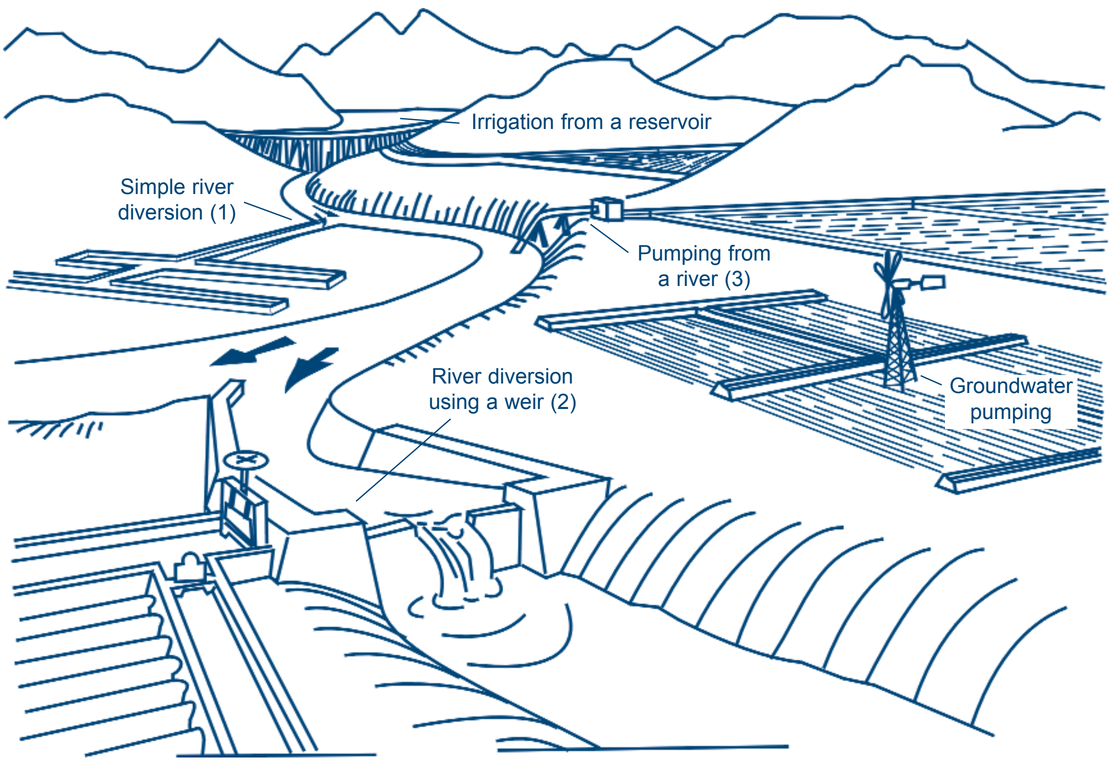
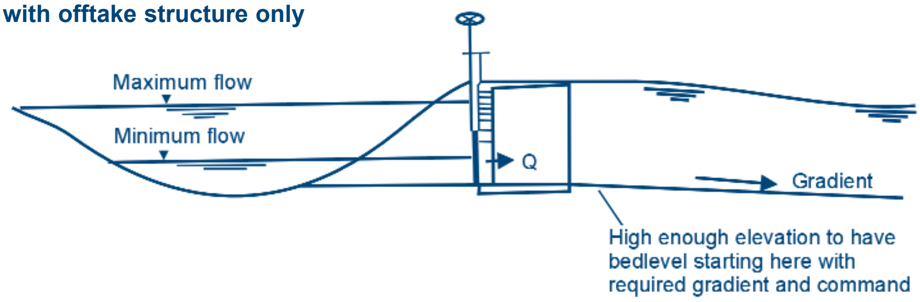


Figure 38
Headwork with offtake structure only



The function of a headwork is to divert the required amount of water at the correct head from the source into the conveyance system. It consists of one or more of the following structures:

- ❖ Offtake at the side of the river
- ❖ Regulating structure across the river or part of it
- ❖ Sediment flushing arrangement

This section concentrates mainly on the headworks for direct river offtake and offtakes using a weir. Some attention is paid to important dam and reservoir aspects, such as the outlet pipe diameter. However, for detailed dam design, the reader is referred to other specialized literature.

6.1.1. Headwork for direct river offtake

In rivers with a stable base flow and a high enough water level throughout the year in relation to the bed level of the intake canal, one can resort to run-off-river water supply (Figure 38 and Example 1 in Figure 37). A simple offtake structure to control the water diversion is sufficient.

The offtake should preferably be built in a straight reach of the river (Figure 39). When the water is free from silt, the centre line of the offtake canal could be at an angle to the centre line of the parent canal. When there is a lot of silt in the system, the offtake should have a scour sluice to discharge sediments or should be put at a 90° angle from the parent canal.

If it is not possible to build the offtake in a straight reach of the river, one should select a place on the outside of a bend, as silt tends to settle on the inside of bends. However, erosion usually takes place on the outside of the bend and therefore protection of the bank with, for example, concrete or gabions might be needed. The offtake can be perpendicular, at an angle or parallel to the riverbank, depending on site conditions, as illustrated in Figure 40.

Figure 39
Offtake possibilities in straight reach of river

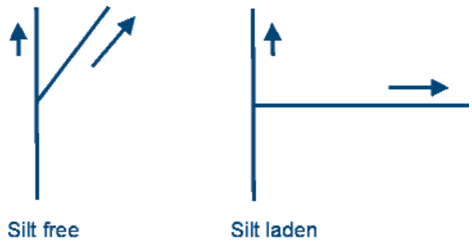


Figure 40
Possible arrangements for offtakes based on site conditions

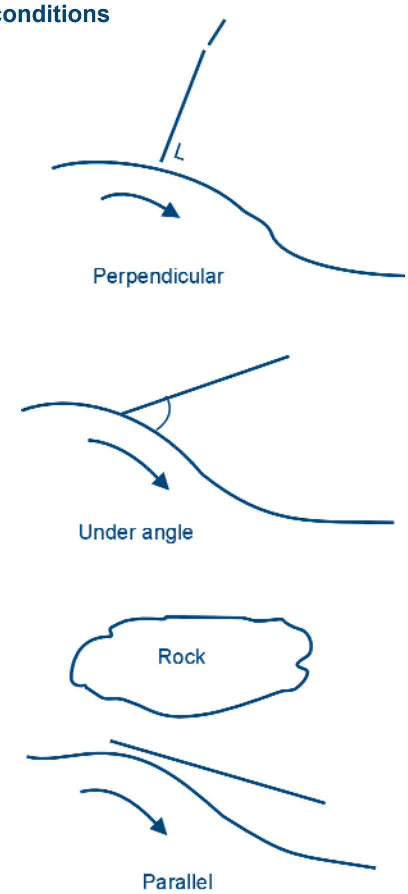
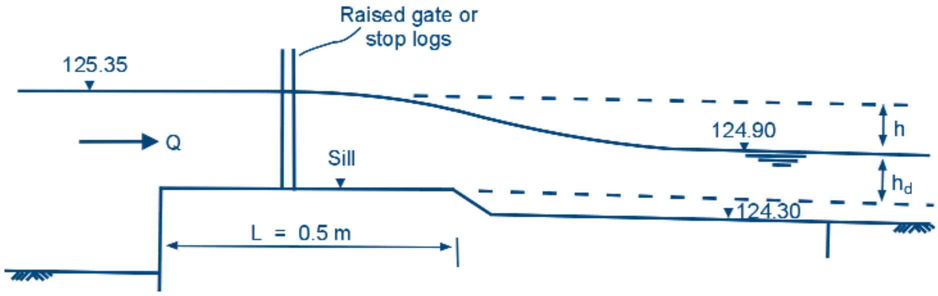


Figure 41
An example of an intake arrangement of a headwork



The functions of the offtake structures are:

- To pass the design discharge into the canal or pipeline
- To prevent excessive water from entering during flood

Considering these functions, the most important aspect of the structure is the control arrangement, which can be a gate, stop logs, or other structures. When the gate is fully opened, the intake behaves like a submerged weir (Figure 41) and its discharge is given by equation 23.

Equation 23

$$Q = C \times B(h + h_d)^{2/3}$$

Where:

Q	=	Discharge in intake (m^3/sec)
C	=	Weir coefficient
B	=	Width of the intake (m)
h	=	Difference between river water level and canal design water level (m)
h_d	=	Difference between canal design water level and sill level of the intake (m)

In some instances, the base flow water level fluctuates greatly over the year and the water level can become so low that the gate opening to the offtake structure will be at a higher elevation than the normal base flow water level. To abstract the required discharge in these situations, one could consider the options below:

- Select an offtake site further upstream. However, site conditions, the increased length of the conveyance canal, and other factors have to be considered carefully.
- Build a cheap temporary earthen dam and temporary diversion structure. This method is especially suitable in unstable rivers, where high expenses for a permanent structure are not warranted because of the danger of the river changing its course.
- Construct a permanent diversion dam or structure (weir or gate) across the river, where the design elevation of the weir should relate to the design water level in the conveyance canal, similar to the previous example.

6.1.2. River offtake using a weir

Figure 42 shows an example of a river diversion structure, in this case a weir (Example 2 in Figure 37).

Example 11

A discharge of 1.25 m^3/sec has to be abstracted from a river, into an open conveyance canal. The base flow water level of the river is 125.35 m. The design water level in the canal is 124.90 m and the water depth is 0.60 m. The weir coefficient is 1.60. The width of the intake is 1.50 m and the length of the weir is 0.50 m (Figure 41). What will be the sill level?

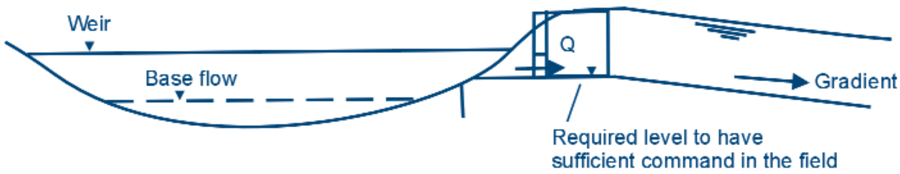
Q	=	1.25 m^3/sec
C	=	1.60
h	=	125.35 - 124.90 = 0.45 m
B	=	1.50 m

The next step would be to substitute these values in Equation 23:

$$1.25 = 1.60 \times 1.50(0.45 + h_d)^{2/3} \Rightarrow h_d = 0.20 \text{ m}$$

Thus the sill level should be at an elevation of 124.90 - 0.20 = 124.70 m

Figure 42
An example of a diversion structure



Structures constructed across rivers and streams with an objective of raising the water level are called cross regulators (see Section 6.4).

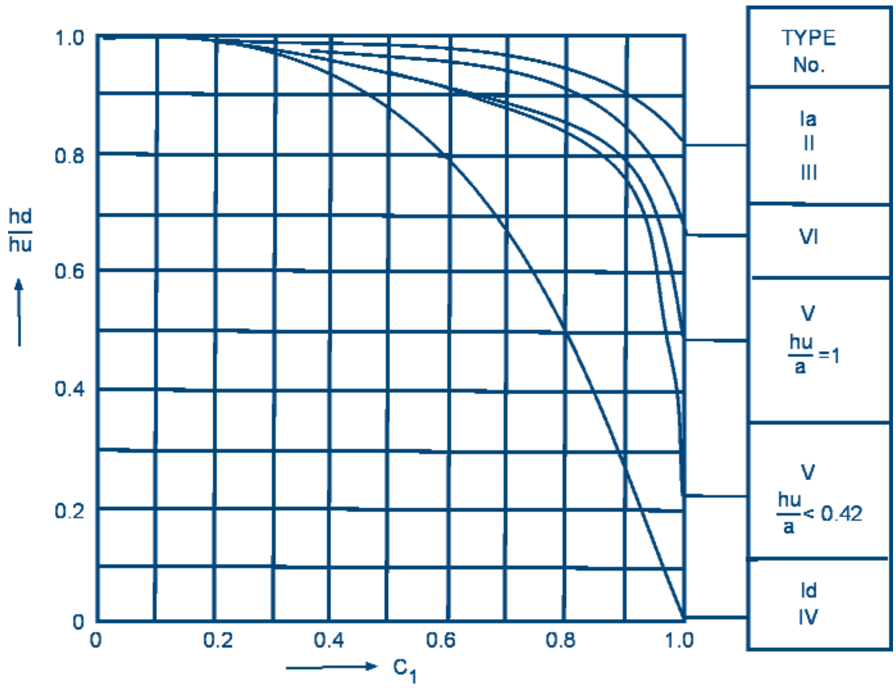
A weir should be located in a stable part of the river where the river is unlikely to change its course. The weir has to be built high enough to fulfil command requirements. During high floods, the river could overtop its embankments and change its course. Therefore, a location with firm, well-defined banks should be selected for the construction of the weir. Where possible, the site should have good bed conditions, such as rock outcrops. Alternatively, the weir should be kept as low as possible. Since weirs are the most common diversion structures, their design aspects will be discussed below.

Design of a weir for flood conditions

The weir height has to be designed to match the design water level in the conveyance canal. The weir length has to be designed to allow the design flood to safely discharge over the weir.

After deciding upon the location of the weir, the design flood, which is the maximum flood for which the weir has to be designed, has to be determined. If data are available, a flood with a return period of 50 or 100 years for example could be selected. If sufficient data are not available, flood marks could be checked, upon which the cross-sectional area can be determined and used, together with the gradient of the river, to calculate the flood discharge. Some formulae have been developed for this purpose, based on peak rainfall intensity and catchment characteristics.

Figure 43
C_1 coefficient for different types of weirs in relation to submergence, based on crest shape



The general equation for all weir types is:

Equation 24

$$Q = C_1 \times C_2 \times B \times H^{3/2}$$

Where:

Q	=	Discharge (m^3/sec)
C_1	=	Coefficient related to condition of submergence and crest shape (Figure 43)
C_2	=	Coefficient related to crest shape (Figure 44)
B	=	Weir length, i.e. the weir dimension across the river or stream (m)
H	=	Head of water over the weir crest (m)

Three general types of weirs are shown in Figure 45. The choice depends, among other aspects, on:

- ❖ Availability of local materials
- ❖ Available funds
- ❖ Local site conditions and floods

As an example, a broad-crested weir would be selected if gabion baskets were available as construction material (Figure 46). Gabion baskets are made of galvanized steel and look like pig netting (see Module 13). However, for the filling of gabion baskets large quantities of stones are required as well as plenty of cheap labour, since the construction method is labour intensive. Stone size is critical for a gabion weir, as large stones leave big spaces between them that allow water to quickly flow through, while too small stones may pass through the mesh.

Figure 44
C_2 coefficient for different types of weirs in relation to crest shape

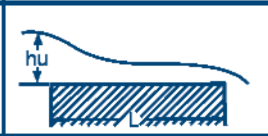
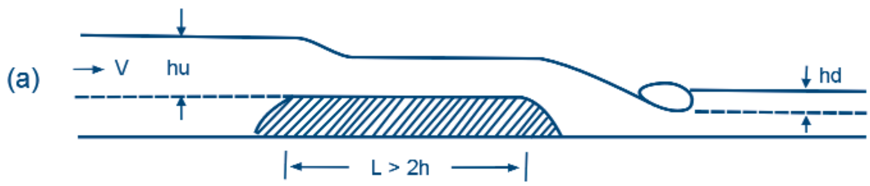
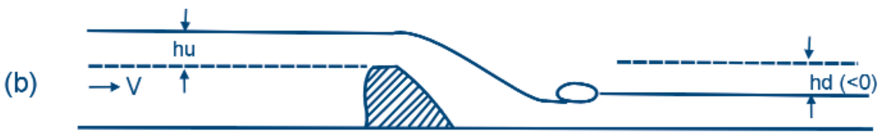
TYPE No	SHAPE	$\frac{L}{hu}$	C_2
Ia Ib Ic Id		≥ 3 2 1 ≤ 0.6	1.4 1.5 1.7 1.9
II		≥ 3	1.6
III		≥ 3	2.0
IV			1.9
V			2.2
VI			2.3
VII			1.75

Figure 45
Types of weirs



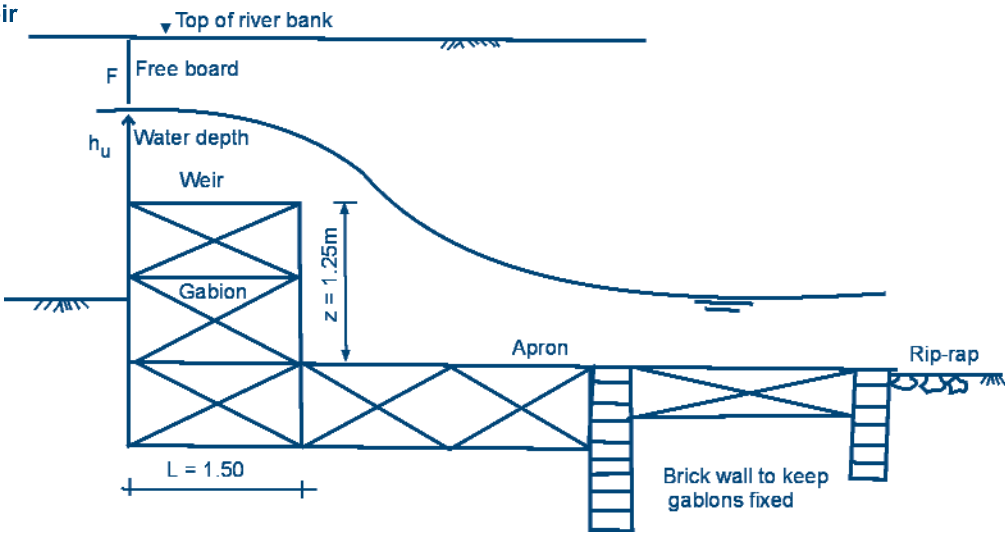
a. Broad-(long)crested weir



b. Short-crested weir

c. Sharp-crested weir

Figure 46
Gabion weir



Example 12

In Example 11 the weir coefficient C, which is the product of C_1 and C_2, was assumed to be 1.60. Can this be confirmed by calculating C_1 and C_2 respectively?

From Example 11 the difference between the water level in the river and the sill elevation can be calculated as follows:

h_u = 125.35 - 124.70 = 0.65m

The weir length L is 0.50 m, thus $\frac{L}{h_u}$ in Figure 44 is: $\frac{L}{h_u} = \frac{0.50}{0.65} = 0.77$

This relates to a weir type between 1c and 1d in Figure 44. By interpolation, C_2 is approximately 1.8.

The difference between the canal design water level and the sill elevation h_d = 0.20 m.

Thus $\frac{h_d}{h_u}$, which is the y-axis in Figure 43, is: $\frac{h_d}{h_u} = \frac{0.20}{0.65} = 0.31$

Using the curve for weir type 1b-d in Figure 43, gives a value for C_1 of approximately 0.9.

Thus $C = C_1 \times C_2 = 0.9 \times 1.8 = 1.62$, which is almost the same as the weir coefficient 1.60 used in Example 11.

Example 13

A broad-crested weir is to be constructed with gabion baskets. The top width L, which is the dimension of the weir in the direction of the river, is 1.50 m. There will be non-submerged conditions, which means that the water level downstream of the weir will be below the weir crest. The design discharge is 37 m³/sec. Due to local site conditions, the head of water over the crest should not exceed 0.75 m. The freeboard (F), which is the distance between the design level of the water and the top of the river bank is 0.70 m (Figure 46). What should be the weir length or the dimension of the weir across the river?

The first step is to determine the values of C_1 and C_2 from Figures 43 and 44 respectively:

h_d is the distance from the crest of the weir to the design water level downstream of the weir (Figure 41). Since there will be non-submerged conditions, h_d will be below the crest of the weir. This means that h_d is 0.

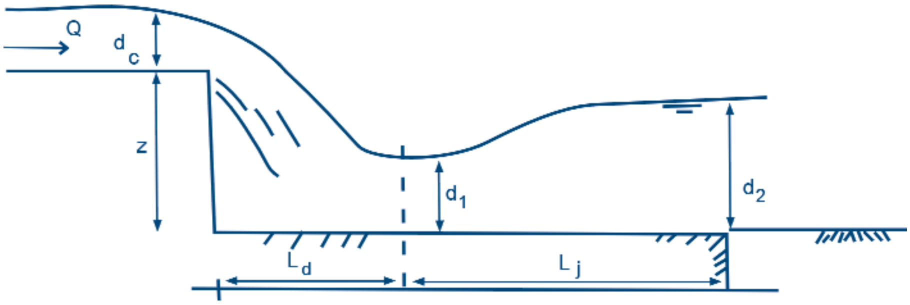
As a result $\frac{h_d}{h_u}$ = 0, thus C_1 = 1 (Figure 43)

L = 1.50 m and h_u = 0.75 m, thus $\frac{L}{h_u}$ = 2, which means that C_2 = 1.5, which is weir type 1b in Figure 44.

Substituting the above data in Equation 24 gives:

$37 = 1.0 \times 1.5 \times B \times 0.75^{3/2} \Rightarrow B = 38$ m.

Figure 47
Typical parameters used in the design of a stilling basin



The downstream side of the weir has to be protected using a stilling basin to dissipate the energy of the dropping water. It could be constructed using masonry, concrete, gabions or Reno mattresses.

Design of a stilling basin

The length of the stilling basin should be correctly determined in order to avoid bed scour and the subsequent undermining of the structure. The parameters used in the design of a stilling basin are shown in Figure 47.

The empirical formulae to use for the design of a stilling basin (apron) are:

Equation 25

$$D = \frac{q^2}{(g \times z^3)}$$

Equation 26

$$\frac{L_d}{z} = 4.30 \times D^{0.27}$$

Equation 27

$$\frac{d_1}{z} = 0.54 \times D^{0.425}$$

Equation 28

$$\frac{d_2}{z} = 1.66 \times D^{0.27}$$

Equation 29

$$L_j = 6.9 \times (d_2 - d_1)$$

Where:

D	=	Drop number (no limit)
q	=	Discharge per metre length of the weir (m^2/sec)
g	=	Gravitational force (9.81 m/sec^2)
z	=	Drop (m)
L_d	=	Length of apron from the drop to the point where the lowest water level d_1 will occur (hydraulic jump) (m)
d_1	=	Lowest water level after the drop (m)
d_2	=	Design water level after the apron (m)
L_j	=	Length of apron from the point of lowest water level to the end of the apron (m)

Example 14

A weir with a length B of 38 m across the river and a design discharge Q of 37 m^3/sec, has a design drop z of 1.25 m. What will be the apron length?

The unit discharge is $\frac{37}{38} = 0.974$ m^3/sec per metre length of weir.

Substituting this value and the drop z in Equations 25 to 29 for the design of a stilling basin dimension gives:

$$D = \frac{0.974^2}{(9.81 \times 1.25^3)} = 0.05$$

$$\frac{L_d}{1.25} = 4.30 \times 0.05^{0.27} \Rightarrow L_d = 2.40 \text{ m}$$

$$\frac{d_1}{1.25} = 0.54 \times 0.05^{0.425} \Rightarrow d_1 = 0.19 \text{ m}$$

$$\frac{d_2}{1.25} = 1.66 \times 0.05^{0.27} \Rightarrow d_2 = 0.92 \text{ m}$$

$$L_j = 6.9 \times (0.92 - 0.19) = 5.04 \text{ m}$$

Thus the total apron length is $(L_d + L_j) = 2.40 + 5.04 = 7.44$ m

Apron floors should have sufficient thickness to counter-balance the uplift hydrostatic pressure and should be sufficiently long to prevent piping action. This is responsible for the removal of the bed material from under the floor, thereby causing its collapse.

Bed material that allows uplift is liable to piping. Piping could be avoided by using sheet piling, which is a method whereby metal or wooden posts are driven vertically into the ground until they reach an impermeable sub-layer. However, this is expensive. Alternatively, horizontal, impermeable layers could be provided. By applying Lane's weighted-creep theory, which is an empirical, but simple and proven method, the length can be determined. This is defined by the following terms:

- The weighted-creep distance L_W of a cross-section of a weir or a dam is the sum of the vertical creep distances (steeper than 45°) plus one-third of the horizontal creep distances (Equation 30).
- The weighted-creep ratio is the weighted-creep distance (L_W) divided by the effective head on the structure, which in this case is the drop (z) (Equation 31).
- The upward pressure may be estimated by assuming that the drop in pressure from headwater to tail water along the line of contact of the foundation is proportional to the weighted-creep distance (Equation 32).

Figure 48 shows the different heights and lengths to be used in determining the weighted-creep ratios.

The weighted-creep distance is as follows:

Equation 30

$$L_w = h_1 + h_2 + h_3 + h_4 + h_5 + \frac{1}{3}(W_1 + L_1 + L_2 + W_2)$$

The weighted-creep ratio is formulated as follows:

Equation 31

$$\frac{L_w}{z}$$

For designing the floor thickness, the uplift pressure P has to be estimated. The uplift pressure at point B of Figure 48 is calculated as follows:

Equation 32

$$P_b = z - \frac{h_1 + h_2 + h_3 + \frac{1}{3}(W_1 + L_1)}{L_w}$$

The thickness of a floor can be determined using the following equation:

Equation 33

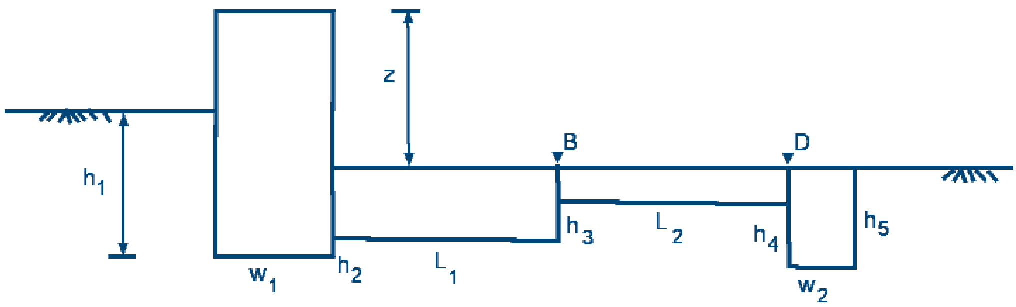
$$t = \text{Uplift pressure x} \frac{\text{Unit weight of water}}{\text{Unit weight of submerged mansonry}}$$

The recommended weighted-creep ratios are given in Table 25.

Table 25
Weighted-creep ratios for weirs, depending on soil type

Bed materials	Weighted-creep ratio
Medium sand	6
Coarse sand	5
Fine gravel	4
Medium gravel	3.5
Coarse gravel	3.0
Boulders with gravel	2.5
Medium clay	2

Figure 48
Schematic view of a weir and apron



Thus, for a given type of bed material, the weighted-creep ratio can be brought within the recommended value by selecting a suitable combination of the floor lengths and vertical cut-offs, as given in Table 25. The materials being used often determine the cut-off walls, though the apron length should also be long enough to dissipate the energy.

Example 15

A masonry weir (Figure 49) has to be built in a coarse sand bed material. The proposed dimensions are as follows (Figure 48):

z	=	1.25 m	h_5	=	1.00 m
h_1	=	1.00 m	W_1	=	1.50 m
h_2	=	0.25 m	L_1	=	3.50 m
h_3	=	0.50 m	L_2	=	3.00 m
h_4	=	0.50 m	W_2	=	1.00 m

Would this structure be safe against piping?

The weighted-creep distance L_w is:

$$L_w = 1.00 + 0.25 + 0.50 + 0.50 + 1.00 + \frac{1}{3}(1.50 + 2.50 + 1.00) = 6.25 \text{ m}$$

The weighted creep ratio is:

$$\frac{L_w}{z} = \frac{6.25}{1.25} = 5.0$$

Comparing this value to the recommended one given for coarse sand in Table 25 shows that the selection of the structure dimensions is acceptable and that no piping should be expected.

Example 16

Using the same data as given in the previous example, calculate the floor thickness at points B and D (Figure 48).

Assuming a masonry floor, the unit dry weight can be taken as 2 400 kg/m^3, while the unit weight of water is 1 000 kg/m^3. The submerged weight of masonry is the difference between the unit dry weight of the masonry minus the unit weight of water, thus: (2 400 – 1 000) = 1 400 kg/m^3.

The uplift pressure at point B is:

$$P_b = 1.25 - \frac{1.00 + 0.25 + 0.50 + \frac{1}{3}(1.50 + 3.50)}{6.25} = 0.70 \text{ m}$$

Thus the required floor thickness at point B is:

$$t = 0.70 \times \frac{1\,000}{1\,400} = 0.50 \text{ m}$$

Similarly for point D, the uplift pressure is:

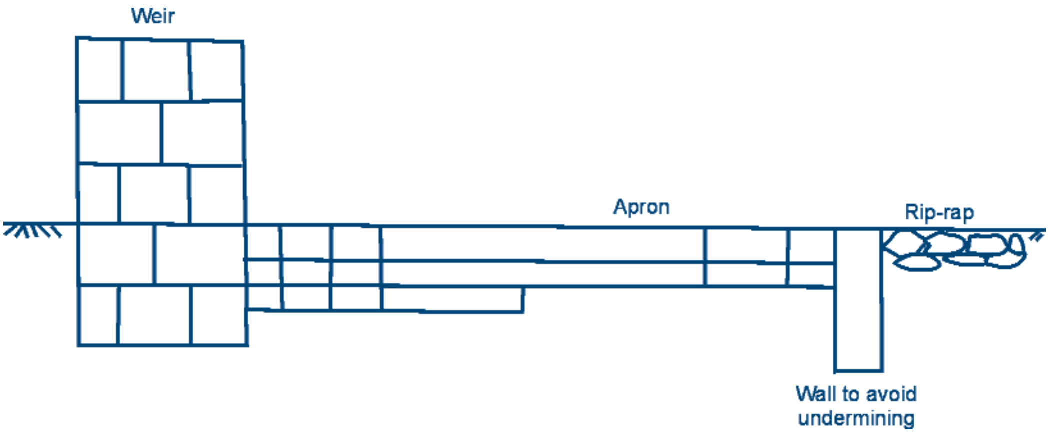
$$P_b = 1.25 - \frac{1.00 + 0.25 + 0.50 + 0.50 + \frac{1}{3}(1.50 + 3.50 + 3.00)}{6.25} = 0.46 \text{ m}$$

The required floor thickness at point D is:

$$t = 0.46 \times \frac{1\,000}{1\,400} = 0.33 \text{ m}$$

As there is a drop in pressure from the head to the tail of the structure, the floor thickness can be less at point D, which is further away from the head of the structure.

Figure 49
Masonry weir and apron



6.1.3. River offtake using a dam

Design of dams will not be discussed in this Irrigation Manual. For this the reader is referred to other specialized literature available. In this section only those aspects of dams that affect irrigation designs will be discussed. A typical dam cross-section is given in Figure 50.

Lowest drawdown level

The lowest drawdown level is the minimum water level in the reservoir that can be abstracted into the irrigation system. The water remaining below the lowest drawdown level is called dead storage. This could be used as drinking water for human beings and animals. The lowest drawdown level often coincides with the latter part of the dry season, when water requirements are high. Even at the lowest draw down level one would like to abstract the design discharge.

A simple relation between the discharge, the difference in height between lowest drawdown level and outlet pipe invert and outlet pipe diameter can be obtained using Equation 34:

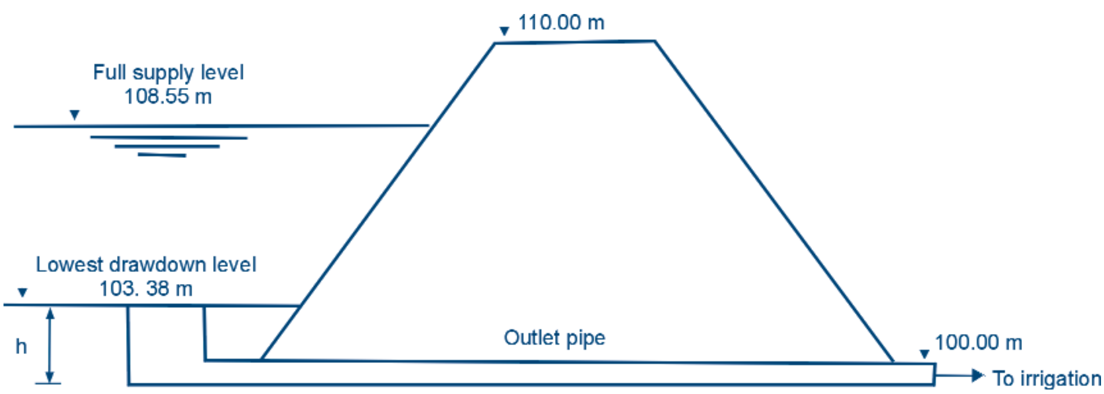
Equation 34

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

Where:

Q	=	Discharge (m^3 /sec)
C	=	Discharge coefficient, approximately 0.5
A	=	Cross-sectional area of pipe (m^2)
g	=	Gravitational force (9.81 m/sec^2)
h	=	Available head (m)

Figure 50
Dam cross-section at Nabusenga



Example 17

The lowest drawdown level of Nabusenga dam is 3.38 m above the outlet pipe (Figure 50). The pipe has a diameter of 225 mm. Would this outlet pipe be able to deliver a discharge of 78.3 l/sec?

Substituting these data in Equation 34 gives:

$$78.3 \times 10^{-3} = 0.5 \times \left(\frac{1}{4} \times \pi \times 0.225^2\right) \times (2 \times 9.81 \times h)^{1/2} \Rightarrow h_{required} = 0.79 \text{ m}$$

The minimum required head of 0.79 m is much less than the $h_{available}$ of 3.38 m. Thus no reduction in discharge should be expected when the water level in the dam reaches its minimum level.

Example 18

Using the data from Example 17, what would be the outlet pipe diameter if a discharge of 78.3 l/sec has to be abstracted at a minimum available head of 1.25 m instead of 3.38 m?

Substituting the data in Equation 34 gives:

$$78.3 \times 10^{-3} = 0.5 \left(\frac{1}{4} \times \pi \times d^2\right) \times (2 \times 9.81 \times 1.25)^{1/2} \Rightarrow d = 200 \text{ mm}$$

Friction losses in outlet pipe

There should be sufficient head available to overcome friction losses in the outlet pipe as well as in the conveyance pipeline in case the conveyance system is a pipe and not an open canal. The available head refers to the water height above the outlet pipe. The friction losses (HL) through a pipe can be calculated using the Hazen-Williams equation, which was given in Equation 21 (see Section 5.2.2):

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

The value of the material constant C depends on the smoothness of the material (Table 24). If the pipe size is small in relation to the discharge, high friction losses are expected, which means that the water head above the pipe outlet should be large. In such a case, the discharge would be reduced at a lower drawdown level than for a larger pipe.

6.1.4. Scour gates for sedimentation control

Many rivers carry substantial sediment loads, especially during the rainy season, in the form of sand, silt, weeds, moss and tree leaves. Approximately 70% of all suspended and bed load sediments travel in the lower 25% of the flow profile.

Example 19

The Nabusenga dam has a 70 m long AC outlet pipe with a diameter of 225 mm. What are the friction losses for discharges of 78.3 l/sec and 32.6 l/sec, including 20% extra for minor losses?

Substituting the above data in Equation 21 gives for Q = 78.3 l/sec:

$$Hf_{100} = \frac{1.22 \times \left[\frac{78.3}{140}\right]^{1.852}}{225^{4.87}} = 1.46 \text{ m per 100 m or HL} = 1.02 \text{ m per 70 m} + 20\% = 1.22 \text{ m}$$

For Q = 32.6 l/sec:

$$Hf_{100} = \frac{1.22 \times \left[\frac{32.6}{140}\right]^{1.852}}{225^{4.87}} = 0.29 \text{ m per 100 m or HL} = 0.20 \text{ m per 70 m} + 20\% = 0.24 \text{ m}$$

Since the minimum head available is 3.38 m (Example 17 and Figure 50) no reduction in discharge is expected, even if the full discharge of 78.3 l/sec has to be delivered.

Example 20

What are the friction losses for a discharge of 78.3 l/sec through a 70 m long galvanized steel pipeline with a 200 mm diameter? The minimum available head is 1.25 m.

$$Hf_{100} = \frac{1.22 \times \left[\frac{78.3}{140}\right]^{1.852}}{200^{4.87}} = 3.44 \text{ m per 100 m or HL} = 2.41 \text{ m per 70 m} + 20\% = 2.89 \text{ m}$$

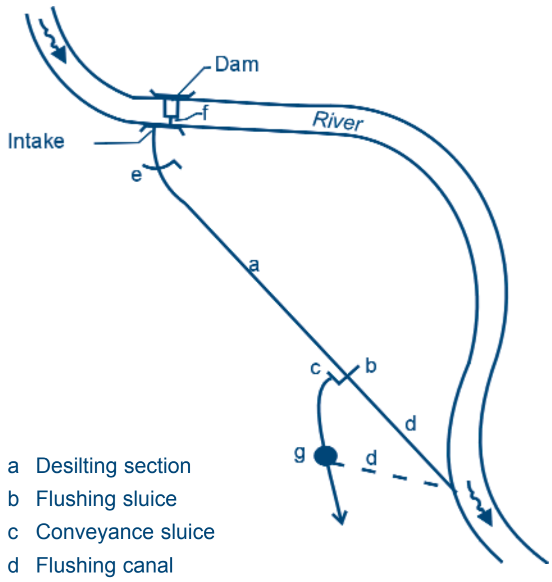
Already at 1.64 m (2.89 - 1.25 m) above the minimum drawdown level, the discharge will be reduced as the water head is insufficient to overcome the friction losses of the design discharge.

It should be noted that with aging the C for galvanized steel pipes drops to 80. This will further increase the head losses in the pipe.

While suspended silt can be beneficial to the scheme by adding nutrients to the farmland, coarse sediments usually cause problems once they are blocked by a weir or other diversion structure. Headworks have to be adapted to these sediment loads to avoid silting of canals and structures. A properly-designed intake should divert only the relatively clean upper part of the water flow into the canal and dispose of the lower part down the river. A sluice should therefore be incorporated into the diversion structure design. It should be placed in line with the weir near the canal intake (Figure 51). Its seal level is generally placed at the river bed level while the floor to the intake gate should be located higher (Figure 52).

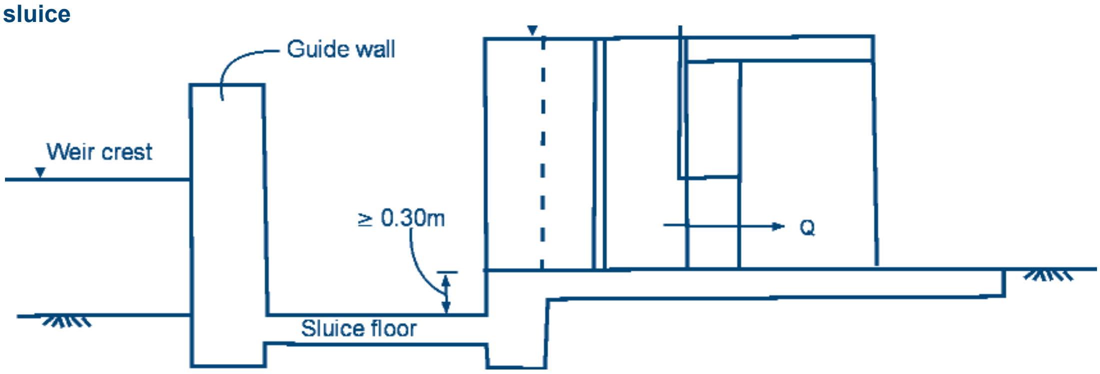
The control arrangement in the scour sluice generally consists of a series of stop logs (timber, concrete) or a sluice gate. This arrangement allows the water to be raised when there are very few or no sediments in the water. During the flood season, the sluice is permanently open or opened at regular intervals so that depositions of sediments can be flushed away. The guide wall prevents lateral movement of sediments deposited in front of the weir and separates the flow through the sluice and the flow over the weir.

Figure 51
Gravity offtake with diversion dam



a Desilting section
b Flushing sluice
c Conveyance sluice
d Flushing canal
e Gravel and sand trap
f Flushing gate
g Regulation basin

Figure 52
Scour sluice



6.2. Night Storage Reservoirs (NSR)

Night storage reservoirs (NSR) store water during times when there is abstraction from the headwork but no irrigation. Depending on the size of the scheme one could construct either one reservoir, located at the top of the scheme as shown in Figure 19, or more than one to command sections of the scheme they are serving.

Night storage reservoirs could be incorporated in the design of a scheme when:

i. The distance from the water source to the field is very long, resulting in a long time lag between releasing water from the source and receiving it in the field.

ii The costs of constructing the conveyance canal or pipeline are very high because of the large discharge it has to convey without a NSR. Incorporating a reservoir means that a smaller size conveyance system can be built.

iii. The discharge of the source of the water is smaller than would be required for the area without storing the water during times of no irrigation.

The following examples illustrate scenarios i to iii.

Example 21

A discharge of 78.30 l/sec has to be delivered through a 7 km long canal with a wetted cross-section of 0.19 m². When should the headwork gate be opened, if water has to reach the field at 07.00 hours?

The water velocity (V) was given by the Continuity Equation 12:

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

Substituting the values in Equation 12 gives:

$$V = \frac{0.0783}{0.19} = 0.41 \text{ m/sec}$$

The time (t) it takes for the water to reach the top of the field is given:

$$t = \frac{\text{distance}}{\text{velocity}} = \frac{7\ 000}{0.41} = 17\ 073 \text{ seconds or 4 hours and 45 minutes.}$$

This would mean that the head gate should be opened at 02.15 hours if irrigation is to start at 07:00 hours. If this is unsuitable for proper management, one should incorporate a night storage reservoir.

Example 22

Water is abstracted from a river with a base flow of less than 78.3 l/sec (required if the delivery period is 10 hours per day), but more than 32.6 l/sec (required if the delivery period can take place 24 hours per day). If abstraction only takes place during daytime the area under irrigation would have to be reduced. Determine the size of the reservoir for the scheme in order to be able to irrigate the whole area.

With a night storage reservoir, one could collect the required discharge of 32.6 l/sec from the water source. At an abstraction rate of 32.6 l/sec, the volume of water accumulated during the 14 hours when there is no irrigation should be stored in the night storage reservoir. Thus the volume (V) to be stored is:

$$V = \frac{32.6 \times 3\ 600 \times 14}{1\ 000} = 1\ 643 \text{ m}^3$$

If 20% is added to cater for evaporation and seepage losses, a night storage reservoir with a capacity of 1 970 m³ could be proposed.

Example 23

If the friction losses in a conveyance or supply pipeline, delivering 78.3 l/sec for a period of 10 hours per day, are kept at around 0.30 m per 100 m, then a 300 mm diameter AC pipe (Class 18) could be used (Figure 35). What pipe size could be used, if a night storage reservoir were built allowing a water flow 24 hours per day?

If abstraction could take place for 24 hours per day, then the discharge would reduce to 32.6 l/sec (= 78.3/(24/10)) and subsequently a pipe size of 225 mm could be selected, considering the same friction loss of 0.30 m per 100 m.

The need for a night storage reservoir should be carefully considered, weighing advantages, such as money saving in water delivery works, against disadvantages, such as cost of reservoir construction, maintenance, seepage and evaporation losses and disease vector control costs.

6.2.1. Types of reservoirs

Reservoirs can be classified on the basis of:

- The material used in construction, such as bricks, concrete or earth
- Their shape, which can be circular, square or rectangular

Earthen reservoirs

Earthen reservoirs are the most common, as they are usually cheaper to construct. Figure 53 shows a design of a typical square earthen reservoir, including the inlet, the outlet and the spillway. The embankments should be well compacted. If the original soils are permeable, a core trench should be dug and filled up with less permeable soils.

Circular reservoirs

A circular reservoir is the common shape of a concrete or brick reservoir. It is the most economical, as the perimeter of a circle is smaller than the perimeter of a square or rectangle for the same area. It also does not need heavy corner reinforcement to resist the water pressure, as do square or rectangular reservoirs. The formula for the calculation of the volume (V) of a circular reservoir is:

Equation 35

$$V = \frac{1}{4}\pi d^2 h$$

Where:

V = Volume of reservoir (m^3)

d = Diameter (m)

h = Water depth (m)

Example 24

If the reservoir in Example 23 has water depth, h, of 2.0 m (the maximum recommended depth for brick reservoirs), what would be the required diameter for the reservoir?

Using Equation 35:

$$1\,970 = \frac{1}{4} \times 3.14 \times d^2 \times 2 \quad \Rightarrow \quad d = 35.42 \text{ m}$$

The best site for a reservoir is on a flat area with firm, uniform soils. It is not recommended to build a reservoir on made-up ground, unless the compaction is extremely well done.

6.2.2. Reservoir components

Foundation and floor

A foundation of 450-600 mm in width and 225-300 mm in depth for a 250 mm wall thickness should be adequate for circular reservoirs on firm, solid ground. Normally, foundations do not need reinforcement, except when placed on unstable soils.

A floor thickness of 100 mm should be adequate. Often a reinforcement grid with 200-300 mm interval is placed in the concrete floor. Joints, meant to control cracking, are placed in the reservoir floor and the reinforcement grid should not cut across these joints. The concrete panels should not exceed 6 m in either length or width. Long narrow panels should be avoided.

Reinforcement

The pressure of the water in a circular reservoir produces tension in the wall. Pressure exerted by the water is directly proportional to the water head (depth) from the surface to the depth considered. Tension produced in the wall of a circular reservoir is directly proportional to the water depth and the diameter of the reservoir.

The tension is taken up, to some degree, by the material of the wall. However, concrete and bricks are weak in tension, therefore reinforcement should be provided. In a

Figure 53

Design of a typical earthen night storage reservoir

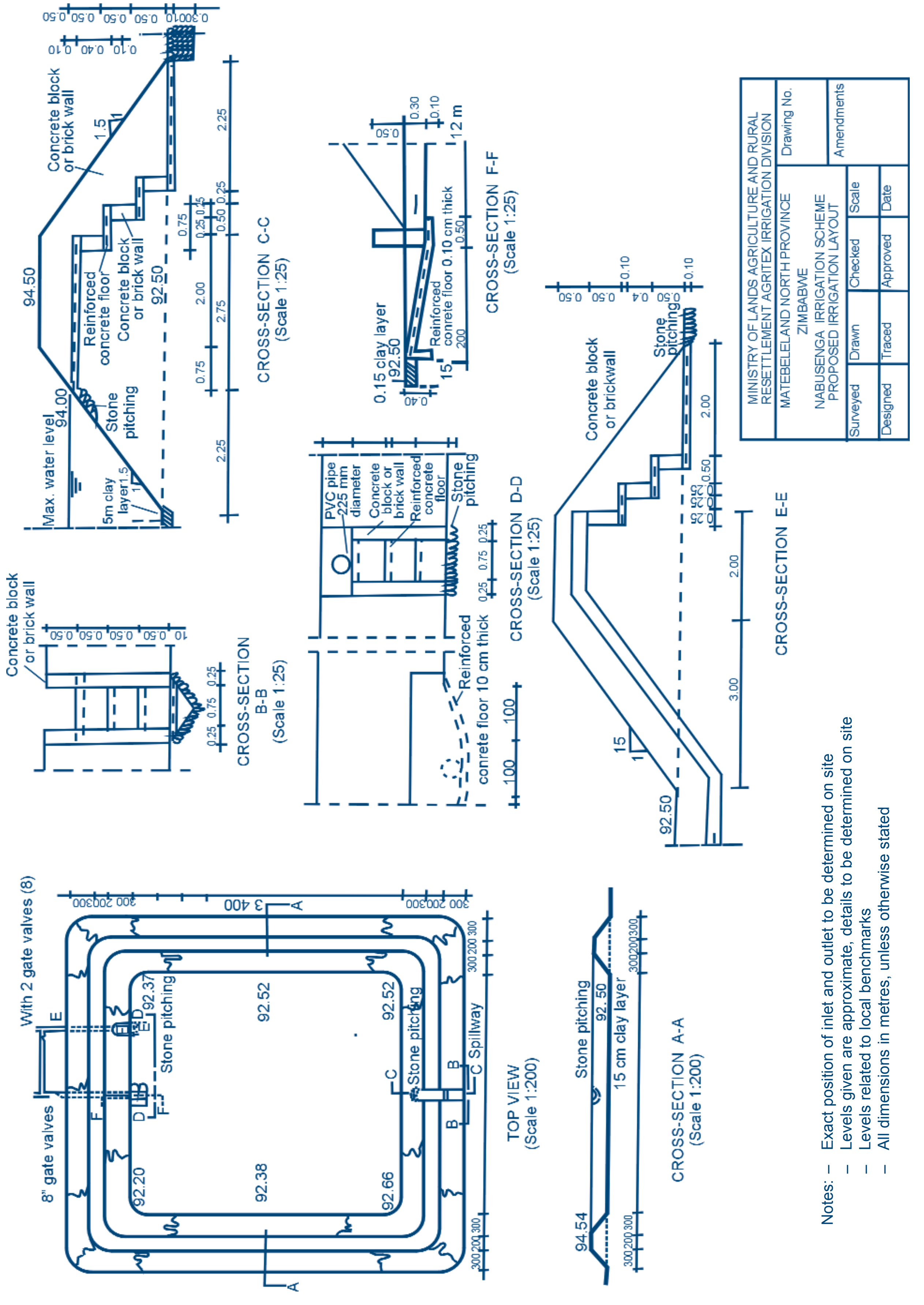
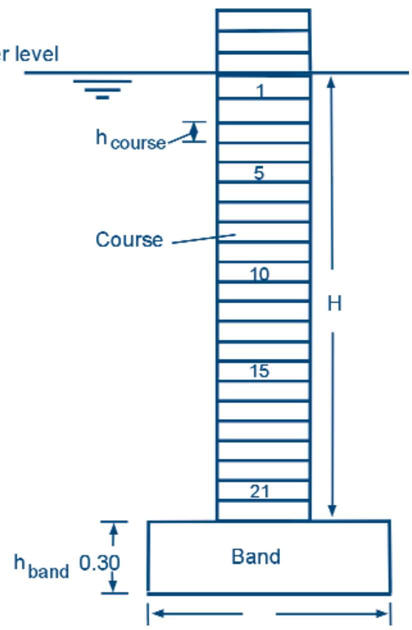


Figure 54
Courses in brick wall of a reservoir



brick or concrete block wall the reinforcement rods are best placed within the mortar of the horizontal joints between courses. A course is a continuous level line of bricks or stones in a wall (Figure 54). A simple formula to calculate the cross-sectional area (A) of reinforcement needed per course is:

Equation 36

$$A = 44.6 \times d \times H \times h$$

Where:

A	=	Cross-sectional area of reinforcement needed for the course or band under consideration (mm^2)
d	=	Diameter of the reservoir (m)
H	=	Distance down from the design water level to the bottom of the course (m)
h_{course}	=	Height of a course (m)
h_{band}	=	Height of band (m)

The band for a concrete wall is assumed to be 300 mm. The course for a brick wall is 90 mm (including 15 mm for the mortar) and the course for a concrete block is assumed to

Example 25

What are the reinforcement requirements for a brick reservoir of 2 m high and 36 m in diameter, with a wall as shown in Figure 54?

The calculations of the steel rod requirements for courses 1, 5 and 21 are given below. Calculations for all other courses are similar and summarized in Table 26. Cross-sectional areas of the different rod sizes are given in Table 27.

Using Equation 36:

Course 1: $A = 44.6 \times 36 \times (1 \times 0.09) \times 0.09 = 13\ mm^2$ ⇒ 1 steel rod of 4 mm diameter is required

Course 5: $A = 44.6 \times 36 \times (5 \times 0.09) \times 0.09 = 65\ mm^2$ ⇒ 3 steel rods of 6 mm diameter are required

Course 21: $A = 44.6 \times 36 \times (21 \times 0.09) \times 0.09 = 273\ mm^2$ ⇒ 6 steel rods of 8 mm diameter or 4 steel rods of 10 mm diameter are required

Table 26
Reinforcement requirements in a clay brick wall of a reservoir

Course	Reinforcement (mm^2)	Number of rods and diameter	Course	Reinforcement (mm^2)	Number of rods and diameter
1	13.0	1 x 4 mm	12	156.1	3 x 8 mm
2	26.0	1 x 6 mm	13	169.1	4 x 8 mm
3	39.0	2 x 6 mm	14	182.1	4 x 8 mm
4	52.0	2 x 6 mm	15	195.1	4 x 8 mm
5	65.0	3 x 6 mm	16	208.1	5 x 8 mm
6	78.0	3 x 6 mm	17	221.1	5 x 8 mm
7	91.0	4 x 6 mm	18	234.1	5 x 8 mm
8	104.0	4 x 6 mm	19	247.1	5 x 8 mm
9	117.0	5 x 6 mm	20	260.1	6 x 8 mm
10	130.1	5 x 6 mm	21	273.1	6 x 8 mm
11	143.1	5 x 6 mm			

Note :It is not recommended to have more than 8 steel bars in a course for a wall thickness of 250 mm. If need be, then it would be necessary to use smaller diameter bars.

be 160 mm high (including 20 mm for the mortar). The height could differ, depending on the actual block or brick used and should be confirmed on site.

Table 27
Cross-sectional areas of reinforcement steel rods

Diameter of rod (mm)	Cross-sectional area $= \frac{1}{4} \pi d^2$ (mm^2)
4	12.6
6	28.3
8	50.3
10	78.5
12	113.1

Pipe requirements

The main pipe requirements are:

- A supply pipe for filling the reservoir
- An outlet pipe
- An overflow pipe
- A scour pipe

The supply pipe generally discharges into the reservoir over the wall from the outside, although it could also be brought under the foundation and up through the floor. The pipe should have a gate valve so that supplies can be shut off when necessary. The diameter depends on the design discharge.

The outlet pipe may be installed into the reservoir wall about 150 mm above floor level or under the foundation and up through the floor. By positioning the pipe above floor level, sludge and sediments are prevented from entering the delivery system. The pipe should be fitted with a screen as a precaution against blockage. The diameter of the outlet pipe depends on the design discharge and the available head. This pipe should also have a gate valve to be able to shut it off.

An overflow pipe should be installed in the wall with its bottom at the same height as the full supply level of the reservoir. A 100 mm diameter pipe usually suffices.

A scour pipe should be provided at a level slightly below floor level, so that the reservoir can be regularly cleaned of sediments. It could have a 100 mm diameter and should be fitted with a gate valve.

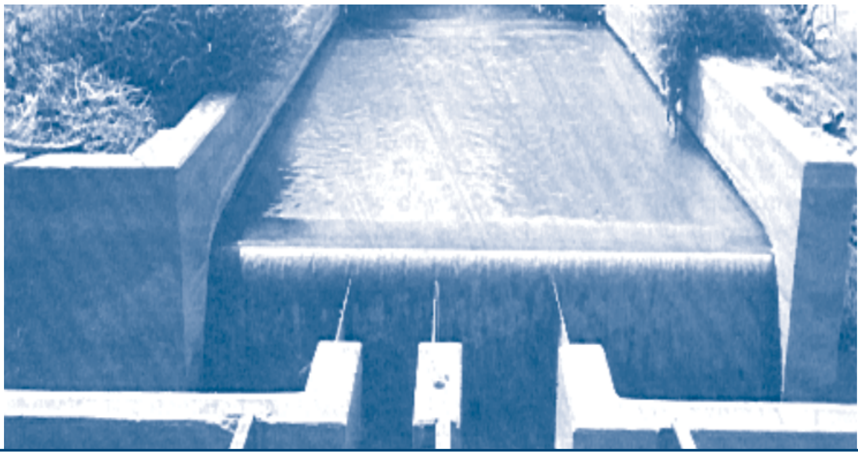
6.3. Head regulators

A head regulator is a structure used to control, and usually also to measure, the discharge of water into the irrigation system. It should be designed in such a way that head losses are kept as low as possible.

On large schemes needing large quantities of water, head regulators can be very large and would usually be built in concrete. The use of concrete will result in strong structures, but can be expensive. The thickness of the floor and the walls should be between 10 cm and 15 cm. A cheaper structure could be a concrete block or brick structure, which would be suitable for smaller structures with low walls. Wooden diversion structures could be used where discharges are less than 200 l/sec. In small schemes, concrete blocks, bricks or even stones could be used to build the regulators. In this case, they have manual lifting gates or moveable weirs.

In addition to the headwork described in Section 6.1, head regulators could also be located at the top of any canal in the scheme, for example a secondary canal or even a

Figure 55
A simple in-situ concrete proportional flow division structure (Source: Jensen, 1983)

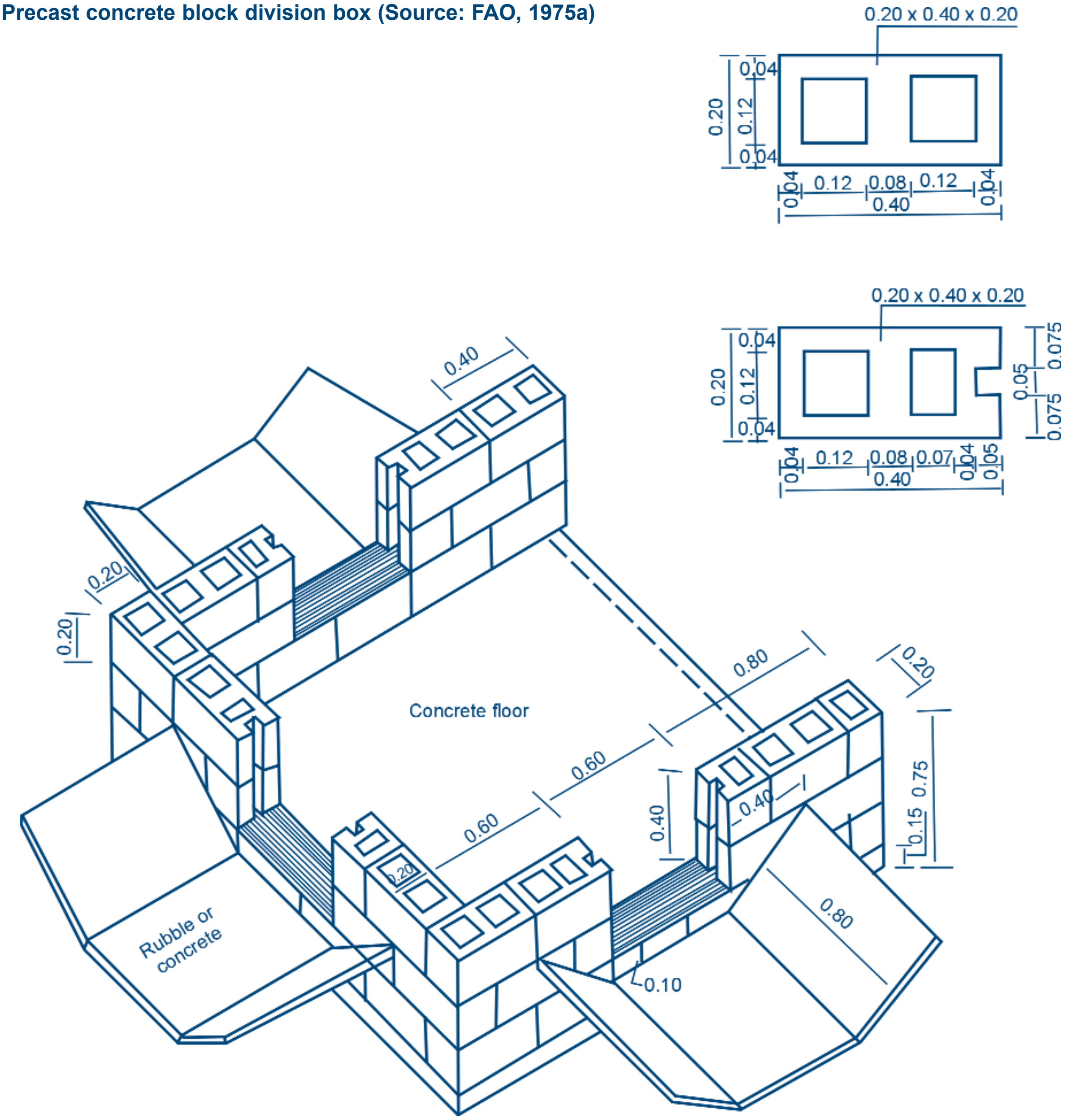


tertiary canal. In these cases, the head regulator is usually called a diversion structure. Weir-type diversion structures have been discussed in Section 6.1. Below diversion structures as regulating structures in general will be discussed.

A diversion structure regulates the flow from one canal into one or more other canals. It normally consists of a box with vertical walls in which controllable openings are provided. The minimum dimensions of the structure depend on its performance in the fully open position. The width of the outlet is usually proportional to the division of water flow to be made. Figures 55-57 show some examples of diversion structures (in-situ concrete, pre-cast concrete and timber structures respectively). The walls can be made either of concrete (10-15 cm thick precast or in-situ) or masonry or even wood.

Large backup of water upstream of the structure, which would result in overtopping of the canal, should be avoided. Since a lined canal is designed to carry water at relatively high velocities, a full gate-opening at the intake to the box, covering approximately the same area as the canal section, should be provided. In earthen canals, gate-opening dimensions can be based on assuming velocities of less than 1.0-1.5 m/sec.

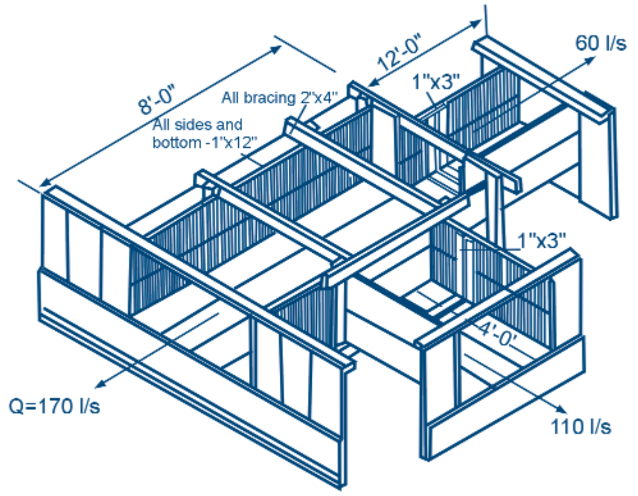
Figure 56
Precast concrete block division box (Source: FAO, 1975a)



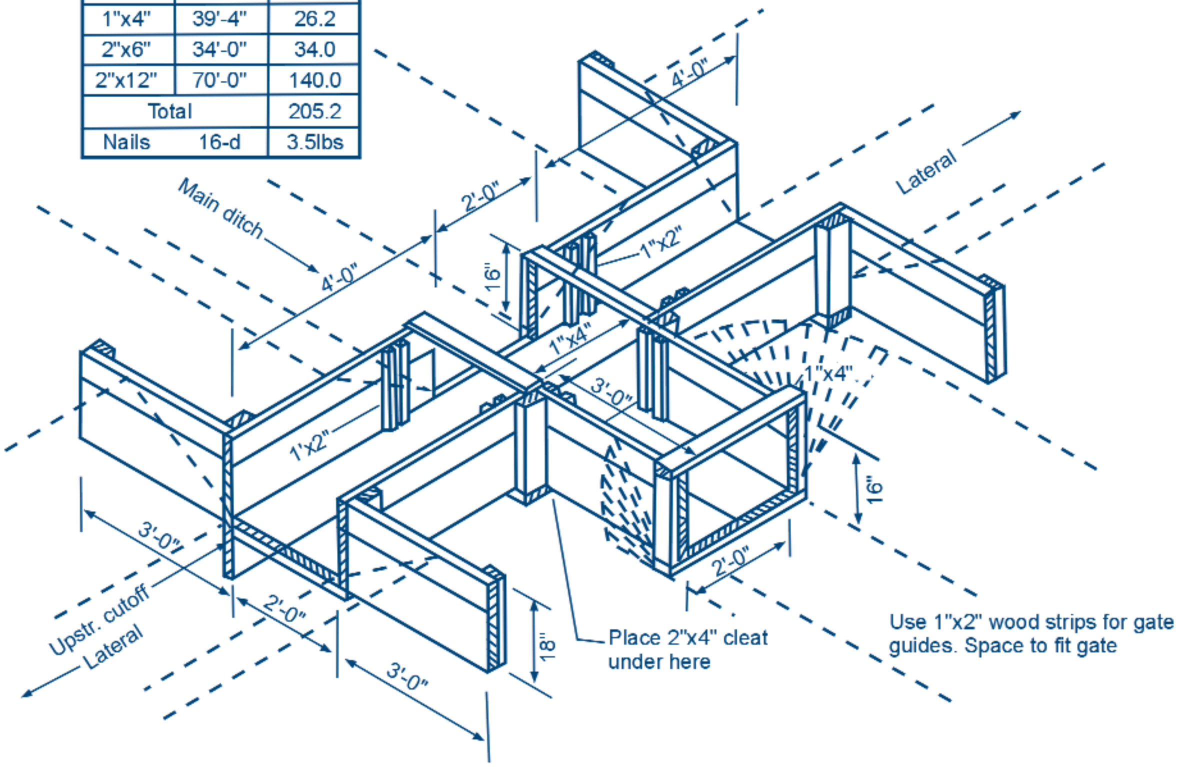
All dimensions are in metres

Figure 57

Timber division structures (Source: FAO, 1975a)



Bill of Materials		
Size	L-F	B-F
1"x4"	15'-4"	5.0
1"x4"	39'-4"	26.2
2"x6"	34'-0"	34.0
2"x12"	70'-0"	140.0
Total		205.2
Nails	16-d	3.5lbs



Example 26

What should be the minimum width of the opening b of a diversion structure, if the discharge Q is equal to 78.3 l/sec and if the water depth in the opening is not to exceed 0.30 m?

Using Equation 12:

$Q = A \times V = 0.30 \times b \times V$

Assuming a $V_{max} = 1.50$ m/sec for concrete lining and substituting it into Equation 12 gives:

$0.00783 = 0.30 \times b \times 1.50 \Rightarrow b_{min} = 0.18$ m

Example 27

What should be the water depth over a weir crest, if the discharge Q is equal to 78.3 l/sec, the weir length B is equal to 0.40 m and C is equal to 1.75?

Using Equation 24:

$Q = C \times B \times H^{3/2} \Rightarrow H = 0.23$ m

The structure should be designed in such a way that the water velocity will not cause erosion in the earthen canal. Thus, the water velocity should reduce to its canal design value after the opening and before the water re-enters the earthen canal.

There is a relationship between the width of the opening of the gate and the head loss. Hydraulic losses through a properly designed structure are small. When the ground slope is very gentle, head losses should be kept to a minimum so as to maintain command in the canals. This would be achieved by making a wider and larger diversion structure.

6.4. Cross regulators

A cross regulator is a structure built across the canal to maintain the water level at the command level required to irrigate the fields. Cross regulators could be simple timber stop logs, check plates, weirs or expensive automatically operated gates, which automatically control a constant water level.

In Section 6.1, weirs have been discussed as headwork structures. In the context of cross regulation, examples of common weirs are duckbill and diagonal weirs, which control the water level at a given height, (Figure 58, 59 and 60). Detailed explanations of weirs as discharge measurement structures are given in Section 6.6.

6.5. Drop structures and tail-end structures

Drop structures and chutes are flow control structures that are installed in canals when the natural land slope is too steep compared to the design canal gradient (see Section 5.1) to convey water down steep slopes without erosive velocities. If a canal were allowed to follow a steep natural gradient, the velocities would be too high. This in turn would cause erosion and make water management difficult. For this, the canal is divided into different reaches over its length. Each reach follows the design canal gradient. When the bottom level of the canal

Figure 58
Duckbill weir photograph (Source: FAO, 1975b)

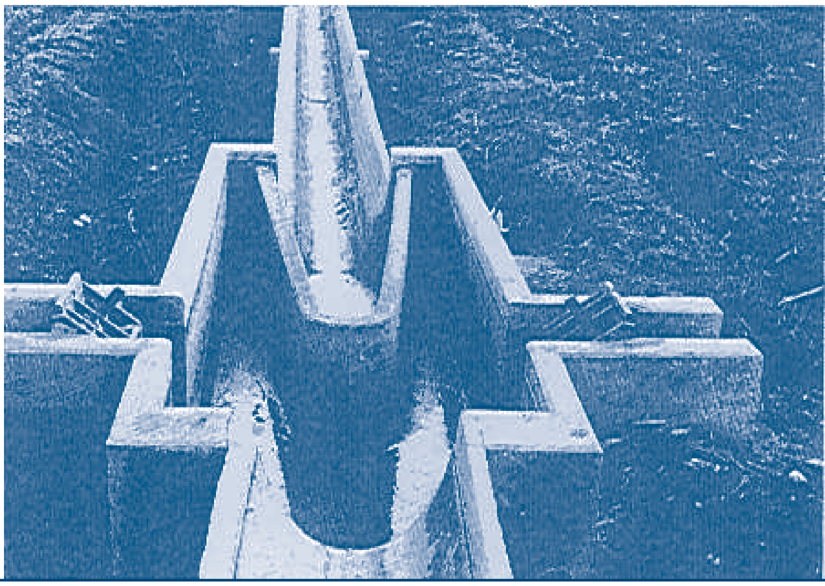
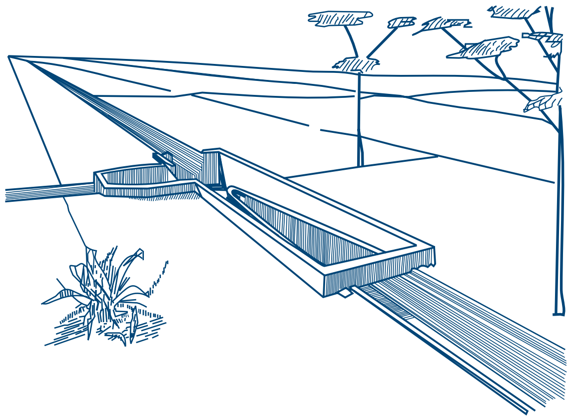


Figure 59
Duckbill weir design (Source: FAO, 1975b)

Basic characteristics

Canal capacity upstream of weir l/s	Dimensions in cm					
	a	b	c	d	e	f
110	7	150	300	13	10	0.08
160	7	220	370	12	9	0.08
210	7	290	440	12	9	0.10
270	8	310	460	11	9	0.10
320	8	370	520	11	9	0.12
370	8	430	580	11	9	0.12



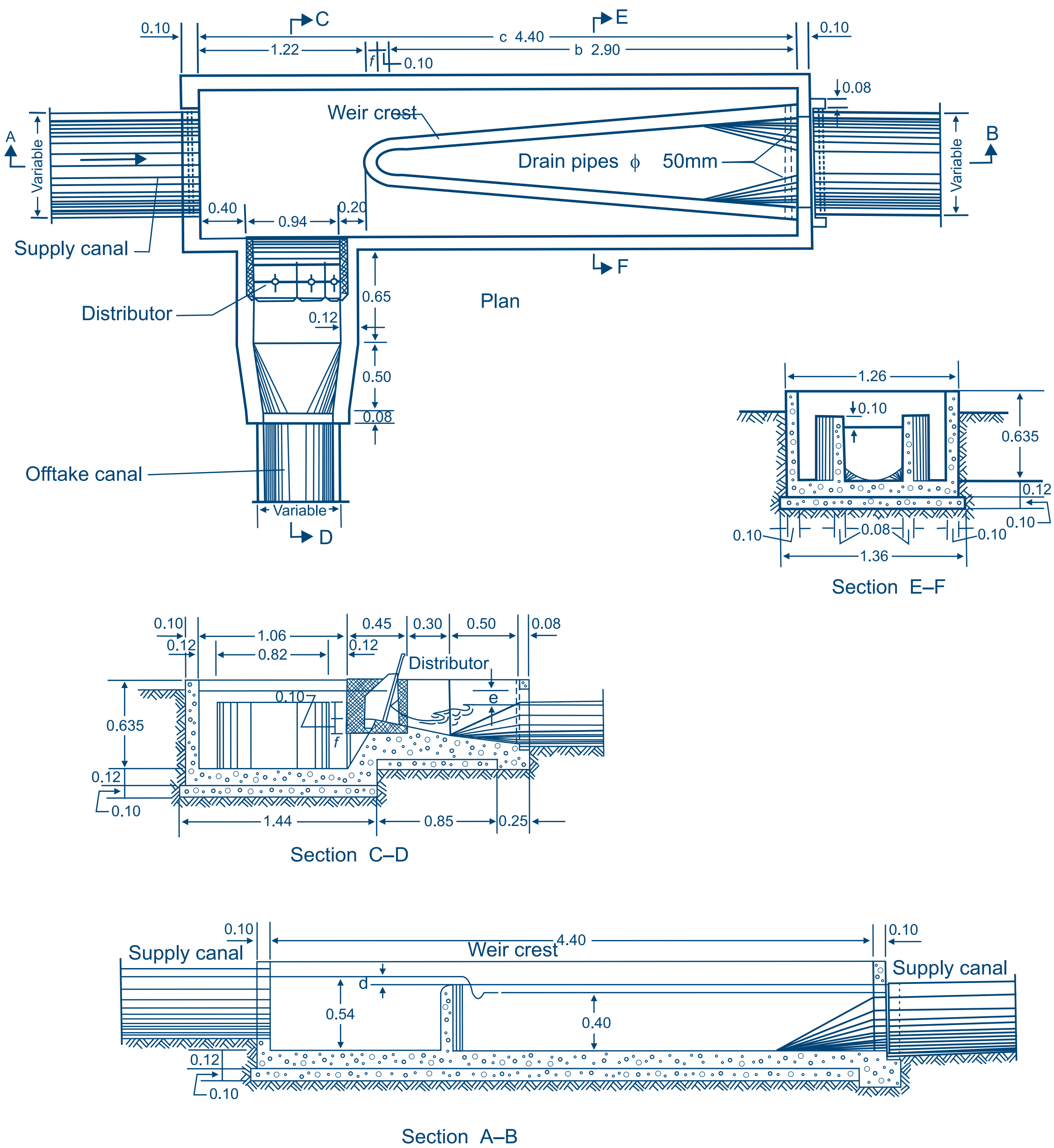
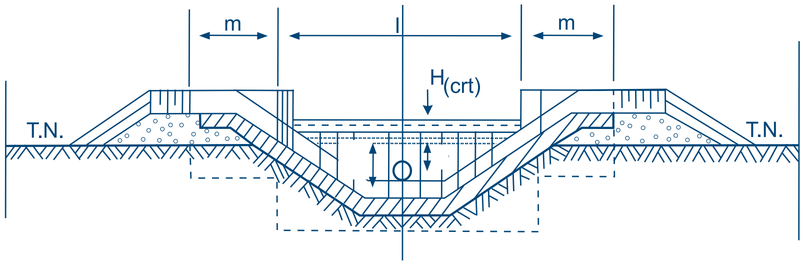
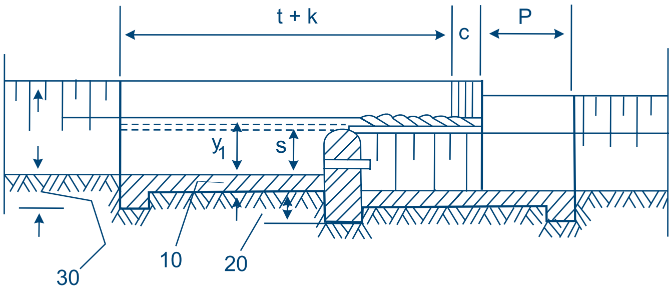


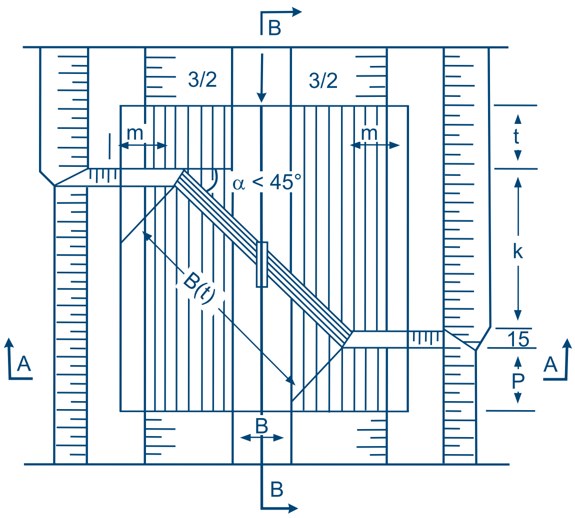
Figure 60
Diagonal weir (Source: FAO, 1975b)



Section A–A



Section B–B



Plan

Range of suitable dimensions for capacities up to 500 l/s

B = 0.20 to 1.00

f = 0.20 to 1.00

y_1 = 0.10 to 0.70 (upstream water depth)

H(crt) = 0.05 to 0.15 (difference between upstream water level and crest level)

s = 0.10 to 0.60

c = 0.15 (thickness of weir)

l = (width of available upstream water surface)

B(t) = (crest length) = $l \times \frac{1}{\cos\propto}$

∝ = (angle between weir crest and cross-section of channel)

m = 1.5f - 1.5s + 0.20

k = Lsin∝

p = f_n

t = f

becomes too high compared to the natural ground level, drop structures are installed. Vertical drops are normally used for the dissipation of up to 1 m head for unlined canals and up to 2 m head for lined canals. For larger drops, chutes are usually used.

For canals that do not require command, the position of drops is determined by considering the cost of canal construction, including balancing the cuts and fills and the cost of the structure. Where there is need for command, the drops should be located in such a way that the canal banks are not too high, but still keeping enough command at the same time.

Figures 61 and 62 show examples of drop structures built with different materials.

6.5.1. Vertical drop structure

An important aspect of a drop is the stilling basin, required to avoid downstream erosion. The floor of the stilling basin should be set at such a level that the hydraulic jump occurs at the upstream end of the basin floor in order to avoid erosion at the unprotected canal bed downstream. A common straight drop structure is shown in Figure 63.

Figure 61
Some drop structures used in open canals (Source: James, 1988)

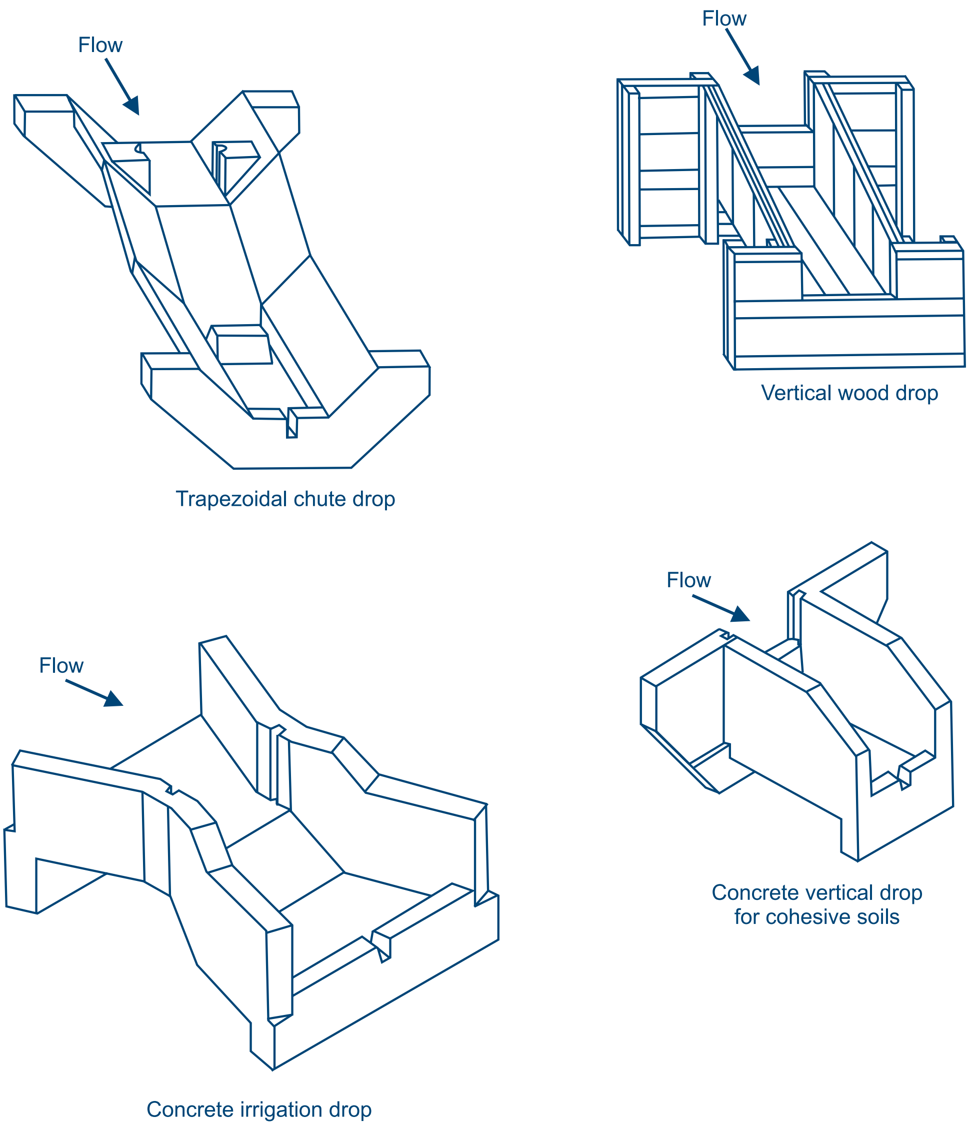


Figure 62
Standard drop structure without stilling basin

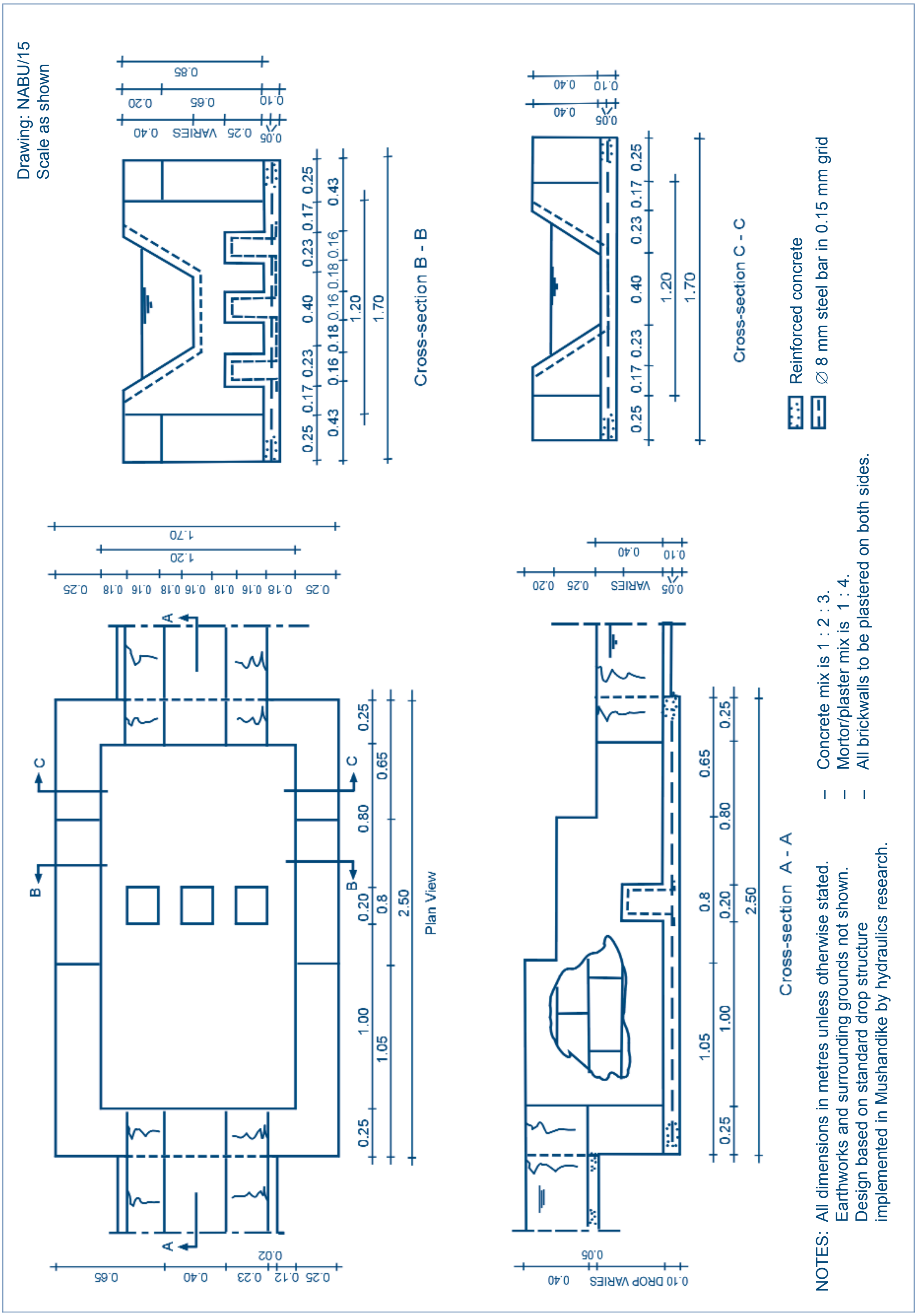
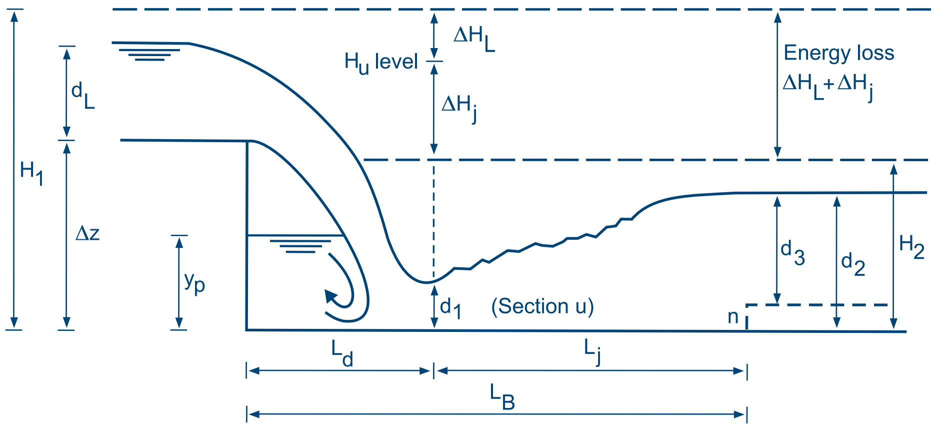


Figure 63
A vertical drop structure



Example 28

Given a discharge of 0.0783 m^3/sec, a drop height of 0.50 m and a drop width of 0.30 m, what would be (Figure 63 and 47):

– *The length of the apron from the drop to the hydraulic jump, where the lowest water level will occur (L_d)?*

– *The height of the jump or the lowest water level after the drop (d_1)?*

– *The design water level after the drop (d_2)?*

– *The total length of the apron (L_B)?*

Q = 0.0783 m^3/sec ⇒ Q = 0.261 m^3/sec per 0.30 m width

Using Equation 25:

$$D = \frac{0.261^2}{(9.81 \times 0.05^3)} \Rightarrow D = 0.0556$$

Substituting the data in Equations 26 to 29 respectively gives:

L_d = 0.50 x 4.30 x $0.0556^{0.27}$ = 0.99 m

d_1 = 0.50 x 0.54 x $0.0556^{0.425}$ = 0.15 m

d_2 = 0.50 x 1.86 x $0.0556^{0.27}$ = 0.38 m

L_j = 6.9 x (0.38 - 0.15) = 1.59 m ⇒ L_B = 0.99 + 1.59 = 2.58 m

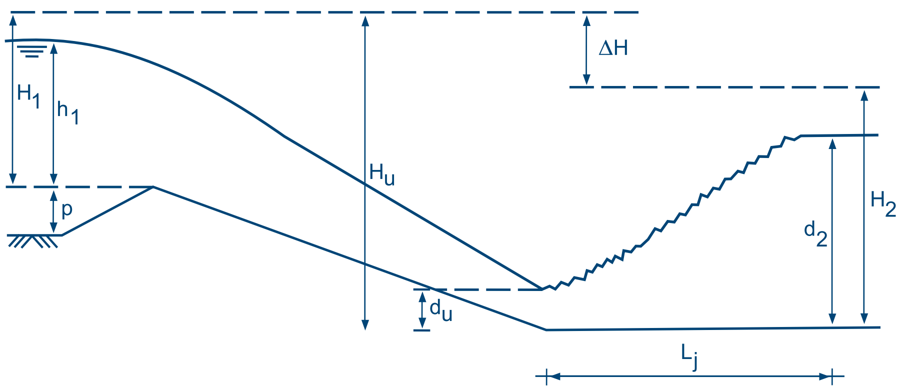
Due to the impact of the water flow on the basin floor and the turbulent circulation, an amount of energy (ΔH_L) is lost. Further energy is lost in the hydraulic jump downstream of the section U in Figure 63. Experiments have shown that the energy head (H_2) is equal to about 2.5 x d_1 (that is 2.5 times the critical depth). This provides a satisfactory basis for design.

An upward step is often added at the end of the basin floor in order to be sure that the hydraulic jump occurs immediately below the drop. This step has the disadvantage of retaining standing water when the canal is not in use, thereby posing a danger to health.

6.5.2. Chutes

An example of a chute structure is given in Figure 64. Chutes are normally rectangular, although they are also made in a trapezoidal shape. They have an inlet, a steep-sloped section of a lined canal, a stilling basin or some other energy dissipating devices, baffle blocks, and an outlet. The energy dissipation is usually effected by the creation of a

Figure 64
A chute structure



hydraulic jump at the toe of the steep-sloped section of the structure. Baffle blocks could be used to facilitate the creation of a hydraulic jump.

The slope of the downstream face (steep-sloped section) usually varies between 1 in 4 and 1 in 6. The length of the stilling basin (often called cistern), L_j, can be estimated with the following equation:

Equation 37

$$L_j = 5 \times d_2$$

6.5.3. Tail-end structures

Most canals need some way of getting rid of water. A tail-end structure should be provided at the end of the canal so that excess water can flow safely into the drain. It normally consists of a drop structure to bring the water level from a command canal level to the drain level from where it will be taken to the main drainage system of the project.

6.6. Discharge measurement in canals

Discharge measurement in irrigation schemes is important for the following reasons:

- ❖ To ensure the maintenance of proper delivery schedules
- ❖ To determine the amount of water delivered for water pricing, where it is applicable
- ❖ To detect the origin of water losses and to estimate the quantity
- ❖ To ensure efficient water distribution
- ❖ To conduct applied research

Almost any kind of obstacle that partially restricts the flow of water in an irrigation canal and provides a free fall, to ensure that upstream and downstream flow are independent, can be used as a measuring device, provided that it can be calibrated. Standard structures, which have already been accurately described and calibrated, exist. Weirs, flumes and orifices are the devices that are normally used for discharge measurement.

6.6.1. Discharge measurement equations

The three fundamental equations used to solve discharge problems in canals are based on the principles of conservation of mass, energy and momentum. For our purposes, only the conservation of mass and energy equations will be dealt with.

Conservation of mass

Conservation of mass leads to the Continuity Equation 12 to be constant:

$$Q = A \times V = \text{Constant}$$

Conservation of energy

Conservation of energy applied along a streamline results in the Bernoulli Equation:

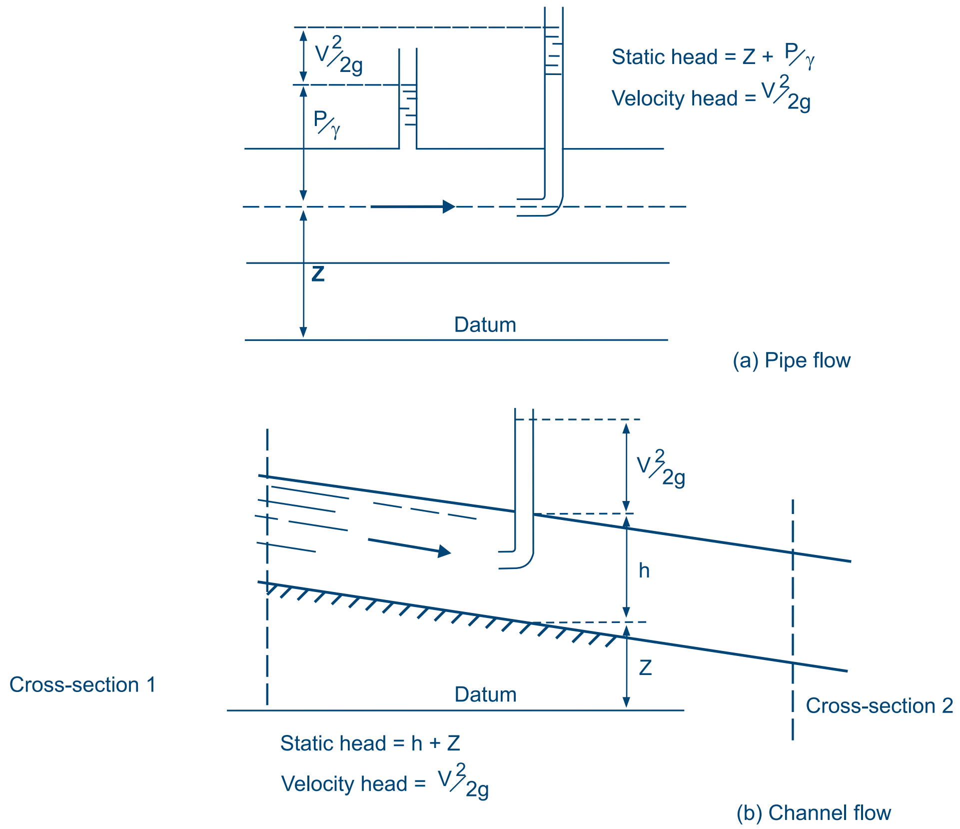
Equation 38

$$\frac{P}{\gamma} + \frac{V^2}{2g} + z = \text{Constant}$$

Where:

P = Pressure (kgf/m^2)
γ = Density of water (kg/m^3)
V = Water velocity (m/sec)
g = Gravitational force (9.81 m/sec^2)
z = Elevation above reference line (m)

Figure 65
Static and velocity heads



Equation 38 sums up the pressure head, velocity head and gravitational head to give the total head. For an open canal, the pressure head equals the water depth h (Figure 65).

When there is frictional loss along the flow path, an expression for frictional head loss must be included. Thus applying the Bernoulli Equation to two successive cross-sections along a flow path results in:

Equation 39

$$h_1 = \frac{V_1^2}{2g} + z_1 = h^2 + \frac{V_2^2}{2g} + z_2 + HL$$

The numbers 1 and 2 refer to the first and second cross-section in Figure 65. HL is the frictional head loss.

Specific energy

The concept of specific energy is used in the analysis of critical flow. At any cross-section of a canal, the energy with respect to the canal bed is referred to as specific energy. It is derived from the Bernoulli Equation according to the following equation:

Equation 40

$$E = \frac{h}{2g} + V^2$$

Where:

E = Specific energy (m)
h = Depth of flow (m)
g = Gravitational force (9.81 m/sec^2)
V = Water velocity (m/sec)

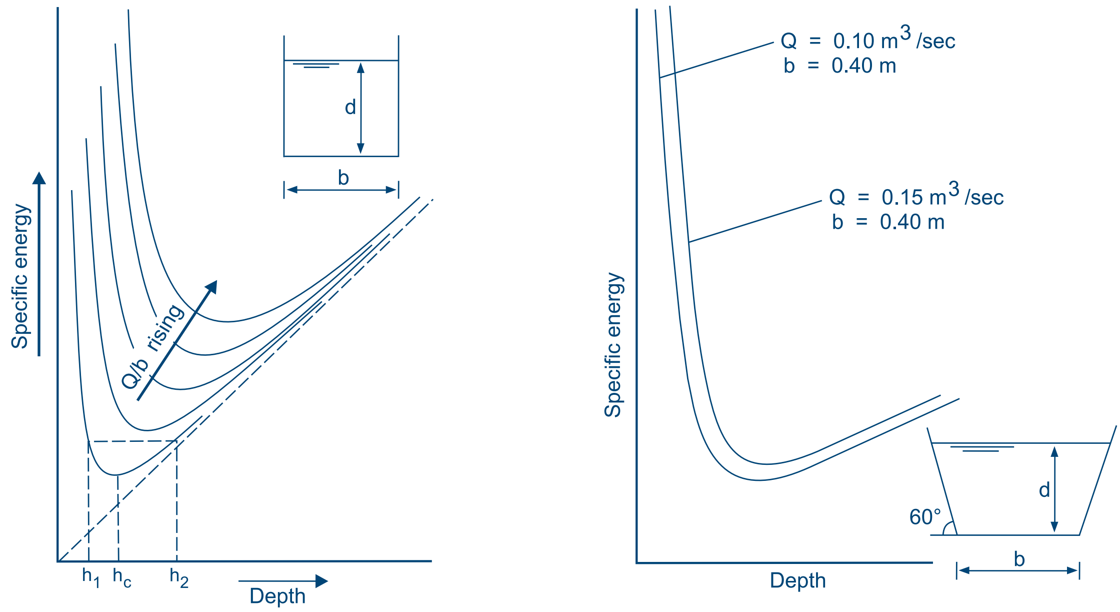
Assuming a uniform velocity distribution, the specific energy is constant across the section. Combining the above equation and the Continuity Equation gives:

Equation 41

$$E = \frac{h}{2g} + \left[\frac{Q}{A}\right]^2$$

The cross-sectional area varies with the depth of flow only if the geometry of the canal is constant. Therefore, for a given discharge the specific energy is a function of depth alone.

Figure 66
Variation of specific energy with depth of flow for different canal shapes



Specific energy can be determined for different structures:

Rectangular canal

$$A = b \times h$$

$$E = \frac{h}{2g} + \left[\frac{Q}{b \times h}\right]^2$$

Trapezoidal canal (with a side slope of 60°)

$$A = b \times h + 0.58 \times h^2$$

$$E = \frac{h}{2g} + \left[\frac{Q}{b \times h + 0.58\ h^2}\right]^2$$

Plotting E against h for different values of (Q/b) gives curves as shown in Figure 66.

The curves show that, for a given discharge and specific energy, there are two alternate depths of flow, which coincide at a point where the specific energy is a minimum for a given discharge. Below this point, flow is physically not possible. At this point flow is critical and it occurs at critical depth and velocity. At a greater depth, the velocity is low and flow is sub-critical. At the lesser depth, the velocity is high and flow is super-critical.

For sub-critical flow, the mean velocity is less than the velocity of propagation of stream disturbances such as waves. Thus, stream effects can be propagated both upstream and downstream. This means that downstream conditions affect the behaviour of flow. When flow is super-critical, the velocity of flow exceeds the velocity of propagation. Consequently, stream effects (for example, waves) cannot be transmitted upstream, and downstream conditions do not affect the behaviour of the flow.

For critical flow, the specific energy is a minimum for a given discharge. In this case, a relationship exists between the minimum specific energy and the critical depth. This relationship is found by differentiating Equation 41 with respect to h, while Q remains constant. This gives:

Equation 42

$$V_c = \left[\frac{g \times A_c}{b_c}\right]^{1/2}$$

Froude Number

The Froude Number is calculated according to Equation 19 (see Section 5.1.2):

$$Fr = \frac{v}{(g \times h)^{1/2}}$$

Where:

Fr = 1 for critical flow
Fr = > 1 for super-critical flow
Fr = < 1 for sub-critical flow

If a structure is built in a canal which has sub-critical flow, it may cause the flow to pass through the critical to the super-critical state. This means that the state upstream of the structure becomes independent of the state downstream. This can either be achieved if the structure narrows the canal, which means increasing the (Q/b)-ratio without altering the specific energy, or if it raises the canal bed, which means reducing the specific energy without altering the discharge per unit width. That is how critical flow is obtained with a measuring device. A control section in a canal is a section that produces a definitive relationship between water depth and discharge.

Hydraulic jump

If, through a structure, super-critical flow is introduced in a canal where the normal flow is sub-critical, flow adjusts back to the sub-critical state through a hydraulic jump in which the water level rises over a short distance with much visible turbulence. This situation occurs, for example, downstream of a sluice gate or a flume. It is undesirable to have a hydraulic jump in an unlined canal because of the risk of scour. In such cases, a jump is usually induced over a concrete apron by means of a sill or baffle blocks set in the floor, as shown in Figure 67.

The relationship between depths just upstream and downstream of a hydraulic jump is found by the application of the momentum theory to the simplified situation shown in Figure 68. It is assumed that boundary frictions are negligible over the length of the jump. For a rectangular canal it can be shown that:

Equation 43

$$h_2 = -\frac{h_1}{2} + 0.5 \times \left[h_1^2 + \left(8 \times V_1^2 \times \frac{h_1}{9} \right) \right]^{1/2}$$

6.6.2. Weirs

The weir is the most practical and economical device for water measurement. Weirs are simple to construct, easy to inspect, robust and reliable. Discharge measurement weirs can either be sharp-crested (Figure 69, 70, 71) or broad-crested (Figure 72).

Sharp-crested weirs

Sharp-crested weirs, also called thin plate weirs, consist of a smooth, vertical, flat plate installed across the channel and perpendicular to the flow (Figure 69). The plate obstructs flow, causing water to back up behind the weir plate and to

Figure 67
Hydraulic jump over a concrete apron

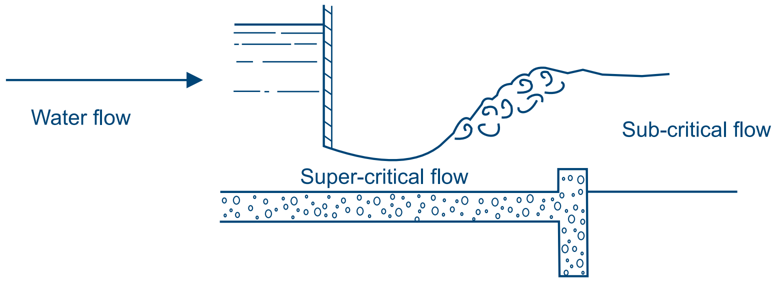


Figure 68
The form of a hydraulic jump postulated in the momentum theory

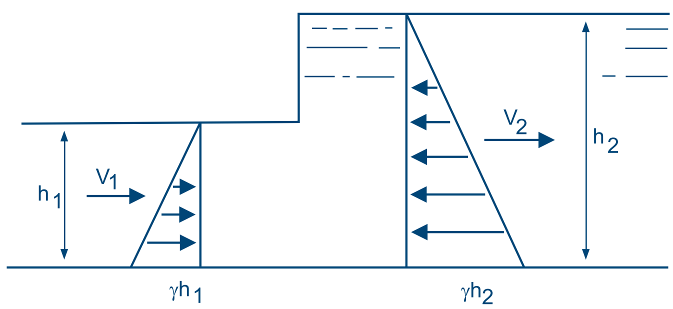
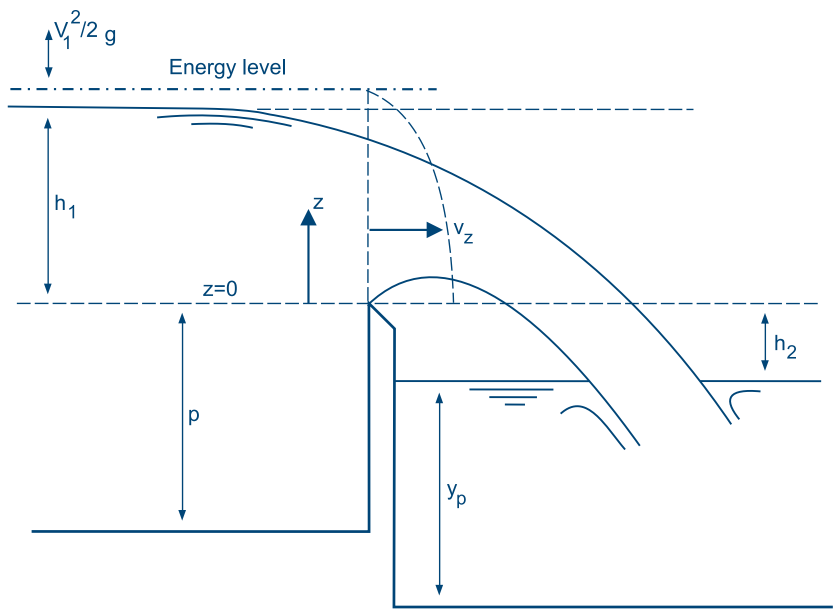


Figure 69
Parameters of a sharp-crested weir



flow over the weir crest. The distance from the bottom of the canal to the weir crest, p, is the crest height. The depth of flow over the weir crest, measured at a specified distance upstream of the weir plate (about four times the maximum h_1), is called the head h_1. The overflowing sheet of water is known as the nappe.

Thin plate weirs are most accurate when the nappe springs completely free of the upstream edge of the weir crest and air is able to pass freely around the nappe. The crest of a sharp-crested weir can extend across the full width of channel or it can be notched. The most commonly used notched ones are:

- Rectangular contracted weir
- Trapezoidal (Cipoletti) weir (Figure 70)
- Sharp sided 90° V-notch weir (Figure 71)

The type and dimensions of the weir chosen are based on the expected discharge and the limits of its fluctuation. For example, a V-notch weir gives the most accurate results when measuring small discharges and is particularly adapted to the measurement of fluctuating discharges. Calibration curves and tables have been developed for standard weir types.

The conditions and settings for standard weirs are as follows:

i. The height of the crest from the bottom of the approach canal (p) should preferably be at least twice the depth of water above the crest and should in no case be less than 30 cm. This will allow the water to fall freely, leaving an airspace under and around the jets.

ii. At a distance upstream of about four times the maximum head a staff gauge is installed on the crest with the zero placed at the crest elevation, to measure the head h1.

iii. For the expected discharge, the head (h1) should not be less than 6 cm and should not exceed 60 cm.

iv. For rectangular and trapezoidal weirs, the head (h_1) should not exceed 1/3 of the weir length.

v. The weir length should be selected so that the head for the design discharge will be near the maximum, subject to the limitations given in (ii) and (iii).

vi. The thickness of the crest for sharp-crest weirs should be between 1-2 mm.

In sediment-laden canals, a main disadvantage of using weirs is that silt is deposited against the upstream face of the weir, altering the discharge characteristics. Weirs also cannot be used in canals with almost no longitudinal slopes, since the required difference in elevation between the water levels upstream and downstream side of the weir is not available.

Discharge equations for weirs are derived by the application of the Continuity and Bernoulli Equations (Equation 12 and 38 respectively). In each case, a discharge coefficient is used in order to adjust the theoretical discharge found by laboratory measurements.

Rectangular contracted weir

A rectangular contracted weir is a thin-plate weir of rectangular shape, located perpendicular to the flow. To allow full horizontal contraction of the nappe, the bed and sides of the canal must be sufficiently far from the weir crest and sides.

Many practical formulae have been developed for computing the discharge, amongst which are the following:

Equation 44

Hamilton-Smith formula:

$$Q = \left[0.616 \times \left(1 - \frac{0.1h}{b}\right)\right] \times \frac{2}{3} \times (2g)^{1/2} \times b \times h^{3/2}$$

Equation 45

Francis formula:

$$Q = 1.838 \times (b - 2h) \times h^{3/2}$$

Where:

Q = Design discharge over weir (m^3/sec)

b = Length of weir crest (m)

h = Design water depth measured from the top of the weir crest (m)

Table 28 gives discharge data related to length of crest, b, and water head, h, over a weir.

Trapezoidal (Cipoletti) weir

The trapezoidal weir has a trapezoidal opening, the base being horizontal. The Cipoletti weir is a trapezoidal weir

Example 29

A rectangular contracted weir has to be placed in a lined canal. The design discharge is 0.0783 m^3/sec and the maximum allowable water depth, h, at the measuring gauge can be 0.15 m. What should be the minimum weir crest length, b, calculated using the Francis formula?

Using Equation 45:

$$Q = 0.0783 = 1.838 \times (b - 0.2 \times 0.15) \times 0.15^{3/2} = 0.1068 \times b - 0.0032 \Rightarrow b = 0.76 \text{ m.}$$

Table 28
Discharge Q (m^3/sec) for contracted rectangular weir, depending on h and b

Head h (m)	Length of crest b (m)						
	0.30	0.40	0.50	0.75	1.00	1.25	1.50
0.0025	0.0001	0.0001	0.0001	0.0002	0.0002	0.0003	0.0003
0.015	0.0010	0.0013	0.0017	0.0025	0.0034	0.0042	0.0051
0.030	0.0028	0.0038	0.0047	0.0071	0.0095	0.0119	0.0143
0.045	0.0051	0.0069	0.0086	0.0130	0.0174	0.0218	0.0262
0.060	0.0078	0.0105	0.0132	0.0199	0.0267	0.0335	0.0402
0.075	0.0108	0.0145	0.0183	0.0278	0.0372	0.0466	0.0561
0.090	0.0140	0.0190	0.0239	0.0363	0.0487	0.0612	0.0736
0.105	0.0175	0.0237	0.0300	0.0456	0.0612	0.0769	0.0925
0.12	0.0211	0.0287	0.0364	0.0555	0.0746	0.0937	0.1128
0.15	0.0288	0.0395	0.0502	0.0769	0.1036	0.1303	0.1570
0.18		0.0511	0.0651	0.1002	0.1353	0.1704	0.2055
0.21			0.0810	0.1253	0.1695	0.2137	0.2580
0.24			0.0977	0.1517	0.2058	0.2598	0.3139
0.27				0.1795	0.2440	0.3085	0.3730
0.30				0.2084	0.2840	0.3595	0.4350
0.36				0.2692	0.3685	0.4678	0.5671
0.42					0.4584	0.5835	0.7086
0.48					0.5527	0.7055	0.8584
0.54						0.8331	1.0155
0.60						0.9655	1.1791

with the sides having an outward sloping inclination of 1 horizontal to 4 vertical (Figure 70). This side slope is such that the water depth-discharge relationship is the same as that of a full width rectangular weir.

The discharge equation for a Cipoletti weir is:

Equation 46

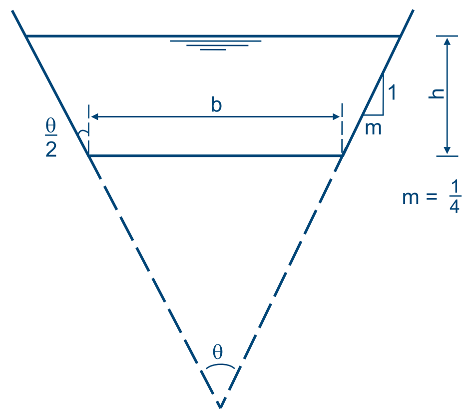
$$Q = 1.859 \times b \times h^{3/2}$$

Where:

Q = Design discharge over weir (m^3/sec)
b = Length of weir crest (m)
h = Design water depth measured from the top of the weir crest (m)

Table 29 shows discharge data, related to the design water depth, h, and weir length, b.

Figure 70
Trapezoidal (Cipoletti) weir



Example 30

A Cipoletti weir has to be placed in a lined canal. The design discharge is 0.0783 m^3/sec and the maximum allowable head, h, at the measuring gauge is 0.15 m. What should be the minimum weir crest length, b?

Using Equation 46:

$0.0783 = 1.859 \times b \times 0.15^{3/2} = 1.108b \Rightarrow b = 0.73$ m

Table 29
Discharge Q (m^3/sec) for Cipoletti weir, depending on h and b

Head h	Length of crest b (m)						
(m)	0.30	0.40	0.50	0.75	1.00	1.25	1.50
0.0025	0.0001	0.0001	0.0001	0.0002	0.0002	0.0003	0.0003
0.015	0.0010	0.0014	0.0017	0.0026	0.0034	0.0043	0.0051
0.030	0.0029	0.0039	0.0048	0.0072	0.0097	0.0121	0.0145
0.045	0.0053	0.0071	0.0089	0.0133	0.0177	0.0222	0.0266
0.060	0.0082	0.0109	0.0137	0.0205	0.0273	0.0341	0.0410
0.075	0.0115	0.0153	0.0191	0.0286	0.0382	0.0477	0.0573
0.090	0.0151	0.0201	0.0251	0.0376	0.0502	0.0627	0.0753
0.105	0.0199	0.0253	0.0316	0.0474	0.0632	0.0791	0.0949
0.12	0.0232	0.0309	0.0386	0.0580	0.0773	0.0966	0.1159
0.15	0.0324	0.0432	0.0540	0.0810	0.1080	0.1350	0.1620
0.18		0.0568	0.0710	0.1065	0.1420	0.1774	0.2129
0.21			0.0894	0.1342	0.1789	0.2236	0.2683
0.24			0.1093	0.1639	0.2186	0.2732	0.3278
0.27				0.1956	0.2608	0.3260	0.3912
0.30				0.2291	0.3054	0.3818	0.4582
0.36				0.3011	0.4015	0.5019	0.6023
0.48					0.3060	0.6325	0.7590
0.54					0.6182	0.7727	0.9273
0.60						0.9220	1.1065
						1.0799	1.2959

V-notch weir

A V-notch weir has two edges that are symmetrically inclined to the vertical to form a notch in the plane perpendicular to the direction of flow. The most commonly used V-notch weir is the one with a 90° angle. Other common V-notches are the ones where the top width is equal to the vertical depth (1/2 x 90° V-notch) and the one where the top width is half of the vertical depth (1/4 x 90° V-notch) (Figure 71). The V-notch weir is an accurate discharge-measuring device, particularly for discharges less than 30 l/sec, and it is as accurate as other types of sharp-crested weirs for discharges from 30 to 300 l/sec (U.S. Department of Interior, 1975).

To operate properly, the weir should be installed so that the minimum distance from the canal bank to the weir edge is at least twice the head on the weir. In addition, the distance from the bottom of the approach canal to the point of the weir notch should also be at least twice the head on the weir (U.S. Department of Interior, 1975).

The general and simple discharge equation for a V-notch weir is:

Equation 47

$$Q = 1.38 \times \tan\left(\frac{1}{2} \times \theta\right) \times h^{5/2}$$

Where:

Q = Design discharge over the weir (m^3/sec)

θ = Angle included between the sides of the notch (degrees)

h = Design water depth (m)

Table 30 gives discharge data for the three common V-notches related to water depth (head) and angle°.

Example 31

A design discharge of 0.0783 m^3/sec has to pass through a V-notch weir with an angle θ of 90°. What will be the water depth over the weir?

Substituting the above data in Equation 47:

$$0.0783 = 1.38 \times \tan\left(\frac{1}{2} \times 90\right) \times h^{5/2} \Rightarrow h^{5/2} = 0.0783 \Rightarrow h = 0.317 \text{ m.}$$

Figure 71
V-notch weirs

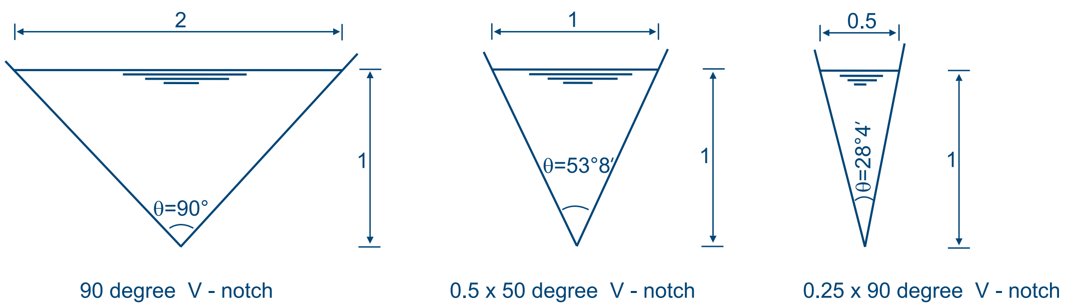


Table 30
Discharge Q (m^3/sec x 10) for a 90° V-notch weir, depending on h

Head (m)	Discharge (m^3/sec x 10)	Head (m)	Discharge (m^3/sec x 10)	Head (m)	Discharge (m^3/sec x 10)
0.050	0.008	0.160	0.142	0.270	0.523
0.055	0.010	0.165	0.153	0.275	0.548
0.060	0.012	0.170	0.165	0.280	0.573
0.065	0.015	0.175	0.177	0.285	0.599
0.070	0.018	0.180	0.190	0.290	0.626
0.075	0.022	0.185	0.203	0.295	0.653
0.080	0.025	0.190	0.217	0.300	0.681
0.085	0.029	0.195	0.232	0.305	0.710
0.090	0.034	0.200	0.247	0.310	0.739
0.095	0.039	0.205	0.263	0.315	0.770
0.100	0.044	0.210	0.279	0.320	0.801
0.102	0.050	0.215	0.296	0.325	0.832
0.110	0.056	0.220	0.313	0.330	0.865
0.115	0.062	0.225	0.332	0.335	0.898
0.120	0.069	0.230	0.350	0.340	0.932
0.125	0.077	0.235	0.370	0.345	0.966
0.130	0.084	0.240	0.390	0.350	1.002
0.135	0.093	0.245	0.410	0.355	1.038
0.140	0.102	0.250	0.432	0.360	1.075
0.145	0.111	0.255	0.454	0.365	1.113
0.150	0.121	0.260	0.476	0.370	1.152
0.155	0.131	0.265	0.499	0.375	1.191
				0.380	1.231

Broad-crested weir

A broad-crested weir is a broad wall set across the canal bed. The way it functions is to lower the specific energy and thus induce a critical flow (Figure 72).

One of the most commonly used broad-crested weirs for discharge measurements is the Romijn broad-crested weir, which was developed in Indonesia for use in relatively flat areas and where the water demand is variable because of different requirements during the growing season (FAO, 1975b). It is a weir with a rectangular control section, as shown in Figure 73.

The Romijn weir consists of two sliding blades and a movable weir crest, which are mounted in one steel guide frame (Figure 74). The bottom blade, which is locked under operational conditions, acts as the bottom terminal for the movable weir. The upper blade, which is connected to the bottom blade by means of two steel strips placed in the frame grooves, acts as the top terminal for the movable weir. Two steel strips connect the movable weir to a horizontal lifting beam. The horizontal weir crest is perpendicular to the water flow and slopes 1:25 upward in the direction of the flow. Its upstream nose is rounded off in such a way that flow separation does not occur. The operating range of the weir equals the maximum upstream head (H_{crt}) which has been selected for dimensioning the regulating structure.

Figure 72
Broad-crested weir

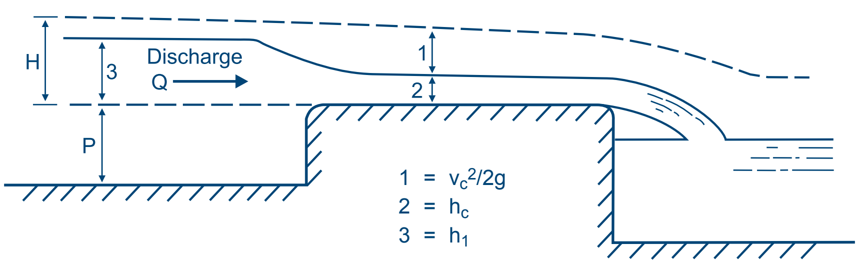


Figure 73
Romijn broad-crested weir, hydraulic dimensions of weir abutments (Source: FAO, 1975b)

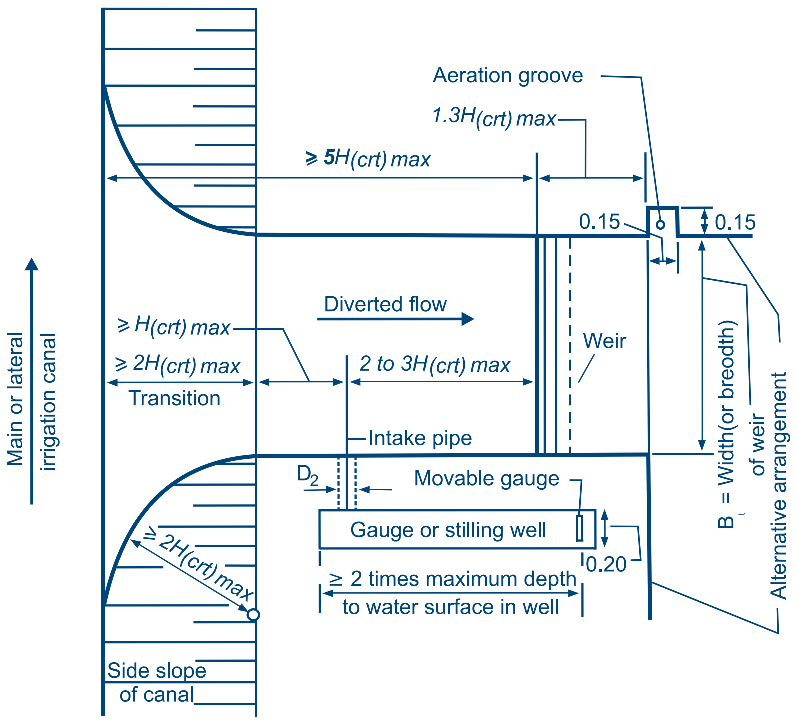
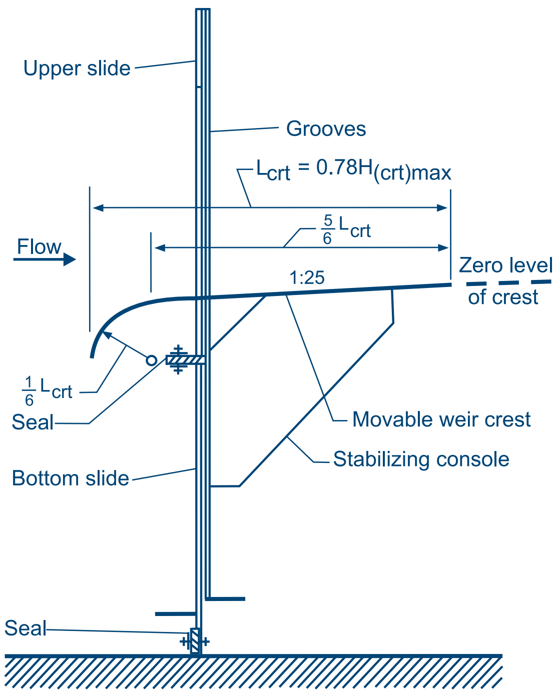


Figure 74
Romijn broad-crested weir, sliding blades and movable weir crest (Source: FAO, 1975b)



The discharge equation for the Romijn broad-crested weir is written as:

Equation 48

$$Q = \frac{2}{3} \times C_d \times C_v \times \left[\frac{2}{3} \times g\right]^{1/2} \times B_t \times H_{crt}^{3/2}$$

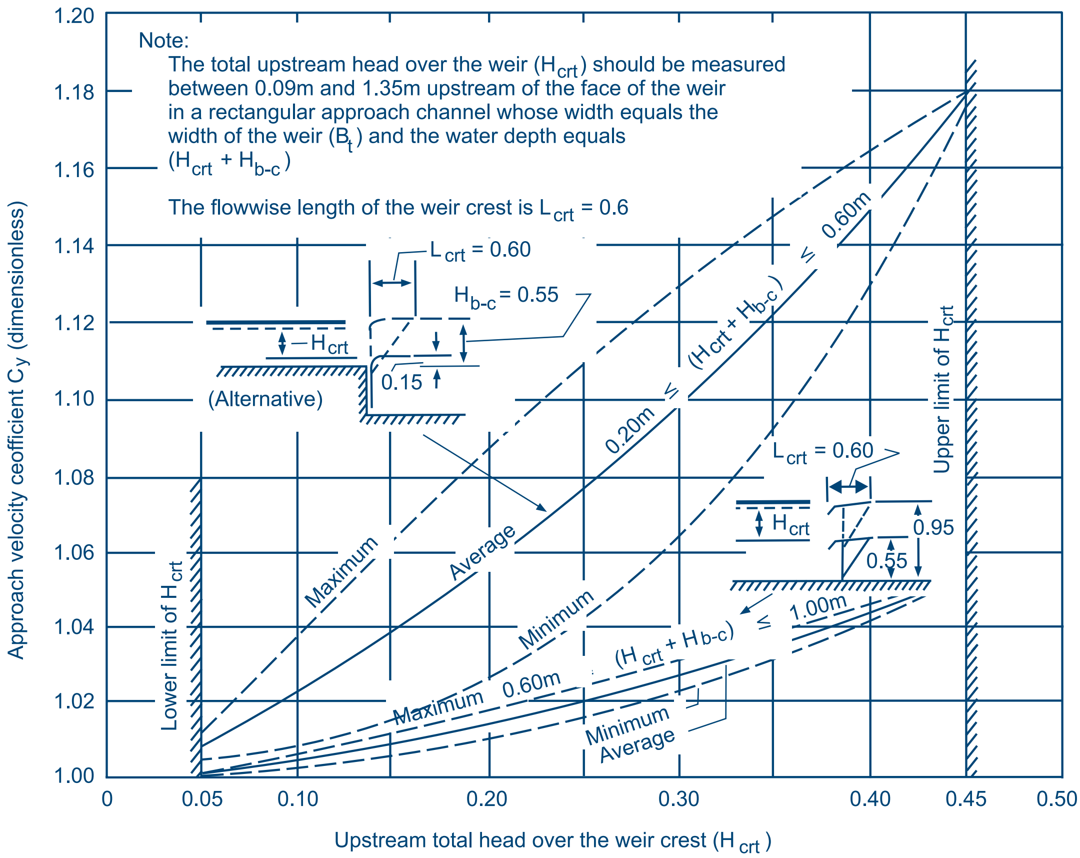
Where:

Q	=	Design discharge over the weir (m^3/sec)
C_d	=	Discharge coefficient
C_v	=	Approach velocity coefficient
g	=	Acceleration due to gravity (= 9.81 m/sec^2)
B_t	=	Width (or breadth) of the weir across the direction of flow (m)
H_{crt}	=	Design upstream water depth over the weir (m)

The value of the discharge coefficient, C_d, has been determined in laboratory tests. For field structures with concrete abutments, it is advisable to use an average value of C_d = 1.00. The value of the approach velocity coefficient, C_v, ranges between 1.00 and 1.18, depending on H_{crt} (Figure 75).

Figure 75

Approach velocity coefficient, C_v, as a function of the total head over the movable weir crest, H_{crt} (Source: FAO, 1975b)



Where both C_d and C_v are considered to be 1.00, substituting these values and the value for g in Equation 48 gives Equation 49:

Equation 49

$$q = 1.7 \times B_t \times H_{crt}^{3/2}$$

More details on the Romijn weir can be found in FAO (1975b).

6.6.3. Flumes

Discharge measurement flumes are extensively used in irrigation schemes mainly because they:

- Can be used under almost any flow condition
- Have smaller head-losses than weirs, thus are more accurate over a large flow range
- Are insensitive to the velocity of approach
- Are relatively less susceptible to sediment and debris transport

Example 32

A Romijn broad-crested weir has to discharge 0.0783 m³/sec. The maximum allowable water depth over the weir can be 0.15 m. What should be the minimum width of weir?

Considering a C_d value of 1.00 and an average C_v value of 1.04 (Figure 75), Equation 48 gives:

$$0.0783 = \frac{2}{3} \times 1.00 \times 1.04 \times \left[\frac{2}{3} \times 9.81\right]^{1/2} \times B_t \times 0.15^{3/2} \Rightarrow Bt = 0.76 \text{ m}$$

Using the simplified Equation 49 would give:

$$0.0783 = 1.7 \times B_t \times 0.15^{3/2} \Rightarrow B_t = 0.79 \text{ m}$$

However, major disadvantages of flumes include the relative large sizes and the accurate manufacturing/construction workmanship required for optimum performance (James, 1988).

A canal section that causes flow to pass from sub-critical through critical to the super-critical state forms a control and the discharge is a single valued function of the upstream water level. Critical flow can be achieved by raising the canal bed, thereby reducing the specific energy, or by decreasing the canal width, thereby increasing the discharge per unit width (see Section 6.6.1). This latter technique is the one used by flumes.

A flume has:

- A convergent section, in which the flow accelerates
- A throat, in which critical flow occurs
- A divergent section, in which the flow returns to normal

Super-critical flow passing from the throat will return to sub-critical flow downstream of the flume. This occurs due to the development of a hydraulic jump, which is induced within the divergent section by a sill or other barrier. Where there is sufficient head available, the divergent section of the flume could be avoided as the flow could fall freely in a stilling basin. In this case, weirs could also be used. However, if canals are expected to carry a lot of sediment, the flume should be the better choice.

Flumes are most commonly rectangular or trapezoidal in cross-section. The former type is the most simple to construct, but if the canal cross-section is not rectangular there is a risk that unpredictable flow patterns will result from an abrupt change of cross-section.

The most commonly used flumes are:

- Parshall flume
- Trapezoidal flume
- Cut-throat flume

Figure 76
Parshall flume

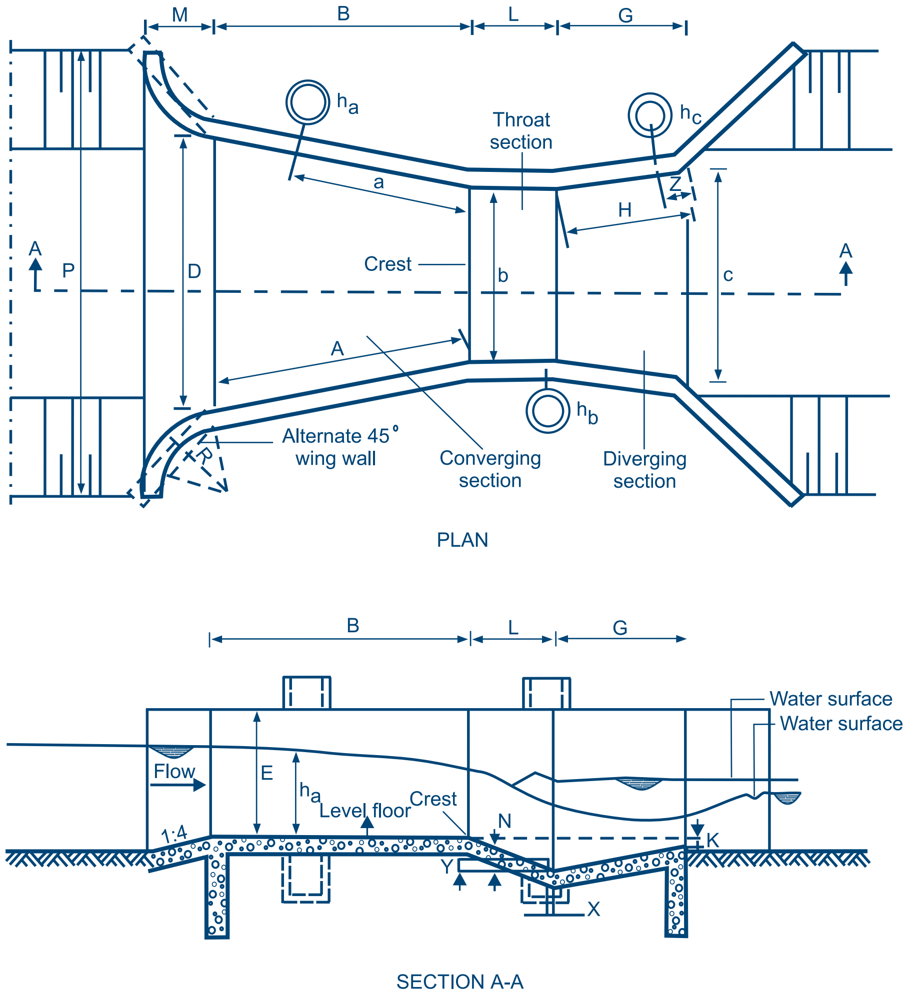


Table 31

Standard dimensions of Parshall flumes (the letters are shown in Figure 76) (Adapted from: FAO, 1975b)

b		A	a	B	C	D	E	L	G	H	K	M	N	P	R	X	Y	Z
' + "	mm	mm																
1"	25.4	363	242	356	93	167	229	76	203	206	19	-	29	-	-	8	13	3
2"	50.8	414	276	406	135	214	254	114	254	257	22	-	43	-	-	16	25	6
3"	76.2	467	311	457	178	259	457	152	305	309	25	-	57	-	-	25	38	9
6"	152.4	621	414	610	394	397	610	305	610	-	76	305	114	902	406	51	76	-
9"	228.6	879	587	864	381	575	762	305		-	76	305	114	1080	406	51	76	-
1"	304.8	1372	914	134	610	845	914	610	914	-	76	381	229	1492	508	51	76	-
1'6"	457.2	1448	965	1419	762	1026	914	610	914	-	76	381	229	1676	508	51	76	-
2'	609.6	1524	1016	1495	914	1206	914	610	914	-	76	381	229	1854	508	51	76	-
3'	914.4	1676	1118	1645	1219	1572	914	610	914	-	76	381	229	2222	508	51	76	-
4'	1219.2	1829	1219	1794	1524	1937	914	610	914	-	76	457	229	2711	610	51	76	-
5'	1524.0	1981	1321	1943	1829	2302	914	610	914	-	76	457	229	3080	610	51	76	-
6'	1828.8	2134	1422	2092	2134	2667	914	610	914	-	76	457	229	3442	610	51	76	-
7'	2133.6	2286	1524	2242	2438	3032	914	610	914	-	76	457	229	3810	610	51	76	-
8'	2438.4	2438	1626	2391	2743	3397	914	610	914	-	76	457	229	4172	610	51	76	-
10'	3048	-	1829	4267	3658	4756	1219	914	1829	-	76	-	343	-	-	305	229	-
12'	3658	-	2032	4877	4470	5607	1542	914	2438	-	152	-	343	-	-	305	229	-
15'	4572	-	2337	7620	5588	7620	1829	1219	3048	-	152	-	457	-	-	305	229	-
20'	6096	-	2845	7620	7315	9144	2134	1829	3658	-	305	-	686	-	-	305	229	-
25'	7620	-	3353	7620	8941	10668	2134	1829	3962	-	305	-	686	-	-	305	229	-
30'	9144	-	3861	7925	10566	12313	2134	1829	4267	-	305	-	686	-	-	305	229	-
40'	12192	-	4877	8230	13818	15481	2134	1829	4877	-	305	-	686	-		305	229	-
50'	15240	-	5893	8230	17272	18529	2134	1829	6096	-	305	-	686	-	-	305	229	-

Parshall flume

The Parshall flume is a widely-used discharge measurement structure. Figure 76 shows its general form. The characteristics of Parshall flumes are:

- Small head losses
- Free passage of sediments
- Reliable measurements even when partially submerged
- Low sensitivity to velocity of approach

The Parshall flume consists of a converging section with a level floor, a throat section with a downward sloping floor and a diverging section with an upward sloping floor. Flume sizes are known by their throat width.

Care must be taken to construct the flumes accurately if the calibration curves have to be used. Each size has its own characteristics, as the flumes are not hydraulic scale models of each other. In other words, each flume is an entirely different device (see Table 31).

The flow through the Parshall flume can occur either under free flow or under submerged flow conditions. Under free flow the rate of discharge is solely dependent on the throat width and the measured water depth, h_a. The water depth is measured at a fixed point in the converging section.

The upstream water depth-discharge relationship, according to empirical calibrations, has the following general form:

Equation 50

$$Q = K \times (h_a)^u$$

Where:

Q = Discharge (m^3/sec)

h_a = Water depth in converging section (m)

K = A fraction, which is a function of the throat width

u = Variable, lying between 1.522 and 1.60.

Table 32 gives the values for K and u for each flume size.

When the ratio of gauge reading h_b to h_a exceeds 60% for flumes up to 9 inches, 70% for flumes between 9 inches and 8 feet and 80% for larger flume sizes, the discharge is reduced due to submergence. The upper limit of submergence is 95%, after which the flume ceases to be an effective measuring device because the head difference between h_a and h_b becomes too small, such that a slight inaccuracy in either head reading results in a large discharge measurement error.

Table 32
Discharge characteristics of Parshall flumes

Throat width b feet + inches	Discharge range (m³/sec x 10⁻³) Minimum	Maximum	Equation $Q = K \times h_a^u$ (m³/sec)	Head range (m) Minimum	Maximum	Modular limit h_b/h_a (m)
1”	0.09	5.4	$0.0604\ h_a^{1.55}$	0.015	0.21	0.50
2”	0.18	13.2	$0.1207\ h_a^{1.55}$	0.015	0.24	0.50
3”	0.77	32.1	$0.1771\ h_a^{1.55}$	0.030	0.33	0.50
6”	1.50	111	$0.3812\ h_a^{1.58}$	0.030	0.45	0.60
9”	2.50	251	$0.5354\ h_a^{1.53}$	0.030	0.61	0.60
1’	3.32	457	$0.6909\ h_a^{1.522}$	0.030	0.76	0.70
1’6”	4.80	695	$1.056\ h_a^{1.538}$	0.030	0.76	0.70
2’	12.1	937	$1.428\ h_a^{1.550}$	0.046	0.76	0.70
3’	17.6	1 427	$2.184\ h_a^{1.566}$	0.046	0.76	0.70
4’	35.8	1 923	$2.953\ h_a^{1.578}$	0.060	0.76	0.70
5’	44.1	2 424	$3.732\ h_a^{1.587}$	0.060	0.76	0.70
6’	74.1	2 929	$4.519\ h_a^{1.595}$	0.076	0.76	0.70
7’	85.8	3 438	$5.312\ h_a^{1.601}$	0.076	0.76	0.70
8’	97.2	3 949	$6.112\ h_a^{1.607}$	0.076	0.76	0.70
	m³/sec					
10’	0.16	8.28	$7.463\ h_a^{1.60}$	0.09	1.07	0.80
12’	0.19	14.68	$8.859\ h_a^{1.60}$	0.09	1.37	0.80
15’	0.23	25.04	$10.96\ h_a^{1.60}$	0.09	1.67	0.80
20’	0.31	37.97	$14.45\ h_a^{1.60}$	0.09	1.83	0.80
25’	0.38	47.14	$17.94\ h_a^{1.60}$	0.09	1.83	0.80
30’	0.46	56.33	$21.44\ h_a^{1.60}$	0.09	1.83	0.80
40’	0.60	74.70	$28.43\ h_a^{1.60}$	0.09	1.83	0.80
50’	0.75	93.04	$35.41\ h_a^{1.60}$	0.09	1.83	0.80

The discharge under submerged conditions is:

Equation 51

$$Q_s = Q - Q_c$$

Where:

Q_c = Reduction of the modular discharge due to submergence.

Figure 77 gives the corrections Q_c for submergence for flumes with 6 inch, 9 inch and 1 foot throat width. The correction for the 1 foot flume is made applicable to other sizes by multiplying the correction Q_c for the 1 foot by the factors given in Figure 77 (1 foot flume).

Usually the smallest practical size of flume is selected because of economical reasons. In general the width should vary between 1/3 to 1/2 of the canal width. Often the head loss across the flume is the limiting factor.

The procedure for selecting the appropriate flume is as follows:

Step 1: Collect site information: maximum and minimum canal discharges, corresponding normal flow depths and canal dimensions.

Step 2: List flumes capable of taking the given discharge, using Table 32.

Then, for free flow at the maximum canal discharge:

i) List values of h_a for the maximum canal discharge passing through the flumes.

ii) Apply the submergence limit appropriate to the flume to find the value of h_b corresponding to the values of h_a (Table 32).

iii) Subtract h_b from the normal flow depth at maximum discharge to give the vertical distance from the canal bed to the flume crest level. This assumes that at maximum submergence the downstream stage is the same as that at h_b, and that the flow downstream of the flume is not affected by it.

iv) Find the head loss across the flume at maximum discharge (Figure 78). Add this to the downstream water depth to obtain the water depth upstream of the flume.

v) Select the smallest size of flume for which the upstream stage is acceptable.

Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

a. Parshall flume with a throat width b of 6 inch or 15.2 cm

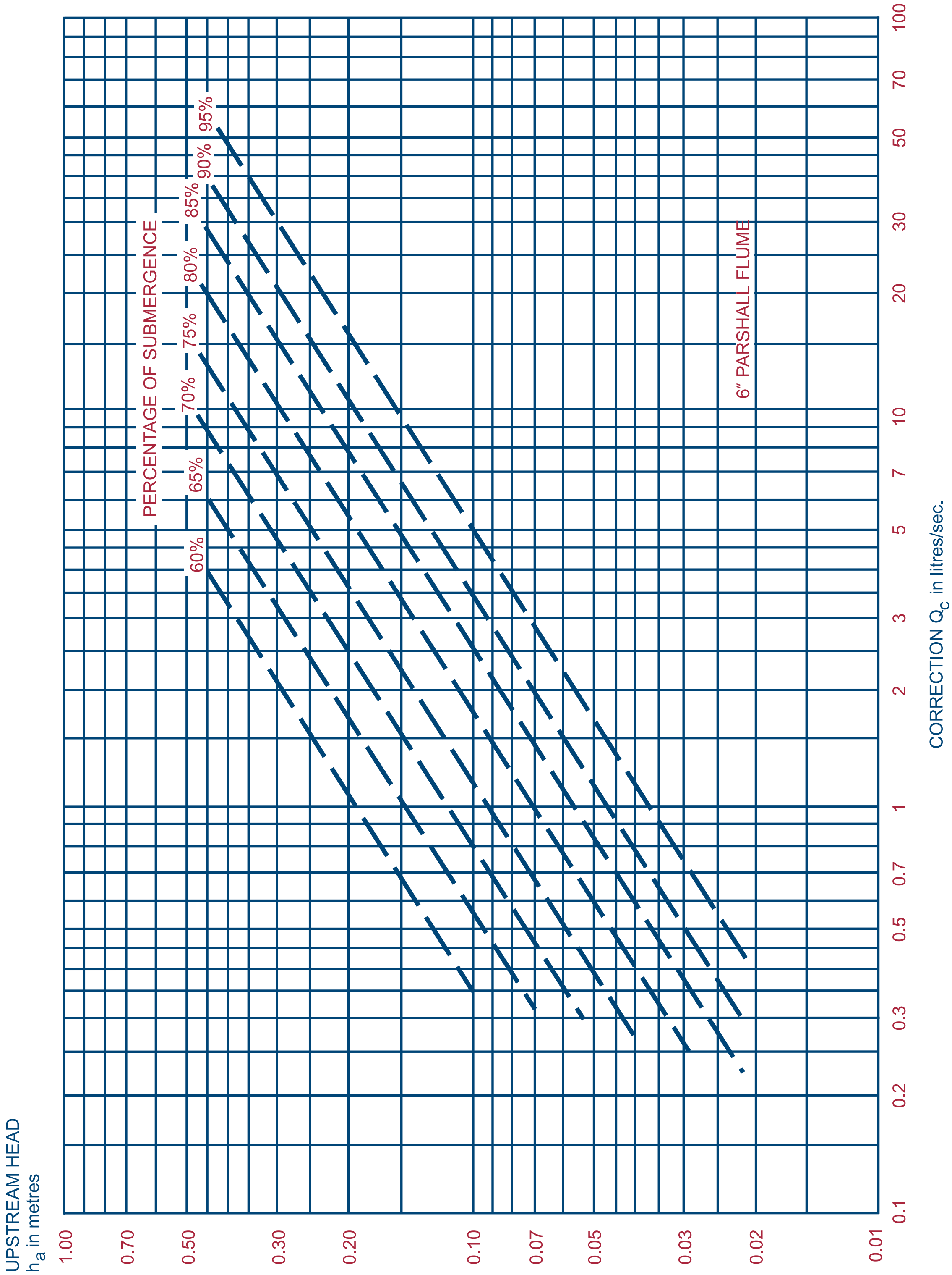


Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

b. Parshall flume with a throat width b of 9 inch or 22.9 cm

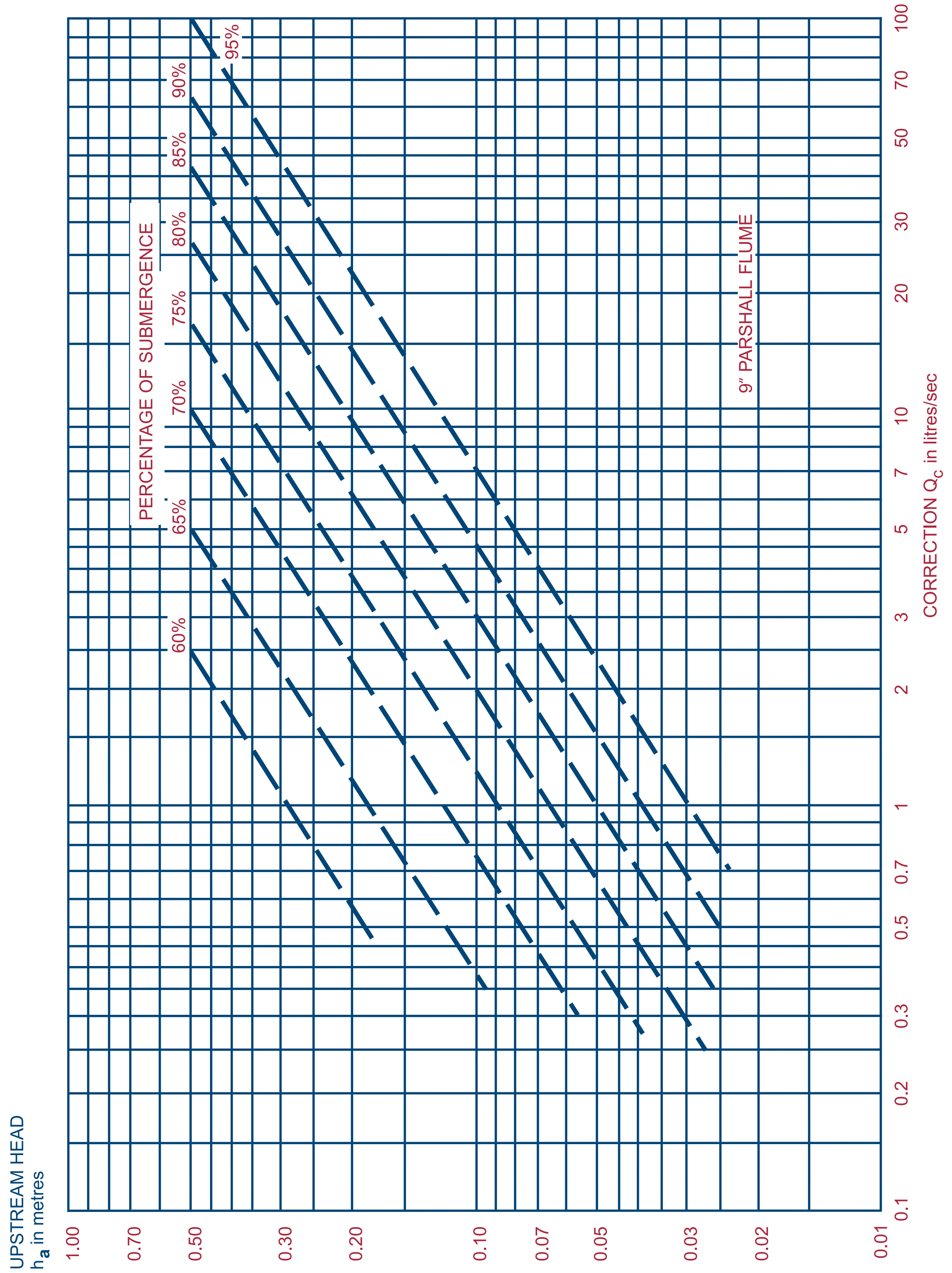


Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

c. Parshall flume with a throat width b of 1 foot or 30.5 cm

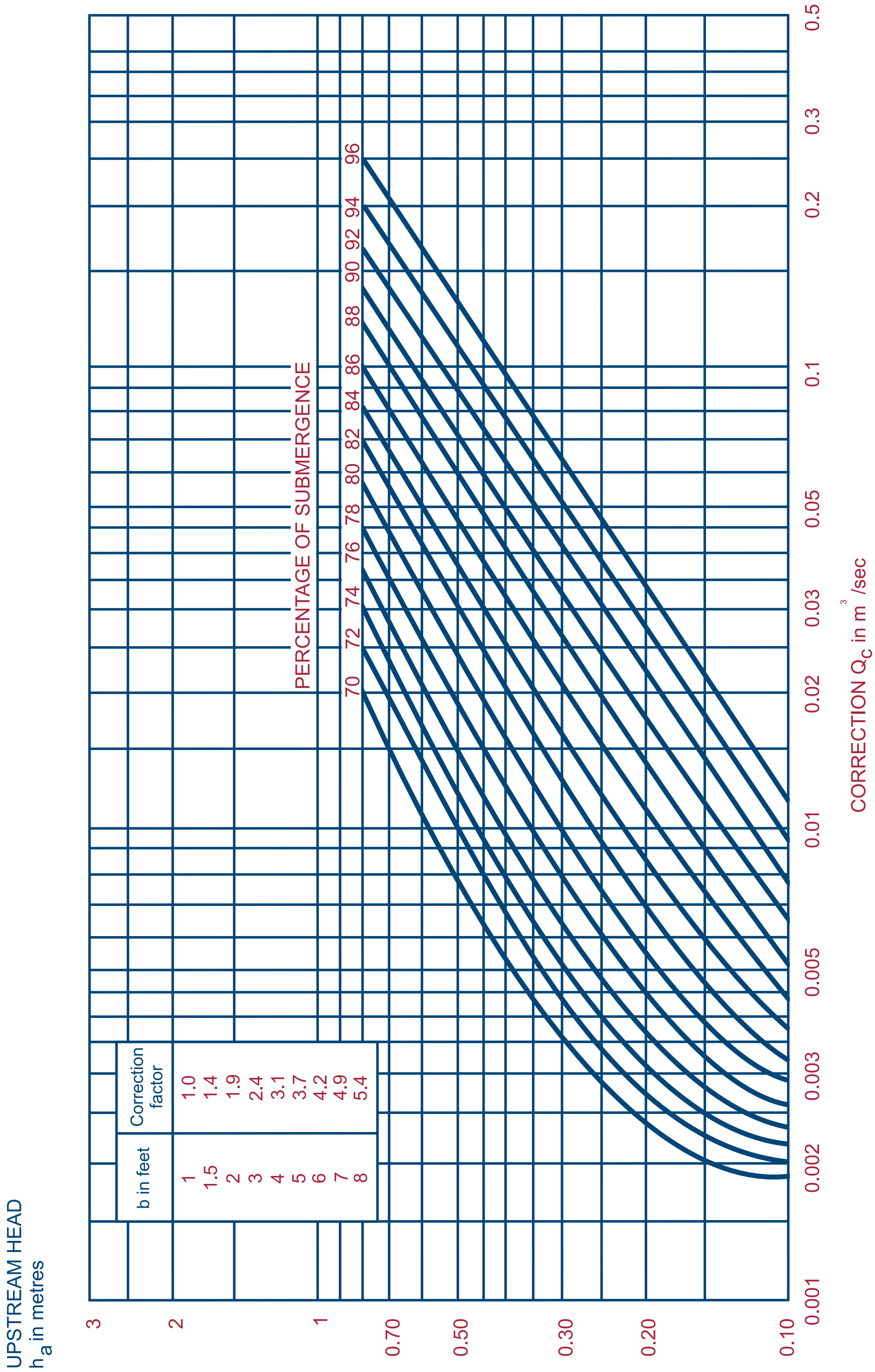
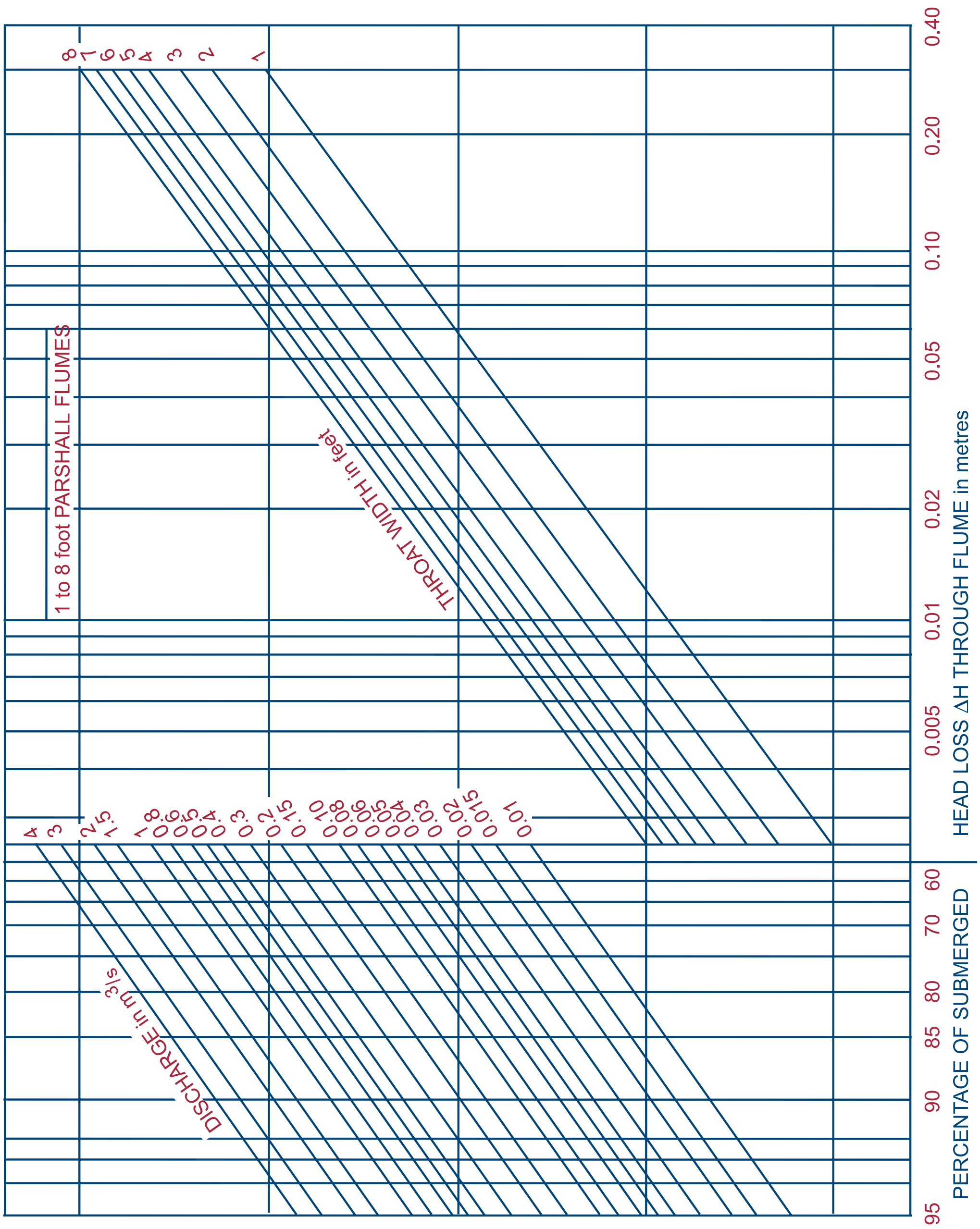


Figure 78
Head loss through Parshall flumes



Example 33

Select the most appropriate flume to be placed in a canal with the following characteristics:

Maximum discharge = 0.566 m^3/sec

Canal water depth = 0.77 m

Canal banks at 3 m apart

The freeboard of the canal = 0.15 m

a. Consider the flumes with a throat width of 3 and 4 foot. Table 32 gives discharge equations for the different flume sizes.

The discharge equation for the 3 foot flume is:

$Q = 2.184 \times h_a^{1.566} \Rightarrow 0.566 = 2.184 \times h_a^{1.566} \Rightarrow h_a = 0.43$ m

The discharge equation for the 4 foot flume is:

$Q = 2.953 \times h_a^{1.578} \Rightarrow 0.566 = 2.953 \times h_a^{1.578} \Rightarrow h_a = 0.35$ m

b. Assume that the submerging of 70% must not be exceeded. This means that $h_b = 0.70 \times h_a$ (Table 32). Thus for the 3 foot flume the water depth h_b = 0.30 m and for the 4 foot flume h_b = 0.25m.

c. The elevation of the crest above the bottom of the canal (K in Figure 76) equals the design water depth minus h_b. Thus K = 0.77 m - 0.30 m = 0.47 m for the 3 foot flume and K = 0.77 m - 0.25 m = 0.52 m for the 4 foot flume.

d. From Figure 78 it can be seen that the head loss is 0.16 m for the 3 foot flume and 0.13 m for the 4 foot flume. Thus the upstream water depth becomes 0.77 m + 0.16 m = 0.93 m and 0.77 m + 0.13 m = 0.90 m for the 3 foot and 4 foot flume respectively.

e. The upstream water depth of the 3 foot flume just exceeds the sum of the normal water depth and freeboard, thus overtopping would result. The 4 foot flume is just within the available limit of depth. Thus this flume could be selected for implementation. If there was sufficient freeboard available for either of the flumes, considering the rise in water level upstream of the flume, one should select the 3 foot flume because this is cheaper.

Trapezoidal flume

Whenever the canal section is not rectangular, trapezoidal flumes such as those shown in Figure 79, are often preferred, especially for measuring smaller discharges. A typical trapezoidal flume has an approach, a converging section, a throat, a diverging and an exit section. A minimum transition will be required. An additional advantage is the flat bottom, which allows sediment to pass through fairly easily. Furthermore, the loss in head may be less for comparable discharges.

Trapezoidal flumes are particularly suited for installation in concrete-lined canals. The flume should normally be put on top of the lining, thus constricting the flow section to the extent required for free flow conditions over a whole range of discharges up to the canal design discharge. As a rule of thumb, one can say that the lower the canal gradient the higher the elevation of the flume above the canal bed level.

The flow characteristics of the flume can be determined experimentally. This allows for the calibrations of the flume. As an example, a flume with dimensions such as those given in Figure 79 can be located in a canal with a bed width of 0.30 m (1 foot), having side slopes of 1:1. The range of calibrated water depth is 6-37 cm and the range of calibrated discharge is 1.4-169 l/sec. This will suit most conditions in a typical small-scale irrigation canal.

Cut-throat flume

The cut-throat flume has a converging inlet section, throat and diverging outlet section. The flume has a flat bottom and vertical walls (Figure 80).

It is preferable to have the cut-throat flume operating under free flow conditions. This facilitates measurements and ensures a high degree of accuracy. Free flow conditions through the cutthroat flume are described by the following equations:

Equation 52

$$Q = C \times (h_a)^n$$

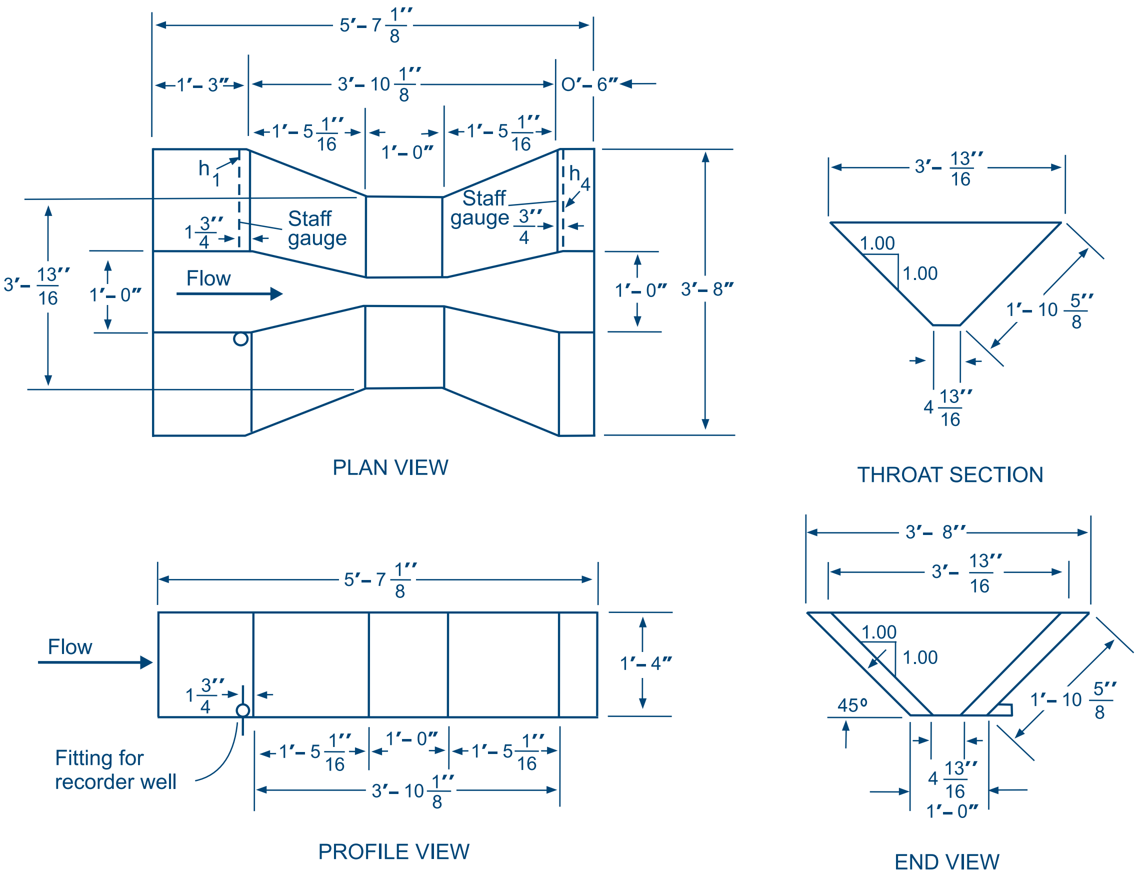
Equation 53

$$C = K \times W^{1.025}$$

Where:

Q	=	Discharge (m^3/sec)
C	=	Free flow coefficient
h_a	=	Upstream water depth (m)
K	=	Flume length coefficient
W	=	Throat width (m)

Figure 79
Trapezoidal flume (Source: FAO, 1975b)



For a given flume length, the values of n and K are obtained from Figure 81. In order to ensure free flow conditions, the ratio between the water depths h_a and h_b should not exceed a certain limit, which is called the transition submergence, S_t.

Example 34

A cut-throat flume is to be installed with a length L = 1.22 m and throat width W = 0.36 m. The maximum discharge through the structure is 0.20 m^3/sec. How should it be installed in order to operate under free flow conditions?

From Figure 81, it follows that for a flume length L = 1.22 m:

S_t = 68.2%

K = 3.1

n = 1.75

Using Equations 53 and 52 respectively:

$C = 3.1 \times 0.36^{1.025} = 1.088$

$Q = 1.088 \times h_a^{1.75} = 0.200 \Rightarrow h_a = 0.38$ m

$$S_t = \frac{h_b}{h_a} = 0.682 \Rightarrow h_b = 0.682 \times 0.38 = 0.26 \text{ m}$$

Therefore the floor of the flume should be placed not lower than 0.26 m below the normal water depth, in order to let pass the maximum discharge of 0.20 m^3/sec. under free flow conditions.

Figure 80

Cut-throat flume (Source: FAO, 1975b)

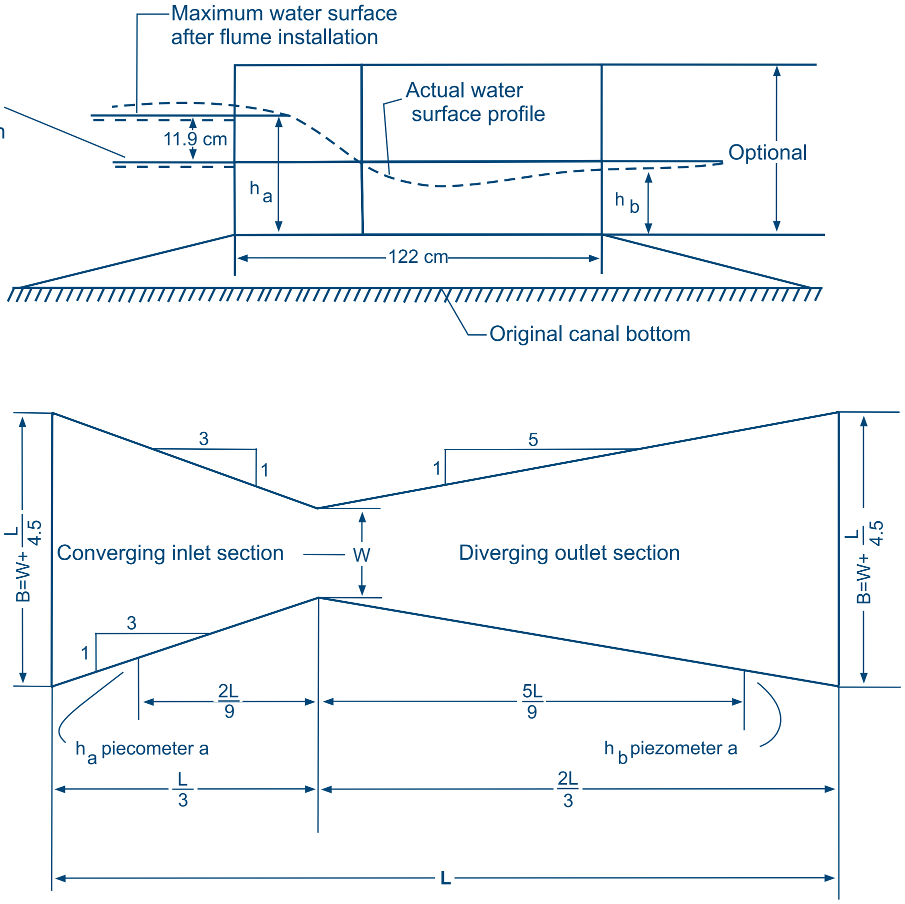
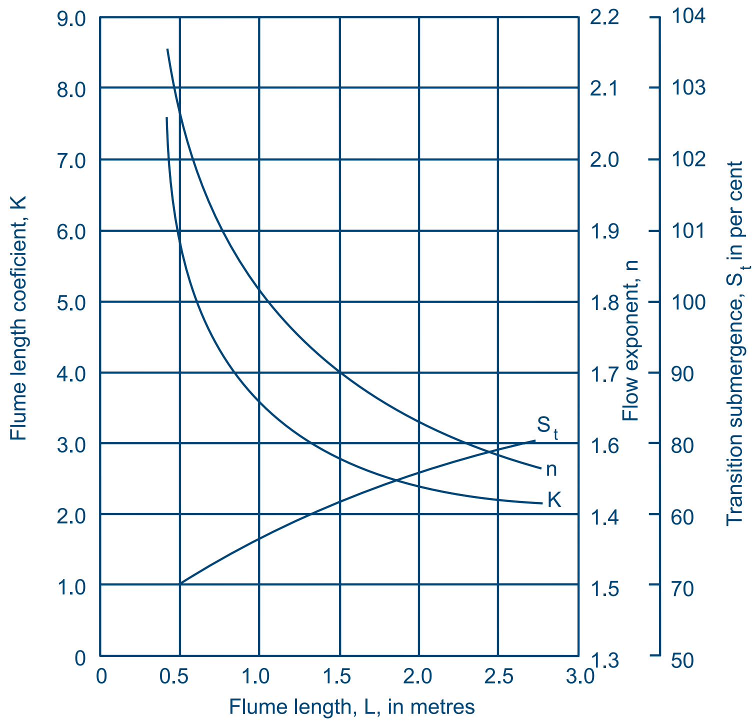


Figure 81

Cut-throat flume coefficients (Source: FAO, 1975b)



6.4.4. Orifices

Orifices, such as gates and short pipes, are also used as water measuring devices (Figure 82). However, they do not offer any advantage over the use of weirs or flumes. Furthermore, their calibrations are not as accurate nor as stable as other types of measuring devices.

For weirs the discharge is proportional to the head above the crest raised to the power 3/2 (Equations 44, 45, 46, 48). Therefore, they are sensitive to the fluctuations in the upstream water level. For orifices, including gates and short pipes, the discharge is proportional to the head of water above the crest raised to the power 1/2, as shown by Equation 34 (see Section 6.1.3). Therefore, they are less sensitive to small fluctuations of the upstream water level.

Under submerged conditions both the upstream and downstream sides of the structure need water level recordings. For free flow conditions, the discharge is a function of the upstream water depth alone.

Figure 82
Examples of orifices

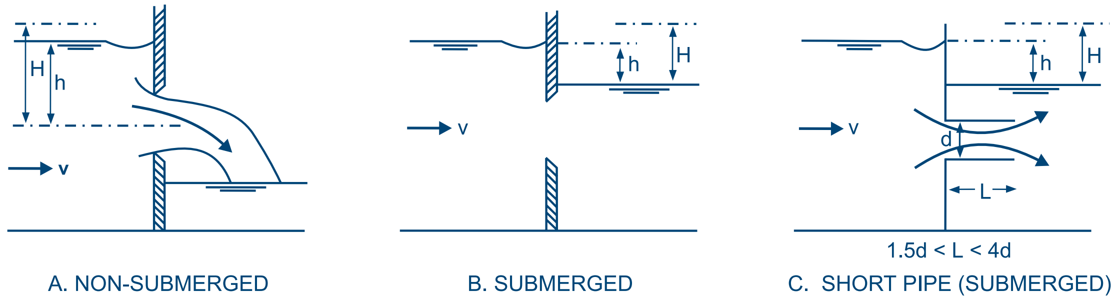
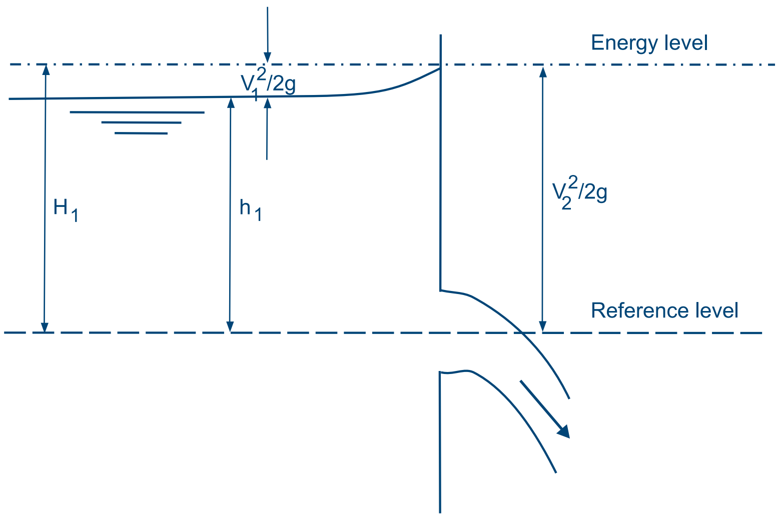


Figure 83
Free discharging flow through an orifice



Example 35

A circular orifice is placed in a canal, which discharges 0.0783 m³/sec. The maximum allowable water depth over the centre of the orifice is 0.25 m. What should be the opening of the orifice?

Substituting the above data in Equation 34 gives:

$$0.0783 = 0.6 \times (\frac{1}{4} \times \pi \times d^2) \times (2 \times 9.81 \times 0.25)^{1/2} \Rightarrow d^2 = \frac{0.0783}{1.0437} = 0.075 \Rightarrow d = 0.27 \text{ m.}$$

Thus the diameter of the orifice should be 0.27 m

The general discharge equation for a free flow orifice is (Equation 34):

$Q = C \times A \times (2gh_1)^{1/2}$

Where:

Q = Design discharge through orifice (m^3/sec)
C = Design coefficient (approximately 0.60)
A = Cross-sectional area of the orifice (m^2)
g = Gravitational force (9.81 m/sec^2)
h_1 = Water depth upstream of orifice over reference level (m) (Figure 83)

Partially-opened sluice gates could be used for discharge measurements, in which case they will be acting like submerged orifices (Figure 84).

For partially-opened sluice gates and submerged orifices the discharge equation reads:

Equation 54

$Q = C \times A (2g[h_1 - h_2])^{1/2}$

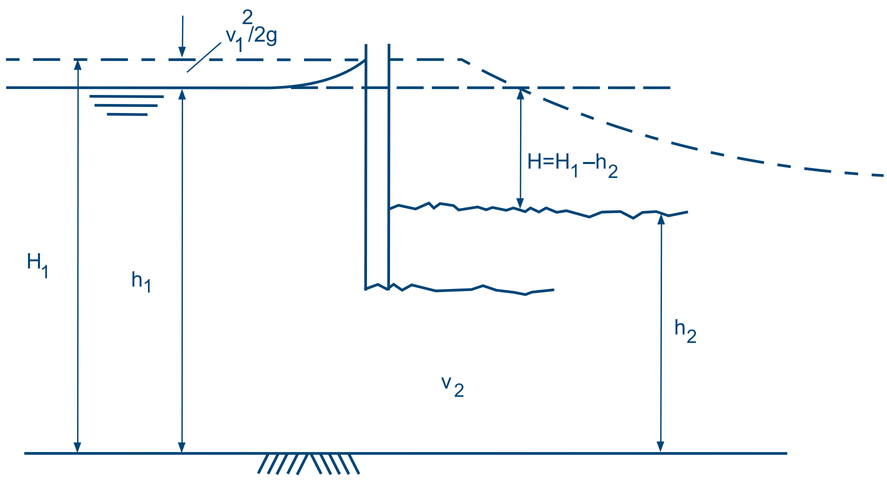
Where:

Q = Design discharge through orifice (m^3/sec)
C = Discharge coefficient, which is 0.63 for sluice gates and submerged orifices and 0.85 for short pipes
A = Cross-sectional area of the orifice (m^2)
g = Gravitational force (9.81 m/sec^2)
h_1 = Water depth upstream of orifice over reference level (m)
h_2 = Water depth downstream of the structure (m)

6.6.5. Current meters

Current meters are used to measure the velocity in a canal, from where the discharge can be calculated using the Continuity Equation 12 (see Section 5.1). Most current meters have a propeller axis in the direction of the current. The flowing water sets the propeller turning. On a meter,

Figure 84
Sluice gate under submerged conditions



Example 36

A sluice gate is installed in a canal with a design water depth of 0.30 m. The canal discharges 0.0783 m^3/sec. The maximum allowable rise in water level upstream of the sluice gate is 0.25 m. The width of the gate opening is 0.40 m. What should be the height d of the opening?

h_2 being 0.30 m and the allowable rise in water level upstream of the gate being 0.25 means that:

$h_1 = 0.30 \text{ m} + 0.25 \text{ m} = 0.55 \text{ m}.$

Substituting the above data in Equation 54 gives:

$0.0783 = 0.63 \times (0.40 \times d) \times (2 \times 9.81 \times [0.55 - 0.30])^{1/2} \Rightarrow d = 0.14 \text{ m}.$

forming part of the equipment, the number of revolutions per time unit can be read and, by means of a calibrated graph or table, the velocity can be determined. A well-known type of current meter is the Ott instrument C31 for velocities up to 10 m (Figure 85). Propeller meters are reliable and accurate, but rather expensive.

In measuring the velocities, the number of points per vertical and the number of verticals per cross-section should be determined. For this purpose, the quantity of work and the time required should be weighed against the degree of accuracy (Euroconsult, 1989). For example, measurements can be taken at 10 cm horizontal distance over the cross-section and at 0.2h and 0.8h depth at each 10 cm (h is the water depth). The velocity is the average of the velocity at 0.2h and 0.8h depth. If the water depth is less than 0.5-0.6 m, one reading can be done at 0.6h. Then, for each vertical the flow per unit width can be calculated according to $q = v_{average} \times h$ (Figure 86a). These q_s are distributed over the total width (Figure 86b) and the area between the q-line and the water surface gives the total discharge. It is also possible to establish the discharge per section and to consider the sum of the discharges in the sections as the total discharge.

Figure 85
Ott C31 propeller instrument

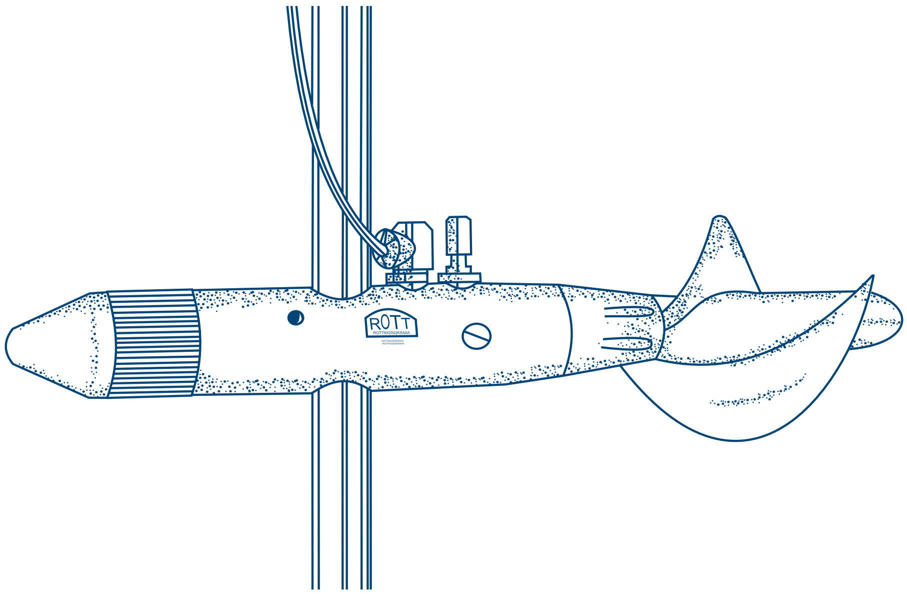
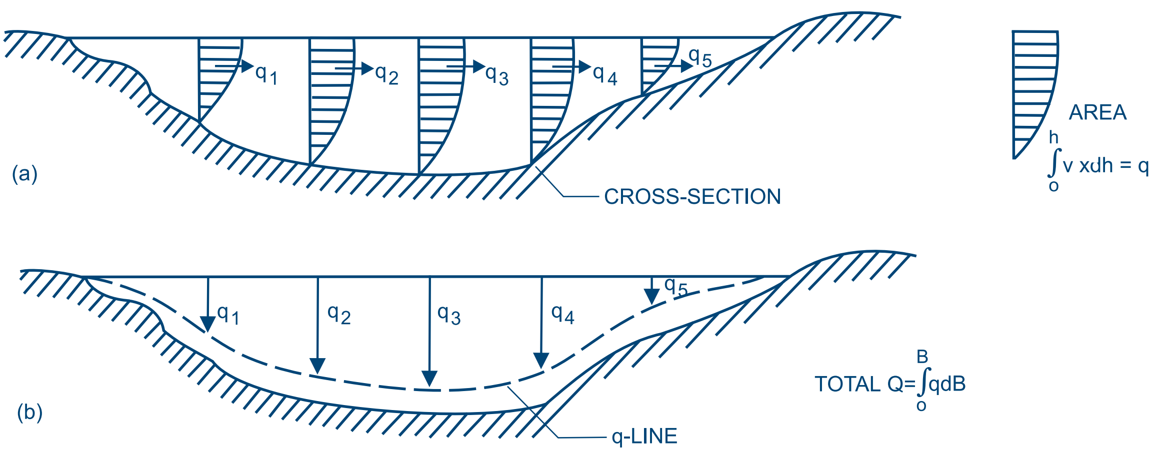


Figure 86
Depth-velocity integration method



6.7. Discharge measurement in pipelines

Several types of devices can be used to measure the discharge in pipelines. This section will discuss differential pressure and rotating mechanical meters, as they are the ones commonly used.

6.7.1. Differential pressure flow meters

Differential pressure flow meters create a pressure difference that is proportional to the square of the discharge. The pressure difference is created by causing flow to pass through a contraction. Manometers, bourdon gauges, or pressure transducers are normally utilized to measure the pressure difference. One good example of a differential pressure flow meter is the Venturi tube (Figure 87).

Venturi tube

The pressure drop between the inlet and throat is created as water passes through the throat. In the section downstream of the throat, the gradual increase in cross-sectional area causes the velocity to decrease and the pressure to increase. The pressure drop between the Venturi inlet and the throat is related to the discharge, as follows:

Equation 55

$$Q = \frac{Cd^2K(P_1 - P_2)^{1/2}}{[1 - (d/D)^2]^{1/2}}$$

Where:

Q = Discharge (l/min)
C = Flow coefficient
D = Diameter of upstream section (cm)
d = Diameter of contraction (cm)
P_1 = Pressure in upstream section (kPa)
P_2 = Pressure in contraction (kPa)
K = Unit constant (K is 6.66 for Q in l/min, d and D in cm, and P_1 and P_2 in kPa)

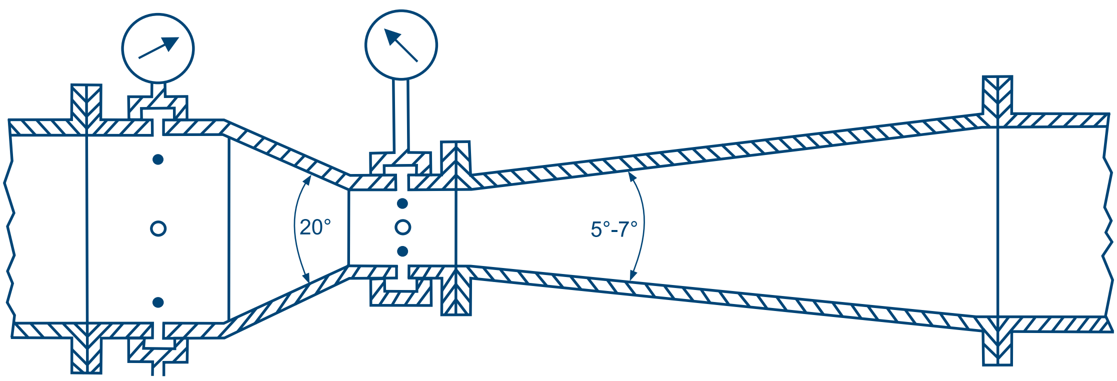
The flow coefficient C for a Venturi metre is 0.97.

6.7.2. Rotating mechanical flow meters

There are many types of rotating mechanical flow meters used in pipelines. These flow meters normally have a rotor that revolves at a speed roughly proportional to the discharge and a device for recording and displaying the discharge and total volume. The rotor may be a propeller or axial flow turbine, or a vane-wheel with the flow impinging tangentially at one or more points.

Calibration tests are usually needed to accurately relate rotor revolutions to the flow. The lowest discharge that can be accurately measured by a rotating mechanical flow meter depends on the amount of bearing friction that can be tolerated while the occurrence of cavitation often establishes the largest flow rate that can be measured (see Module 5). Head loss through most rotating mechanical discharge meters is moderate.

Figure 87
Venturi flow meter



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

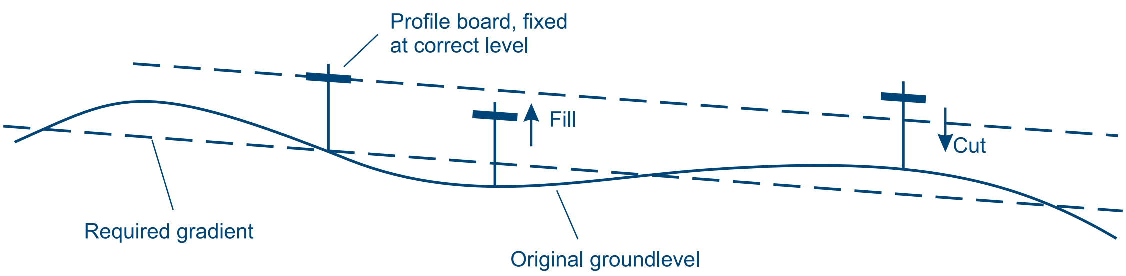
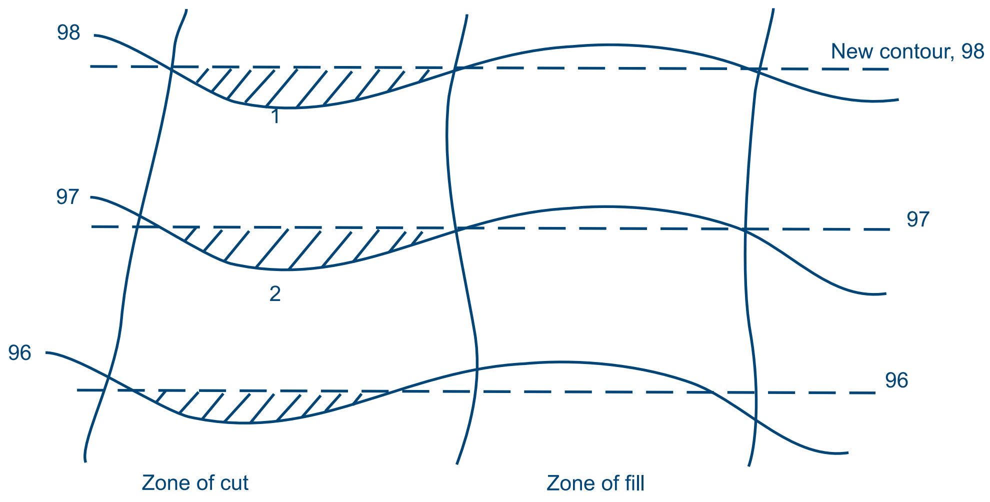


Figure 89
The contour method of land levelling



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:

$$\frac{(3.5 \times 88.678) + (7 \times 88.846) + (7 \times 88.919) + (3.x\ 89.126)}{3.5 + 7 + 7 + 7 + 3.5}$$

$$= 88.905 \text{ m}$$

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

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

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

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

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:

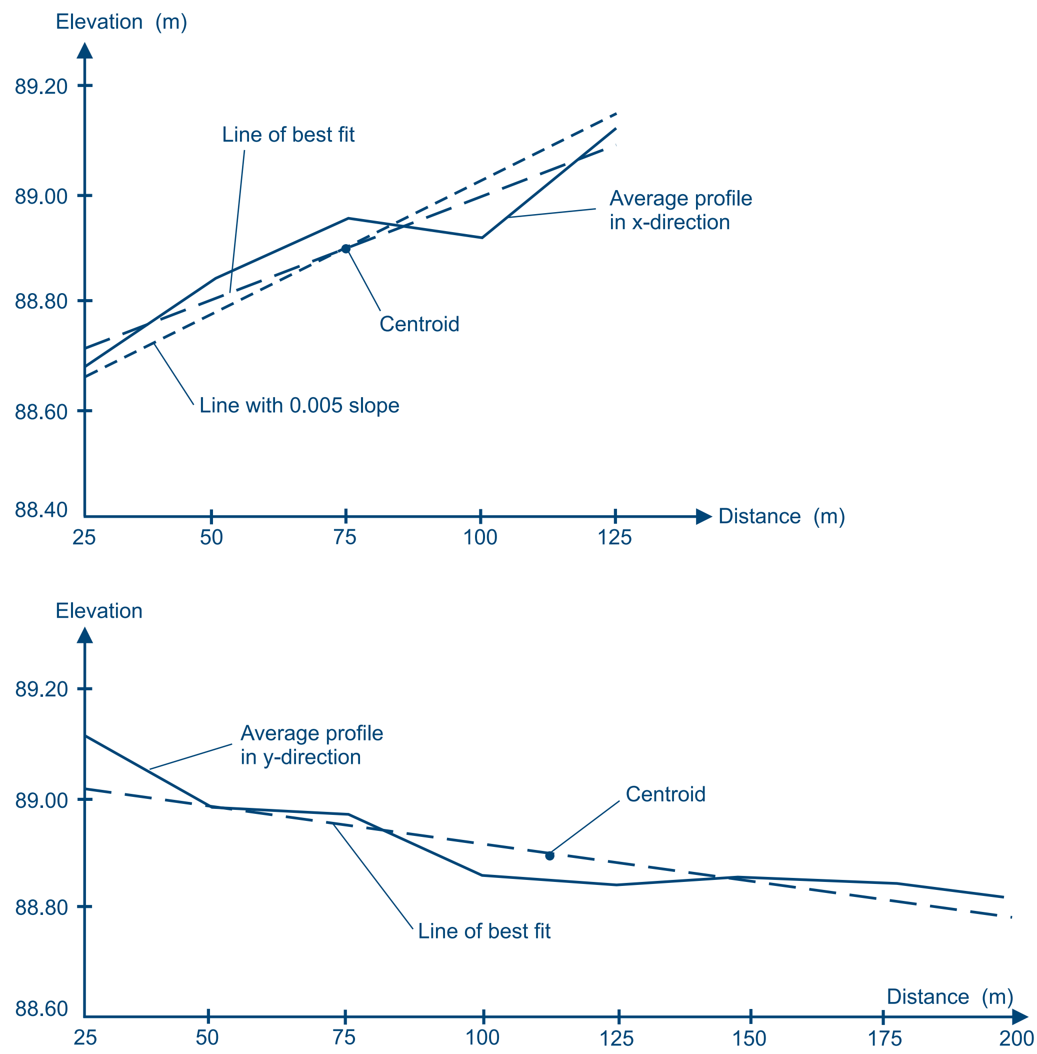
$$G_X = \frac{33\ 363.525 - \dfrac{(375 \times 444.524)}{5}}{34\ 375 - \dfrac{140\ 625}{5}} = \frac{24.225}{6\ 250} = 0.0039$$

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):

$$EL(X,Y) = (0.0039 \times X) - (0.0015 \times Y) + 88.781$$

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:

$$EL(25,25) = (0.0039 \times 25) - (0.0015 \times 25) + 88.781$$
$$= 88.841 \text{ m}$$

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:

$$EL(X,Y) = (0.005 \times X) - (0.0015 \times Y) + 88.698$$

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

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

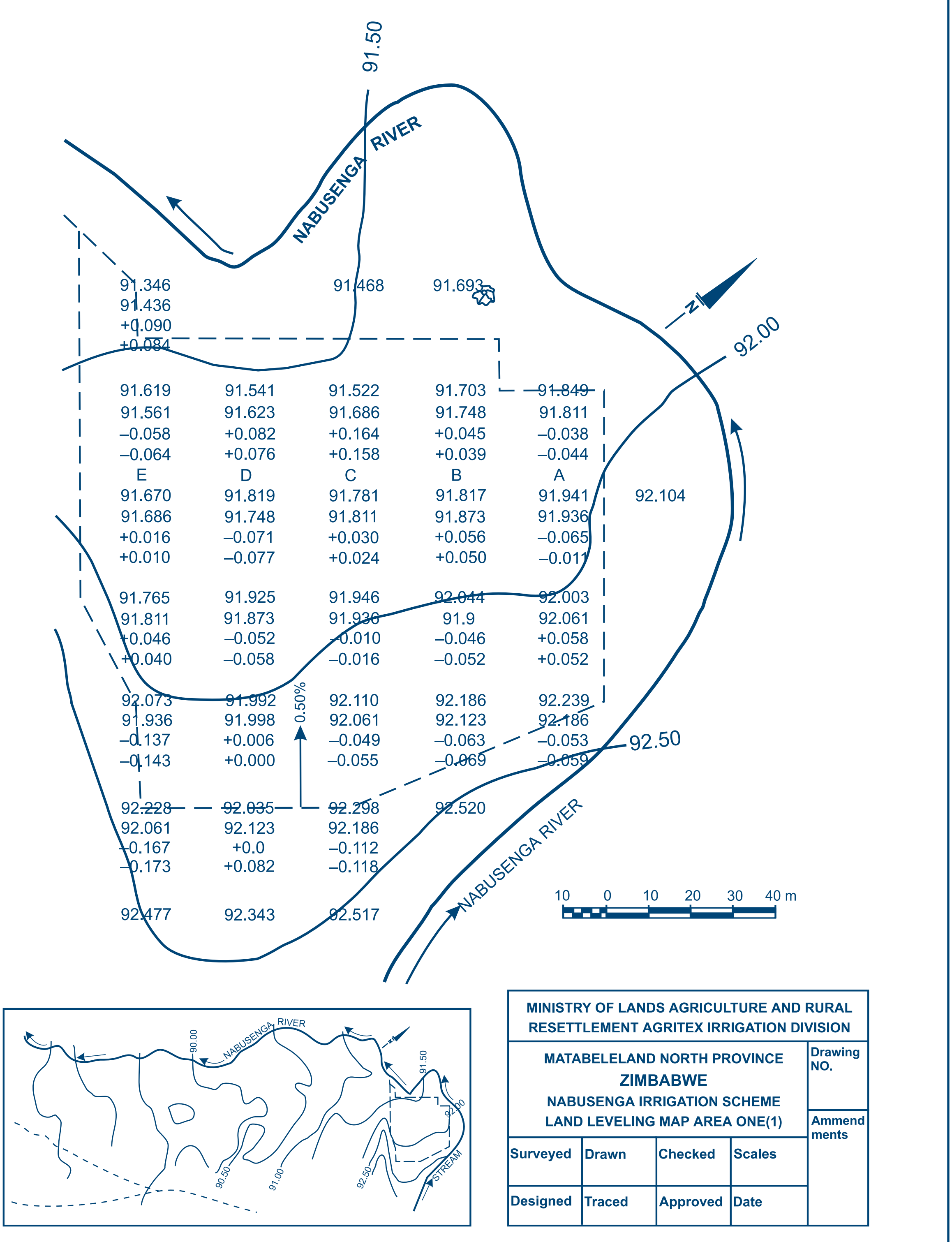
Equation 65

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

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):

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

$$= 0.0075 \text{ m}$$

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 m^3

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 m^3

Table 38b

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

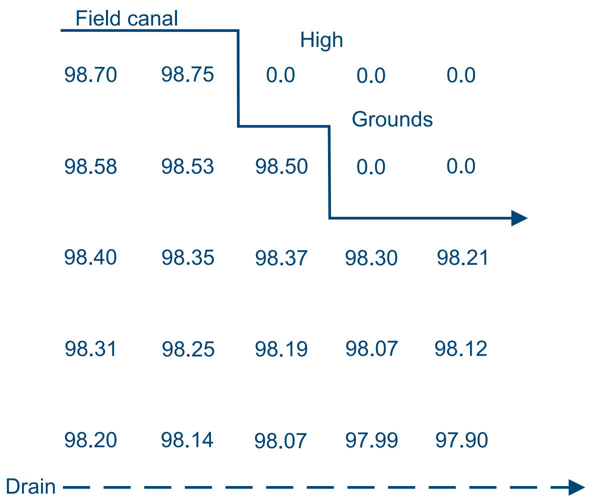
Location N	Location M	Elevation (m)	Ground elevation (m)	Operation (m)
1	1	9.70	10.01	F 0.31
1	2	10.02	9.99	C 0.03
1	3	10.03	9.97	C 0.06
1	4	10.06	9.95	C 0.11
1	5	9.96	9.94	C 0.02
1	6	10.08	9.92	C 0.16
1	7	9.94	9.90	C 0.04
1	8	10.03	9.88	C 0.15
1	9	9.97	9.86	C 0.11
1	10	10.04	9.85	C 0.19
1	11	9.81	9.83	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

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

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:

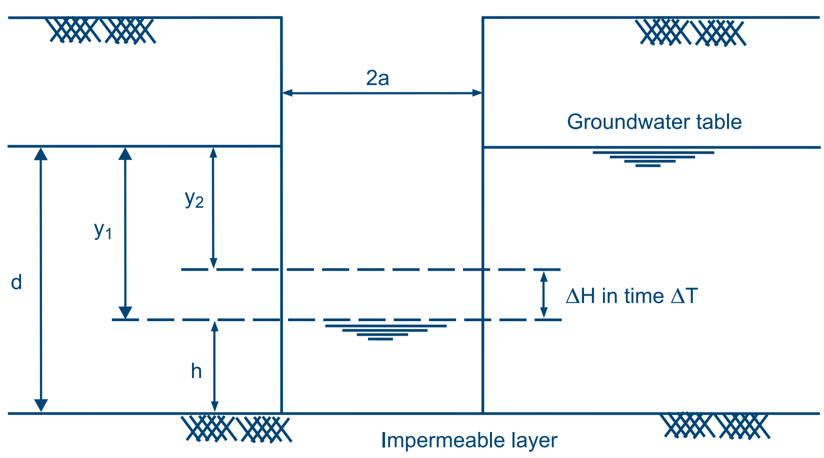
$$k = \frac{3\,600 \times 0.04^2}{(1.26 + 10 \times 0.04) + \left[2 - \frac{0.0284}{1.26}\right] \times 0.284} \times \frac{5.6}{50}$$

$\Rightarrow$ k = 0.77 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

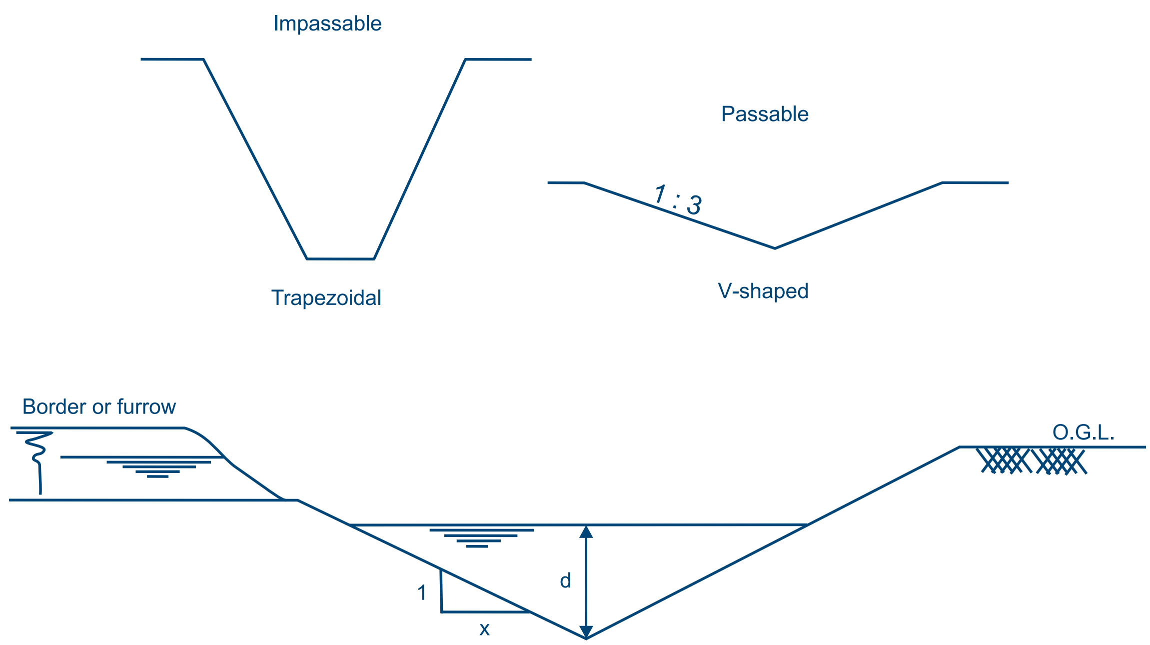
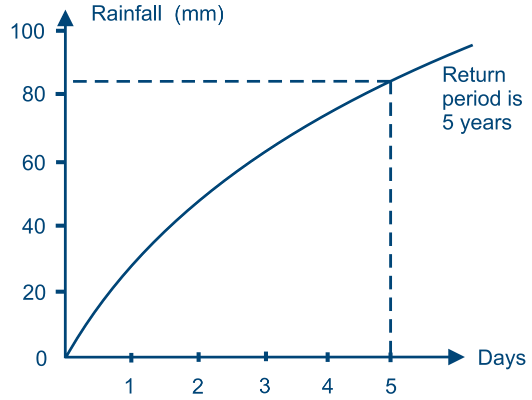


Figure 96
Rainfall-duration curve



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:

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

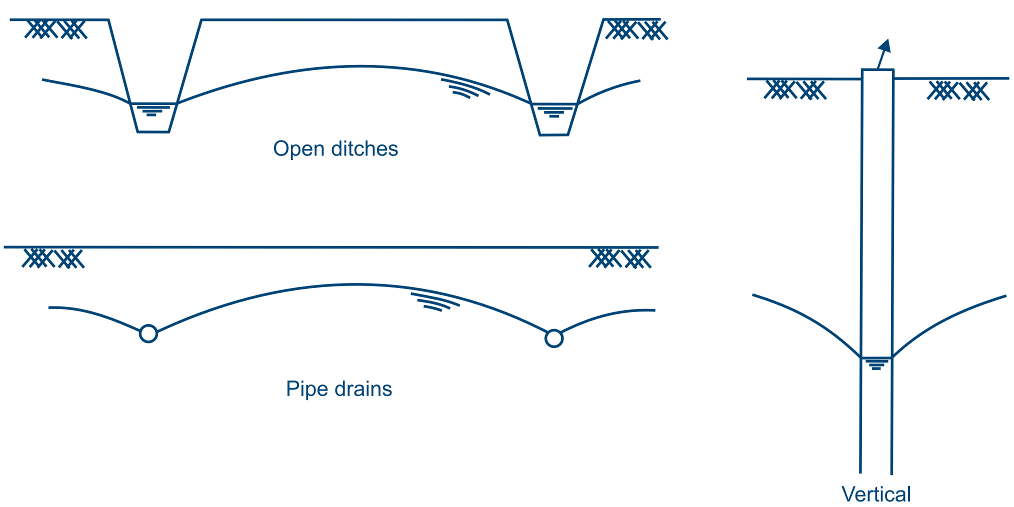
$$\Rightarrow Q = 0.625 \text{ m}^3\text{/sec or } 6.25 \text{ litres/sec per ha}$$

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:

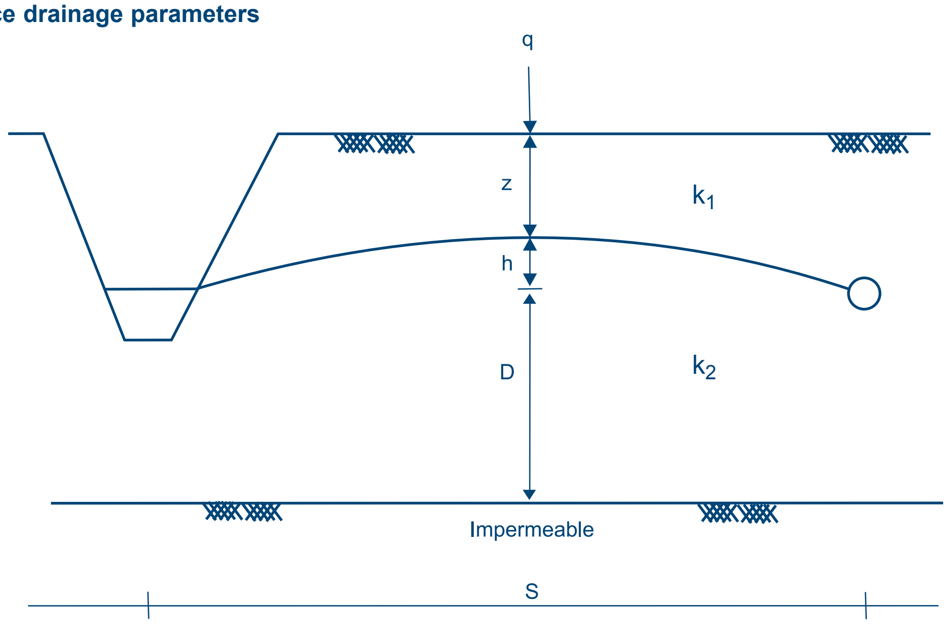
Equation 72

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

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:

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

and

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

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:

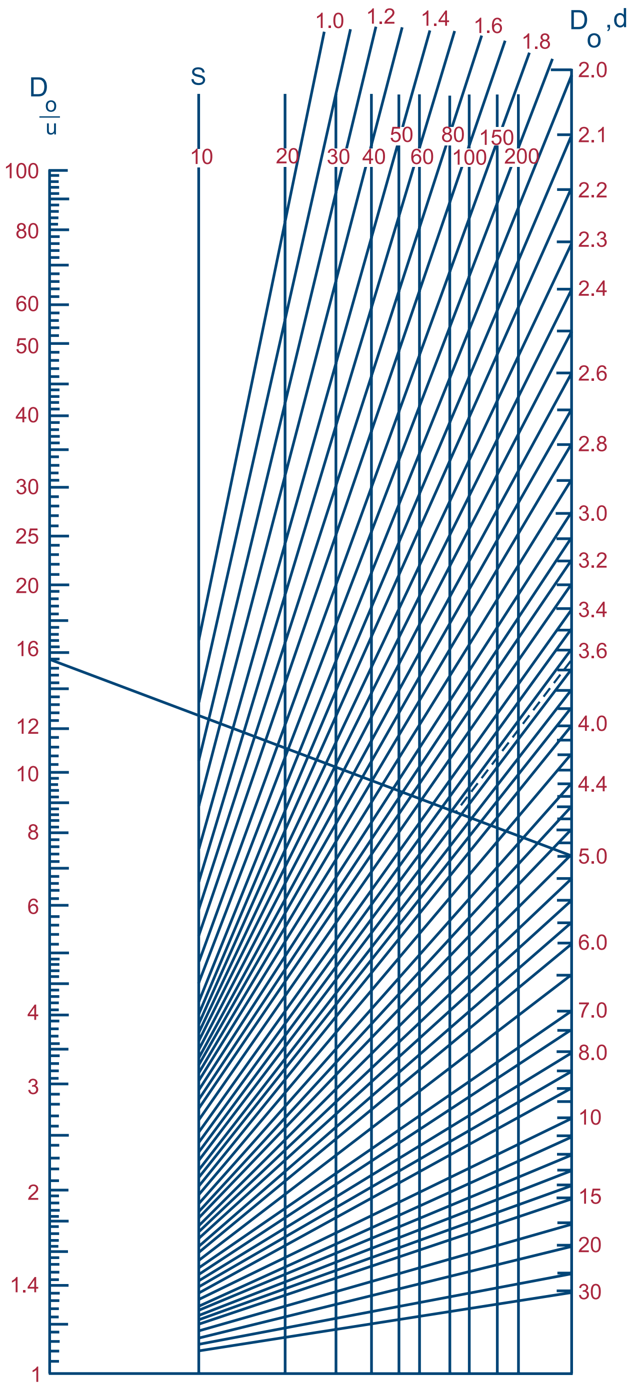
- Draw a line from D = 5 on the right y-axis to D/u = 15.6 on the left y-axis
- Determine the intersection point of the above line with the S = 90 line
- Draw a line from this point to the right y-axis, as shown by the dotted line
- The point where it reaches the right y-axis gives the d value

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

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

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



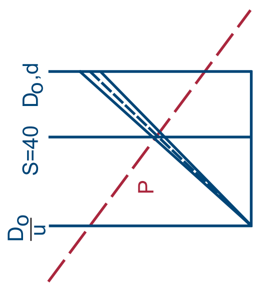
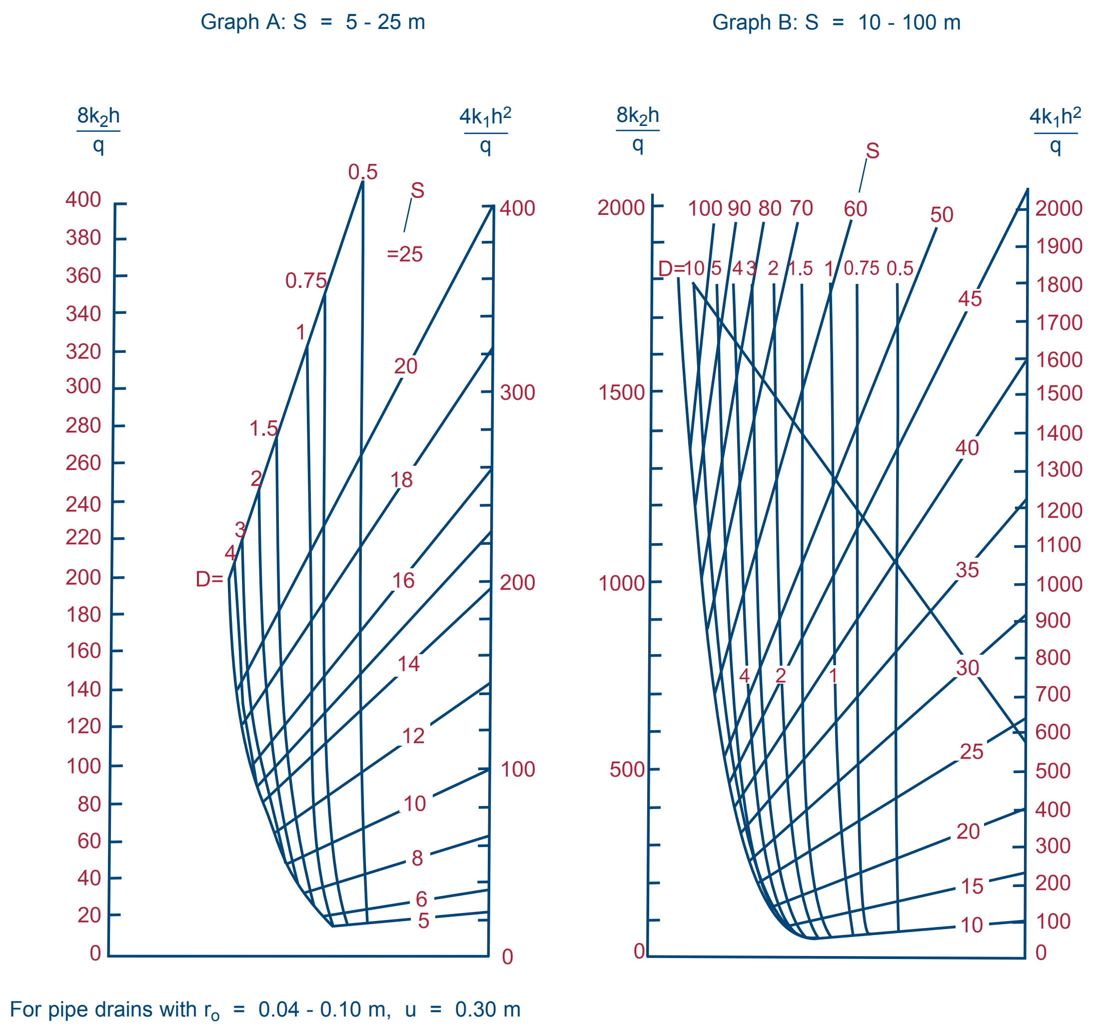


Figure 100

Nomograph for the solution of the Hooghoudt drain spacing formula



8.4.2. Vertical subsurface drainage

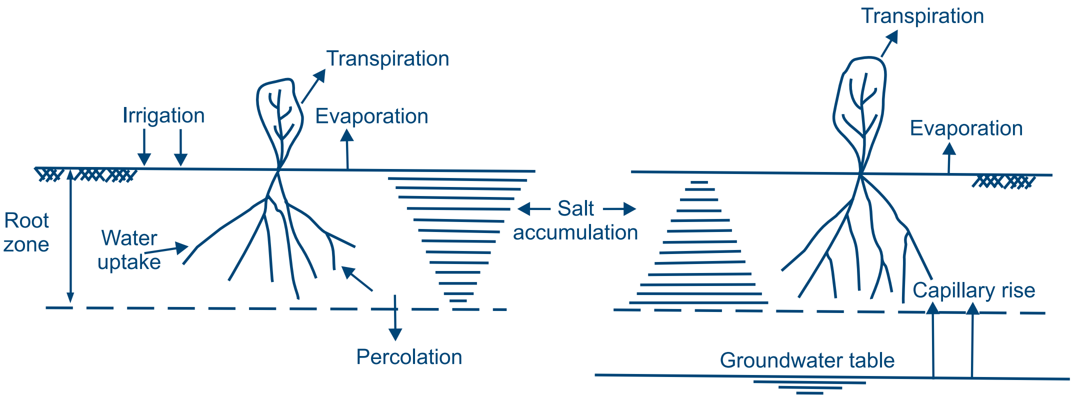
Where soils are of high permeability and are underlain by highly permeable sand and gravel at shallow depth, it may be possible to control the water table by tubewells located in a broad grid, for example at one well for every 2-4 km^2. Tubewells minimize the cost of and disturbance caused by field ditches 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

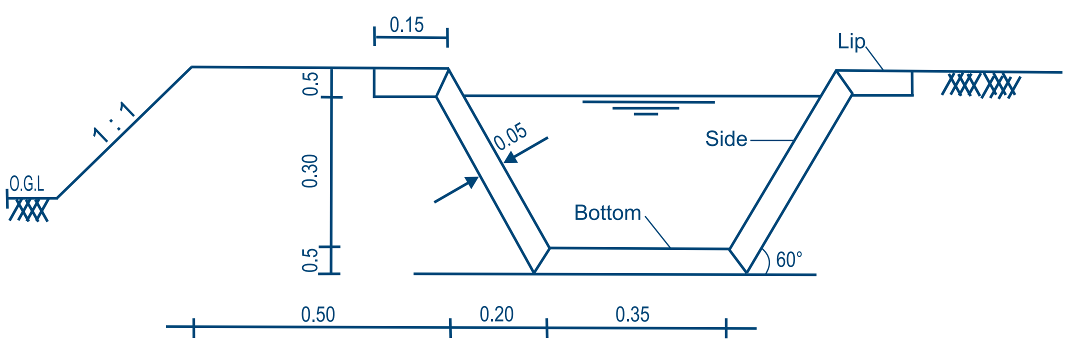
  → for two sides, the area is 2 x 0.023 = 0.046 m^2

- 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.

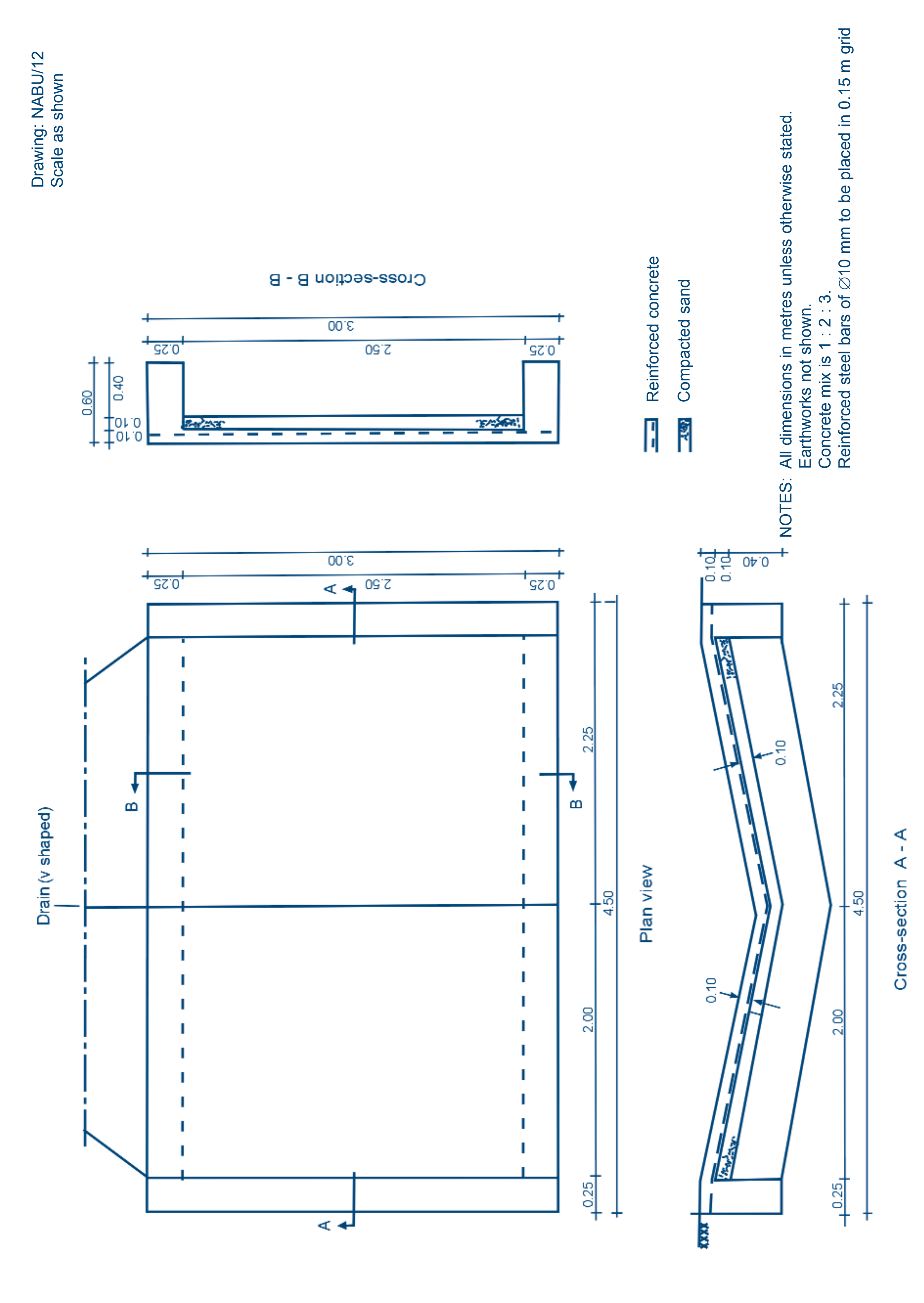
9.1.2. The construction of a saddle bridge

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

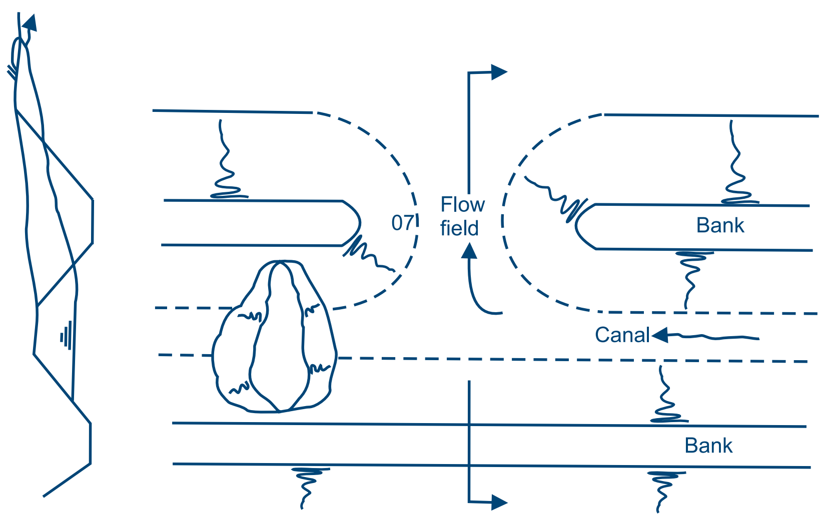


Figure 105 shows a permanent outlet structure.

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

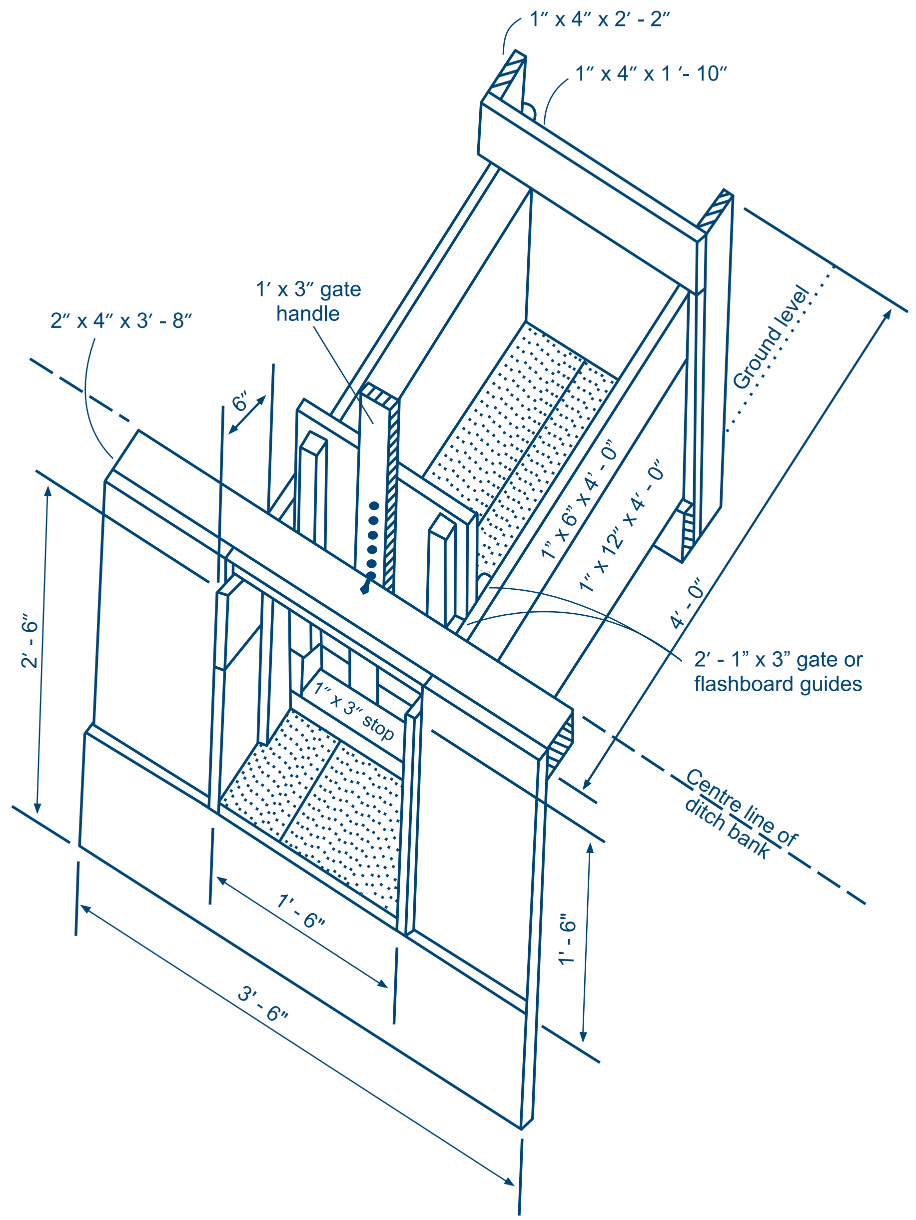
Depth of water over the sill at the intake (cm)	Discharge per 10cm width of the sill (l/sec)
10	6
15	11
20	17
25	22

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)

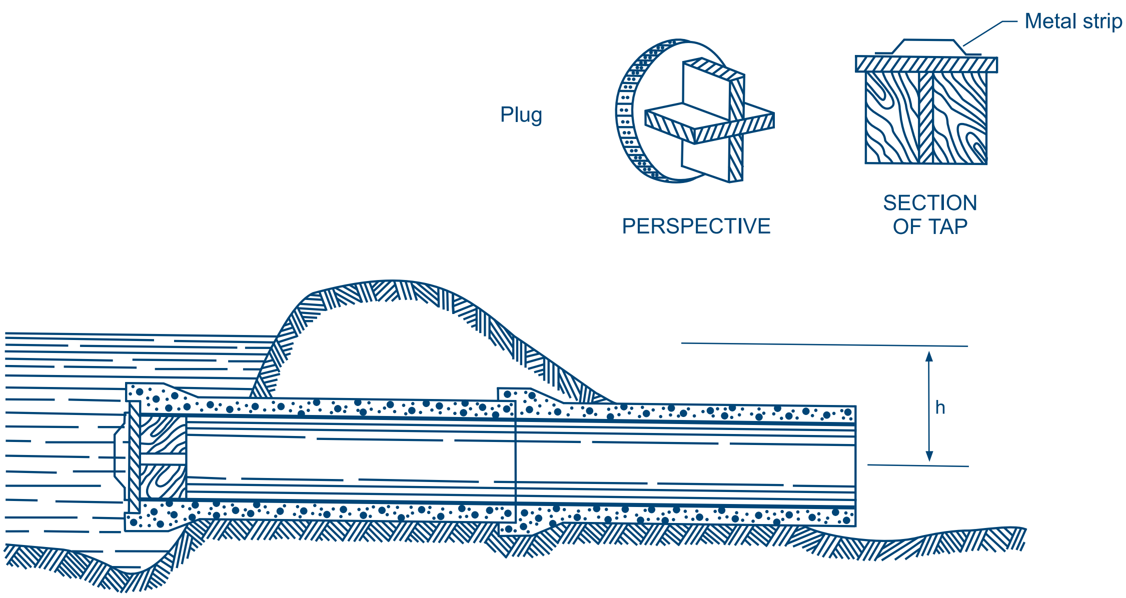
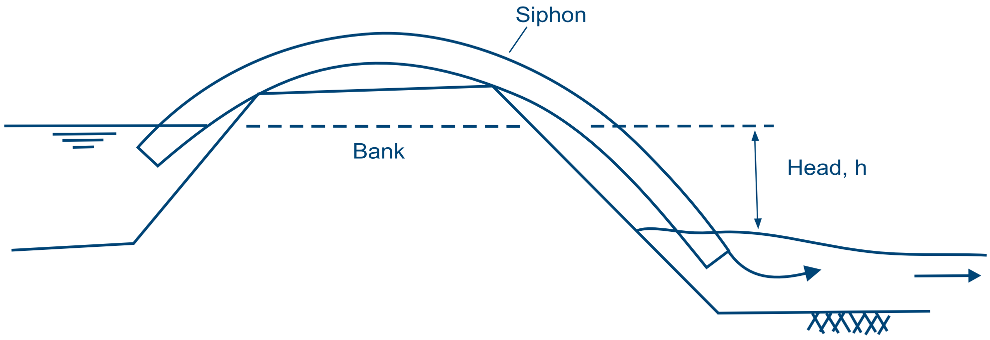


Figure 107

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

References

Addink, *et al*. 1989. *Design and operation of farm irrigation systems*. ASAE.

Agritex. 1990. *Nabusenga irrigation scheme: feasibility and design report.* Unpublished.

Ball, J.S. 1974. *Night storage dams, metric version.* Department of Conservation and Extension, Zimbabwe.

Ball, J.S. 1983. *Design and construction of screeded concrete irrigation furrows*. Agritex, Zimbabwe.

Bassett, D. L. *et al*. 1980. Hydraulics of surface irrigation. In: Jensen, M.E. (ed). *Design and Operation of Farm Irrigation Systems.* ASAE Monograph 3. St Joseph, MI, 447-498 pp.

Benami, A. and Ofen, A. 1984. *Irrigation engineering - Sprinkler, trickle, surface irrigation principles, design and agricultural practices*. Irrigation engineering scientific publications, Israel.

Boumans, J.H. 1963. Een algemeen nomografische oplossing van het stationaire ontwateringsvraagstuk, met toepassingen voor isotroop en anisotroop doorlatede gronden. In: *Polytechnisch Tijdschrift* 14B, 545-557 pp.

Dhawan, S.K. 1978. *Guidelines for design of small irrigation intakes*. Rural Development Department, Kabul, Afghanistan.

Euroconsult. 1989. *Agricultural compendium, for rural development in the tropics and subtropics*. Elsevier, The Netherlands. 740 p.

FAO. 1974. *Surface Irrigation*. By: Booher, L. J. Rome, Italy.

FAO. 1975a. Small hydraulic structures, Volume I. FAO *Irrigation and Drainage Paper 26/1.* By: Kraatz, D.B. and Mahajan, I.K. Rome, Italy.

FAO. 1975b. Small hydraulic structures, Volume II. FAO *Irrigation and Drainage Paper 26/2.* By: Kraatz, D.B. and Mahajan, I.K. Rome, Italy.

FAO. 1988. Irrigation methods. *Irrigation water management training manual No. 5*. By Brouwer, C., Prins, K., Kay, M. and Heibloem, M. Rome, Italy.

FAO. 1989. Guidelines for designing and evaluating surface irrigation systems. FAO *Irrigation and Drainage Paper 45*. By: Walker, W.R. Rome, Italy.

FAO. 1992. Scheme irrigation water needs and supply. *Irrigation water management training manual No 6*. By: Brouwer, C., Hoevenaars, J.P.M., van Boasch, B.E., Hatcho, N. and Heibloem. M. Rome, Italy.

Goldsmith, J. and Mathews. *Construction site management notes*. The Zimbabwe Institution of Engineers in association with the Construction Federation of Zimbabwe.

ILRI. 1978. *Discharge measurement structures*. ILRI Publication No. 20. Wageningen, The Netherlands.

James, L.G. 1988. *Principles of farm irrigation system design.* John Wiley & Sons.

Jensen, M.E. 1983. *Design and operation of farm irrigation systems*. American Society of Agricultural Engineers, U.S.A.

Kay, M. 1986. *Surface irrigation - systems and practice*. Cranfield Press, Bedford, U.K.

Keller, J. and Bliesner, R.D. 1990. *Sprinkler and trickle irrigation*. Chapman and Hall, New York.

Kraatz, D.B. and Stoutjesdijk, J.A. 1984. *Improved headworks for reduced sediment intake.* Proceedings African Regional Symposium on Small Holder Irrigation, Harare.

Larry, J. 1988. *Principles of farm irrigation system design*. John Wiley and Sons.

Michael, A.M. 1994 (reprint from 1978). *Irrigation theory and practice*. Vikas Publishing House Pvt Ltd.

Portland Cement. 1963. *Concrete structures for farm water supplies.*

Rycroft, D.W. and Smedema, L.K. 1983. *Land drainage – planning and design of agricultural drainage systems*. Batsford, London.

Sir William Halcrow and Partners. 1986. *Fuve Panganai Irrigation Scheme, Engineering Report.* Agritex, Zimbabwe.

South African Bureau of Standards. 1976. *Standard specifications for components of unplasticized Polyvinyl Chloride (uPVC) pressure for potable water*. Revision 1. Pretoria.

Stoutjesdijk, J.A. 1993. LONSEC - *Calculation of longitudinal sections of standard Agritex field canals*. Users Manual Version 1.1: Volume 1. Harare. 508 p.

U.S. Department of Interior. 1975. *Water Measurement Manual.* U.S. Bureau of Reclamation, U.S. Government Printing Office, Washington, D.C.

Walker, W.R. and Skogerboe, G.V. 1987. *Surface irrigation - theory and practice.* Practice-Hall, Inc., New Jersey, USA.

Withers, B. and Vipond, S. 1974. *Irrigation - design and practice.* Batsford, London.