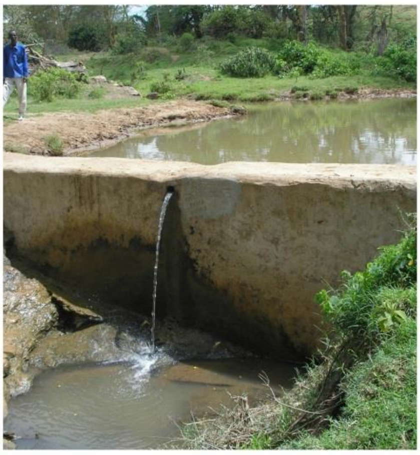
PRACTICE MANUAL FOR SMALL DAMS, PANS AND OTHER WATER CONSERVATION STRUCTURES IN KENYA

2nd Edition

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Disclaimer	The material contained in this product is intended to provide general guidelines for the design and development of small dams, pans and other water conservation structures in Kenya. Any person using the material in this product is expected to exercise due diligence and apply professional judgement on its applicability and appropriateness in the particular circumstances. Use of the material in this product does not make the authors or publishers of this product liable for the design or performance of any structure developed with the use of this material.

PRACTICE MANUAL FOR SMALL DAMS, PANS AND OTHER WATER CONSERVATION STRUCTURES IN KENYA

FOREWORD BY PRINCIPLE SECTRETARY, STATE DEPARTMENT FOR WATER

DRAFT

The State Department for Water in the Ministry of Water and Irrigation Services has its fundamental goal and purpose as conserving, managing and protecting water resources for socio-economic development. Its aim is to improve the living standards of people by ensuring proper access to available water resources and water services. The ministry was created in 2003 following a separation from the Ministry of Environment and Natural Resources.

- **Our Vision:** To be a regional leader in the sustainable management and development of water resource
- **Our Mission:** To facilitate sustainable management and development of water resources for national development.
- **Our Mandate:** Our Mandate is formulation, review and implementation of policy on the water sector and reclamation of degraded lands for sustainable development of our Nation.

The GOK published the Kenya Vision 2030 in 2007, which is the country's new development blueprint covering the period from 2008 to 2030. The Vision 2030 aims to transform Kenya into a newly industrialised, "middle-income country providing a high quality of life to all its citizens by the year 2030". The Vision 2030 is based on three pillars of development namely, economic, the social and the political. The economic pillar aims to achieve an average GDP growth rate of 10% per annum beginning in 2012. The social pillar seeks to build a just and cohesive society with social equity in a clean and secure environment. The political pillar aims to realise a democratic political system, and protects the rights and freedoms of every individual in Kenyan society.

The national development targets on the water sector in the Vision 2030 are as follows:

a) Water and sanitation - to ensure that improved water and sanitation are available and accessible to all by 2030,
b) Agriculture - to increase the area under irrigation to 1.2 million ha by 2030 for increase of agricultural production,
c) Environment - to be a nation that has a clean, secure and sustainable environment by 2030, and
d) Energy - to generate more energy and increase efficiency in energy sector.

The National Water Master Plan 2013 addresses the water resource management challenges in Kenya and sets out plans to support the realisation of Vision 2030. The NWMP anticipates the development of a total of 17,860 small dams and water pans adding an additional 893 MCM. This represents a significant investment in water storage which will be made by national and county government departments and agencies, non-governmental organisations, communities, private sector and with support from development partners. In order to realise the benefits of this investment the structures must be designed and constructed professionally and be properly maintained.

The *Practice Manual for Small Dams, Pans and Other Water Conservation Structures in Kenya* provides the required information to support the proper planning, design and construction of these structures. Adherence to the material in the Manual will ensure that structures provide the intended benefits for the expected lifespan of the structures.

The new Kenyan Constitution devolves the mandate for water services to the county governments and places emphasises on the need for thorough stakeholder engagement by development agents whether from private, public or civil society, to consult with stakeholders on project development. The *Practice Manual for Small Dams, Pans and Other Water Conservation Structures in Kenya* is a document to be used by all those planning and developing small dams and pans.

The Ministry of Water and Irrigation Services wishes to convey its gratitude to all who have been involved in the revision, updating and publication of the Manual and in particular to the Swiss Agency for Development and Cooperation (SDC) for their financial and technical assistance.

James Teko Lopoyetum, HSC

Principle Secretary
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Nairobi
May 2015

FOREWORD BY DIRECTOR, WATER STORAGE AND LAND RECLAMATION

DRAFT

Small dams, pans and other water storage structures provide a critical source of water for many Kenyans for both domestic and commercial purposes particularly for livestock and irrigation. Kenya is now facing a future where water security is a major challenge. The increasing population places a higher demand for water services and climate induced hydrological extremes makes the availability of the resource more uncertain. These two factors mean that improved water storage has a fundamental role to play in building a more water secure future for Kenya.

The First Edition of the *Guidelines for the Design, Construction and Rehabilitation of Small Dams and Pans in Kenya* was published through the Kenyan Ministry of Water Development in 1992 with assistance from the Kenya-Belgium Water Development Programme. The Guidelines have been widely used by development agents, engineers, technicians and contractors from both the public and private sectors. However it has now become evident that the Guidelines need to be updated to take on board experience over the last 25 years, current best practice, new technologies, and new legislation.

The Second Edition, titled *Practice Manual for Small Dams, Pans and Other Water Conservation Structures in Kenya* is an important reference document for those involved in the development of water storage structures. The document covers aspects of project planning, stakeholder engagement, legal and environmental compliance, and catchment conservation in addition to the technical material on design and construction of the water conservation structures.

The preparation of the 2nd Edition has involved participation from the Ministry of Water and Irrigation Services, other Ministries and Government Institutions, Non-Governmental Organizations, the Institution of Engineers of Kenya, the National Construction Authority, contractors, professional engineers, hydrologists and geologists handling water conservation activities. It therefore presents the collective body of experience and knowledge from diverse stakeholders.

The role of the Ministry of Water and Irrigation Services is to facilitate the development of water conservation structures that are safe, robust, economic, environmentally and legally compliant and which support the goals of sustainable economic and social development as envisaged in the Vision 2030 plan to transform Kenya into a middle income economy. The *Practice Manual for Small Dams, Pans and Other Water Conservation Structures in Kenya* is an important contribution towards Vision 2030 as it provides information needed by engineers, technicians, investors, communities, and government agents to ensure that water conservation structures are designed and built to acceptable standards.

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Nairobi
May 2015

PREFACE

Since their first publication in 1992 by the Ministry of Water Development, the "Guidelines for the Design, Construction and Rehabilitation of Small Dams and Pans in Kenya" has been used by engineers, technicians and others for the design and development of many water storage structures. This experience by practionners and various advances in technology and methods for the design and development of small water structures has motivated the need to revise and update the original manual to ensure that the Guidelines remain useful for the future.

This document (Practice Manual for Small Dams, Pans and Other Water Conservation Structures in Kenya) is essentially the 2nd Edition of the "Guidelines for the Design, Construction and Rehabilitation o Small Dams and Pans in Kenya". However the title of the document has been changed to align with the "Practice Manual for Water Supply Services" published in 2005 by the Ministry of Water and Irrigation. In addition, the title reflects the fact that water conservation structures other than small dams and pans have been included.

The arrangement of chapters and their content has been modified from the 1st Edition. As new water conservations structures have been introduced, it was considered useful to distinguish generic aspects of project development that would apply to any type of structure, from the design and construction aspects that are structure specific. Consequently material related to the hydrology, site and material investigations, and reconnaissance and feasibility surveys have been developed into their own chapters.

Chapter 1 – Introduction – is similar to the 1st Edition in that it covers the purpose of the document and a description of the intended users. The 2nd Edition includes a description of the principle reasons for the revision to the 1st Edition. It also introduces the readers to the complementary web-site. The section in the 1st Edition on common problems in construction and rehabilitation of dams has been incorporated in Chapter 3 of the 2nd Edition which addresses project planning and management.

Chapter 2 – Definitions and Classifications – draws on material from the 1st Edition on restrictions and definitions but accommodates a broader discussion on the classification of the structures covered in the 2nd Edition. The components in the 1st Edition on water requirements have been transferred to Chapter 3 in the 2nd Edition which covers project planning and management and the sections on storage requirements have been transferred to Chapter 7 of the 2nd Edition that deals with hydrology and sediment analysis.

Chapter 3 – Project Planning and Management – presents revised material from the 1st Edition on general issues related to the planning and management of any type of storage project. The chapter sets out the key elements of the project cycle approach which include defining project objectives, establishing water demands, and establishing a project team with clearly identified roles and responsibilities.

Chapter 4 – Policy and Legal Compliance – provides new material on the policy and legal framework for storage development to reflect the substantial changes in both policy and legislation since 1992 which have been part of the water sector reform process as captured in the Water Act (2002) and subsequent subsidiary legislation. This chapter also covers the requirements of Environmental Impact Assessments as required under the Environment Management Coordination Act (1998).

Chapter 5 – Stakeholder Engagement – draws on material previously covered in Chapter 4 of the 1st Edition but expands the material to include a discussion on the legislative framework under the Constitution of Kenya (2010) that requires stakeholder consultation as part of the devolution of mandates to the counties and local levels. The chapter is less prescriptive in terms of process but provides more information on techniques for stakeholder analysis.

Chapter 6 – Environmental and Social Impact Assessment – is new material that expands on what was formerly in Chapter 3 of the 1st Edition. The EIA process is an important and mandatory step in most of the projects within the scope of the 2nd Edition and so it was felt that more details would be useful to strengthen this process with respect to water conservation structures.

Chapter 7 – Erosion Control and Catchment Conservation - covers the issues related to reducing siltation of reservoirs through erosion control and catchment conservation measures. It draws heavily on material from Chapter 9 in the 1st Edition.

Chapter 8 – Hydrology and Sediment Analysis – incorporates elements of Chapter 5 in the 1st Edition related to the estimation of flood flows but substantially expands the material to cover data requirements and treatment, rainfall analysis, flood frequency analysis, alternative methods to determine storage requirements and the determination of the minimum environmental releases (Reserve).

Chapter 9 – Site Surveys and Investigations – sets out the approach and requirements for site surveys and material investigations which were formerly covered under geotechnical investigations in Chapter 5 of the 1st Edition. The chapter now covers topographical surveys, geotechnical investigations and material testing.

Chapter 10 – Reconnaissance Survey – presents the factors that should be considered in site selection and to determine site suitability. This material was formerly in Chapter 3 of the 1st Edition. A reconnaissance survey is an essential part of the project development process and many potential problems can be avoided or identified and incorporated into the subsequent project phases.

Chapter 11 – Feasibility Study – outlines the details needed to make a proper assessment of the project feasibility. The emphasis is to make an holistic assessment of the technical, environmental, legal, social and economic feasibility of the proposed project.

Chapter 12 – Earthfill Dams – covers the design, construction, maintenance and rehabilitation of small earth embankment dams. It draws on material in the 1st Edition from Chapter 5 related to the design considerations and Chapter 8 on rehabilitation and maintenance.

Chapter 13 – Mass Gravity Dams – presents new material on the design, construction and maintenance of mass gravity dams.

Chapter 14 – Pans and Lagoons - covers the design, construction, and maintenance of pans and lagoons. It draws on material from Chapter 6 in the 1st Edition but also incorporates new material related to the popular use of synthetic liners for lagoons.

Chapter 15 – Sand Dams - covers the design, construction, and maintenance of sand dams. It draws on material from chapter 6 in the 1st Edition.

Chapter 16 – Subsurface Dams - covers the design, construction, and maintenance of subsurface dams based on material from chapter 6 in the 1st Edition.

Chapter 17 – Rock Catchments - covers the design, construction, and maintenance of rock catchment dams based on material from chapter 6 in the 1st Edition but also incorporates new approaches to rock catchments which favour covered storage.

Chapter 18 – Ancillary Structures - covers the design, construction, and maintenance of ancillary structures that are applicable to any of the storage structures covered under the 2nd Edition. It includes material from Chapter 5 of the 1st Edition.

Chapter 19 – Technical Reports – mimics Chapter 7 in the 1st Edition but presents updated material based on requirements from the Water Act 2002.

Chapter 20 – Bibliography – provides the list of references referred to in earlier chapters and includes useful references for those involved in the development of the type of structures covered in this document.

One of the new developments related to the 2nd Edition is the development of a complementary web-site to enable the public to have access to the document, drawings and related materials in digital format. The web-site also provides an opportunity for updates and other relevant material to be posted for users.

Future revisions to the document are anticipated as part of the process of strengthening the standards for design and development of water conservation structures in Kenya. It is anticipated that these revisions will include the development of specific design guidelines for the different types of storage structures in future.

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ABBREVIATIONS

ALDEV	African Land Development Program
AMC	Antecedent (Soil) Moisture Condition
ARF	Area Reduction Factor
ASAL	Arid and Semi-Arid Lands
B	Billion
B.M	Bench Mark
BoQ	Bill of Quantities
BPT	Break Pressure Tank
BS	British Standard
CAAC	Catchment Area Advisory Committee
CAD	Computer Aided Design
CBO	Community Based Organizations
CFA	Community Forest Association
CFU	Composite Filtration Unit
CIG	Common Interest Groups
CN	Runoff Curve Number
CSR	Corporate Social Responsibility
CW	Crest Width
CWP	Communal Water Point
DO	Dissolved Oxygen
DOS	Determine On Site
EAP	Environmental Action Plan
EDM	Electronic Distance Measurer
EIA	Environmental Impact Assessment
EMCA	Environmental Management and Coordination Act
EMMP	Environmental Management and Monitoring Plan
ENNDA	Ewaso Nyiro North Development Authority
ESIA	Environmental and Social Impact Assessment
ESP	Exchangeable Sodium Percentage
EV	Extreme Value
FAO	Food and Agriculture Organization
FDC	Flow Duration Curve
FWL	Flood Water Level
G.F	Gross Freeboard
GI	Galvanised Iron
GIS	Geographic Information Systems
GoK	Government of Kenya
GDP	Gross Domestic Product
GPS	Global Positioning System
HAR	Hydrological Assessment Report
HDPE	High Density Polyethylene
HVA	Height-Volume-Area Relationship
IC	Individual Connection
ICE	International Council of Engineers
ICOLD	International Committee on Large Dams
IDF	Inflow Design Flood
IWM	Integrated Watershed Management
IWRM	Integrated Water Resources Management
JICA	Japan International Cooperation Agency
KARI	Kenya Agricultural Research Institute
KEBS	Kenya Bureau of Standards
KenGen	Kenya Electricity Generating Company
KEWI	Kenya Water Institute

km^2	Square kilometres
KMD	Kenya Meteorological Department
KMS	Kenya Meteorological Services
KNBS	Kenya National Bureau of Statistics
KS	Kenya Standard
KWS	Kenya Wildlife Service
LDPE	Low Density Polyethylene
LN	Legal Notice
LSU	Livestock Unit
M	Million
masl	meters above sea level
MCA	Member of County Assembly
MCM	Million Cubic Metres
MDD	Maximum Dry Density
MWIS	Ministry of Environment Water and Natural Resources
MoWI	Ministry of Water and Irrigation
N.F	Net Freeboard
NC	Non-individual Connection
NCA	National Construction Authority
NEMA	National Environment Management Authority
NGOs	Non-Governmental Organizations
NIB	National Irrigation Board
NRW	Non Revenue Water
NWCPC	National Water Conservation and Pipeline Corporation
NWL	Normal Water Level
NWMP	National Water Master Plan
O&M	Operation and Maintenance
OMC	Optimum Moisture Content
OSHA	Occupational Safety and Health Administration
PAP	Project Affected Person
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PPE	Personal Protective Equipment
PPPs	Private-Public Partnerships
PVC	Polyvinyl Chloride
RAP	Resettlement Action Plan
RDF	Rainfall Duration Frequency
SCS	Soil conservation Service (US)
SCMP	Sub-catchment Management Plan
SDC	Swiss Agency for Development and Cooperation
SEP	Stakeholder Engagement Plan
SHG	Self Help Groups
SIDA	Swedish International Development Cooperation Agency
SSD	Sub-Surface Dam
SWOT	Strengths, Weaknesses, Opportunities and Threats
T_C	Time of Concentration
TNA	Training Needs Assessment
TRRL	Transport and Road Research Laboratory
UFW	Un-accounted For Water
UNICEF	United Nations Children’s Fund
uPVC	Unplasticized Polyvinyl Chloride
USDA	United States Department of Agriculture
VES	Vertical Electrical Sounding
WAB	Water Appeals Board
WASH	Water, Sanitation and Hygiene

WB	World Bank
WQ	Water Quality
WRM	Water Resource Management
WRMA	Water Resources Management Authority
WRUA	Water Resource User Association
WSB	Water Service Board
WSP	Water Service Providers
WASREB	Water Services Regulatory Board
WSB	Water Supply and Sanitation Services Board
WSP	Water Service Provider
WSS	Water Supply and Sanitation
WSTF	Water Services Trust Fund

UNIT CONVERSION TABLES

Length

	m	ft	in	km	miles
1 metre (m)	1	3.281	39.37	1 x10^{-3}	6.214 x10^{-4}
1 foot (ft)	0.3048	1	12.00	3.048 x10^{-4}	1.894 x10^{-4}
1 inch (in)	0.0254	0.0833	1	2.54 x10^{-5}	1.578 x10^{-5}
1 kilometre (km)	1000	3281	3.937 x10^{4}	1	0.6214
1 mile (mi)	1609	5280	6.336 x10^{4}	1.609	1

1 nautical mile (UK) = 1.853 km　　　1 degree of latitude is about 111 km

Area

	m^2	ft^2	acre	hectare
1 m^2	1	10.76	2.471 x10^{-4}	1.0 x10^{-4}
1 ft^2	9.29 x10^{-2}	1	2.29 x10-5	9.29 x10^{-6}
1 acre	4.047 x10^{3}	4.356 x10^{4}	1	4.047 x10^{-1}
1 hectare (ha)	1.0 x10^{4}	1.076 x10^{5}	2.471	1

Volume

	m^3	l	Imp. gal	US gal	ft^3
1 m^3	1	1.0 x10^{3}	2.200 x10^{2}	2.642 x10^{2}	35.32
1 litre (l)	1.0 x10^{-3}	1	0.220	0.2642	3.532 x10^{-2}
1 Imp. gallon	4.546 x10^{-3}	4.546	1	1.200	0.1605
1 US gallon	3.785 x10^{-3}	3.785	0.8326	1	0.1337
1 ft^3	2.832 x10^{-2}	28.32	6.229	7.480	1

Discharge (Volume rate of flow)

	l/s	m^3/s	m^3/d	UK gal/day	ft^3/s
1 l/s	1	1.0 x10^{3}	86.40	1.901 x10^{4}	3.531 x10^{-2}
1 m^3/s	1.0 x10^{3}	1	8.640 x10^{4}	1.901 x10^{7}	35.313
1 m^3/d	1.157 x10^{-2}	1.157 x10^{-5}	1	2.200 x10^{2}	4.087 x10^{-4}
1 UK gal/day	5.262 x10^{-5}	5.262 x10^{-8}	4.546 x10^{-3}	1	1.858 x10^{-6}
1 ft^3/s	28.32	2.832 x10^{-2}	2.447 x10^{3}	5.382 x10^{5}	1

Note: the cubic metre per second (m^3/s) is also known as the 'cumec'
The cubic foot per second (ft^3/s) is also known as the 'cusec'

Velocity

	m/s	km/h	mi/h	ft/s
1 m/s	1	3.6	2.237	3.281
1 km/h	0.2778	1	0.6214	0.9113
1 mi/h	0.447	1.609	1	1.467
1 ft/s	0.3048	1.097	0.6818	1

Mass

	kg	lb	t	UK ton
1 kilgram (kg)	1	2.205	1 x10^{-3}	9.842 x10^{-4}
1 pound (lb)	0.454	1	4.536 x10^{-4}	4.464 x10^{-4}
1 metric tonne (t)	1000	2205	1	0.984
1 UK ton	1016	2240	1.016	1

1 US ton = 2000 lb

Density

	g/cm^3	kg/m^3	lb/in^3	lb/ft^3
1 g/cm^3	1	1000	0.0361	62.43
1 kg/m^3	1×10^{-3}	1	3.61×10^{-5}	0.0624
1 lb/in^3	27.68	27.68×10^{3}	1	1728
1 lb/ft^3	0.016	16.02	5.787×10^{-4}	1

Pressure

	kgf/cm^2	bar	kN/m^2	lbf/ft^2 (psi)
1 kgf/cm^2	1	0.981	98.1	14.223
1 bar	1.02	1	100	14.504
1 kN/m^2	0.01	0.0098	1	0.145
1 lbf/ft^2 (psi)	0.07	0.0689	6.89	1

1 Pa (pascal) = 1 N/m^2
1 N/mm^2 = 1 MN/m^2 = 1 MPa
101325 Pa = 1 standard atmosphere (atm) = 1.01325 bar
100 kPa = 1 bar
10.33m head of water = 1 atm
2989 Pa = 1 ft head of water = 22.42 mm of mercury (mmHg)
1 mmHg = 0.0394 inch of mercury (inHg)
1 MPa = 145 lbf/in^2 (psi)

Force

	N	kgf	lbf
1 Newton (N)	1	0.1019	0.2248
1 kilogram-force (kgf)	9.8066	1	2.2046
1 pound-force (lbf)	4.4482	0.4536	1

1 N = 1 kg m/s^2

Torque (Moment of force)

	Nm	kgf m	lbf ft	lbf in
1 Newton metre (Nm)	1	0.1020	0.7376	8.8507
1 kilogram-force metre (kgf m)	9.8066	1	7.2330	86.7962
1 pound-force foot (lbf ft)	1.3558	0.1382	1	12.0000
1 pound-force inch (lbf in)	0.1130	0.0115	0.0833	1

Temperature
F°= 9/5 C° + 32°
C°= 5/9 (F° - 32°)

Metric Prefixes

Prefix	Symbol	Factor by which the unit is multiplied	Equivalent term in common usage
giga	G	1 000 000 000	Billion
mega	M	1 000 000	Million
kilo	k	1 000	Thousand
hecto	h	100	Hundred
deca	da	10	Ten
deci	d	0.1	
centi	c	0.01	
milli	m	0.001	
micro	μ	0.000 001	

CHAPTER 1

INTRODUCTION

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1 INTRODUCTION

This manual is the 2nd Edition of “The Guidelines for the Design, Construction and Rehabilitation of Small Dams and Pans in Kenya” (MOWD, 1992).

1.1 Background

The First Edition of the Guidelines for the Design, Construction and Rehabilitation of Small Dams and Pans in Kenya was published through the Kenyan Ministry of Water Development in 1992 with assistance from the Kenya-Belgium Water Development Programme. The Guidelines have been widely used by engineers, technicians and contractors from both the public and private sectors.

In 2014, the Swiss Agency for Development and Cooperation decided to support a process of reviewing and updating the Guidelines to bring them up-to-date with current practice, to capture the experience of the last 25 years, and to provide a useful reference document for the design and development of water conservation structures into the future.

The updates to the first edition concentrate on the following areas:

1. **Changes in Technology:** There have been significant improvements in survey equipment and availability of maps and terrain data since the original manual was published. In addition a much greater variety of construction equipment is now available in Kenya than in 1992.
2. **Wider Scope:** The original manual limited itself to dams of less than 10m height and less than 100,000 cubic meters in storage volume. The revised edition bring this in line with the current Water Act 2002 classification for low and medium risk dams (i.e. less than 15m height, less than 1,000,000 cubic meters of storage and catchment area of less than 1,000 square kilometres).
3. **Different Types of Structures:** The original manual concentrated mainly on earth embankments with a few brief sub-sections on other structures. The revised manual looks at alternative structures in more detail.
4. **New Legislation:** The revised manual brings the original manual up to date with regards to current NEMA Regulations, the Water Act 2002 regulations and other relevant legislation.
5. **New Issues (e.g. Climate Change):** The revised manual cover issues that have emerged since the original manual was published in 1992. These include climate change and web oriented design tools.
6. **Incorporate Lessons Learned from use of the 1st Edition:** The revised manual makes use of the experience gained from using the 1st Edition.
7. **Standardization:** Generic drawings, Bills of Quantities (BOQs) and calculation worksheets related to this Practice Manual have been made available through the complementary website.

The Practice Manual has been developed for general application on sites where the structures are within the boundaries of the limitations and restrictions described below. However, for larger and more complicated structures, especially those creating a significant hazard, or where there is doubt, reference should be made to other internationally recognised design handbooks, such as "Design of Small Dams" (United States Department of the Interior - Bureau of Reclamation, 1987), and use made of experienced professional dam engineers.

References to design manuals and internationally recognized textbooks providing in-depth coverage of various scientific and technical disciplines related to design and construction of dams form part of the bibliography attached to this publication.

1.2 Objective/Purpose

The Practice Manual is intended to provide a general reference for the design, construction and rehabilitation of environmentally appropriate small dams, pans and other water conservations structures in Kenya, with special emphasis on the specific problems encountered in relation to the establishment of small water conservation structures in the rural and ASAL (Arid and Semi-Arid Lands) areas of the country.

1.3 Layout of Manual

The manual has been laid out into 20 Chapters which address a full range of water storage projects. Chapters 1 to 11 deal with issues that are common to all storage projects. Chapters 12 to 18 look at specific types of storage structures and storage options. Chapter 19 provides the outline for various technical reports associated with water storage structures. Chapter 20 presents a detailed bibliography.

The manual does not contain any photographs. Photographs are available through the online version of the manual.

The manual contains a limited number of drawings. More detailed drawings are available through the online version of the manual.

1.4 How to Use Manual

The manual should be used as a general reference for anyone considering low and medium risk water storage projects. It is not necessary to use the entire manual, although the first 11 chapters should be of interest to all water storage projects. Individual chapters can be used as stand alone guides for a variety of water storage structures.

While the hard copy version provides details and formulas for calculations and design, the online version provides a selection of spreadsheets that allow fast, accurate and standardised design calculations and reporting.

1.5 Target Readers

The Practice Manual is intended for use by engineers, artisans, surveyors, developers, owners, and other practitioners involved in the development of safe, economic and environmentally appropriate small dams, pans and other water conservation structures.

1.6 Complementary Website

A complementary web-site is accessible through a portal on the ministry website (www.water.go.ke). The website contains additional materials, references and worksheets related to the content of the Guidelines. A soft copy of the manual and supporting material is also availed in a CD enclosed at the back of this document.

CHAPTER 2

DEFINITION, CLASSIFICATION AND LIMITATIONS

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2 DEFINITION, CLASSIFICATION AND LIMITATIONS

This chapter clarifies the scope of the manual and elaborates on the specific size storage projects for which the manual is relevant.

2.1 Definitions

There is some confusion with regard to many terms used to describe dams, reservoirs, lagoons, pans, etc. For the sake of this manual, the following definitions have been adopted.

A **dam** is a barrier or wall designed and developed on a water course to confine and then control the flow of water. It will retain water upstream of the structure.

A **pan** is a structure developed through excavation or a natural depression to retain water. Water is retained below natural ground level.

A **lagoon** is a structure developed both through excavations below ground level and through construction of a retaining wall above ground level in order to retain water. They are typically lined with an HDPE or LDPE lining. Water is retained both above and below natural ground level.

A **reservoir** is the water retained by a structure.

As noted above, the terms are frequently used without precise application of the definitions. The critical aspect from an engineering and risk perspective is whether the structure is designed to retain water above natural ground level as this requires the application of engineering design to withstand the hydraulic pressures. In such a case, the retaining part of the structure should be treated as a dam.

Dams are frequently described by the purpose(s) for which the dam is built e.g. fish dam or multipurpose dam. The size and structure type (earth fill, rock fill, concrete arch...) can also be used to describe a dam.

Pans are also frequently described by the shape of the structure e.g. hafir or turkey nest dam.

Other terms that are increasingly coming into use with regards to water storage in Kenya are:

> **Sand Dams:** Stone masonry or concrete walls designed to store water by retaining sand on the upstream side of the wall. The water is stored in the voids in the sand. Technically they are dams with mass gravity walls.
> **Sub-surface Dams:** These dams, variously referred to as groundwater dams, make use of a stone masonry, concrete or compacted earth wall which is constructed across a sandy water course, to artificially raise the water level within the sandy medium on the upstream side of the wall. The wall acts as a retaining wall.

2.2 Classification of Dams

Classification of dams by a single physical characteristic (embankment height, storage volume….) is straightforward to do but does not necessarily capture all the areas of concern with respect to the dam. Instead, a hazard based classification is generally used. A dam can be considered a hazard as it may cause inundation, physical and environmental damage and loss of life. As such classifying dams into different hazard classes helps to define the design specifications and acceptable risk associated with the different scale of hazard.

The ICOLD Bulletin 157 of 2011 "SMALL DAMS: Design, Surveillance and Rehabilitation" (ICOLD, 2011) develops a PHC (Potential Hazard Classification) for small dams that looks at physical characteristics, life safety risk, economic risk, environmental risk and social disruption that could occur in the event of a catastrophic failure. The system provides for three PHC classes, namely (i) low risk, (ii) medium risk and (iii) high risk.

The Water Resource Management Rules (2007) use a hazard classification system based on three factors, namely depth, volume and catchment area that result in three classes of dams as shown in Table 2-1. The factor that places the dam in the highest hazard class prevails. This system, which is roughly consistent with the ICOLD classification system described above has been adopted for this Practice Manual as its application is straightforward and not based on any subjective risk analysis.

Table 2-1: Classification of Dams

Class of Dam	**Maximum Depth of Water at NWL (m)**	**Impoundment and NWL (m^3)**	**Catchment Area (km^2)**
A (Low Hazard)	0-4.99	<100,000	<100
B (Medium Hazard)	5.00-14.99	100,000 to 1,000,000	100 to 1,000
C (high Hazard)	>15.00	>1,000,000	>1,000

NWL = Normal Water Level

When using the table above, it is important to note that only one factor is necessary to place a dam into a higher hazard class. For example, a 3m deep reservoir with 20,000 cubic meters of storage and a 1,500 square kilometre catchment area would classify as a high hazard dam due to its large catchment area. Similarly, a 15.5m tall mass gravity wall with 25,000 cubic meters of storage and a 3 square kilometre catchment area would be a high hazard dam due to the maximum water depth.

2.3 Scope of Manual

This manual is intended for use with medium and low risk dams. As such it should only be used for Class A and B dams. The manual describes design procedures and provides minimum requirements for planning, design and construction of small dams, pans and other water conservation structures. The guidelines were developed to provide uniform criteria in order to ensure that these structures can be designed, constructed and operated in a standardized way in order to ensure consistent performance.

As new experience, materials, and knowledge become available, this document will need to be revised.

As with all manuals, a degree of judgement is necessary when applying the procedures, guidelines and requirements that are presented.

It is strongly recommended that for all structures with more than 10m of water depth that detailed soil and geotechnical studies are carried out.

Site specific factors should also be considered when using this manual. For example, a Class A dam located immediately above a populated area might be treated as a Medium Hazard or even as a High Hazard structure simply because of the risk to life in the event of a failure.

2.4 Limitations and Restrictions

This manual applies to all low and medium hazard structures. Requirements stated are minimum limits and more conservative requirements may be more appropriate in some situations. In some cases, problems may arise where proven solutions are not available or alternate procedures may need to be evaluated before the best solutions can be developed and selected. Experience, laws and regulations, investigations, analysis, expected maintenance, environmental considerations, and/or safety laws may dictate more conservative criteria to ensure satisfactory performance.

As with all engineering works, final responsibility for design and construction supervision rests with the engineers and organizations responsible for each specific project.

CHAPTER 3

PROJECT PLANNING AND MANAGEMENT

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3 PROJECT PLANNING AND MANAGEMENT

The degree of project planning required for a water storage project can vary widely depending on if the project is a Government project, an NGO project, a community sponsored project or a private project. There are, however many common items that should be considered.

3.1 Project Cycle Approach

A project cycle approach that encompasses the following should be adopted.

1. Needs assessment;
2. Specific planning and design;
3. Implementation;
4. Evaluation and monitoring.

Stakeholder engagement and needs assessment are discussed in Chapter 5. Other general planning and implementation related topics are discussed below.

3.2 Definition of Objectives

The development of small dams, pans and other water conservation structures is an investment towards improving water access and security for one or multiple types of users e.g. domestic, livestock, agriculture, industry, recreation, tourism, flood control, hydro-power and environment. The different types of users may have different, and possibly competitive, interests in the water conservation structure. Defining the objectives is important for the design process as it will guide aspects of reliability, water quality, drawoff works, and the need for ancillary structures.

When planning the construction and/or rehabilitation of small water conservation structures, the following considerations regarding the various possible uses of the stored water should be taken into account:

3.2.1 Domestic Water Supply

Reservoir water, being an open water source is generally of poor quality with respect to drinking water standards and should not be used for domestic purposes without treatment. If the purpose of the structure is domestic water supply, then adequate attention should be given to:

- Minimising the likelihood of contamination from industrial, agricultural, livestock or human pollution (see Chapter 7 on Catchment Conservation);
- Minimising direct livestock, wildlife and human access to the reservoir;
- Providing water drawing facilities (e.g. water kiosk) so that domestic users can obtain the water without having to enter the reservoir area;
- Providing water treatment facilities. Where the capital and operation and maintenance requirements rule out the inclusion of a full water treatment facility measures to improve water quality as much as possible should be considered. In addition, domestic users should be guided on how to acquire and use household water treatment options (UNICEF, FAO and Oxfam GB, 2012). Simple methods for improving the quality of water from small dams and pans will be limited to reducing the turbidity of the water by use of horizontal gravel/sand filters (dams) or dug wells (pans). Prolonged light free storage may also be beneficial (Twort, Ratnayaka, & Brandt, 2000): it reduces turbidity by sedimentation and it reduces pathogenic bacteria as well. However, prolonged storage will usually lower the oxygen content of the water. For killing the schistosome larvae (bilharzia) a storage period of 48 hours is adequate (Cairncross & Fleachem, 1993).

By virtue of being an open water source, the provision of water from a dam or pan without treatment to meet domestic water requirements does not meet the government criteria of an improved supply and therefore does not improve access to water for that population. Consequently adequate consideration of alternative options to meet domestic water demand should be given which may provide an adequate quantity and quality of water for domestic purposes.

3.2.2 Livestock Supply

The use of water of lower quality than required for domestic water supply is permitted. In general, livestock water supply is the most common purpose of the construction and rehabilitation of small dams and pans in the ASALs. It should however be noted, that in order to avoid severe pollution of the reservoir water, straight watering of livestock from the reservoir should be discouraged. (This is in particular the case if the water from the reservoir is also to be used for domestic supply!) Therefore, draw-off facilities (e.g. cattle troughs) should, where possible, be provided.

The intimate relationship between water supply and rangeland for livestock implies that other issues other than water quality are of paramount importance for livestock supply:

- Location of the site and the impact of additional livestock watering on the rangeland;
- The reliability of the water supply. One way to limit the risk of overgrazing in the vicinity of the dam/pan is to restrict the reliability of the water supply so that the structure is expected to dry out two to four months after the end of the rains.

3.2.3 Irrigation

The use of a dam/pan for irrigation requires careful consideration of the following points:

- Irrigation requires more water than domestic or livestock uses and so careful analysis of the water storage capacity and reliability is required to see what scale of irrigation can be sustained. Small dams and pans have been used successfully to support micro-irrigation activities for smallholder irrigation of high value crops, tree nurseries, establishing early planting material and “kitchen gardens” which provide significant livelihood, nutritional and food security benefits.
- Detailed analysis of the topography to determine whether the reservoir water can be used by gravity supply or whether a pumped supply system is required.
- Irrigation efficiency; in general, more water efficient irrigation techniques should be employed to maximize the productive use of the stored water.
- Type of soils; soils must be suitable for long term irrigated farming.
- Water quality is an issue if drip irrigation systems are to be employed.

3.2.4 Wildlife Supply

The same remarks as for livestock supply apply to wildlife. Prevention of wildlife from entering a dam or pan is very difficult and any prevention system should be species specific. In general, wildlife that is likely to damage the structure, pose a risk to other users, or materially affect the water quality, should be denied access. Provision of wildlife water away from the structure, possibly through a gravity draw off pipe, is one way to alleviate pressure on the structure itself but can only be used in combination with fencing the site. Both the fence and distribution works should be wildlife-proof.

The most notorious wildlife vandals for water structures are elephants. Stone wall, bees and electric fences are some of the options that have been tried and can be considered.

Wildlife supply is usually difficult to combine with domestic water supply. The nature and extent of possible wildlife interference should be investigated during the planning and design stages of the dam/pan and consultation with Kenya Wildlife Services or wildlife experts is advisable.

3.2.5 Fish Breeding

Fish breeding can be combined with most other purposes on condition that the water level and water quality throughout the year are sufficient to sustain the aquatic life.

3.2.6 Water Conservation

While there may seem to be an inherent value in conserving runoff and flood waters, the expense and environmental risk does not justify a project in which the purpose is so ambiguous. Careful review of the purpose of the project should be undertaken.

3.2.7 Flood Control and Stream Flow Regulation

Flood control and stream flow regulation implies that the intended structure has sufficient storage capacity to attenuate peak flood flows and sufficient capacity to enable the controlled release of the flood water during low flow periods. For a dam to provide this function, the water level is likely to fluctuate rapidly over the year with releases structured to result in low water levels at the start of the rainy season.

In general dams established for this purpose are large and require specialised investigations and analysis and are beyond the scope of this manual.

3.2.8 Hydropower

The development of a dam for hydro-power purposes is a specialised topic which is beyond the scope of this manual. In general, the size and scope of the dams described in this manual are not large enough for hydro-power development.

3.3 Water Demand Analysis

3.3.1 Initial Estimate

The RELMA Manual "*Water from ponds, dams and pans*" (Lindqvist A.K, 2005) recommends the following initial calculations for small scale reservoirs.

Table 3-1: RELMA Recommended Calculations for Small Scale Water Demand

Item	Population	Consumption Rate (litres/day)	Total (litres/day)
People		x 20	
Camels		x 15	
Cattle		x 15	
Sheep/Goats		x 3.5	
Donkeys		x 15	
Irrigation		x 20 litre buckets/day	
Other		+10% (seepage and evaporation losses)	
Total (Litres/day)			
Total (cubic meters/day) Divide total litre by 1,000)			

For a more detailed estimation of the water demand, reference should be made to the Practice Manual for Water Supply Services in Kenya – Part A (Ministry of Water and Irrigation, 2005). A brief summary of the key points from this manual is presented in the following section.

3.3.2 Design Period

The water demand should be estimated for the initial, future and ultimate periods to provide some anticipation of future demands. The initial period covers the current demand, the future period covers the demand after 10 years and the ultimate period covers the demand after 20 years. Storage structures should be designed to meet the ultimate demand where possible.

3.3.3 Supply Area

In order to establish the demand, the supply area must be defined. In an ASAL area where there are no other water sources, the following guidelines can be used:

a) 5 km radius for domestic users;
b) 5 km radius for sheep and goats;
c) 10 km radius for cattle and wildlife;
d) 15 km radius for camels.

Where other water sources exist, the demand can be attributed to the different sources based on proximity, water quality preferences, water quantity or other relevant local conditions.

3.3.4 Domestic Water Demand

1) Population Projections

Population estimates should make use of the most current census data provided by the KNBS. The smallest unit for which data is provided is the sub-location. This information can be cross checked with current information from the local administration.

The population to be served is based on the sub-location population data. The proportion of the supply area falling in each sub-location should therefore be established. Superimposing a map of the supply area boundaries over a map of the administrative boundaries will provide the supply area within each sub-location. This can be done easily using GIS software.

The initial population within each sub-location is established using local key informants (e.g. local administration), field surveys or is based on the supply area within each sub-location using Equation 3-1.

Equation 3-1 $$\boldsymbol{P_i = \frac{P_T}{A_T} \times A_i}$$

Where: P_i = Population of sub-location “i” in the supply area
P_T = Total population in sub-location “i”
A_T = Total area of sub-location “i” [km^2]
A_i = Supply area within sub-location “i” (as established by GIS (e.g. ARCGIS, MAPINFO), manually from a map or through Google Earth in km^2.

Future population estimates can established based on Equation 3-2.

Equation 3-2 $$P_{yn} = P_{y0}(1 + \frac{r}{100})^n$$

Where: n = number of years projecting forward from year 0.
P_{yn} = Population in year n.
P_{y0} = Population in year 0 i.e. year of census or year data collected.
r = projected annual population growth rate [%] as defined by KNBS or other reliable sources. The national population growth rate in 2014 was approximately 2.7%.

It should also be kept in mind that especially in the ASALs rapid population increases can occur, due not only to a high natural population growth rate, but also through migration from densely populated higher potential areas. Population projections should try to take this phenomenon into account. It should be noted that any significant improvement in the water supply in a certain area might actually induce further migration of people and livestock towards that area. Allowing for this is best done by using an adjusted population growth rate that allows for an influx of people and livestock. Determining a reasonable adjustment is a very subjective task that should be clearly identified and described in any preliminary calculations.

2) Service Level

The service level has a direct bearing on the consumption rate as those with individual connections (IC) will generally consume more that those without (NC). In order to estimate water demand, an estimate should be made of the proportion of the population that will be supplied through individual connections. Table 3-2 and Table 3-3 provide classes and values that can be used. However, where no distribution system is designed or anticipated, then the population can be expected to remain with no individual connections. For the purposes of this table, the following descriptions can be used.

Table 3-2: High, Low and Medium Potential/Class Brackets

Category	**Description**
Urban High Class	Low density housing on 0.2 ha or larger plots, houses with internal hot water systems
Urban Medium Class	Low density housing on 0.1 ha or smaller plots. Houses with internal cold water
Urban Low Class	High density housing, houses with internal cold water but many external facilities
Rural High Potential	Areas with rainfall over 1,000mm/year
Rural Medium Potential	Areas with rainfall 500mm to 1,000mm/year
Rural Low Potential	Areas with rainfall less than 500mm/year

Table 3-3: Proportion of Population Service with Different Service Level

	Proportion of Population with Individual Connections (IC) [%]			Proportion of Population without Individual Connections (NC) [%]		
	Initial	**Future**	**Ultimate**	**Initial**	**Future**	**Ultimate**
Urban Areas						
High & medium Class housing	100	100	100	0	0	0
Low class housing	10	30	50	90	70	50
Rural Areas						
High potential	20	40	80	80	60	20
Medium Potential	10	20	40	90	80	60
Low Potential	5	10	20	95	90	80

(Source: MWI Practice Manual for Water Supply Services, 2005)

3.3.5 Livestock Water Demand

1) Livestock Population

The present livestock population should be based on government records which include:

- Livestock census data;
- Water Master Plans (County or National).

Where no government records are available, the livestock population can be estimated based on the annual rainfall as indicated in Table 3-4.

Table 3-4: Livestock Units per Hectare

Annual Rainfall (mm)	Livestock Units per ha
Less than 400	0.4
400 - 600	0.6
600 – 800	0.8
800 - 1000	1.0
1000 - 1200	1.3
1200 - 1700	1.7
Over 1700	2.5

(Source: MWI Practice Manual for Water Supply Services, 2005)

2) Future Livestock Populations

Unless there is reliable information that the livestock data represented a period of unusually high or low livestock numbers, future livestock populations are expected to remain fairly constant and are hard to predict as the numbers may vary with rainfall, disease, security and other external factors.

3) Livestock Units

A convenient unit of measurement for livestock is known as the Livestock Unit (LSU). According to the National Water Master Plan 2030 one LSU consumes 50l/head/day of water. Conversion of stock numbers to livestock units is achieved using Table 3-5.

Table 3-5: Conversion of Stock to Livestock Units

Stock Type	Equivalent LSU
1 Grade Cow	1 LSU
3 Indigenous cows	1 LSU
15 Sheep or goats	1 LSU
5 Donkeys	1 LSU
2 Camels	1 LSU
3 Pigs	1 LSU
50 Rabbits	1 LSU
165 Poultry	1 LSU

The NWMP, 2013 considers livestock water demand for pigs and poultry as negligible and does not include figures for them (JICA; Nippon Koei Co.Ltd., 2013). The figures for pigs, rabbits and poultry above have been estimated based on experience.

3.3.6 Wildlife Water Demand

In general, wildlife water demand can be extremely difficult to estimate due to the movements of wildlife populations. Water storage for wildlife use can use figures based on the following table. The NWMP, 2013 classifies wildlife water consumption rates into two groups, depending on their water requirements as shown below:

Group A: Elephant, zebra, wildebeest, kudu, warthog and buffalo (these species require relatively more water

Group B: Giraffe, gazelle, gerenuk, impala, hartebeest, topi, eland, oryx and ostrich (these species require relatively less water)

The respective unit water consumption rates are given in Table 3-6 below:

Table 3-6: Unit Water Consumption Rates of Wildlife

Group	Unit Water Consumption	Remarks
Group A	5 l/100kg/day	About 50% of standard water consumption of one livestock unit
Group B	2.5 l/100kg/day	About 25% of standard water consumption of one livestock unit

(Source: NWMP 2030.)

Alternatively, Table 3-7 below is based on local experience and gives estimated wildlife water use figures for a variety of wildlife.

Table 3-7: Wildlife Water Use Figures

Species	Body weight, kilogrammes	Demand, litres per day[1] (litres per animal per day)
Wildebeest	200	9
Zebra	400	18
Buffalo	800	36
Elephant	4,000	182
Eland	600	27
Lion& other predators	varies	10 (but varies)
Waterbuck	200	9
Bushbuck	50	2
Reedbuck	80	5
Impala	80	4
Grants Gazelle	60	3
Thomson's Gazelle	20	1
Warthog	80	4
Rhino	1,000	45
Giraffe	750	36
Baboon	15	7
Ostrich	80	4
Kongoni	110	5
Hippopotamus	2,500	114

3.3.7 Institutional Water Demand

a) Schools

Unless specific information is gathered from government records or field surveys, it may be assumed that 30% of the population attend primary and/or secondary school. The County Integrated Development Plan or more localised development plans may have relevant updated details.

b) Health Centres

Unless specific information is gathered from government records or field surveys, it may be assumed that one health centre and two to four dispensaries will serve about 35-40,000 people with one hospital bed per 1250 people.

3.3.8 Water Consumption Rates

Consumption rates are presented in Table 3-8. A provision of 20% allowance for water losses from leakage and wastage should be factored in.

[1]Based on data presented in IRA & NORDECO 1996: NCAA 1996: Loth & Prins 1986: Douglas-Hamilton 1975. The Trust considers 200 lcd per adult elephant to be reasonable.

Table 3-8: Consumption Rates

CONSUMER	UNIT	RURAL AREAS			URBAN AREAS		
		High potential	**Medium potential**	**Low potential**	**High Class Housing**	**Medium Class Housing**	**Low Class Housing**
People with individual connections	1/head/ day	60	50	40	250	150	75
People without connections	1/head/ day	20	15	10	-	-	20
Livestock unit	1/LSU/ day	50					
Boarding schools	1/head/ day	50					
Day schools with WC without WC	1/head/ day	25 5					
Hospitals Regional District other	1/bed/ day	400 + 20 1 per outpatient and day (minimum 5000 1/day) 200 + 20 1 per outpatient and day (minimum 5000 1/day) 100 + 20 1 per outpatient and day (minimum 5000 1/day)					
Dispensary and Health Centre	1/day	5000					
Hotels High Class Medium Class Low Class	1/bed/ day	600 300 50					
Administrative offices	1/head/ day	25					
Bars	1/day	500					
Shops	1/day	100					
Unspecified industry	1/ha/day				20,000		
Coffee pulping factories	1/kg coffee	25 (when re-circulation of water is used).					

(Source: MWI Practice Manual for Water Supply Services, 2005)

3.3.9 Irrigation Water Demand

The reader is referred to the Practice Manual for Water Supply Services in Kenya – Part B (Ministry of Water and Irrigation, 2005) for a detailed methodology to establish the irrigation water demand.

The values presented in Table 3-9 are based on experience and can be used as a rough guide for planning purposes.

Table 3-9: Irrigation Water Use

Type of Irrigation	Irrigation Water Requirement [m^3/ha/day]
Drip	60
Overhead sprinkler	90
Surface	120

These values broadly reflect the peak irrigation supply requirements and include conveyance and field application efficiencies. The values do not consider effective rainfall and so should not be used to establish annual water supply requirements. These values are useful for establishing how many days of irrigation supply can be provided by the reservoir. For example, the 90 day storage requirement (typically required by WRMA to support a water permit for irrigation purposes) responds to the need for an irrigator to be able to meet his/her irrigation demands for the entire duration of the dry season (roughly three months or 90 days).

3.3.10 Evaporation Losses

At the planning stage of the project an estimate is required of the likely loss from evaporation from the water surface. Monthly open water evaporation estimates for average and dry (1 in 5) conditions are provided in Table 3-10 and Table 3-11 based on *Studies of Potential Evaporation in Kenya* (Woodhead, 1968).

Evaporation pan data are a fair estimate of open water evaporation and can be obtained from the government institutions such as KMS, KARI and WRMA.

Maximum daily evaporation loss can be estimated using Equation 3-3.

Equation 3-3 $Evol = A_{max} \times E_o \times 10$

Where: E_{vol} = Maximum evaporative losses [m^3/day]
A_{max} = Maximum reservoir surface area [ha]
E_o = Open water evaporation [mm/day] as defined by the average over the dry season months.

Table 3-10: Average Monthly Open Water Evaporation [mm]

	Station	Altitude	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
		m													
1	Ahero	1200	205	195	212	179	178	169	167	175	194	200	180	182	2236
2	Ainabkoi	2600	165	162	174	126	126	98	110	85	142	150	116	136	1590
3	Archers Post	865	210	210	230	208	215	210	215	230	240	230	182	185	2565
4	Bachuma	400	200	185	198	182	160	158	156	162	177	190	182	189	2139
5	Baricho	70	195	195	215	185	165	165	165	180	185	195	190	195	2230
6	Busia	1180	182	170	184	170	170	158	152	164	183	186	164	173	2056
7	Chebloch	1200	185	176	191	169	171	156	151	164	174	179	164	170	2050
8	Eldoret	2100	182	177	195	160	148	126	118	123	148	170	168	168	1883
9	Equator	2760	179	177	192	151	140	117	104	111	139	161	153	164	1788
10	Garissa	130	201	191	216	203	207	183	188	199	206	219	182	179	2374
11	Gede	30	189	165	191	178	155	137	148	155	176	192	181	185	2052
12	Habaswein	200	246	257	277	248	275	273	272	282	291	286	205	208	3120
13	Hola	90	198	202	221	191	191	168	169	182	191	198	190	192	2293
14	Isiolo	1100	209	208	230	206	216	209	215	231	241	228	181	187	2561
15	Kabondori	1140	180	165	164	146	125	98	120	119	163	157	129	138	1704
16	Kapenguria	2130	145	153	157	131	131	124	101	117	133	131	123	142	1588
17	Kapsabet	2000	177	176	198	162	152	136	138	148	166	176	171	169	1969
18	Kaputir	700	205	200	200	175	180	165	165	175	195	200	185	190	2235
19	Katumani	1600	181	165	166	136	145	126	116	125	153	171	136	170	1790
20	Kedong	1900	176	161	176	147	129	117	111	124	147	171	150	152	1761
21	Kericho	2070	160	152	166	125	130	125	121	120	124	125	121	141	1610
22	Kiambu	1730	192	178	180	138	129	98	109	117	158	166	151	165	1781
23	Kibos	1170	203	197	217	191	188	174	174	187	202	217	192	198	2340
24	Kimakia	2500	150	149	160	132	116	105	89	99	122	143	131	132	1528
25	Kipkabus	2400	178	183	199	152	149	116	124	128	156	177	152	165	1879
26	Kisumu	1140	187	182	195	164	157	143	144	156	165	182	167	176	2018
27	Kitale	1900	180	170	192	167	151	139	131	147	161	169	155	163	1925
28	Kitui	1180	189	191	200	169	168	152	149	162	183	203	163	167	2096
29	Koru	1600	182	174	180	152	148	144	140	145	163	163	158	170	1919
30	Lamu	9	219	199	220	182	173	162	166	188	193	214	206	205	2327
31	Lamuria	1850	132	133	144	136	156	140	146	138	165	147	115	115	1667
32	Lodwar	500	227	210	232	204	235	221	221	226	239	255	220	224	2714
33	Likichokio	1050	200	200	200	175	200	175	175	175	200	210	190	197	2297
34	Likitaung	700	255	255	270	221	232	235	234	242	261	257	238	239	2939
35	Machakos	1650	190	174	182	151	140	129	128	140	169	180	158	166	1907
36	Magadi	613	230	227	246	201	194	185	196	204	223	238	218	223	2585
37	Makindu	1000	175	179	182	160	151	139	139	153	179	191	154	149	1951

	Station	Altitude	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
38	Malindi	20	210	197	215	186	171	156	156	175	191	202	195	205	2259
39	Mandera	330	233	234	257	210	213	222	223	234	238	205	193	215	2677
40	Maralal	1950	161	159	173	151	151	132	130	132	151	157	139	150	1786
41	Marigat	1070	205	195	212	187	190	173	167	182	193	199	182	189	2274
42	Marsabit	1360	176	168	175	138	155	153	154	162	173	168	134	147	1903
43	Masara	1200	193	184	191	163	171	157	165	179	196	200	172	185	2156
44	Meru	1565	155	155	170	140	150	130	130	150	155	165	135	130	1765
45	Molo	2500	149	147	159	133	127	110	108	110	127	140	123	137	1570
46	Mombasa	60	211	204	221	180	152	148	144	162	181	198	200	204	2205
47	Moyale	1110	220	207	218	160	150	147	144	161	175	165	164	184	2095
48	Muguga	2100	173	171	186	141	116	107	96	109	143	171	149	152	1714
49	MweaTebere	1160	197	192	200	173	166	140	123	148	176	196	183	188	2082
50	Mwingi	1050	185	185	190	170	167	143	137	164	180	190	163	163	2037
51	Nairobi Kab.	1737	173	176	183	146	125	113	108	116	140	158	141	159	1738
52	Nairobi Sth	1675	195	189	192	157	144	122	119	132	166	184	169	179	1948
53	Naivasha	1900	167	160	169	134	137	123	126	133	153	160	139	153	1754
54	Nakuru	1890	137	156	163	133	139	132	138	141	145	142	121	146	1693
55	Nanyuki	1950	156	155	158	128	129	125	125	138	150	146	118	135	1663
56	Narok	1900	149	148	156	127	122	113	112	122	143	157	142	147	1638
57	Ngao	15	205	193	220	190	178	165	165	180	191	205	190	200	2282
58	Nyeri	1800	182	171	179	153	138	118	94	120	148	164	133	145	1745
59	OlJoroOrok	2380	129	131	152	117	122	109	94	101	117	122	110	108	1412
60	Oloitokitok	1850	160	123	116	124	117	107	91	104	128	170	150	148	1538
61	P. Victoria	1200	180	170	184	170	150	145	150	150	175	180	164	173	1991
62	Ruiru	1610	160	151	171	125	115	104	105	107	136	181	150	116	1621
63	Rumuruti	1860	181	177	196	171	168	149	150	158	178	186	167	178	2059
64	Sigor	1050	145	155	170	130	145	135	110	120	125	125	135	165	1660
65	Sth Kinangop	2600	116	113	129	110	99	88	81	86	100	119	105	105	1251
66	Subukia	2100	140	152	165	132	125	119	116	127	137	142	129	137	1621
67	Taveta	770	175	175	175	150	140	135	135	145	165	185	175	175	1930
68	Thika	1460	193	193	195	156	145	124	113	114	153	177	155	167	1885
69	Voi	560	183	187	198	176	166	158	156	162	174	189	182	175	2106
70	Wajir	240	233	225	238	205	205	199	201	206	213	207	187	208	2527
71	Wayu	160	203	190	209	190	190	167	173	187	191	193	182	198	2273
72	Yatta	1220	197	192	200	173	166	140	123	148	176	196	183	188	2082

* Note: Data from "Studies of Potential Evaporation in Kenya", T. Woodhead

Table 3-11: Monthly Open Water Evaporation for Dry Conditions (1 in 5) [mm]

	Station	Altitude	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
		m													
1	Ahero	1200	238	218	235	202	201	188	197	198	223	227	212	216	2555
2	Ainabkoi	2600	186	178	189	139	139	107	126	94	156	166	133	156	1769
3	Archers Post	865	223	226	246	225	234	226	240	250	265	251	204	208	2798
4	Bachuma	400	223	201	214	199	175	171	175	177	197	209	205	215	2361
5	Baricho	70	218	211	232	201	181	178	186	197	205	214	214	221	2458
6	Busia	1180	209	190	200	187	188	172	176	181	206	208	187	199	2303
7	Chebloch	1200	202	187	202	180	184	166	165	176	188	194	181	188	2213
8	Eldoret	2100	205	193	212	176	164	138	134	136	166	189	192	194	2099
9	Equator	2760	202	193	208	167	155	128	119	122	156	179	175	189	1993
10	Garissa	130	224	207	233	222	226	198	211	218	228	241	205	203	2616
11	Gede	30	211	179	206	194	170	148	166	169	195	211	204	210	2263
12	Habaswein	200	275	279	298	271	301	295	306	308	322	315	231	236	3437
13	Hola	90	221	219	238	208	209	181	190	199	212	219	214	218	2528
14	Isiolo	1100	232	224	246	223	235	225	240	251	266	249	203	210	2804
15	Kabondori	1140	205	181	180	162	139	107	138	132	184	176	149	161	1914
16	Kapenguria	2130	167	170	173	147	147	137	117	132	152	147	144	167	1800
17	Kapsabet	2000	200	192	215	178	168	148	157	164	186	195	196	195	2194
18	Kaputir	700	224	212	211	187	193	175	181	187	211	216	204	210	2411
19	Katumani	1600	204	181	180	150	161	138	132	138	172	190	155	196	1997
20	Kedong	1900	204	180	195	166	145	130	130	140	168	194	176	180	2008
21	Kericho	2070	186	169	183	141	146	139	142	136	143	142	143	167	1837
22	Kiambu	1730	216	195	196	153	142	107	125	129	177	185	173	190	1988
23	Kibos	1170	232	217	238	213	210	192	201	209	229	244	222	232	2639
24	Kimakia	2500	171	165	176	147	129	115	102	111	138	161	152	156	1723
25	Kipkabus	2400	201	200	216	168	165	127	141	141	175	196	174	190	2094
26	Kisumu	1140	213	201	214	183	175	157	167	174	187	205	193	205	2274
27	Kitale	1900	207	189	211	187	169	154	152	165	184	191	181	192	2182
28	Kitui	1180	212	208	216	185	184	165	168	178	204	224	185	190	2319
29	Koru	1600	209	193	199	170	166	159	163	163	186	184	184	200	2176
30	Lamu	9	245	215	237	198	189	175	186	206	214	235	232	232	2564
31	Lamuria	1850	150	146	157	151	173	153	167	154	186	164	144	134	1879
32	Lodwar	500	253	227	250	223	257	239	247	247	265	281	248	254	2991
33	Likichokio	1050	223	217	215	191	219	189	196	191	222	230	214	224	2531
34	Likitaung	700	284	277	291	241	253	254	263	265	289	283	268	271	3239
35	Machakos	1650	214	190	198	167	155	141	146	155	190	200	181	191	2128
36	Magadi	613	260	248	267	221	214	202	223	226	250	265	250	257	2883
37	Makindu	1000	197	195	197	176	166	151	157	166	200	211	175	170	2161

	Station	Altitude	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
38	Malindi	20	234	213	231	203	187	168	175	191	212	222	220	232	2488
39	Mandera	330	260	253	277	229	232	240	250	256	264	225	218	244	2948
40	Maralal	1950	178	171	185	163	164	142	145	144	167	172	155	169	1955
41	Marigat	1070	224	207	224	200	204	184	183	195	209	215	201	209	2455
42	Marsabit	1360	196	182	188	150	170	165	173	177	192	185	151	166	2095
43	Masara	1200	222	205	210	182	192	173	191	210	223	226	200	218	2452
44	Meru	1565	176	171	186	155	166	142	150	171	175	184	156	151	1983
45	Molo	2500	172	164	176	149	143	122	127	124	146	159	145	162	1789
46	Mombasa	60	235	220	238	196	164	160	162	177	200	217	226	231	2426
47	Moyale	1110	246	224	235	175	164	159	161	176	194	181	185	209	2309
48	Muguga	2100	203	193	208	160	132	120	114	125	166	197	179	182	1979
49	MweaTebere	1160	223	210	217	191	183	152	140	164	198	218	209	217	2322
50	Mwingi	1050	208	202	205	186	183	155	155	184	201	210	185	185	2259
51	Nairobi Kab.	1737	195	192	199	161	137	123	122	128	156	175	160	182	1930
52	Nairobi Sth	1675	220	207	208	173	160	133	136	145	186	205	193	206	2172
53	Naivasha	1900	191	177	185	150	153	135	145	149	174	179	162	179	1979
54	Nakuru	1890	156	172	`	149	155	145	160	157	164	159	140	170	1727
55	Nanyuki	1950	177	171	173	142	143	137	144	154	169	163	136	157	1866
56	Narok	1900	172	164	172	142	137	125	130	137	163	177	165	173	1857
57	Ngao	15	219	206	237	207	194	178	186	197	212	225	214	228	2503
58	Nyeri	1800	207	188	195	169	154	130	108	133	167	183	153	168	1955
59	OlJoroOrok	2380	148	145	167	130	136	120	109	113	133	137	128	127	1593
60	Oloitokitok	1850	180	135	126	136	129	117	103	115	144	190	173	171	1719
61	P. Victoria	1200	203	186	200	187	166	160	172	166	196	201	187	199	2223
62	Ruiru	1610	182	167	187	138	127	114	121	119	153	202	173	135	1818
63	Rumuruti	1860	200	191	210	186	183	161	167	171	196	203	186	201	2255
64	Sigor	1050	167	172	187	146	163	149	127	135	143	140	158	194	1881
65	Sth Kinangop	2600	130	124	141	121	109	96	62	94	112	132	121	121	1363
66	Subukia	2100	160	168	181	147	140	131	134	142	155	159	150	160	1827
67	Taveta	770	195	190	189	164	153	146	151	158	183	204	197	198	2128
68	Thika	1460	219	212	213	173	161	136	129	126	172	198	179	193	2111
69	Voi	560	204	203	214	192	182	171	175	177	193	208	205	198	2322
70	Wajir	240	260	244	257	224	224	215	225	225	236	228	211	236	2785
71	Wayu	160	226	206	225	207	208	180	194	204	212	212	205	225	2504
72	Yatta	1220	223	210	217	191	183	152	140	164	198	218	209	217	2322

Source: Woodhead, T. 1968. Studies of Potential Evaporation in Kenya. East Africa Agriculture and Forestry Research Organization. Nairobi.

3.3.11 Seepage Losses

Seepage losses can occur through the floor of the reservoir area, and beneath or through the embankment. The seepage is a function of the hydraulic head, soil properties, the embankment design and construction techniques. At the planning stage of a project, a fair estimate of the seepage losses is needed. Table 3-12 provides hydraulic conductivity values for different soil conditions. Maximum daily seepage losses can be approximated using Equation 3-4 which assumes a unit hydraulic gradient and uses the surface area rather than the wetted surface area.

Table 3-12: Hydraulic Conductivity

Water Depth (m)	Hydraulic Conductivity (m/s)	
	Lower Limit	Upper Limit
Permeable	2×10^{-7}	2×10^{-1}
Semi-permeable	1×10^{-11}	1×10^{-5}
Impermeable	1×10^{-11}	5×10^{-7}

Equation 3-4 $Svol = K \times A_{max} \times 86400$

Where: S_{vol} = Maximum seepage losses [m^3/day]
K = Hydraulic Conductivity [m/s]
A_{max} = Maximum reservoir surface area [ha].

3.3.12 Environmental Flows

Any requirement for downstream environmental flows should be factored into the water demand calculations. Environmental flows for perennial rivers and streams should be in line with the Q95 flows at the proposed dam site. Environmental flows for seasonal rivers are much harder to quantify and are generally not included in water demand calculations for storage on seasonal rivers.

3.4 Project Team

The planning, design, implementation and operation of a small dam, pan or water conservation structure requires a project team to bring together the expertise and skills required so that the project delivers sustainable benefits.

The roles and responsibilities for different project members are described below to facilitate teamwork and to help minimise disputes.

3.4.1 Project Proponent/Owner

The dam proponent or owner is any individual or body corporate who wishes to construct a water storage facility and who has legal access to the land on which the proposed structure is to be built.

The dam proponent must undertake the following:

- Submit a duly completed and signed Water Permit Application to WRMA, including form WRMA 001A and 001C;
- Pay the appropriate permit assessment fee which is dependent on the class of the permit application;
- Commission at his/her own cost a Dam Design Report carried out by a Qualified Water Resource Professional as set out in Section 57 of the WRM Rules (2007);
- Commission at his/her own cost an Environmental Impact Assessment (EIA) in accordance with the Environmental Management and Coordination Act 1999;

- Register the proposed project with the National Construction Authority within 30 days after awarding of the contract (NCA Regulations, 2014).

Once the WRMA has issued an Authorisation to Construct, the proponent must:

- Commission at his/her own cost a Qualified Contractor;
- Commission at his/her own cost a Qualified Water Resource Professional to supervise construction;
- Ensure that the construction is inspected at the milestones stated in the Authorisation to Construct;
- Ensure all risks (including third party) liability coverage for the duration of the construction work;
- Apply for an extension to the Authorisation to Construct in the event that the works are not completed within the allotted time.

Once the works are complete the proponent must submit a Completion Certificate (WRMA Form 008) to WRMA. This will provoke a final inspection by WRMA, and on satisfactory completion, WRMA will issue a Water Permit.

If an Emergency Action Plan (EAP) exists for the project, the proponent is responsible for ensuring that the EAP is adhered to. EAPs are discussed in detail in Section 3.6.

An owner of an existing water conservation structure must:

1. Ensure that the structure has a valid water permit. If the permit has expired then the dam owner must apply for a renewal;
2. Ensure third party liability coverage to cover any injuries or damages that result from use of the structure or activities on the structure or failure of the structure;
3. Ensure dam safety inspection in accordance with Section 59 of the WRM Rules (2007);
4. Ensure operation of the dam in accordance with the operating rules set out in the Dam Design Report and as may be required by WRMA and stipulated in the Conditions of Authorisation;
5. Report any dam failure or damage to the WRMA in accordance with Section 68 of the WRM Rules (2007).

3.4.2 Operator

In the event that the dam operator is not the same person as the owner, then the operator must be provided with the authority and complimentary responsibilities for operation through a proper lease or contract. This document must be vetted and lodged with the WRMA. The contract must be clear on the following items:

- Any financial costs or benefits that derive from operating the dam, including the costs for inspection, water use charges and permit fees as may be appropriate;
- Responsibility for operations and maintenance;
- Responsibility for repairs;
- Responsibility for safety and emergency procedures;
- Liability and insurance against the same;
- Procedures for the resolution of disputes;
- Contract determination.

3.4.3 Qualified Water Resource Professional

The role of the Qualified Water Resource Professional is to provide technical advice to his/her client and to WRMA.

A Qualified Water Resource Professional is a person who is licensed by the MWIS and who is either:

- A person who has graduated with a degree from any recognized university, and
- Who has had at least five years practical experience in the relevant profession, and
- Who is a registered member of the relevant professional institution of that profession.

The registered Qualified Water Resource Professionals are gazetted annually by the MWIS. A professional who is not licensed can only work under the supervision of a licensed professional.

The duties of the Qualified Water Resource Professional revolve around:

1) Building compliance to the water regulations;
2) Supporting the dam proponent to fulfil statutory requirements in relation to the structure;
3) Undertaking the site investigations, design and developing a dam design report;
4) Supporting his/her client to obtain a qualified dam contractor and in the management of the construction contract;
5) Supervising construction to ensure that the contractor follows the design requirements and specifications;
6) Facilitating the inspection by WRMA at the milestones laid out in the Authorisation to Construct;
7) Undertaking dam safety inspections and submitting the required report to WRMA.

Failure by the Qualified Water Resource Professional to adhere to the water regulations can be cause for disciplinary action against the Qualified Professional.

License requirements for dam design engineers, based on WRM Rules (2007) are as shown in Table 3-13.

Table 3-13: License Requirements for Dam Designs

Class of Dam	Category of Qualified Water Resource Professional
A (Low Risk)	Panel II C, Panel I C1& Panel I C2
B (Medium Risk)	Panel I C1 & Panel I C2
C (High Risk)	Panel I C2

3.4.4 Technical Team

Depending on the scale of the project, the technical team undertaking the design work may include the following:

- Dam Design Engineer
- Hydrologist
- Surveyor
- Geotechnical Engineer/Engineering Geologist
- Electro-mechanical Engineer
- Environment/Social Specialist

3.4.5 Qualified Contractor

The role of the Qualified Contractor is to provide construction services to his/her client. These duties revolve around building water storage structures that comply with the approved design and specifications.

Failure by the Qualified Contractor to adhere to the water regulations can be cause for disciplinary action against the Qualified Contractor.

The National Construction Authority requires that all contractors be duly registered and accredited by the body. (National Construction Authority Regulations, 2014).The project proponent/ owner can only engage a contractor registered with the NCA.

3.4.6 Role of WRMA and Other Regulators

WRMA is the regulator for water resources which includes regulating water storage structures and water allocations. In this regard, WRMA is responsible for ensuring public safety and therefore has to review dam designs and construction activities to ensure they meet acceptable standards and are compliant with regulations. The permit application and inspection process provides WRMA with the information it needs to authorise and permit the structure and water use.

It is not expected that WRMA will be directly involved in the design or construction of any structure, except in the provision of water resource information held on its databases and providing advice relating to the application/approval process.

Other government regulators (e.g. NEMA) have the responsibility to ensure that the project is compliant with their respective legislations.

The National Construction Authority (NCA) was established to oversee the construction industry and coordinate its development. Part of its functions include: accreditation and registration of contractors and regulation of their works, as well as accreditation and certification of skilled construction workers and construction site supervisors. All contracts, construction works projects and projects are required by law to be registered with the Authority (National Construction Authority Regulations, 2014).

3.4.7 Role of the Ministry

The Ministry, represented by the State Department for Water, is responsible for policy, coordination and monitoring sector performance, including setting out standards. The Ministry is responsible for registering Qualified Water Resource Professionals and Qualified Contractors. Any questions related to the registration or competence of a Qualified Water Resource Professional or Qualified Contractor should be channelled to the Registrar in the State Department for Water and the National Construction Authority.

3.4.8 Role of County Government

The main role of the county government is to ensure that the proposed project fits within the county development plan. There should be close liaison with County Government during the planning stages of any water storage project.

County Government input is often required to resolve land disputes.

3.4.9 Project Beneficiary

Water conservation structures designed and built for public benefit will have beneficiaries who are not necessarily the project proponent. In this case, it is useful to distinguish the project beneficiaries from the project proponent. The project beneficiaries have the responsibility to:

a) Pay for the water supply, if agreed, in a timely manner;
b) Make efficient use of the water;
c) Ensure that those responsible for implementing and maintaining the structure are held accountable for the financial and technical performance;
d) Report any problems with the structure, water quality or service to the relevant authorities.

Chapter 5 provides further details on community engagement.

3.4.10 Development Partner

The development partner is a local or international agency that is supporting the design and/or implementation of the project with technical and/or financial resources. The development partner is not the project proponent, the owner, operator or beneficiary. In general the development partner will cease to play a role in the project once the structure is built.

The development partner may however place conditions on the project as may be agreed with the government agencies and the project proponent that relate to legal compliance, environmental compliance, fiduciary controls, and distribution of benefits.

3.4.11 Role of the Community

As the overall beneficiary, the community's role would be to participate throughout the entire project implementation process. More discussion on community participation is detailed in Chapter 5. The community needs to see itself in one or more of the roles that have been set out above.

3.4.12 Role of Other Stakeholders

Other stakeholders may play the role of watch-dog in the public interest, reporting or engaging on any issues that might affect the public or environment.

3.5 Project Timeline

The project timeline sets out the phases and main tasks in order to establish the overall project duration and sequencing of main tasks. Figure 3-1 provides a typical project timeline which should be customised to suit each individual project. A detailed construction plan is specific to the construction phase for each individual project. Examples of a construction plan are provided later in the relevant chapters.

The timeline in Figure 3-1 assumes a fairly short construction period of 20 weeks and covers a total period of 70 weeks. The total time taken for a project is highly dependent on both construction time and sourcing funding.

Important considerations regarding the timing of the project are:

- Stakeholder engagement should commence at the early phase of the project and continue throughout the project.
- Sufficient time should be provided for WRMA and NEMA approval. Experience has shown that WRMA approval can take up to 6 months and NEMA approval for small scale projects can take up to 45 days.

- The construction phase should ideally be conducted during the dry season to avoid disruption to construction by inclement weather. However, in arid areas, construction must also be scheduled when water is available and often must occur at the onset or tail end of the rainy season. Water is especially important for proper compaction of earth embankments and generally a water volume of 30% of the embankment volume will be needed during the construction phase.

3.6 Dam Safety Planning

The impoundment of water particularly by a dam forms a hazard so due consideration is required to the nature of the hazard, the risk of harm and/or damage, and mitigation measures that can be undertaken to minimise the risks.

The Emergency Action Plan (EAP) is a useful tool which helps to identify preventive measures which can reduce the scale of harm and damage in the event of a dam failure. The preparation of an EAP is now considered good practice for small dams that fall into a medium or high hazard class. The EAP should be developed by the dam owner/operator, in collaboration with other relevant parties as described below and should be maintained in a ready-state.

The EAP involves an analysis of the risks and anticipates an emergency that would necessitate immediate notification of government officials and downstream communities to minimise harm and damage downstream.

This material is drawn heavily from the “Federal Guidelines for Dam Safety: Emergency Action Planning for Dam Owners”, (Interagency Committee on Dam Safety, April 2004)

The EAP is a site specific document which covers the following components.

3.6.1 Notification Flow Chart

The notification flowchart provides the name, contacts, organisation, position and priority for those who should be notified and the cascade of responsibility for onward notification to other parties (See Figure 3-2). The notification list should consider the following individuals and organisations:

- Dam owner and/or operator;
- Local emergency management offices;
- Local county and administration officials;
- Local police station;
- Water resource user associations;
- Downstream residents, water users and downstream dam owners/operators;
- Local Red Cross offices;
- Media.

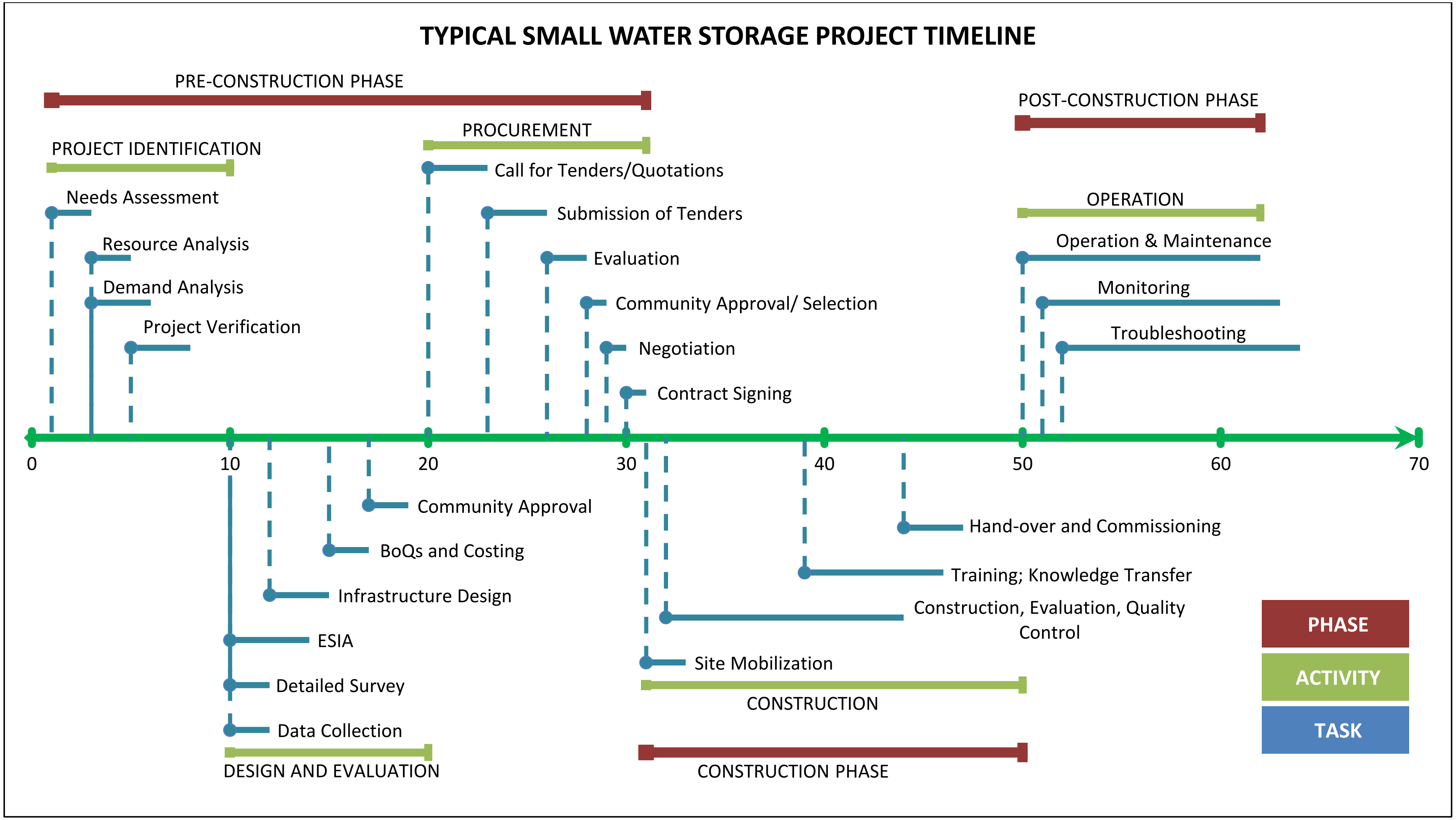

Figure 3-1: Typical Project Time Line

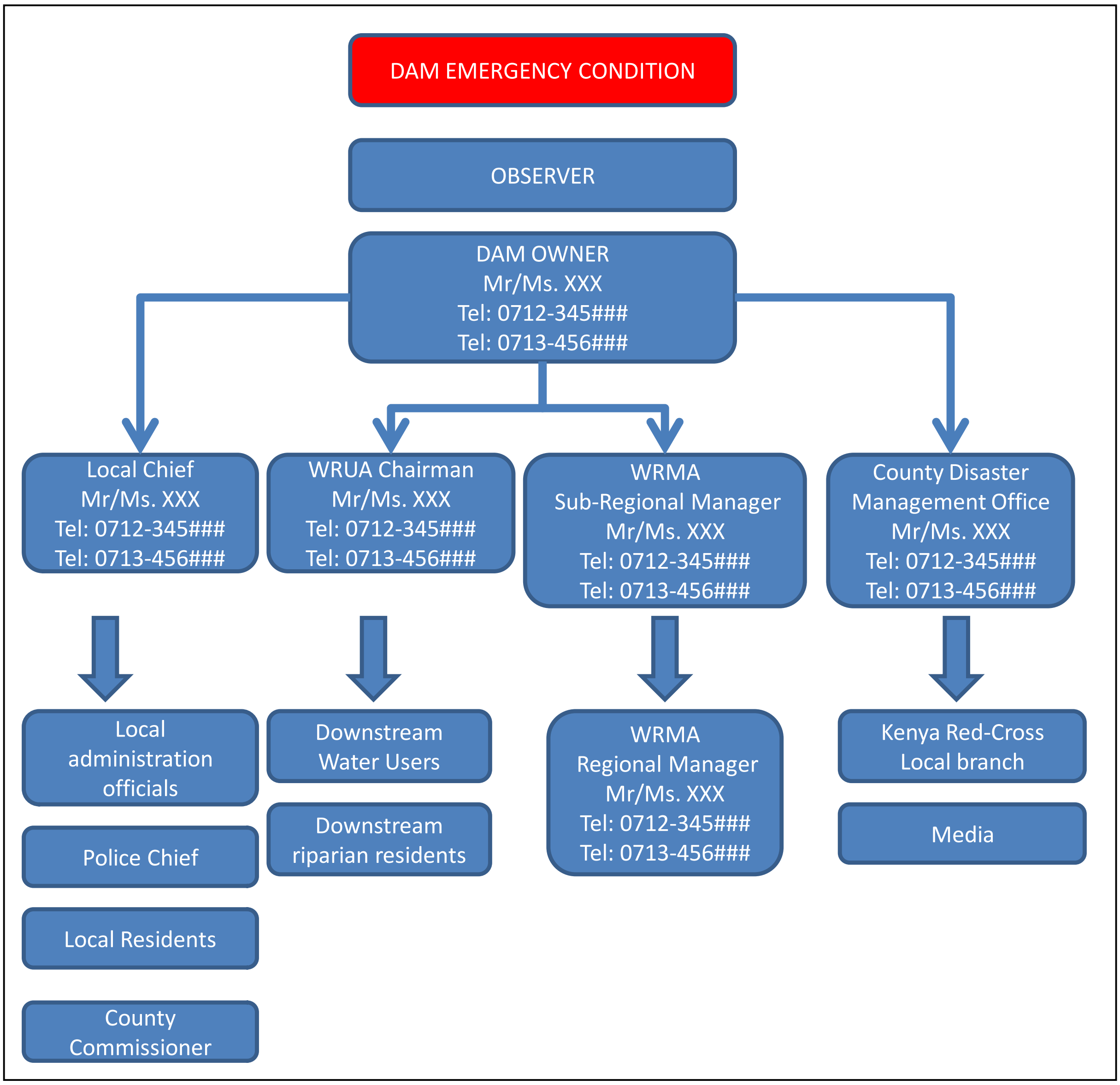

Figure 3-2: Sample Notification Flowchart

3.6.2 Project Description

A description of the nature, scale and location of the dam is provided, including a detailed map indicating access routes to the site. In addition, any upstream or downstream dams should be identified.

3.6.3 Emergency Detection, Evaluation and Classification

The EAP should include a description of the inspection and monitoring systems which are needed to ensure timely **detection** of an existing or potential emergency. These systems should ensure that competent persons, able to identify a problem, are involved in the inspection and monitoring procedures. In addition, the data and information required to help identify abnormal conditions should be provided.

Once an existing or potential emergency has been detected, it should be **evaluated** against the emergency classification system to establish the level of threat. The emergency classification system should use terms and conditions that are agreed by the dam owner/operator in conjunction with the county disaster emergency officials and which clearly indicate the urgency of the emergency condition. As the declaration of an emergency can be a controversial decision, the aim of the classification system is to provide a framework for quick decisions and actions.

Emergencies can be classified into two categories:

A. **Alarm** Category. Failure is imminent or has just occurred. This condition implies that time has run out and immediate steps must be taken to notify and evacuate vulnerable downstream people. The implication is that corrective measures cannot prevent failure and the focus of attention should be on safeguarding lives.
B. **Alert** Category. The alert condition applies if conditions are developing that could result in flooding downstream either due to dam failure or extreme floods. This condition implies that there is time to implement pre-planned actions to prevent or control failure, notify downstream inhabitants and protect infrastructure. A report of an Alert category emergency should indicate the expected time period before an Alarm situation arises.

3.6.4 Responsibilities

The EAP should clearly indicate the person and organisation responsible for the maintenance and operation of the dam and the persons responsible for implementing different components of the EAP, including co-ordinating the response.

A. Dam Owner/Operator

The dam owner/operator holds the primary responsibility to see that the EAP is developed, up-to-date and that preparedness activities have been undertaken with all parties concerned.

B. Responsibility for Notification

The notification flowchart indicates the person and details to be contacted when an emergency has been observed. The person contacted should have the authority to provide onward notification to third parties without having to seek authority as this can delay the dissemination of the notice.

C. Responsibility for Evacuation

Evacuation is typically undertaken at the direction and supervision of government authorities. In this case the County Disaster Management Office will coordinate with the County administration to implement any evaluation plans. However, the dam owner/operator should be prepared to assist with evacuation activities, particularly in the vicinity of the dam.

D. Responsibility for Duration, Security, Termination and Follow-Up

The EAP should provide details on the person responsible for real-time monitoring of the situation at the dam site to provide local authorities with up-dated information during and after the emergency. This information will help local authorities to be able to terminate the emergency.

E. EAP Coordination

The EAP should identify the EAP Coordinator who should be a person with sufficient authority to revise the EAP should the need arise and be able to share emerging information with other parties.

3.6.5 Preparedness

The EAP should specify the preparedness actions that are planned and implemented under normal operating conditions. These preparedness actions may include:

- Providing real-time monitoring of reservoir levels and/or spillway releases in combination with pre-defined thresholds that trigger the emergence of an alert or alarm situation;
- Identification and preparation of normal and alternative access routes to the site and/or downstream inundation areas, including options to be followed during adverse weather conditions;
- Identification of backup systems to illuminate the site;
- Identification of alternate systems of communication should the mobile phone network be inoperative;
- Pre-positioning of emergency supplies, equipment and machinery that may assist in the prevention of or during an emergency;
- Anticipating emergency scenarios and providing specific information on steps to be taken within each scenario to reduce the threat and protect lives and infrastructure.

3.6.6 Inundation Maps

Inundation maps provide an estimate of the areas that may be inundated should a dam failure occur. The maps should provide specific information to the county disaster management offices regarding any settlements or specific infrastructure (roads, power lines, power stations, etc.) within the inundation areas. The maps should clearly indicate the emergency scenario covered by each inundation map. For example, the inundation map should clearly show whether the scenario is a dam failure scenario or an extreme flood event as the inundation areas may be different depending on the scenario analysed.

3.6.7 Appendices

A. Investigation and Analysis of Dam Break Floods

The EAP should identify and briefly describe the method and assumptions used to identify the potential inundation areas. Typically these assumptions will include items related to the nature of breach, the storage condition, time to breach, prevailing weather and inflow conditions, and flood routing.

B. Plans for Training, Exercising, Updating and Posting the EAP

1. **Training.** The EAP should include a training plan in which the individuals to be trained are identified and the curriculum showing the specific information and tasks that the individuals are expected to undertake. Scenario simulation provides a useful method to familiarise trainees with roles, information and responsibilities under the EAP.

2. **Exercising.** A simulation of an emergency is a useful way to test out whether emergency procedures are ready for use. An evaluation of the simulation exercise is required to identify bottlenecks, areas of confusion or lack of appropriate information.

3. **Updating.** The EAP should include a schedule for regular updating to ensure that all the contact information on the notification flow chart is correct and that all revised copies of the EAP are circulated.

4. **Posting.** The most recent version of the EAP Notification Flowchart should be posted in prominent places at the dam site, at the owner/operators office and at local offices for the county disaster management office, WRUA, and chief.

C. Site Specific Concerns

This section of the EAP should include any detailed information or drawings that relate to the dam structure and which may be useful during or after an emergency.

D. Approval

The EAP should be approved by the dam owner/operator, WRMA, the County Disaster Management Office and local chief. This indicates that the main parties to the EAP have understood their responsibilities under the EAP.

3.6.8 Suggested EAP Format

The EAP should be structured to enable quick and easy reference to key information. A suggested format and outline is given in Chapter 19.

3.7 Common Problems in the Design, Construction and Rehabilitation of Small Dams

The purpose of the present Practice Manual is among others to address a number of frequently occurring problems regarding construction and rehabilitation of small dams, pans and other water conservation structures, which in most cases can be avoided or dealt with without necessitating the use of sophisticated means.

Therefore, the following points merit particular attention.

3.7.1 Hydraulic Failures

Hydraulic failures are reservoir failures caused by overtopping or surface erosion.

- **Overtopping:** When the free board of the dam or capacity of the spillway is insufficient, the flood water will pass over the dam and erode the embankment. The most important single technical reason why embankment failures occur in small earth dams in Kenya (and elsewhere) is insufficient spillway capacity (caused by either under-estimation of the flood flows, under-dimensioning of the spillway structure, lack of maintenance of the spillway or changes in the catchment characteristics). Insufficient spillway capacity can cause overtopping of the embankment with subsequent erosion of the downstream slope resulting finally in embankment failure. In order to avoid this problem, it is necessary to conduct a proper investigation of flood flows, and to determine the spillway dimensions accordingly.

  In many cases the proper functioning of the spillway is seriously hampered by pronounced erosion, especially in the outlet channel. Apart from adequate design, including erosion protection measures where required, it is obvious that (especially for earth lined channels) regular inspection of the structure and prompt remedial action are indispensable to ensure correct functioning of the spillway.

- **Erosion of downstream toe:** The toe of the dam at the downstream side may be eroded due to i) heavy cross-current from spillway flows, or ii) tail water. When the downstream toe is eroded, it will lead to failure of the dam. This can be prevented by providing a downstream slope pitching or a riprap up to a height above the tail water depth Also, the side wall of the spillway should have sufficient height and length to prevent possibility of cross flow towards the earth embankment.

- **Erosion of upstream surface:** During winds, the waves developed near the top water surface may cut into the soil of the upstream dam face which may cause slip of the upstream surface leading to failure. To prevent against such failure, the upstream face should be protected with stone pitching or riprap.

- **Erosion of downstream face by gully formation:** During heavy rains, the flowing rain water over the downstream face can erode the surface, creating gullies, which could lead to failure. Erosion by wildlife and livestock can also lead to gully formation. To prevent such failures, the dam surface should be properly maintained; all cuts filled on time and surface well grassed. Berms could be provided at suitable heights and surface well drained.

3.7.2 Seepage Failure

Seepage always occurs in dams. If the magnitude is within design limits, it may not harm the stability of the dam. However, if seepage is concentrated or uncontrolled beyond limits, it will lead to failure of the dam. Following are some of the various types of seepage failure.

- **Piping through the dam body:** When seepage starts through poor soils in the body of the dam, small channels are formed which transport material downstream. As more materials are transported downstream, the channels grow bigger and bigger which could lead to wash out of the dam. Piping is often caused by inadequate choice of construction material (soil). Location and choice of borrow areas should be carried out by experienced personnel. Heavy clays as well as soils containing a large percentage of sand are in principle not suitable for the construction of homogeneous embankment dams. Soils to be used as construction materials should also systematically be tested for dispersivity (See Section 9.4.4).

- **Piping through the foundation:** When highly permeable cavities or fissures or strata of gravel or coarse sand are present in the dam foundation, it may lead to heavy seepage. The concentrated seepage at high rate will erode soil which will cause increase flow of water and soil. As a result, the dam will settle or sink leading to failure.

- **Sloughing of the downstream side of the dam:** The process of failure due to sloughing starts when the downstream toe of the dam becomes saturated and starts getting eroded, causing a small slump or slide of the dam. The small slide leaves a relative steep face, which also becomes saturated due to seepage and also slumps again and forms more unstable surface. The process of saturation and slumping continues, leading to failure of dam.

3.7.3 Structural Failure

This is mainly due to shear failure causing slides along the slopes. The failure may be due to:

- **Slide in the embankment:** When the slopes of the embankments are too steep, the embankment may slide causing failure. This might happen when there is a sudden drawdown, which is critical for the upstream side because of the development of extremely high pore pressures, which decreases the shearing strength of the soil. The downstream side can also slide especially when the dam is full.

- **Foundation slide:** When the foundation of an earth dam is composed of fine silt, clay, or similar soft soil, the whole dam may slide due to water thrust. If seams of fissured rocks, such as soft clay, or shale exist below the foundation, the side thrust of the water pressure may shear the whole dam and cause its failure. In such failure the top of the dam gets cracked and subsides, the lower slopes moves outward and forms large mud waves near the dam heel.

- **Faulty construction and poor maintenance:** When during construction, the compaction of the embankment is not properly done, it may lead to structural failure.

3.7.4 Operational Failure

Failure of a reservoir to fill or excessive siltation can be considered as operational failures.

- **Rapid siltation:** Siltation of reservoirs can best be addressed through a catchment wide program of erosion prevention and soil conservation, but a considerable number of problems can be avoided (or at least alleviated) by a considered choice of the dam location, avoiding wherever possible rivers with excessive silt loads.

- **Evaporation:** Significant water losses through evaporation can be a concern. As a general rule, and with the possible exception of reservoirs solely intended for the supply of livestock during part of the year, care should be taken not to construct very shallow reservoirs (water depth <3-4 metres) in areas of high potential evaporation. Other possibilities (e.g. sub-surface dams) should be considered carefully if excessive water losses through evaporation are expected.

- **Lack of inflow:** Lack of inflow can result from inaccurate estimation of catchment runoff or from construction of storage or other water infrastructure upstream in the catchment. Future catchment development plans should be considered before committing to storage projects.

- **Lack of maintenance:** The operation, maintenance and training aspects of completed and ongoing small earth dam projects often receive insufficient attention, while public awareness and participation of the involved communities is often inadequate. This results in rapid degradation - and even partial destruction - of completed structures due to easily avoidable causes: cattle is left to wander up earth embankments and into the reservoir thus wearing out the embankment and damaging fences, while severely polluting the reservoir water; trees are left to grow on the embankment (large roots in the embankment will create preferential seepage paths for the water): nothing is done about beginning erosion -by rainfall or run-off in the spillway and on the embankment; high vegetation is left thriving in the spillway channel, thus hampering the spillway discharge capacity etc.

CHAPTER 4

POLICY AND LEGAL COMPLIANCE

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4 POLICY AND LEGAL COMPLIANCE

This chapter sets out the policy and legal framework for the development of water conservation structures. These are principally defined by the policy and laws governing the water and environment sectors.

4.1 Policy on Water Storage

The National Water Harvesting and Storage Management Policy[1] (May 2010) is the current MWIS's policy on water storage. Notwithstanding that the policy has not been gazetted as a sessional paper which provokes debate regarding its legal validity, the policy does set out the MWIS approach to issues related to water storage development.

The policy objectives are to:

- Provide a framework for expansion of infrastructure for national water storage capacity from the current 124 Mm^3 to 4.5 Bm^3 to ensure an increase in per capita storage from 5.3m^3 to 16m^3 over the next ten years;
- Improve participation in planning, financing and investment by communities, development partners, NGOs, PPPs, and other stakeholders' contributions;
- Create an enabling environment for the participation of farmers and/or land owners, water user groups, and all water sector stakeholders in planning, implementation and management of water harvesting, storage and flood infrastructure;
- Enhance flood mitigation preparedness in affected areas;
- Build human resource capacity to enhance innovation, research, science and technology, adoption and management of water harvesting and storage systems and flood control structures;
- Enhance a stakeholders-driven multi-sectoral approach to sustainable water harvesting and storage systems and flood control structures, as well as expansion and protection of water catchment areas;
- Ensure integrated coordination of stakeholder activities for development of water harvesting, storage and flood control infrastructure; and
- Establish a responsive institutional, legal, and regulatory framework for water harvesting, storage and flood control.

The policy's guiding principles are as follows:

On the development of infrastructure, the policy principle is that the Ministry and other stakeholders shall undertake water harvesting and storage infrastructure planning, design, development and management based on the latest innovations, research, science, technology, information, and management and make use of the most appropriate and cost-effective best practices to optimize sustainability.

On regulation the policy principle is that every water harvesting, storage and flood control project shall be registered with the government institution responsible for water storage and flood control.

On the issue of effective management the policy principle is that the implementation of water harvesting, storage and flood control systems shall be optimally and efficiently managed to ensure sustainable economic returns and social enhancement.

[1] The National Water Harvesting and Storage Management Policy has not been gazetted as a Sessional Paper, which is not an uncommon status for Ministerial policy documents.

On licensing the policy principle is that every individual and institution responsible for design, development, implementation and management of water harvesting, storage and flood control structures shall acquire relevant permits, authorizations and licenses (where applicable) from relevant government agencies or any such agency as may be proscribed by law from time to time.

On general responsiveness, equality and equity the policy principle is that planning and implementation of water harvesting and storage systems and flood control programmes shall embrace equality and equity while being sensitive to gender and the specific needs of the youth, minority groups including persons with disabilities, orphans, and other vulnerable and marginalized groups in communities in targeted areas.

On partnerships the policy principle is that effective partnerships shall be developed in all stages of planning and implementation within the framework of an Integrated Water Resources Management (IWRM) approach.

On ecological stability the policy principle is that when implementing water harvesting, storage and flood control programmes, water institutions and water-related agencies shall take into account ecosystems' integrity and resilience, and biodiversity and environmental conservation.

On access to water resources the policy principle is that every household and institutional needs for water including domestic, livestock, crop agriculture, aquaculture, irrigated agriculture, commercial, industrial, social, environmental services and other uses shall be taken into account.

On access to health services, the policy principle is that every individual has a right to have access to safe drinking water and adequate sanitation, in an environment of reduced incidences of water-borne and water-related diseases and incorporation of public health aspects in the development of water harvesting, storage and flood control systems.

On disaster responsiveness the policy principle is that planning and implementation of water harvesting, storage and flood control structures shall incorporate disaster preparedness and management to enable households and institutions to cope with and mitigate the impacts of cumulative climate variability, and natural disasters.

On ethics the policy principle is that planning and implementation of water harvesting, storage and flood control programmes shall be ethically executed within recognized and proscribed institutional and legal frameworks, to be created under this policy.

On governance the policy principle is that planning and implementation of water harvesting, storage and flood control systems shall recognize cross-cutting aspects and shall be guided by the principles of transparency, accountability, and good governance inscribed within the rule of law.

Finally the policy is to be implemented through the existing water laws and regulations governing the management of water resources.

The national water harvesting and storage policy is supplemented by policies on water resources management, land use planning, irrigation, forests, land degradation, livestock and environment, all of which must be taken into account in considering the applicable policy framework.

4.2 The Legal Framework

The legal framework comprises laws and regulations governing the natural resources sectors. The overarching law is the Constitution of Kenya 2010. Article 43 deals with economic and social rights and includes the right to clean and safe water in adequate quantities as a fundamental human right. The state therefore has an obligation to ensure that every Kenyan has access to clean and safe water, which makes it imperative that the state put in place measures and frameworks for making water accessible, through dams and other storage infrastructure among other measures.

The Constitution also puts in place the applicable institutional framework. It establishes two levels of government: the national government and county governments. The Fourth Schedule allocates functions to the two levels of government. The national government has the mandate over:

- the use of water resources;
- water protection;
- securing sufficient residual water;
- hydraulic engineering;
- the safety of dams; and
- disaster management.

The county government has the mandate over soil and water conservation and county public works and services including water and sanitations services under which would fall the construction of small dams and small storage facilities for water services purposes.

The current water law is the Water Act 2002 which governs the water sector. The Government has however presented to Parliament the Water Bill 2014 to align the provisions of the water law to the Constitution of Kenya 2010. The Government has also developed a revised water policy which has not yet been adopted as a sessional paper.

Other components of the legal framework are laws governing the sectors on environment, forest, wildlife, agriculture, land use planning and regional development, county government and the management of public finances.

Regarding the private sector there are laws governing the provision of services by professional engineers, contractors and other service providers. The Government has put in place a legal framework for procurement of professional and other services and this procurement system would apply if the dams and small structures are being financed by a public agency or out of public funds.

4.2.1 Water Allocation

Water allocation is governed by the provisions of the Water Act 2002. The responsibility for water allocation lies with the Water Resources Management Authority (WRMA). Under Section 8 of the Water Act, 2002 one of WRMA's functions is to develop principles, guidelines and procedures for the allocation of water resources.

The Water Act itself sets certain key principles which set the parameters for water allocation as follows:

a. Through the national water resources management strategy WRMA shall determine the requirements of the reserve for each water resource. This determines the water available for allocation;
b. Water resources are to be classified. The class of water resource is important in determining the use to which it may be put, including whether and the extent to which impoundment and abstraction is permissible;
c. Provision is made for designating areas as protected areas and ground water conservation areas, which limits the allowable activities within a particular area.

In addition to the national water resources management strategy there will be for each catchment a catchment area management strategy (CMS) which will, among other aspects:

- Contain water allocation plans and set out the principles for allocating water in the catchment.

- Contain mechanisms for stakeholder consultation.

Any project to construct facilities for small dams and other water conservation structures needs to take account of both the national water resources management strategy and the applicable catchment area management strategy in so far as they relate to the allocation of water resources for the project.

Other key principles which give priority in the allocation of water resources are:

1. The use of water for public purposes (including storage or impoundment of water for bulk distribution take priority in the allocation of water resources).
2. The use of water for domestic purposes takes precedence in the allocation of water over the use of water for other purposes.

Finally it is important to note that water resources will only be allocated on the basis of availability; compliance with conditions set by WRMA to secure the similar right of other users downstream; construction of structures which meet the standards set by WRMA; and the payment of a water use charges.

4.2.2 Requirement for a Permit

The Water Act 2002 imposes a requirement for a permit on any person wishing to acquire a right to use water from a water resource. Section 27 makes it an offence to construct or use works to abstract water without a permit. There are however three exceptions to the permit requirement. These are cases of:

- minor uses of water resources for domestic purposes (where abstraction is conducted without works or equipment) ;
- uses of underground water in areas not considered to face groundwater stress and therefore not declared to be groundwater conservation areas; and
- uses of water drawn from artificial dams or channels, which – being artificial rather than natural - are not considered to be water resources of the country.

The application for the permit is made to WRMA. Section 32 stipulates the factors to be taken into account in considering an application for a permit. These include:

a) The existing lawful uses of the water;
b) Efficient and beneficial use of the water in the public interest;
c) The likely effect of the proposed water use on the water resource and on other water users;
d) The strategic importance of the proposed water use;
e) The probable duration of the activity for which the water use is required;
f) Any applicable catchment management strategy; and
g) The quality of water in the water resource, which may be required for the reserve.

It is expected that WRMA will also take into account any requirements imposed by international agreements related the catchment relevant to the water permit application.

These considerations are designed to enable WRMA balance the demands of competing users, but also to take into account the need to protect the general public interest in the use of water resources as well as the imperative to conserve water resources.

Further guidance is given to the Authority in deciding on allocation of the water resource as follows:

a. That the use of water for domestic purposes shall take precedence over the use of water for any other purpose and, in granting a permit, the Authority may reserve such part of the quantity of water in a water resource as is required for domestic purposes. It is to be recalled that, in rural settings, use of water for domestic purposes typically includes use for small dams, water pans and similar structures; and

b. That the nature and degree of water use authorized by a permit shall be reasonable and beneficial in relation to others who use the same sources of supply.

Permits are given for a specified period of time. Additionally, WRMA is given power to impose a charge for the use of water. Details of the charges to be imposed, including the amounts to be charged, and the uses for which a charge may be imposed are spelt out in the Water Resources Management Rules, 2007, which should be consulted before proceeding.

Permits run with the land. Where the land is transferred or otherwise disposed of, the permit also passes to the new owner of the land. Section 34 requires that a permit specify the particular portion of any land to which the permit is to be appurtenant. Where the land on which the water is to be used does not abut on the watercourse the permit holder must acquire an easement over the lands on which the works are to be situated. It is thus not possible, under the law, to obtain a permit in gross (i.e., which is not linked to particular land).

In all cases of construction of new small dams or pans, an application for a Water Permit should be filed with WRMA. Where small dams and pans are rehabilitated, it should be examined on a case to case basis whether the existing Water Permit (if any) is still relevant to the situation after rehabilitation. Where no Water Permit has been issued or when the situation after rehabilitation will vary from the terms and conditions on which the original permit was issued, the required steps must be taken to either obtain a Water Permit or to have the old Water Permit revised.

Water Permits are issued by WRMA to which applications must be submitted. Every applicant for a Water Permit must complete and file with WRMA the following documents in triplicate, accompanied by the prescribed fee:

- An application in the prescribed form together with plans or drawings, which will allow all requisite details to be legibly recorded;
- An application to construct the required works.

The following forms are prescribed by the Water Act for the purpose of filing the above mentioned applications:

- Form No. WRMA 001A: Application for a Water Permit;
- Form No. WRMA 001B: Surface Water covering Diversion, Abstraction, In-stream and Conveyance Works;
- Form No. WRMA 001C: Storage Dams.

4.2.3 Permit Classes

The Water Resources Management Rules 2007 provide for the following classes of water use permits:

- Class A: low risk water use;
- Class B: potential to make a significant impact on a resource;
- Class C: significant impact on water resource;
- Class D: involves two different catchment areas, is of a large scale or complexity.

An application for a water permit for a dam or storage reservoir will need to ascertain the category of dam as described in Table 4-1 from the Fourth Schedule, Water Resource Management Rules (2007).

Table 4-1: Classification of Dams

Class of Dam	Maximum Depth of Water at NWL (m)	Impoundment at NWL (m^3)	Catchment Area (km^2)
A (Low Risk)	0 – 4.99	< 100,000	< 100
B (Medium Risk)	5.00 – 14.99	100,000 to 1,000,000	100 to 1,000
C (High Risk)	> 15.00	> 1,000,000	> 1,000

Having decided on the category of dam, the applicant must determine the appropriate class of permit using Table 4-2.

Table 4-2: Relationship between Category of Dam and Class of Permit

Type of Structure	Category of Dam	Class of Permit
Pan (tank, lagoon, etc) whose capacity exceeds 10,000 m^3		A
Dam with normal water level no greater than 1.5m above ground level		A
Small Dam	Class A	B
Medium Dam	Class B	C
Large Dam	Class C	D

(Source: WRMA 2009. Guidelines for Determination of Permit Classification for Water Storage Structures)

4.2.4 Water Permit Application Process

The required forms may be obtained from the office of the nearest Regional or Sub-regional Office of WRMA. The applications must be completed and filed in triplicate together with all relevant plans and drawings as well as a copy of the design report. A fee covering the examination of the application and the issue of a permit must be paid in accordance with the quantity of water for which the application is made. The actual fee is dependant on the Category of Permit.

Upon receipt of an application, WRMA may amend or vary the application, maps or plans. The law requires stakeholder consultation before the grant of the application. This usually will involve the input of the Water Resources Users Association in cases where there is a WRUA in the area. The WRUA comments should be documented on WRMA Form 003 and submitted to WRMA. In contentious cases a full public hearing of the application may be required.

There may also be a need for an Environmental Impact Assessment, depending on whether the structure requires a full EIA study under the Environmental management and Coordination Act, 2009. This will require consultation with the NEMA office in the county.

WRMA may, after consideration finally approve, refuse or approve the application in part. If the application is finally approved, WRMA shall consequently authorize the construction of the works. In such case, a copy of the application, with maps and plans as approved shall be returned to the applicant with the authorization.

Any works authorized may be inspected, during construction, by officers of WRMA. Upon completion of the works authorized, the operator shall submit a completion certificate (Form No. WRMA 008), upon which an inspection may be made by an officer appointed by WRMA.

Upon completion of the works and in accordance with the terms of the authorization, WRMA shall issue on such terms and conditions as it may deem necessary a permit to divert, abstract, obstruct, use or store the quantity of water for which the application was finally approved.

4.2.5 Requirement for Hydrological Assessment Report/Dam Design Report

Rule 64 of the Water Resources Management Rules 2007 requires a dam design report to be submitted as part of the application documentation. The form of the design report is provided in the Second Schedule. The report shall be approved by WRMA prior to construction. The rules state that the level of detail of the report depends on the class of dam and so will vary from case to case.

In addition to the dam design report the Rules require a hydrological assessment report. The specifications of the report are provided in the Second Schedule and these should be reviewed by the professional preparing the report before embarking on the assignment.

In addition it is important that an analysis is undertaken in the context of the EIA approval of the potential environmental impacts of the dam. This analysis may also include the determination of the Reserve or environmental flow requirements.

Rule 57 states that a dam shall be designed and supervised by the appropriate category of qualified water resource professional as set out in Table 2 of the Fourth Schedule.

Rule 58 states that a dam shall be constructed by the appropriate category of contractor as set out in Table 3 of the Fourth Schedule.

Finally Rule 66 requires the filing of a dam completion report with WRMA. The form of the report is set out in the Second Schedule and a completion certificate is to be issued by WRMA.

4.2.6 Requirement for Qualified Water Resource Professional

Rule 57 of the WRM Rules 2007 specifies that a qualified water resource professional shall be used to design and supervise the construction of a dam as specified in Table 2 of Schedule 4. This applies regardless of whether the project is a state, community or private endeavour.

The registration of water resource professionals is dealt with in Rules 132 to 140 of the WRM Rules 2007. It gives the criteria to be met by individuals who wish to be registered as qualified water resources professionals, the application process, licensing and regulation of professionals, including the procedure for lodging complaints against professionals.

The MWIS maintains a register of qualified professionals and this should be inspected before engaging a professional to undertake an assignment involving a dam construction. The use of qualified professionals enhances the quality of the work and minimises the risk of dam failures.

4.2.7 Requirement for Qualified Contractor

Rule 58 of the WRM Rules 2007 also require that qualified contractors are used to construct dams. The category of contractor is set out in Table 3 of the Fourth Schedule. This applies regardless of whether the project is a state, community or private endeavour.

The registration status of a contractor should be verified before engaging the contractor. Additionally the National Construction Authority Act, 2012 requires contractors to be registered with the Authority in order to undertake construction. The register of contractors showing the category of works they are registered for is published in the Gazette. It is possible to obtain a copy of the register from the Authority but additionally the contractor should be asked for evidence of registration.

4.2.8 Requirement for Regular Inspections

During construction, a dam construction progress report is required to be submitted to WRMA at such intervals as WRMA requires, as per Rule 65 of the WRM Rules 2007. Rule 93 gives WRMA the authority to inspect the works at any time, prior to, during or after construction.

4.2.9 Fines and Penalties for Offences under WRM Rules

The Third Schedule of the WRM Rules (2007) sets out the fines and penalties for offences committed against the Rules.

4.3 Policy on Environmental Management

Dam construction is also governed by environmental policies and laws. The current environmental policy is the National Policy on Environment and Development, 1999. The key policy principle relates to sustainability in the use of natural resources. The policy was given effect by the National Environmental Management and Coordination Act, 1999.

4.4 Environmental Laws and Regulations

The National Environmental Management and Coordination Act, 1999 is the main legal instrument on environmental management. The Act establishes the National Environmental Management Authority (NEMA) as the key coordinating institution on environmental management in the country. The Act also provides that other agencies with statutory functions in the natural resources sector are lead agencies, meaning that they will take the lead in implementation of environmental policies and laws, within the context of their mandate. WRMA is therefore a lead agency under EMCA, 1999 for purposes of issuing permits and regulating dams and small water structures.

Under the Act regulations have been gazetted to give effect to the provisions of the act. The key ones are:

- The Environmental Management and Coordination (Environmental Impact Assessment and Audit) Regulations LN No 101 of 2003
- The Environmental Management and Coordination (Waste Management) Regulations LN No 121 of 2006 which deals with the handling of waste products and
- The Environmental Management and Coordination (Water Quality) Regulations LN No 120 of 2006 which stipulates water quality standards.

4.4.1 EIA Permit

Section 58 of the Environmental Management and Coordination Act, 1999 imposes a requirement for any project proponent to obtain an EIA licence from NEMA before undertaking a development project. Dams would therefore require an EIA licence. The EIA report is to be prepared by an EIA expert who is registered with NEMA. The proponent of the dam project is required to choose a NEMA registered EIA expert to prepare the report. Additionally a fee is paid to NEMA for the licence.

The EIA process should identify the potential environmental impacts of the dam and propose mitigation measures which can address these impacts. The EIA process also requires stakeholder consultation.

4.4.2 EIA Permit Application Process

The application for an EIA licence begins with the preparation of a project report which gives a brief description of the project. The report enables NEMA to determine whether the project will have significant impacts on the environment. If so then a full EIA study will be required otherwise an approval can be issued based on the project report.

Where a full EIA study is required the project proponent is required to engage a registered EIA expert to undertake the study according to Terms of Reference agreed with NEMA. The study involves gathering data to establish the baseline, carrying out public consultation and analysing the information to determine impacts. Where adverse impacts are anticipated then mitigation measures need to be proposed.

NEMA is required to send the report to lead agencies for their input. Additionally NEMA will make public the fact that a licence has been applied for and invite comments. Where the application raises objections a public hearing may be held. Where there are complex issues NEMA may appoint a technical advisory committee to advise it.

If approved NEMA will issue a provisional approval and subsequently issue a licence for the works.

The requirement for an EIA licence must be complied with in addition to the requirement for a water use permit. The issues arising in the two processes may be very similar but in the case of EIA licensing ecological and environmental sustainability issues may arise which go beyond the water use issues.

NEMA maintains offices at the county level where there is a county and sub county environmental officer. These officers should be consulted to facilitate the process of EIA licensing.

4.5 Land and Trespass Laws and Regulations

The current land laws are found in the Land Act, 2012 and the Registration of Land Act 2012. Under the Land Act land is either public land, private land or community land. Private landowners have evidence of ownership, which is either a certificate of title or a certificate of lease. A formal search from the land registry to confirm ownership is important.

For a dam project access to and use of land is critical. Therefore an agreement ought to be reached with the landowner regarding the use of land. The landowner may sell the land to the project, in which case a transfer is registered in the name of the dam owner. Even if a transfer is not agreed to some formal documentation recording the landowner’s agreement to the use of the land is essential.

In several cases the landowner is the government, the county government or the community who have agreed to allow the land to be used for the dam. In these cases a transfer of the title to the dam owner is not likely but some letter of authority to use the land should be obtained.

Securing land access and rights of investigation and development may require arranging for a wayleave (or easement). Where access is required over another person’s land then a wayleave will be necessary. This is formal permission to cross over someone’s land, using pipes of some other infrastructure. Wayleaves are often paid for and recorded on the title so that they bind the owner. Obtaining wayleaves can at times be problematic. The Water Act 2002 provides a procedure for seeking WRMA’s intervention in cases where the landowner proves unreasonably difficult to grant a wayleave.

4.6 Liability and Indemnity

Where there is damage to another’s property, say from failure of a dam, the dam owner may face a claim and be liable to pay compensation. This may require that the dam owner take out insurance. Additionally the dam owner may require the professionals and the contractor involved in the project to provide an indemnity in the event that the damage has arisen from professional negligence in the design or construction or similar failing. For this purpose the professionals will be required to take out professional indemnity insurance cover. The contractor will also take out an appropriate insurance cover.

4.7 Procurement Regulations

If public money is being used to carry out the project it is necessary to follow public procurement regulations. These are found in the Public Procurement and Disposal Act, 2006. These require that the public agency undertaking the project use competitive tendering to secure the service providers. There are limited cases in which competition is not required but it is important to remain complaint as the contract award can be nullified if the law is not complied with.

If the project is supported by donor funds, the project proponent should check if public or donor procurement laws apply. At times donor procurement laws can be quite stringent and it is important to familiarise oneself with them if the funds are being provided by a donor.

If the project is a private endeavour, using private funds, the owner can decide how she/he procures an engineer and contractor but he/she must still contract engineers and contractors that fulfil the legal requirements.

4.8 Other Relevant Laws and Regulations

Labour laws will govern the labour practices that the contractor must comply with during construction. Therefore the contractor must be familiar with the requirements of the labour laws and occupational safety and health laws. These can lead to criminal and civil liability if they are not complied with and delay the project.

CHAPTER 5

STAKEHOLDER ENGAGEMENT AND COMMUNITY PARTICIPATION

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5 STAKEHOLDER ENGAGEMENT AND COMMUNITY PARTICIPATION

5.1 Introduction

Successful development of safe, economically and environmentally appropriate small dams, pans and other water conservation structures, like any other development project, demands the active participation of stakeholders to ensure proper coordination, planning, smooth implementation, and sustainable benefits. The importance of actively developing and sustaining relationships with affected communities and other stakeholders throughout the life of such projects has proved beneficial in risk management and has delivered better project outcomes.

This chapter details the basis, requirements, importance and process of stakeholder engagement that can be applied in developing cooperation with stakeholders during the design, construction, operation and rehabilitation of water conservation structures. It is however important to ensure that such community participation and engagement takes cognizance of how participation is captured in the diverse communities in Kenya. In some cultures in the ASAL regions of Kenya, for example the *Deda* system for range management and the *Abba Herega* at a point source, have legitimacy at community level but often lack official recognition and hence cannot enforce their rules.

Additionally, it is important to note that since the 1990's great strides have been made in the area of public participation. This has culminated in the development of legislation, policies, strategies and tools for stakeholder engagement that were either previously not in place or not well developed. There now exist diverse strategies and tools for stakeholder engagement and thus the description of a strategy or tool in this manual should not in any way limit the users of the manual from using or applying other available, effective strategies and tools; the idea is to ensure that the key steps of stakeholder identification, analysis, engagement and monitoring are done.

Reference should be made to other documents[1] that provide more detail on building capacity within communities to manage their water supplies which include the local dams, pans and other water conservation structures.

This chapter is primarily orientated towards public projects in which the local community are the beneficiaries of the structure being developed. However, the process of stakeholder analysis and engagement is equally applicable to private projects even though the local community may not be the direct beneficiaries of the proposed water conservation structure.

Additional reference can be made to the WSTF documents associated with the Community Project Cycle (CPC), the WRUA Development Cycle (WDC) and the UNICEF, FAO and Oxfam GB (2012) document *A Trainers Manual for Community Based Water Supply Management in Kenya* which provide tools for stakeholder engagement.

5.1.1 Objectives of Stakeholder Engagement

The principle objectives of stakeholder engagement are to:

- Ensure effective co-ordination of the project with different sectors and players;
- Introduce a range of ideas, experiences and expertise from stakeholders which motivates development of lasting and improved water conservation structures that incorporate all stakeholder interests;
- Build consensus between the projects and community members to mitigate potential risk of conflicts which can be detrimental to the success of the water conservation project;

[1] e.g. UNICEF, FAO and Oxfam GB 2012. A Trainer's Manual for Community Managed Water Supplies in Kenya. UNICEF-Kenya County office, FAO & Oxfam GB, Nairobi, Kenya.

- Utilize existing opportunities and relationships to enhance project outcomes;
- Foster local pride and ownership among project stakeholders.

5.1.2 Classification of Stakeholders

A stakeholder is any person, group or organization who can be positively or negatively impacted by or cause a positive or negative impact on a proposed water conservation project. There are two key classifications of stakeholders as follow:

Primary Stakeholders:- These are persons, groups or organizations that are directly affected by the project either as beneficiaries (positively impacted) or de-beneficiaries (negatively impacted), sometime referred to as a Project Affected Person (PAP). This could for example include various water users of the proposed structure.

Secondary Stakeholders:- Persons, groups or organizations that are not directly affected by the project but have an intermediary role in the project and may thus have an effect on the project outcome. This could include stakeholders such as government ministries and departments, regulatory bodies, development organization, among others who have a stake in the proposed water conservation structure.

Persons, groups or individuals from either group who can significantly influence or are important to the success of the project are further referred to as **Key Stakeholders**. This group may include local opinion leaders such as local political leaders (e.g. MCAs) and ward administrators.

5.2 Legislative Basis for Community and Stakeholder Participation

Stakeholder engagement is not only a positive strategy for enhancing water conservation projects success but also a legal and ethical requirement. The following is a brief description of some of the existing laws and policies that demand stakeholder engagement.

1. **The Constitution of Kenya 2010** has captured public participation of its citizens under national values and principles of governance (Chapter 2). The Bill of Rights (Chapter 4) further provides for among others the right to freedom of expression (Article 33) and the right to access to information, and the protection of all right and fundamental freedoms.

   The Constitution of Kenya 2010 also introduced the county government structure necessitating changes in engagement with a new set of stakeholders. The county ministries or departments for the time being charged with the responsibility of water supplies and land at county level will need to be fully involved in the siting, designing, planning and bestowing of community ownership of the water conservation facilities within the context of the county laws and in line with the national laws.

   Participation of the community in the processes of planning and design will then be ensured through the legal and social framework overseen by these departments. The rights and responsibilities of the community must be discussed and agreed in a structured way and formal registration of a community based organisation (or dam management committee) ensured within the prevailing legal framework.

   In light of the dictates of the Constitution of Kenya 2010 governance of WRUAs and other water associations has also changed. While previously a group of community members could register themselves as owners of a community asset such as a dam and take charge, the new dispensation now requires that such an arrangement is officially sanctioned by the county government department in charge of water supplies. The mandate will be a delegated responsibility by the county government especially if the location of the dam or other water conservation structure is on public or community owned land and if the group is formed to manage the dam on behalf of the wider community. Each county will have its own process of registering and recognizing such a group and how to monitor its management of the dam as a public asset.

2. **Water Act 2002** makes provisions for the formulation, through public consultation, of a catchment management strategy for the use, development, conservation, protection and control of water resources within a catchment area (Section 15). Section 16 further provides for the constitution of a catchment area advisory committee (CAAC) with membership of diverse stakeholders within the catchment areas. At a local level, the WRUA membership should reflect the key stakeholders of water users within a sub-catchment and should therefore be considered during stakeholder analysis and consultation for the project.

3. **The Environmental Management and Coordination Act (EMCA)** imposes the need for an Environmental and Social Impact Assessment (ESIA) for all public and private water storage infrastructure projects. The ESIA is dealt with in more detail in Chapter 6. However, in certain projects a public or non-government body has the role of project owner, project manager, and/or project beneficiary and must fulfil the requirements of the Environment Management and Coordination Act (EMCA).

4. **The Community Land Bill, 2013** deals with structures erected on community land. The small dams, pans and other water conservation structures dealt with in this manual will be located on land that is either communally owned (held in trust by county government) or privately owned. Article 14(3) of the **Community Land Bill, 2013** states that, until any parcel of community land has been registered in accordance with this Act (Community Land Act – yet to be enacted), such land shall remain unregistered community land and shall be **held in trust by the county government** on behalf of communities pursuant to Article 63(3) of the Constitution. However, once an unregistered community land is registered in accordance with this Act, the trusteeship role of the county government shall lapse and the community group registered in relation to such land shall, through its relevant committee, assume the management and administrative functions provided in this Act. The Bill provides a mechanism for the management of community land. Article 18(1) states that every community shall through election by the community assembly, establish a Community Land Management Committee. On land use planning, Article 36 (1) of the draft Bill states that a Committee may, on its own motion or at the request of the county government, submit to the government for approval a plan for the development, management and use of the community land vested in the management of the Committee.

   In light of the above, where the small dams, pans and other water conservation structures are to be constructed on community or public land, due process should be followed as contained in the laws above to ensure that the ownership is secured for the said asset before construction begins. If the dam – a public asset - is to stand on privately owned land, the community should insist on the owner of the land signing a lease with the community before construction works begin.

5. **The Dublin Statement on Water and Sustainable Development** are also known as the Dublin Principles. This statement recognizes the increasing scarcity of water as a result of conflicting water uses and the overuse of water. The statement sets out the following four guiding principles as recommendations for action at local, national and international levels to reduce water scarcity:

   i. Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment;
   ii. Water development and management should be based on a participatory approach, involving users, planners and policy makers at all levels;

iii. Women play a critical role in the provision, management and safeguarding of water;
iv. Water has an economic value in all its competing uses and should be recognized as an economic good.

5.3 Key Concepts in Community Participation

5.3.1 Sustainability

In order to achieve sustainable projects (in the case of small earth dams, pans and other water conservation structures, the structure should have a useful lifetime of 20 to 25 years) the beneficiaries should be actively involved in the planning, construction and particularly operation and maintenance of the facility. Sustainability for a small earth dam or pan is being achieved when:

1) The water sources are not over-exploited but are naturally replenished;
2) Water systems are maintained in a condition which ensures a reliable and adequate water supply;
3) The benefits of the supply continue to be realized by all users indefinitely;
4) The service delivery process demonstrates a cost-effective use of resources that can be replicated;
5) The water supply system is maintained in a condition which is able to provide water services to meet the needs of the growing population and increasing water demand without external support.

5.3.2 Community Empowerment

Community members must develop and maintain structures for holding their leadership accountable as part of self-governance. But this also comes with community members' understanding of their own responsibilities for the management of the asset. Some possible structures of accountability are included in Table 5-1:

Table 5-1: Possible Accountability Structures

Issue	Management Indicators	Community Action/checks
Financial Management		
Accountability	Proper book keeping: issue receipts against payment for water, invoices for all payments made, stock book etc	Establish this system from onset and develop system of auditing by members (users)
Appropriation of funds	Budgets and proposals to donors and potential benefactors	Discuss and agree actions Review income against expenditure
Water charging	Up to date records	Ensure the committee maintains paperwork and make it available for inspection
Leadership		
Elections	Registered Community Dam Constitution Fair election procedures defined within constitution and followed	Insist on term limit for office bearers and democratize and ensure regular election of leaders
Communication	Minutes of meetings shared	Insist that decisions taken by committee are minuted and disseminated to users by posting on a public place. Annual general meeting where community can question the leadership decisions taken on their behalf
Equity in leadership	Gender & stakeholder balance in the committees	Insist on one-third gender rule in leadership as the very minimum
Lack of legal redress in dealing with corruption	Bylaws high lighting action against misuse of office	Insist that WUA registers as Society under the Societies Act and initiates process to become a WSP.
Poor service levels/user dissatisfaction		
Lack of equitable access to water	Byelaws provisions in regard to equity	Ensure that byelaws are appropriate, have been agreed by the whole community and are followed
Larger livestock owners not paying in proportion to the amount of water they use or abusing the facilities	Byelaws provisions in regard to equity; Develop specific facilities (troughs) for livestock and enforce their use	Public auditing of accounts and comparing revenue against production by use of water meters as a means of quantifying unaccounted for water.
Conflict between users	Byelaws provisions in regard to conflict resolution	Mechanisms for conflict resolution should be articulated within byelaws.

5.3.3 Inclusivity and Gender Balance

Inclusivity is another essential aspect of the new constitutional dispensation. While prior to 2010, it would be acceptable to put together a leadership team of any community or national institution without due regard to gender balance, the constitution now puts a threshold for participation of all gender in leadership. Especially in rural and ASAL communities, women are traditionally the most knowledgeable regarding domestic water needs as well as the main family providers of domestic water and yet are most often overlooked when leadership is being constructed. Where tradition works against women engaging in leadership with men, it is important to interrogate the traditions and find a way to ensure the constitutional minimum threshold of two-third gender rule ensuring that marginalized groups do not only belong to the committees but are elected to positions of authority within the committees.

5.4 Stakeholder Analysis

Effective engagement of beneficiaries in any project starts with a thorough analysis of project stakeholders. Stakeholder activities can either enhance or undermine the operations of a project while the project on the other hand can also impact the stakeholders positively or negatively. It is therefore important that before the project starts, its stakeholders are properly identified and analyzed, and their level of involvement or how the project will affect them or their operations is described and ways of engaging with them developed. Stakeholder analysis involves three key steps:

1. Identification of the key stakeholders from the large array of groups and individuals that could potentially affect or be affected by the proposed intervention.

   This process can be initiated by indiscriminately listing all stakeholders likely to be affected or affect the proposed project. Prioritization of the stakeholders listed can then be done through consultation with experts within and outside the community to come up with the list of key stakeholders.

2. Assessment of stakeholder interests and the potential impact of the project on these interests as well as the influence of the identified stakeholder on the project. Influence in this case refers to the power that a stakeholder has over a project and thus the significance of their involvement in the project.

   Once the key stakeholders have been identified, the possible interest and influence that these groups or individuals may have in the project will then be considered and assessed. Stakeholders are typically categorized into one of four groups as follows:

   - **Stakeholders of high influence and high interest:** They need to be closely engaged throughout the preparation and implementation of the water conservation structure project. Collaborative and empowering approaches should be thus adopted with this group. Caution also ought to be taken of stakeholders in this group who have strong opinions.
   - **Stakeholders of high influence but low interest:** They are not the target of the project but their influence could be used to oppose the project. They therefore need to kept informed and their views on the project taken into consideration through consultation where this is necessary.
   - **Stakeholders of low influence and high interest:** They require special efforts to ensure that their needs are met and that their participation is meaningful through consultation and involvement. Particular attention need to be paid to marginalized groups whose low influence may stem from poor opportunities.
   - **Stakeholders of low influence and low interest:** They are unlikely to be closely involved in the project and require basic level of participation which can be achieved through information.

3. Outlining a stakeholder participation and engagement and communication strategy for the different stages of the project based on the analysis of the stakeholders.

   The assessment of the stakeholder's interest and influence serves to inform the participation method and technique to be applied in the engagement of each of the stakeholders based on where they fall in the assessment.

There are five common approaches of public participation; (1) informing, (2) consulting, (3) involving, (4) collaborating and (5) empowering with each requiring application of unique techniques such websites, focus groups, workshops, committees, and ballots respectively. Stakeholder analysis thus informs the intensity of the engagement required for each group of stakeholders as well as the approach of participation.

Table 5-2 provides a sample of analysis that would be necessary for stakeholder groups of a small community dam, pans, rock catchments, sand dams, and sub-surface dams. Implementers must analyze and fill in all the columns provided in the table to ensure that the interests of any group do not compromise those of another group or the smooth operations and implementation of the project. Ideally, community participation should include meetings with all the stakeholder groups either jointly or separately to understand their concerns and include these concerns in the planning, development and maintenance of the project.

Table 5-2: Stakeholder Analysis

Who are the Stakeholders	How project will affect them	How they will affect the implementation or operation of project	Ways of engaging them
Community Members			
Domestic water users			
Livestock keepers			
Irrigators			
Commercial water vendors			
Commercial water users			
Youth			
National Government Departments			
County Government department			
NGOs			
Faith Based Organizations			
Local politicians			
Local Institutions			
Schools			
Hospitals			
User Groups – WRUAs, WUAs			
Financiers			
Engineers			
Contractors			

5.4.1 Methods and Tools for Stakeholder Analysis

There are various tools for undertaking stakeholder analysis in a community meeting or group. The method helps the community to establish answers to the second and third columns of Table 5-2.

a) Participatory methods

Participatory methods include focus group discussions, workshops, surveys, and polls among others which are used for discussions, scoring, ranking, voting and agreeing on the level of effect each stakeholder is likely to have on the project – positive or negative - and agreeing on whether the particular stakeholder has influence and interest in the project and if they need to be engaged or not.

b) Graphical Stakeholder Analysis Techniques

The Venn diagram is one example of the visual way of undertaking stakeholder analysis. The community (or representatives) sit in a group and agree on a list of stakeholders. The project name is written on a piece of paper and enclosed in a circle. Each stakeholder is represented by its own circle in which the size of the circle represents the potential influence of the stakeholder on the project and the distance from the project circle represents the "closeness" of the relationship. Through discussions the group then decides how to engage with the stakeholders. The group can strategize on how the highly influential stakeholders that are "far" from the project can be moved "closer" to the project. A sample Venn diagram is illustrated in Figure 5-1.

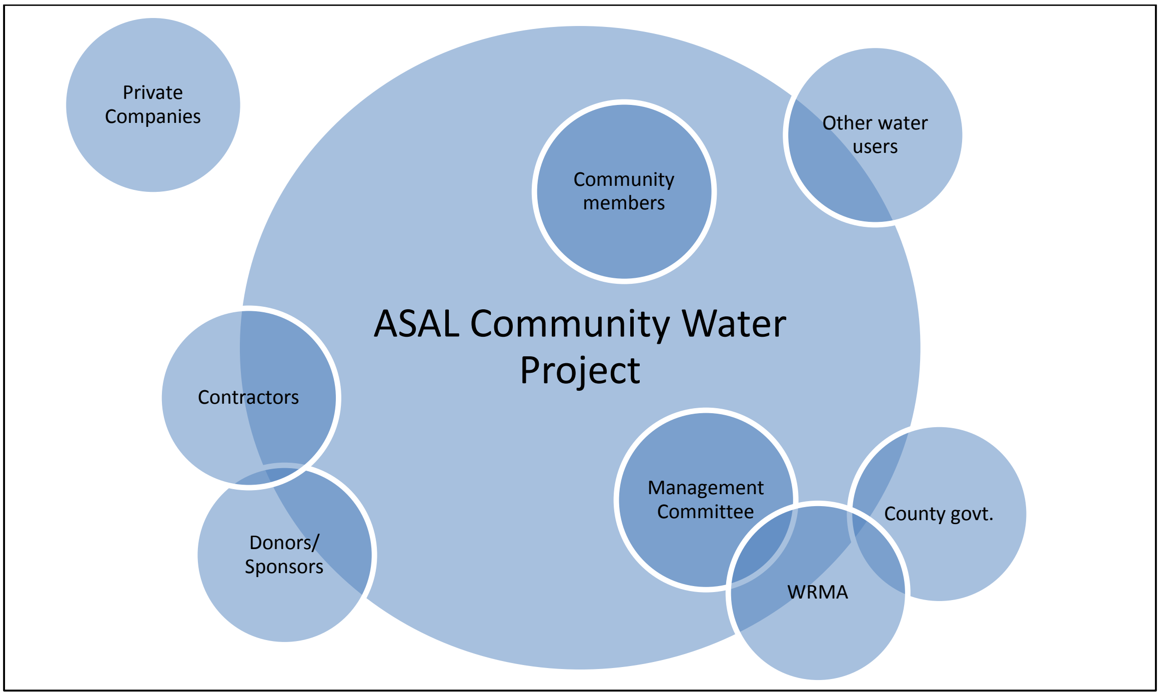

Figure 5-1: A Simple Venn Diagram Illustrating Stakeholder Relationships

5.4.2 Roles and Responsibilities

It is now an accepted practice of development that responsibility through participation enhances ownership and ensures sustainability of projects. Each of the stakeholders identified in the processes above should be assigned roles and responsibilities in the project either individually or through representation. This will ensure they are engaged positively and are therefore unlikely to do anything to undermine the project.

5.4.3 Stakeholder Participation Processes

a) **Initiation of engagement of identified stakeholders:** Following the processes of identifying stakeholders and listing what their interest in the project might be, the project implementers should approach each group by identifying their leadership structures and engaging them. This might take time and is sometimes a question of trial and error as you identify the real people in the group that hold the decision-making powers.

b) **Stakeholder group meetings:** Once these groups are identified and approached, each group should be encouraged to meet and discuss the potential benefits and challenges they foresee in the proposed project. The groups should then choose representatives to carry their case at the all-inclusive stakeholders meeting where their concerns will be heard and addressed.

c) **County stakeholder meetings:** Efforts are required to ensure that relevant county government departments are involved in the coordination and planning of the project to avoid duplication of projects and/or efforts and to ensure the project fits in with the broader development agenda for the area. Counties may have established sector based coordination forum (e.g. County WASH forum).

d) **Local level stakeholder meetings:** Local level water and natural resource management stakeholder groups (e.g. WRUAs, CFAs, rangeland management associations, etc), including traditional rangeland and water management forums, should be identified and involved in project planning and coordination.

e) **Community local leadership:** Communities are often a special group of stakeholders in such an enterprise due to the fact that either by design or default they are often less organized or resourced than other project stakeholders. Special care should thus be taken to ensure their inclusion in the processes without compromising their integrity. Just like the other stakeholders, communities are also not homogenous; it is important for the project leadership to understand the different pockets of leadership within the community. One way is to start with the establishment of formal contacts with the local leadership e.g. MCAs, head teachers, ward administrators, chiefs, religious leaders, etc., as most communities will only participate fully when they know that their leaders are informed of what is going on.

   The leaders should be briefly informed about the plans for the project, what it aims to achieve and what limitations and constraints might exist. They should then be encouraged to convene meetings with their members to discuss the issues that are likely to emerge as the project is rolled out, the potential benefits, possible conflicts and likely areas of participation of the community members. The leaders should then help the community to elect representatives to carry their case in the subsequent meetings. Often communities have a wealth of knowledge about their areas and what can or cannot work but they must be facilitated to share this knowledge and wisdom with project planners.

f) **Agenda of discussions with community members:** Communities should be organized and facilitated to discuss the matters listed below and minutes of the meetings and agreements reached should be recorded and filed with the relevant County departments for reference.

   1. Clarify the role of the community in the project;
   2. Agree on roles of the community during implementation and operation and maintenance;
   3. Define geographical boundaries of the "community" that the structure will serve;
   4. Establish that the construction (or rehabilitation) of the small dam, pan or other water conservation structure is a felt need in the community;
   5. Establish how the construction fits with the wider plans of development in the community;
   6. Discuss in detail the role of the community, agree on aspects of ownership, contributions (if any), tasks expected, responsibilities of all parties etc.;
   7. Counter check population, land and ownership data etc.;
   8. Establish how other development projects in the area have been implemented;
   9. The leadership team should be tasked to develop a draft constitution, MOU or rules of engagement stating the roles and responsibilities of the different players in the project;
   10. Discuss the design of the structure and agree the various technical aspects such as fencing of the structure and reservoir area and the construction of proper draw-off facilities (cattle trough, communal water point, distribution networks, intakes etc.);
   11. Discuss in detail the operation and maintenance aspects of the project so as to enhance ownership;
   12. Introduce the work plan to be followed during the implementation phase. Outline who does what, when and how;

13. Education on the need for proper fencing and draw-off facilities;
14. Further expressing of views concerning the project, and answering the beneficiaries' questions;
15. Establish a community project leadership team such as a "Dam Committee" (if the community is ready, and no committee exists);
16. Set a plan of activities, in terms of contributions (construction materials) and labour required, including a detailed timetable. This should be a joint exercise between the Dam Committee and implementing organisation or administration.

5.4.4 Constitution and Registration of Groups

After the initial community and stakeholders meetings, decisions will be reached on the nature of the structure of the organization necessary to carry forward the work of the project. Some structures lend themselves better to the operations of a small community water conservation project than others. Some possibilities include – Associations, Cooperatives, Societies, Self-help groups/community based organisations, Limited Liability Companies etc. Most communities prefer to work with the Association structure as this suits a membership group. Most important is the need for registration as **a legal entity**, and thus the community should be advised on the alternatives available for registration as such.

Once the community has settled on a form of registration, the next step is to develop a governance document or constitution to guide its operations. Templates exist from which members can draw. However, the group can develop its own constitution from scratch. A good constitution should state:

a. Name of the organisation;
b. Type of organisation;
c. Objects of the organisation;
d. Membership criteria;
e. Governance structures complete with offices and tenure (term limits important);
f. Case for dissolution and what to do when that happens;
g. Sources and uses of organization funds;
h. Organisational structure.

In addition to the constitution, the group will also need to develop site specific by-laws to govern their internal day to day operations.

5.4.5 Group Establishment and Capacity Development

Like children, no organization is born performing. Every organization must go through various stages of formation before they can perform at peak. Even groups composed of highly technical members will need training on the ways of working within the new organization. This may include developing terms of reference, instituting processes for meetings and resolution of conflicts and sharing of opposing views. In most cases this learning curve will require training. Some important training topics for new leaders of a community water structure include:

a) Time and time management;
b) Methods and processes;
c) CBO governance skills (leadership, group dynamics and conflict management, accountability and transparency, compliance to constitutional provisions, planning, budgeting, etc.);
d) Financial management skills;
e) Technical operation and maintenance skills;
f) Resource mobilisation.

5.5 Community Participation Activities

5.5.1 Community Contributions

Community contribution will include but is not limited to time, land, cash, labour, and materials. Supporting agencies often choose different ways to either acknowledge or compensate these contributions by the community and other stakeholders. Some methods that have been used include compensation of the community time and labour through modes such as cash for work, food for work or food for assets. In cases where land belonging to the community is alienated, some agencies recognize this by erecting boards that note that the asset was developed through the contribution of land by a particular community and financial support of a particular agency. Some communities contribute to the project by providing labour for excavation or cash in lieu of labour. Whatever the choice of contribution, the terms should be clearly agreed and documented. Where legal transfers of any assets such as land on which the water conservation structure or access road will stand are involved, the transfers should be legally done [See Chapter 4 for details].

It is important that these issues are clearly brought out and discussed at the initial stakeholders meetings and modes of compensation or recognition agreed and documented before the project starts. Failure often leads to unnecessary conflicts during implementation and in the current dispensation could lead to unnecessary court cases that could delay the project for a long time.

5.5.2 Rehabilitation of Water Conservation Structures

Whenever rehabilitation (particularly where de-silting of water conservation structures is considered) the reasons why the need for rehabilitation arises should be thoroughly examined and discussed with the beneficiaries. This exercise should take place during the first stages of the planning phase. In case the useful lifetime of the reservoir has been less than should reasonably be expected (20 to 25 years) the issue of sustainability after the rehabilitation should be raised in this phase of the dialogue with the community. The rehabilitation should consequently only be carried out if the required conditions to ensure sustainability will be met (e.g. if erosion control measures are introduced in the catchment area). The economics of desilting, compared to other options for storage development, should also be considered.

5.5.3 Assessment of Potential for Community Participation

When selecting sites for the construction (or rehabilitation) of small dams, pans, or other water conservation structures a survey of the existing potential for active involvement of the concerned community in the proposed project should be undertaken. It should be ascertained that a certain implementation capacity exists (within the community) before proceeding with the project. It will generally be difficult to meet objectives and to obtain sustainable results if the project has to be started without such capacity on which to build.

Dialogue with communities regarding possible construction or rehabilitation of small earth dams, pans and other water conservation structures should be carried out by people with field experience in community dialogue situations who may include Community Development Assistants (Ministry of Culture and Social Services), water officers stationed in the field, and Soil and Water Conservation Extension Workers (Ministry of Agriculture).

Areas with the potential for community participation should be identified, quantified and costed during the preliminary stages of project development. Examples of such activities include grass planting, rip-rap placing, construction of check dams and live fencing etc.

5.5.4 Communicating with the Community

One of the basic factors that will guarantee maximum community participation is how information regarding the project is communicated to and from the community. Information to the community should be simple, to the point and as complete as possible. When communities are told about issues they should be invited to respond to those issues and expose how they affect them. While the construction or rehabilitation of the water conservation structure is on-going, certain issues covered in the meetings may be revisited, in order to build up the ownership sense in the community and to establish a base for the future operation and maintenance of the structure. Communication can be structured so the community receives updates on the construction process either by telephone (text messages) or regular updates through radio or further meetings. At the end of a phase of the project – say construction, it is important for the community to have a meeting and review progress and make certain decisions regarding the project.

It is important that the project implementation team allows (requires) the community representatives time to consult the community formally on matters regarding the project. What tends to happen with most communities is once they elect representatives their participation ends and the representatives then take over the decision-making on all matters of the project on their behalf. Such a situation is not only undesirable but often also leads to conflicts later. The community representatives are NOT the community.

5.5.5 The Role of Communities in Construction, Rehabilitation and Maintenance

- **Project Committee**

The selected Project Committee should ideally consist of 9-10 members, representing all ethnic groups in the community. At least not more than 2/3 of the members should be of one gender. This will ensure compliance with the constitutional threshold as well as enrich the leadership. Special groups within the community should also be considered for representation. The committee should elect its office bearers (Chairperson, Secretary, Treasurer) and dearly outline their respective duties. It is important that the elected office bearers be literate, so proper records of the group’s activities can be kept. The office bearers should be joint signatories to any bank accounts, while the treasurer shall be responsible for receiving and disbursing all moneys belonging to the group (under the directions of the group), as well as for the keeping of proper books of accounts.

- **Implementation Phase**

During the construction phase of the project (construction or rehabilitation) the community could be responsible for the following:

- Clearing of water conservation structure, spillway and reservoir area of bushes and trees;
- Fencing of water conservation structure and reservoir area: making or purchase of fencing posts as well as installation of the fence;
- Completion of the draw-off system downstream from the valve chamber: provision of building materials and construction of a cattle trough and a communal water point.

It is essential that cattle are not allowed in the reservoir (water conservation structure), wherever possible. In cases where the community is unwilling to fence the reservoir and to provide cattle troughs and\or communal water points, construction or rehabilitation of the facility should be carefully considered. However, the need for off-site watering facilities such as cattle troughs should be strongly emphasised, to reduce contamination of the reservoir by livestock.

At the end of the construction or rehabilitation of the dam, pan or any other water conservation structure, a small ceremony should be organised to mark the transition from construction to operation and maintenance and specifically to reinforce the community ownership of the facility and responsibility for operation and maintenance tasks.

- **System Components**

As part of developing an operation and maintenance schedule, it is important to consider each component of the project, its function and maintenance requirements. Potential system components specific to a dam project are listed in Table 5-3 below. The purpose of each component is explained:

Table 5-3: System Components

Item	Purpose
Catchment Area	Area above the source where rain falls and the runoff comes from
Source	Where water is taken from, e.g. river or stream
Inlet channel	A channel that conveys water from the source and puts it into the dam or pan
Pan Embankment	Wall of excavated material
Dam Embankment	Wall that is built and compacted to hold the water
Storage area	The volume that is filled with water
Spillway sill	Wall in the spillway to control top water level
Spillway channel	Channel to safely discharge excess water to water course or away from the dam/pan
Outlet/draw-off	Pipe-work to take water out of the dam
Perimeter fence	Constructed to prevent livestock, wild animals and children from entering the dam/pan area and contaminating the water

The most common problems with small pans, dams and most of the other water conservation structures are (1) silting up which reduces the stored volume and therefore the reliability or the period of time that there is water in the dam or pan after the end of the rains and (2) blocking of the spillway with vegetation thereby reducing the capacity of the spillway to discharge floods safely and (3) erosion of the spillway which can reduce the storage capacity of the structure.

- **Operation and Maintenance**

Past experiences indicate that operation and maintenance is probably the most problematic aspect of small dams, pans and other water conservation structures. In this respect it is recommended that cash contributed by the beneficiary community should be converted to materials required for operation and maintenance or deposited into the Group's bank account. This will help avoid temptation to misuse contributed cash resources.

Operation and maintenance of small dams, pans and other water conservation structures is simple and inexpensive, but nevertheless essential in terms of project sustainability, since unattended minor issues can easily develop into major problems which can ultimately reduce the useful life-time of the structure.

It is the responsibility of the project committee to arrange for regular inspections and basic repairs and maintenance works. During the implementation phase of the project (construction or rehabilitation), a "Water Conservation Structure operator", who will be responsible for the operation and maintenance matters should be appointed by the project committee. Remuneration and terms of service regarding this operator should be worked out by the community. It is recommended that the functions of management (Project Committee) be separated from those of operation and maintenance.

During the implementation phase of the project, the operator and selected members of the project committee should receive basic training in operation and maintenance of the structure. Specific issues on which this training should concentrate and guidelines for maintenance of surface water structures are outlined in each chapter of the structures under consideration.

At the conclusion of construction, a Project Completion Report should be presented to the Project Committee. The purpose of this report is to assist the community in correctly operating and maintaining their facility. It is also preferable that the county government department in charge of water affairs is fully briefed on the completion of the project and is then able to carry out follow-up work and provide assistance to the community after completion of the project. The community should be made aware of who to contact in the county government.

- **Plan of Activities**

A clear Plan of Activities for the implementation and operation and maintenance phases should be established by the implementing organisation and the project committee. The Plan of Activities should specify the following items:

a) Labour requirements (to be provided by the beneficiaries) with timetable, details of working times and days.
b) Types and quantities of construction materials required with detailed timetable. The Plan of Activities should clearly specify who will be responsible for the provision of which type of materials and when.
c) The responsibilities for operation and maintenance: technical specifications, as well as who will be responsible for operation and maintenance.

Ideally the Plan of Activities should be the object of a formalised (written) agreement between the implementing organisation and the community.

5.6 Anticipating Some Common Problems

Like any other operation, community participation will often face challenges but with good planning and adequate trouble-shooting, these should be anticipated and dealt with before they reach a crisis level. Some of the common problems and possible mitigation measures include:

- Poor attendance at project meetings (small numbers; not all groups represented; few women).
    1. Improve notice and awareness of proposed meetings;
    2. Schedule meetings at suitable times and in convenient locations;
    3. Provide more opportunity for feedback;
    4. Arrange to meet unrepresented groups separately;
    5. Arrange for a speaker from another community that successfully completed a similar project.

- Resistance to the choice of the water conservation structure site.
    1. Discuss criteria for site selection with the community before finalizing the choice;
    2. Give proper compensation for acquired land;
    3. Deal with right-of-way problems.

- Difficulties with volunteer labour and default in payment (often arising from a poor experience with a previous project).
    1. Organize to pay a stipend to the volunteers;
    2. Agree tariffs with all the members and devise a way to generate and distribute bills and collect revenue;
    3. Arrange labour requirements taking into account other community work, cultural events, migration patterns etc.;

4. Allow choice of labour or cash contribution;
5. Arrange timetable according to the wishes of the community;
6. It is not recommended to mix different labour types (paid labour, self-help labour, food-for-work).

- Misuse and depreciation of the structure.
    1. Make users directly responsible for supervising and cleaning cattle troughs etc.;
    2. Discourage overgrazing in the vicinity of the structure;
    3. Improve hygiene at water points;
    4. Reduce the presence of pools of water that might act as disease breeding points;
    5. Maintain fences in good condition;
    6. Consider ways of reducing vandalism (cutting of fences etc.).

- User unwillingness to contribute construction materials, cash etc.
    1. Make time of contribution convenient e.g. instalments, after harvest etc;
    2. Modify the basis of contribution e.g. based on consumption, ability to pay, distance from the dam etc.;
    3. Give discounts for prompt contributions;
    4. Let communities decide on sanctions and incentives.

- Lack of unity in community decisions.
    1. Talk to different factions separately;
    2. Allow sufficient time for the resolution of differences;
    3. Give clear indication of consequences of delayed decisions.

- Multiple projects within the area of interest.
    1. Work with the local county government office to identify ongoing projects;
    2. Conduct proper assessment of the entire area.

CHAPTER 6

ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

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6 ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT

6.1 Introduction

The Environmental and Social Impact Assessment (ESIA) is a planning process that has become common practice for water storage infrastructure projects. In addition to an ESIA being a legal requirement, past experience has proven that proper public consultation and analysis of potential environmental and social impacts improves the likelihood of sustainable benefits from the project to a wider body of stakeholders and reduces the likelihood of negative impacts of the project on the social and bio-physical environment.

The construction of small dams, pans and other small scale water storage structures usually pose a smaller risk of adverse social and environmental impacts than the construction of large-scale dams and reservoirs. The ESIA process, scaled suitably to the scale and nature of the project in question, however remains an important component of project development particularly where fragile environments and vulnerable people are involved. For example in the case of arid and semi-arid areas, which are characterized by relatively limited surface water resources, care should be taken to ensure that the construction or rehabilitation of a small dam or pan does not upset delicate balances between quantity of water available, existing water uses and sustainable rangeland utilisation.

It is therefore important to note that the exploitation of (surface) water resources by the creation of an artificial reservoir, however limited in size, still constitutes an intervention in the hydrological cycle. It is therefore important that the social and environmental impacts be taken into account and addressed accordingly during the planning process.

This chapter provides an outline of the ESIA methodology as it applies to dams, pans and other water conservation structures. Readers should refer to other documents for a more comprehensive discussion on the ESIA process. The reader is also advised to review the material in Chapter 4 on Legal Compliance which presents the legal aspects of the ESIA process.

6.2 What is an ESIA?

Design and construction of water storage structures is primarily informed by the opportunities for benefits associated with such a structure. But like any other infrastructure project, water storage structure projects also have the potential to trigger negative social and bio-physical impacts.

The Environmental and Social Impact Assessment (ESIA) is thus a project planning process that identifies, predicts and assesses the type and scale of potential social and bio-physical impacts and opportunities for benefits associated with the proposed water storage project.

The ESIA documents the baseline condition and how this is likely to change during construction, operation and decommissioning of a project. It explores alternatives and provides an environmental monitoring and mitigation plan. The process is multi-disciplinary in nature and requires disclosure and consultation with stakeholders.

6.3 Importance of the ESIA Process

The ESIA is primarily a planning rather than a regulatory process although this dimension of the ESIA process is often overlooked. Many project proponents enter the ESIA process with trepidation because the outcome of the disclosure and consultation process is uncertain. The process of exposing the project to the public and stakeholders can invite questions, criticisms and alternatives that can change, derail or delay the project implementation. However, a well designed and implemented ESIA process, in which the stakeholders are provided with adequate information and time to understand the benefits, risks and environmental monitoring and management plan associated with the proposed project, will enhance the project and minimise the risks of future conflict with stakeholders and/or

NEMA. Essentially, the ESIA process, coming prior to project implementation, can reassure the investor that the project has been adequately scrutinised by stakeholders and the regulator and so the project can move into implementation with more confidence. Furthermore, including the benefitting community in the ESIA process ensures understanding and promotes a sense of ownership of the proposed project, which is important for the success and sustainability of the proposed venture.

6.4 Common Shortcomings of the ESIA Process

The full value of the ESIA process to a proposed project is only realised if done well. The following common problems have been identified and attention should be drawn to them so that the shortcomings can be avoided.

1. **Insufficient time**. The ESIA process requires time to gather baseline information, to organise stakeholder meetings, to gather views, and to analyse information from multi-disciplinary fields. This process can take one to two months for small projects and longer for larger projects. In addition, the regulatory review, as provided in the EMCA, can take 45 days. This implies that the ESIA process should be initiated immediately after the Feasibility Phase so that any adjustments to the design can be incorporated into the project design.

2. **Faulty stakeholder identification**. Stakeholders who have been excluded, by intent or omission, from the ESIA process can pose a risk to the project as they can raise legitimate complaints that they were not consulted, even if the project does not pose a risk to them. This can disrupt project implementation.

3. **Inadequate disclosure**. Infrastructure projects are proposed to overcome identified problems and deliver a stream of intended benefits and such benefits are easy to explain. It is however more awkward to justify that the proposed project may also create a hazard, change or induce negative changes to the social and bio-physical environment. On balance, it is expected that the positive impacts outweigh the negative impacts, but this position must be proved, not assumed. Failure to adequately disclose the potential negative impacts breeds suspicion and rumours which can pose a more significant risk to the project implementation. One aspect of disclosure is ensuring that the stakeholders understand both the benefits and risks of the project. This may require the proponent to provide an opportunity for credible experts and community representatives to discuss the project with the public.

4. **Professional independence of EIA expert**. The EIA Expert conducting the ESIA process is recruited and paid for by the project proponent. There is a prevailing perception that the EIA Expert is therefore biased in favour of the project. It is therefore critical that the EIA Expert ensures that the process has credibility with stakeholders, that opposing views are properly documented, and that the implications of negative and positive project benefits are adequately explained. The EIA Expert must maintain professional integrity during the execution of the ESIA process.

6.5 The Problem of Scale

The EMCA has made provision for the scale of the project by requiring an EIA Project Report for small projects and a full EIA process and report for large projects. The EIA Project Report is essentially a brief EIA report that provides more qualitative statements about the baseline condition and the impacts of the project. The majority of projects anticipated in this manual are likely to require an EIA Project Report, rather than the full EIA. Whether a project requires the full EIA process is a decision made by NEMA.

It is presently less clear with very small projects whether the project requires NEMA approval. For example, does a 500 m^3 pan require NEMA approval? Thus any project proponent who is in doubt should seek guidance from the relevant county NEMA office but as a guideline, any water conservation structure that requires a water permit, will also require NEMA approval.

6.6 ESIA Process

Due to the extensive nature of the ESIA process, it is recommended that this phase commence early during the project cycle. The stages of the ESIA process are summarised in the following sections.

6.6.1 Scoping

Scoping is the process of brainstorming on the issues and alternatives that need to be considered in the ESIA process. It helps to determine which impacts are likely to be significant and thus require more focus in the ESIA process. This is a valuable step at the start of the ESIA process and as part of the EIA Project Report development, as it can mitigate against unexpected issues arising later in the project. The scoping analysis also helps to inform on data availability and gaps, determine the appropriate scope of the assessment, suggest suitable survey and research methodologies and help to eliminate issues that could otherwise consume time and resources to investigate.

The scoping process should involve the beneficiary community, as this will encourage buy-in and general acceptance of the proposed project. Social concerns around water needs should also be considered, such the adequacy of the available resource to meet the expected demand which will help to control expectations.

6.6.2 Analysis of Potential Impacts

The scoping process of the ESIA is followed by the analysis of the potential impacts. This involves analysing the potential impacts identified during scoping to determine their exact nature, scale, magnitude, likelihood, extent, effect as well as possibility for reversibility. This analysis promotes better understanding of the potential impacts and provides information on whether the impact is positive or negative and, if negative, whether it is acceptable, requires mitigation or is not acceptable.

This analysis can also help in distinguishing primary and secondary impacts.

Primary impacts are those typically associated with construction, operation and maintenance of a structure and are generally more obvious and easy to quantify. These impacts can be negative as well as positive. Such impacts may include:

- Removal of soil and vegetation impacting on habitats, current productive uses of the land, archaeological or cultural sites and artefacts;
- Increase/decrease in habitat for pests e.g. crocodiles, hippos, waterfowl, fish, aquatic insects (mosquitoes), snails, etc.;
- Displacement of people, livestock, wildlife, public amenities, businesses etc;
- Conflicts between project proponent, regulators, service providers and public;
- Disruption to public services and utilities;
- Change in the natural hydrological pattern which may impact on floods and low flow conditions downstream;
- Degradation of water quality due to erosion, excessive storm water and discharge of contaminated effluent;
- Increase in dust and noise;
- Increase in traffic and risk of accidents;
- Influx of immigrant workers;
- Discovery of rare/unique artefacts.

Secondary impacts are those that are induced by the project or the primary impacts. These might include:

a) Reduction/increase and change in reliability in downstream water availability impacting domestic, agricultural, livestock, wildlife and environmental conditions;
b) Increase/decrease in social cohesion. This can be conflicts between communities or within communities related to control of the structure, and sharing or attributing benefits and impacts;
c) Increase/decrease in local population, demand for land and land prices;
d) Increase/decrease in businesses, employment, commerce and livelihoods;
e) Increase in health risks e.g. drowning, traffic, *malaria*, *schistosomiasis* etc.
f) Increase in local utilities and services;
g) Improvement in road access.

It is generally helpful to consider the different impacts during the different stages of the project (site investigations, construction, operation and maintenance) as it is easier to identify the mitigation measures and attribute responsibility in the mitigation plan.

One way that is commonly used to document the nature and degree of impact is through the use of a matrix, a sample of which is shown in Table 6-1 and Table 6-2 at the end of this chapter.

6.6.3 Identification of Mitigation Measures

The analysis of potential positive and negative impacts is then followed by the identification of mitigation measures to address the potential negative impacts. The aim of mitigation is to either eliminate or reduce negative impacts. Some of the mitigation options include: Avoidance of impact, reduction of impact, restoration to original state, relocation of those affected, and compensation among others.

Table 6-3 at the end of this chapter provides a range of possible mitigation measures but these should be customised to the specific site and conditions of the proposed water conservation structure.

6.6.4 Analysis of Alternatives

After the analysis of potential impacts and the identification of mitigation measures, analysis of options and alternative ways to meet the same objectives can be considered with an aim to identify the least damaging option. At this point, comparison of potential impacts and mitigation options can be made against a series of alternative designs, locations, technologies and operation so as to identify the most desirable combination. It is important that the objectives of the proposed project are clearly articulated otherwise the analysis of alternatives can digress into the consideration of irrelevant options.

For most water conservations structures, an analysis of alternatives should include the following considerations:

1) **Different location**. This issue is of particular importance where there are cultural or special habitats that should be protected, where a particular location might increase the likelihood of conflicts (e.g. over pasture or between domestic users and livestock/wildlife) or increase the likelihood of environmental degradation for example by attracting more livestock than the environment can sustain;
2) **Different design**. This might involve considerations of different ways of supplying water from the structure (e.g. cattle trough or cattle ramp into the water), ways to make the structure safer or to improve water quality, and ways to provide wider public benefit, etc.;
3) **Different way to meet same objective**. This might include a consideration of alternative sources, water treatment of existing sources or additional infrastructure at existing sources.

For example, improved water use efficiency through control of leaks, metered connections and tariffs, control of illegal connections can increase the supply without the need to develop a new source;

4) **No project**. This option essentially provides a basis of comparison with the proposed project and other alternatives. The no-project option is not necessarily a static situation as external factors such as demand for water, employment and livelihoods are dynamic.

6.6.5 Environmental Management and Monitoring Plan

The Environmental Management and Monitoring Plan (EMMP) sets out the indicators, timeframe, cost and responsibility for the management of the impacts and implementation of the mitigation measures. The EMMP should be elaborated to sufficient detail to address the identified adverse impacts. Some of the areas that should be covered in the EMMP include but are not limited to: Description of prioritized mitigation activities, timelines and resources to ensure delivery of the EMMP, a communication plan as well as monitoring strategies.

Table 6-4 at the end of this chapter provides the framework of an EMMP.

6.6.6 Decommissioning Plan

Decommissioning of a small dam, pan or water conservation structure can arise for a number of reasons which may include:

- The structure has filled with sediment or for what ever reason cannot provide the stream of benefits for which it was constructed;
- The structure has become an uncontrolled public safety hazard. This could arise if proper maintenance of the spillway was neglected by the owner and WRMA decides to withdraw the water permit;
- The owner of the structure decides to decommission the structure.

Decommissioning a structure does not necessarily mean removing the structure because the process of decommissioning may cause negative environmental and/or social impacts. Decommissioning implies making the structure safe through a process of analysis of the options and impacts, and establishing a decommissioning plan that aims to secure the best long term beneficial impacts to both the social and bio-physical environment.

In the event that the removal of the structure is inevitable, then breaching, in the case of a dam, may be considered. Gradual emptying the dam or lowering the water level (by cutting down the spillway or opening the scour pipes) to reduce pressure on the embankment should be undertaken before any breaching of the embankment is undertaken.

6.7 Public Consultation, Disclosure and Participation in the ESIA Process

6.7.1 Public Disclosure and Consultation

Public disclosure and consultation is a regulatory requirement but experience has also proven that it adds value to the project and helps mitigate future conflicts and negative impacts.

Public disclosure and consultation is particularly important during the ESIA process firstly because completion of most ESIA processes demand it and cannot be said to have effectively occurred without it and secondly because the ESIA process begins at the initial stages of the project and thus provides a great opportunity to set the pace on public disclosure and consultation and win the trust and collaboration of stakeholders.

Relevant plans for public disclosure and consultation must therefore form part of the ESIA process. It is important that the disclosure process provides time and resources to ensure that the affected communities have an opportunity to understand the implication of potential social and environmental impacts. An individual impact may cause a cascade of other secondary impacts and it is this association of cause, effect and impacts that should be fully disclosed.

6.7.2 Stakeholder Analysis and Consultation

Stakeholder analysis is the process of identifying interested and affected parties and considering how best to consult with these parties. The outcome should be a Stakeholder Engagement Plan (SEP) that documents who, how and when stakeholders will be consulted regarding what aspects of the project throughout the various stages of the project. Refer to Chapter 5 of this Manual which provides a detailed description of the basis, requirement, importance and process of stakeholder engagement.

The goal of stakeholder engagement during the ESIA process is to engage with interested and affected stakeholders in order to provide accurate and timely information on the merits and demerits of the proposed water conservation structure, facilitate discussions to register comments and concerns, and enable stakeholders to participate meaningfully in the ESIA process. The expected outcome of this engagement is a well-informed body of stakeholders, including the project proponent, with an understanding of the potential benefits and impacts of the project, where concerns that they raised have also been addressed. The support of stakeholders provides the project with the social licence for project implementation.

The consultation process should use participatory methodologies and should include:

1. **Public meetings (barazas).** These are appropriate for reaching a larger number of people. Adequate attention must be given to announcing the intended meetings. Notices of proposed public meetings can be posted in public places in the vicinity of the project, channelled through the local day schools, religious groups, local CBOs, WRUAs, and through the local administration office (e.g. chief's office) or announced on vernacular radio stations popular in the project area;
2. **Workshops.** These provide an opportunity to share details of the project in more depth than can usually be achieved in a public meeting. In addition, participants can focus on the issues and provide more considered feedback;
3. **Key informant interviews.** These one-on-one interviews are appropriate for local leaders, thematic experts, and individuals who are likely to be directly affected by the project;
4. **Focus group discussions.** These are appropriate for sharing information and opinions with selected groups (e.g. women, youth, disadvantaged, etc) within a community. It provides an environment in which group members can speak more freely and discuss internally to formulate and voice an opinion that is perhaps contrary to the position of the more powerful and vocal members of the community.

An important part of the stakeholder consultation process is the documentation of who was consulted, what was disclosed, and what opinions were expressed. The following documentation is typically required to substantiate that public consultation was conducted:

- Notice of public meetings and record of announcements;
- Signed participation lists from public meetings and focus group discussions;
- Signed minutes of meetings;
- Signed key informant forms which document the opinions of the informant;
- Copy of materials that were discussed or shared with the public and stakeholders;
- Photographs (and possibly videos).

6.8 ESIA Project Report

The outline of an ESIA Project Report is provided in Chapter 19.

Table 6-1: Example of Impact Matrix for Construction Phase of a Small Dam

	Potential Environmental and Social Impacts													
Activity	Land Use Change	Vegetation & habitats	Public utilities & services	Noise & Vibration	Dust and air pollution	Water quality	Generation of wastes	Social Unrest	Public Safety	Cultural Heritage	D/s flows	Public health risks	Employment & business	etc
Site survey & clearance	---	---					-	-		-			+	
Development of access roads		--		--	--	--	-		-	-			++	
Establishment of Site Camp, Offices & Workshops			-			---	--	--					+	
Stocking fuels, lubricants, chemicals, material						-	--		-					
Borrow pit establishment & use				---	---	--	-		--				+	
Construction of water diversions						--		-	-		---		+	
Construction of Embankment				---					-				++	
Construction of Spillway				---		--			-				++	
Construction of perimeter fence								+	-				+	

(Note: + beneficial impact, - adverse impact)

Table 6-2: Example of Impact Matrix for Operational Phase of a Small Dam

	Potential Environmental and Social Impacts													
Activity	Land Use Change	Vegetation & habitats	Public utilities & services	Noise & Vibration	Dust and air pollution	Water quality	Generation of wastes	Social Unrest	Public Safety	Cultural Heritage	D/s flows	Public health risks	Employment & business	etc
Reservoir filling											---			
Normal operations		++	++			-	-		-		- / +	--	+++	
Emergency releases		-							--					
Maintenance/de-silting						--								

(Note: + beneficial impact, - adverse impact)

Table 6-3: Possible Mitigation Measures

Project Phase	Potential Negative Impact	Possible Mitigation Measure
Project Planning	Social discord and conflict	• Detailed stakeholder analysis • Comprehensive stakeholder and community consultation and disclosure • Careful attention to issues of land ownership and control • Full legal compliance with WRMA and NEMA approval • Community representation in project implementation structures (if appropriate) • Establishment and disclosure of grievance mechanism
	Loss of vegetation during site clearing and excavation of test pits	• Limit clearing of vegetation to facilitate access to and survey of site • Control access to site • Cover test pits and refill after sampling
	Displacement of people, livestock, wildlife, public amenities, businesses	• Identification of alternative and/or improvement of grazing areas • Improvement of alternative • Re-settlement and compensation. These are complex issues and specialist advice and government involvement is advisable to develop a Resettlement Action Plan (RAP) • Establish alternative or rebuild livelihoods • Provide fish ladders if applicable
Construction	Loss of soils, vegetation & habitats	• Restrict clearing to immediate requirements, minimise unnecessary clearing, restrict clearing for fuel wood • Stock pile soil for re-use • Re-vegetate site after construction wherever possible (borrow areas, riparian area, etc) using adequate stockpiled top soil • Rare flora species to be identified and relocated • Ensure reserve flow
	Loss of archaeological and cultural artefacts	• Seek expert advice • Consult with community leaders/ members
	Discovery of unique/ dangerous items	• Seek expert advice
	Disruption to public utilities and services	• Ensure public is notified in advance of road closures, disruptions to water supply and power
	Nuisance from noise and vibration	• Provide workers with PPE • Control working hours and limit noisy activities in proximity to habitations and at

		night • Provide public notification in advance of any rock blasting
	Dust and air pollution	• Control dusty conditions by spreading water as required • Maintain equipment fleet in good working condition
	Water quality degradation	• Control of erosion to limit sediment transport to water course • Provide proper containerised storage of fuels, lubricants and chemicals
	Generation of wastes	• Provide waste collection bins and dispose of waste to designated dump sites. • Incineration, composting and recycling systems to be established. • Toxic chemical containers to be returned to suppliers
	Social unrest	• Maintain open communication with local community • Provide inclusive structure for community participation • Set out clear labour policies that favour local employment where possible • Ensure compliance with labour laws • Notify downstream water users, WRMA and WRUA of likely changes in water quality and reliability
	Public safety	• Provide OSHA training and PPE for workforce and ensure it is used at all times • Control traffic and working hours • Vehicles to be fitted with lights, reversing alarm, and rotating light • Ensure proper lighting at all times; • Provide clean drinking water and safe rest areas • Control public access to site
	Change in downstream flows	• Ensure reserve flow at all times • Monitor downstream water quantity and quality
	Public health risks	• Sensitise workforce • Provide proper water and sanitation facilities for workforce • Ensure adequate and safe housing available on site or within vicinity for workforce
	Conflicts due to access to employment and business	• Provide opportunities for local businesses and service providers to engage with project • Recruit labour locally where possible
Operations	Water quality degradation	• Provide adequate provision for aeration of releases within the project design • Provide proper containerised storage of fuels, lubricants and chemicals

		• Create vegetated buffer within riparian area
	Generation of wastes	• Provide waste collection bins and dispose of waste to designated dump sites. • Incineration, composting and recycling systems to be continued. • Toxic chemical containers to be returned to suppliers
	Social unrest	• Provide inclusive structure for community participation • Set out clear labour policies that favour local employment where possible • Ensure compliance with labour laws • Notify downstream water users, WRMA and WRUA of likely changes in water quality and quantity
	Public safety	• Ensure dam construction meets design specifications and recruit competent contractor • Provide OSHA training and PPE for workforce and ensure it is used at all times • Control public access to site • Establish and implement Dam Safety Plan, including notification of emergency releases • Conduct routine inspection • Ensure timely implementation of maintenance tasks • Raise awareness within community regarding hippo and/or crocodile hazard and alert KWS • Raise awareness within community regarding the risk of drowning and provide accessible safety flotation rings
	Change in downstream flows	• Ensure reserve flow at all times • Monitor downstream water quantity and quality
	Public health risks	• Provide proper water and sanitation facilities for workforce and visitors • Undertake community awareness regarding health risks in relation to *malaria* and *schistosomiasis* • Provide bed nets • Support local health centre with testing facilities
	Employment and business	• Provide opportunities for local businesses and service providers to engage with project • Encourage local entrepreneurs regarding recreational, fisheries businesses

Table 6-4: Partial Environmental Management and Monitoring Plan for a Small Dam

Issue	Mitigation Measure	Indicator	Means of verification	Cost	Responsible
Loss of land, housing & livelihoods	Resettlement Action Plan (RAP)	Resettled households	Observations & Surveys		Proponent's environmental compliance officer
Social conflicts	Grievance redress mechanism	Conflicts resolved	Minutes of meetings & Register of complaints		Proponent's environmental compliance officer
Loss of grazing land	Pasture improvement on remaining grazing land	Pasture Condition	Observations		Proponent's environmental compliance officer
Loss of vegetation and habitats	Re-vegetate	Soil cover Habitat quality	Observations		Contractor's environmental compliance officer
Waste generation	Garbage bins Safe disposal of containers	Litter Returned containers	Observations Stock records		Contractor's environmental compliance officer
Water quality	Control of erosion	Downstream water quality	Water quality report		Contractor's environmental compliance officer
	Containerised storage of fuels, lubricants and chemicals	Passing inspection	NEMA/WRMA Inspection Report Water quality report		Contractor's environmental compliance officer
Public safety	Occupational Health and Safety training and PPE	% of workforce trained PPE infringements	Observations & counts of infringements		Contractor's environmental compliance officer
	Control of traffic	No. of incidences	Health records Incidence report		Contractor's environmental compliance officer
	Dam Safety Plan operational	Approval by professional or WRMA review	Inspection and testing		Owner
Public health	Community environmental health awareness training	% of local community trained Incidence of *malaria* & *schistosomiasis*	Health records at local health centre		Owner in collaboration with local health workers

CHAPTER 7

EROSION CONTROL AND CATCHMENT MANAGEMENT

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7 EROSION CONTROL AND CATCHMENT MANAGEMENT

This chapter is principally concerned with the problem of siltation of small surface water reservoirs and how to reduce this risk. The structures primarily at risk are dams and pans and to lesser extent rock catchments. The functionality of sand and sub-surface dams is generally less vulnerable to adverse siltation due to the fact that the storage volume is filled up by course sandy sediments. However, the issues of erosion control and catchment conservation are applicable to all catchments in order to increase or maintain the productive and ecological capacity of the land.

The ability of sedimentation to seriously reduce the useful lifetime of the reservoir means that erosion control should be included, designed, costed and implemented as an integral part of the project. The additional cost of soil erosion control, the expected life of the project and the potential benefits are factors that should be evaluated in the economic feasibility of the project. It should be noted that erosion control measures will never completely eliminate sedimentation and siltation of reservoirs, although they can if done properly reduce the problem considerably. If the outcome of this analysis is that the sediment sources are numerous, the sediment loading is high, and the likelihood of impacting the sediment loads is minimal, then serious consideration should be given to whether the investment in the structure is justified given the risk of rapid sedimentation.

Erosion control and catchment conservation measures should principally focus on the control of water flow across the land surface and through a catchment. Erosion control measures are in most cases not undertaken solely for the purpose of limiting the quantities of sediments entering surface water reservoirs, but principally as soil conservation measures to conserve and enhance productivity of valuable agricultural and range lands. Therefore; all planning of catchment protection measures should take place in close cooperation with the competent technical assistants in the relevant department in the County Government. In areas with intensive farming activities, erosion control is best managed through the individual farmers.

Catchment protection measures will in many cases require relatively heavy investments in time and labour from the land and water users concerned with the project. It is therefore essential that within the structure representing the local community, a responsible person(s) be appointed for catchment protection works at an early stage. The role of this person will essentially be the coordination between the various parties involved in soil conservation and catchment protection efforts. Once the appropriate conservation measures have been carried out, they will need regular maintenance. It should also be emphasized that measures against erosion in the rangelands can only be successfully implemented if combined with grazing control.

It should be noted that although this chapter deals mainly with erosion and sediment control other water quality issues may also affect water storage structures. Sources of pollution that degrade water quality can also be identified and dealt with during catchment management planning.

7.1 Introduction

The challenge of erosion control for a dam, pan or similar water conservation structure arises from a variety of factors which include:

- Erosion sites and sediment sources are dispersed across a large area and are not particularly easy to identify;
- The catchment area may belong to people who do not benefit from the dam;
- As a water source, the dam or pan may induce higher domestic, livestock and wildlife traffic which can enhance the conditions for accelerated erosion (and possibly conflicts);
- Erosion, sediment transport and deposition are natural geo-morphological processes and identifying accelerated erosion or sediment yield induced by anthropogenic activities from natural processes can be difficult which makes it harder to motivate erosion control activities.

Soil conservation and sediment control in catchment areas demand the implementation of a long term global policy, the principal elements of which can be summarised as follows:

- Use of appropriate farming or rangeland methods;
- Afforestation of hill-tops;
- Terracing of steep agricultural lands;
- Use of cut-off drains and artificial waterways where required;
- Grazing control;
- Control of gully development;
- Riverbank protection and development of vegetated riparian buffers zones;
- Proper disposal of pathway and road runoff.

Labour intensive methods are most suited for implementation of soil conservation in high potential areas which are intensively farmed by small-scale farmers. Mechanised soil conservation is best suited to large-scale farming or large semi-arid areas where no steep slopes occur.

A number of methods for erosion control and soil conservation which bear direct relationship with the sedimentation of reservoirs will be presented briefly hereunder. For more specific and detailed explanations of soil conservation methods in agriculture, reference is made to “Soil Conservation in Kenya, especially in small-scale farming in high potential areas using labour intensive methods”, 7th edition (Wenner, 1981) and “Soil and Water Conservation Manual for Kenya” (Thomas, D.B (ed.), 1997).

7.2 Approach

Various approaches have been adopted over the years to tackle the problem of erosion within catchments. The concept of Integrated Watershed Management (IWM), targeting catchments of 5 km^2 or less, aims to identify sediment sources and work with farmers/pastoralists to implement erosion control measures while developing a holistic understanding of water resource availability and use within the catchment.

The Ministry of Agriculture has adopted an approach that focuses more specifically on land productivity and land husbandry. This approach is directed through Common Interest Groups (CIGs) with the idea that successful farmers and pastoralists who adopt good land use practices that improve soil fertility and animal husbandry will adopt appropriate soil and water conservation methods to achieve improved productivity.

The MWIS has encouraged the formation and establishment of Water Resource User Associations (WRUAs); a voluntary membership organisation of water users and stakeholders focused on the management of a common water resource. The WRUA provides an institutional structure that can reach out across property and administrative boundaries to improve water resource availability and management, including undertaking soil and water conservation activities. WRMA, in collaboration with the WSTF, has established a financing framework (called the WRUA Development Cycle or WDC) to assist WRUAs in the implementation of their sub-catchment management plans (SCMPs).

Essentially the implementing party, be it a community group, WRUA or private entity, should, with the assistance of a soil and water conservation officer from the county government establish a long term soil and water conservation plan that goes beyond identifying technical solutions but also addresses the financing, organisational and monitoring aspects required for an effective plan:

7.3 Identification of Erosion Sites and Sediment Sources

The first step is to identify the actual erosion sites and sediment sources within the catchment. This requires a site walk or tour of the catchment and careful observation of the following features particularly where these features occur in combination:

- Bare areas lacking vegetated soil cover. These may be areas which are over grazed, frequently common grazing areas, areas near watering points where there is increased concentration of livestock traffic, and areas with shallow soils or ploughed farmland;
- Steep and long slopes;
- Areas with cohesion-less soils (sands, sandy silts, etc.) or unconsolidated sediments;
- Man-made drainage systems. These include roads, footpaths and storm drains. These tend to intercept and accumulate runoff and either are eroded themselves or discharge to a site where erosion can occur;
- Gullies and river banks. These are areas where runoff accumulates and erosion can take place at the headwall or along the banks;
- Areas downslope from impermeable areas. e.g. land on the edge of urban settlements where excessive storm water can cause erosion;
- Quarries and construction sites. These sites tend to have stock piles of disturbed and loosened soil which are easily eroded.

Once the main erosion sites and sediment sources have been identified, they should be assessed to determine where and what interventions can be undertaken that will actually impact the sediment load into the water conservation structure. This process of prioritisation with respect to reservoir sustainability may have a different outcome if the objective of the soil and water conservation plan is to maintain land productivity. Both should be developed and discussed with stakeholders prior to the final selection of priority erosion sites and implementation measures.

Once the erosion sites have been selected, specific measures, budgets and timeframes for each site can be drawn up in combination with the land owner(s) and/or user(s).

Roles of all actors in the enforcement of erosion control activities should be discussed and agreed upon.

Table 7-1 provides a typical catchment evaluation checklist that can guide the identification of erosion sites and sediment sources.

7.4 Erosion Control on Agricultural Land

Erosion control on agricultural land requires the willing participation of the farmers. They must see how they benefit from erosion control programmes.

7.4.1 Appropriate Farming Methods

During the growing season, measures which favour the growth of crops (applying manure, fertiliser etc.) will also result in good protection against rain erosion. Outside the growing season, a continuous layer of crop residue left on the ground (mulching) reduces erosion, and will increase the infiltration rate of the rainwater, in addition to improving soil fertility. Minimum tillage, frequently practiced in combination with mulching, is a tillage technique aimed at minimising the soil disturbance and loss of soil moisture to enhance plant growth.

Table 7-1: Catchment Evaluation Checklist

Task	Notes	Tick
Establish catchment map	Start with sketch or existing topographical map. Add in the following details: • Bare areas; • Steep/long slopes; • Areas with erodible soils; • Man-made drainage systems; • Gullies and riverbanks; • Areas with high runoff; • Quarries and construction sites.	
Identify county government officers working on soil conservation in the area	Note names and contact details. Discuss ongoing catchment protection work. Get details for established contacts within the catchment.	
Follow up with important farmers/pastoralists in the area	Contact before field work. Meet during field work. Get their views and suggestions on erosion in the area and steps needed to control it.	
Visit the catchment and update map as needed	Use a GPS to record points of interest (show them on the map). Note areas of immediate concern. Note areas of longer term concern.	
Discuss any previous erosion control programmes in the area	Get as many details as possible on what has been done in the past. Find out what has succeeded, what has failed and reasons for both success and failure. Involve all key actors.	
Visit any existing erosion control infrastructure	Evaluate status of any erosion control infrastructure. Establish when it was installed and what it cost. If possible find out who implemented the project.	
Visit areas/farms with both good and poor land use	Make sure to review both positive and negative situations within the catchment.	
Discuss plans for erosion and sediment control	Establish initial plans and initial budgets. Consider possible funding sources. Include discussions on enforcement of catchment protection.	
Report back on findings to everyone involved	If possible produce a brief report with contact details for key actors and suggested actions (with timelines)	

Maize and other crops should not be cultivated year after year. A three year rotation of maize and grass is recommended for maintaining a soil structure which will decrease the rate of erosion.

All cultivation (ploughing, planting etc.) should be along the contours and not up and down the slopes. Strip cropping (wide strips with alternating crops under rotation) can be used on permeable soils occurring on slopes under 20%.

On slopes > 20% good farming methods may not be sufficient and terracing should be considered.

7.4.2 Classification of Land

Farm land can be classified with regard to slope and soil, with the different categories requiring different measures:

- **<u>Flat Land (slope < 2%)</u>** can usually be farmed without special soil conservation measures except contour farming.

- **Gentle Slopes (2% < slope < 12%)**: In terms of the Agriculture Act terracing is not obligatory. In semi-arid areas and in areas with erodible soils, terracing is however desirable.

- **Slopes exceeding 12%** (but not exceeding 55%): Terraces should be used if the depth of the soil is more than approximately 0.75 m. Developed bench terraces are preferred. Sometimes modified bench terraces (narrow ledges cut into the slope) can be used for planting fruit and other trees, on slopes of 35-55%.

- **Slopes exceeding 55%** should be covered with forest and/or grass. It is permissible to cultivate tea, cane or bananas with a layer of trash on the ground. Sometimes modified bench terraces can be used for fruit and other trees.

- **Soils which are rocky, stony or shallow** should be used for pasture or forest, or should have stone terraces.

7.4.3 Terracing

Types of Terraces: Developed bench terraces (Figure 7-1) are generally preferable, since they will reduce the gradient and length of the slope. They will also retain eroded soil, moisture and nutrients.

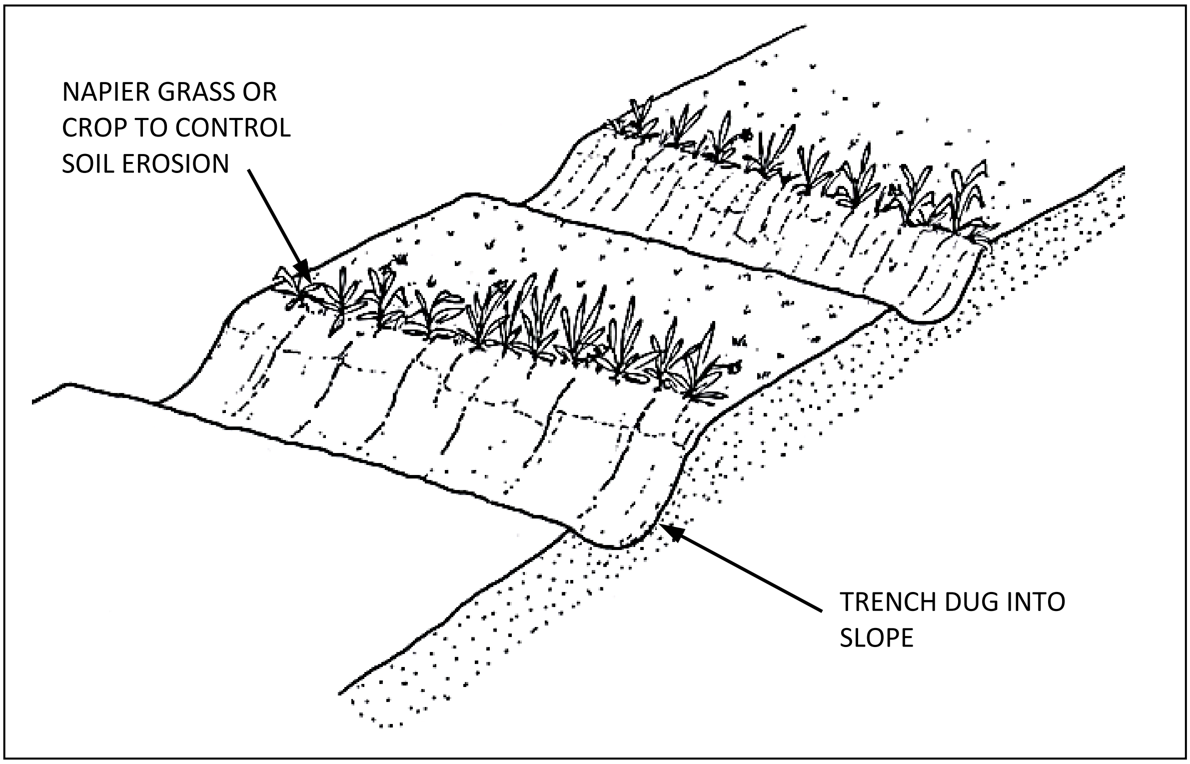


Figure 7-1: Bench Terraces (slope <55%)

Bench terraces can be developed from grass strips, which in their turn can start as either unploughed strips, grass planted in one or two rows (e.g. Napier grass) or trash lines laid along the contours. Figure 7-2 shows how manual labour and erosion together can develop a bench terrace from a grass strip.

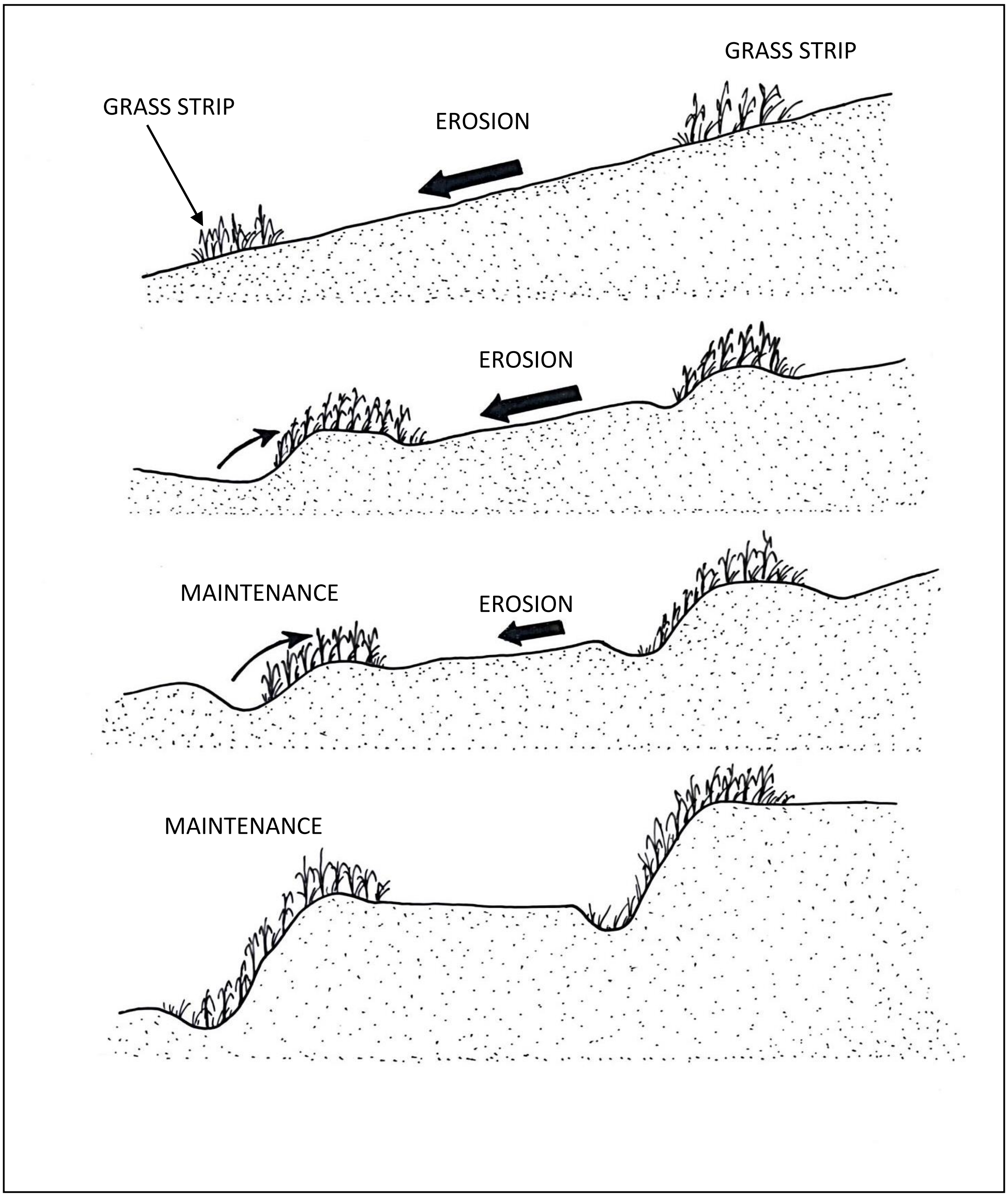


Figure 7-2: Development of Bench Terraces from Grass Strips

To hasten the development of a bench terrace, or on steep slopes, the so called Fanya Juu method can be used (See Figure 7-3). A channel is dug and the soil thrown uphill to form an embankment (ridge). Grass should be planted on the ridge to protect it. Part of the channel should be maintained as a storm water drain. In dry areas an infiltration ditch may be better except on steep slopes or unstable soils. On rocky ground, stones can be collected and set in a small ditch to act as a barrier.

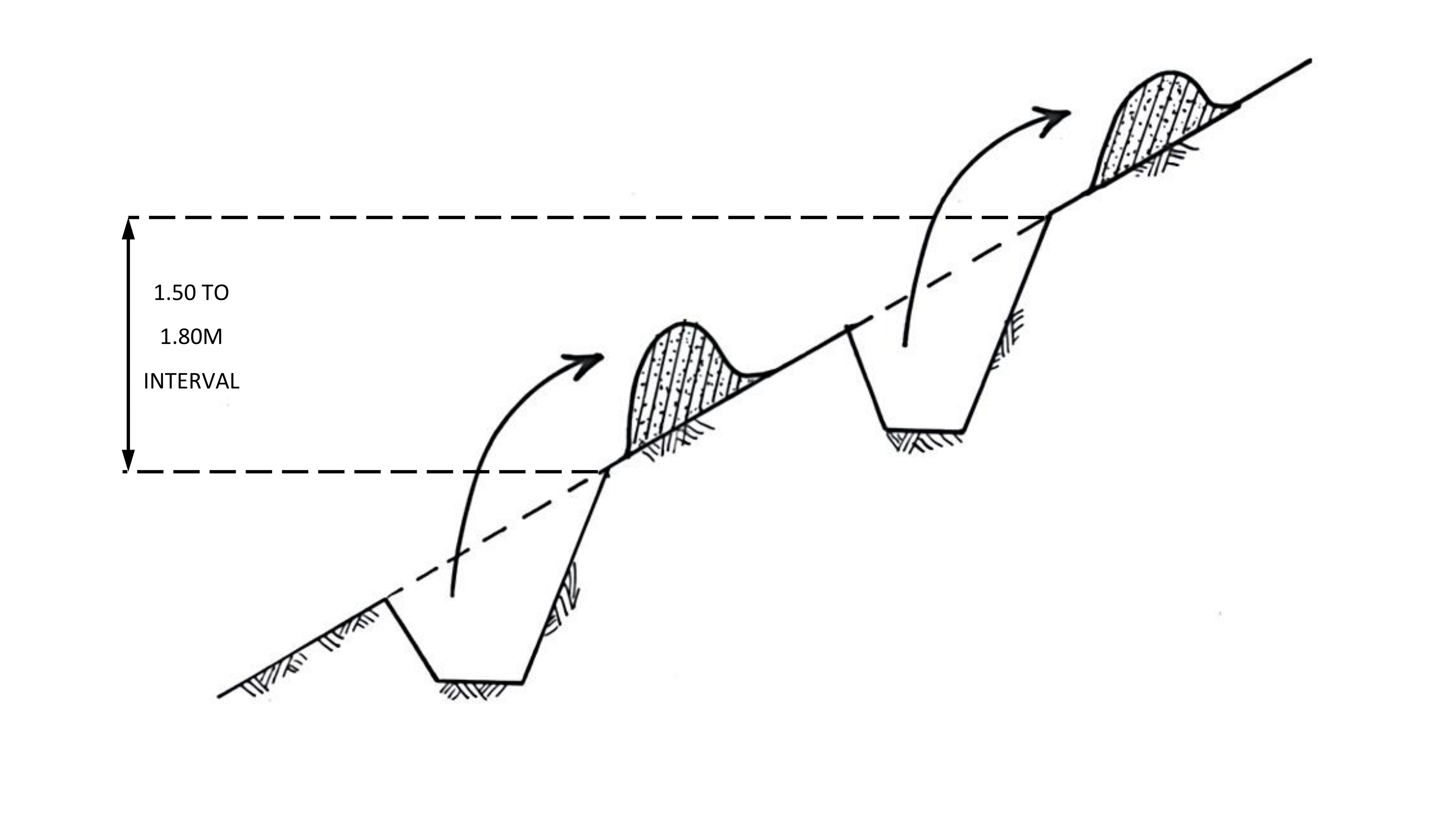


Figure 7-3: "Fanya Juu" Terracing

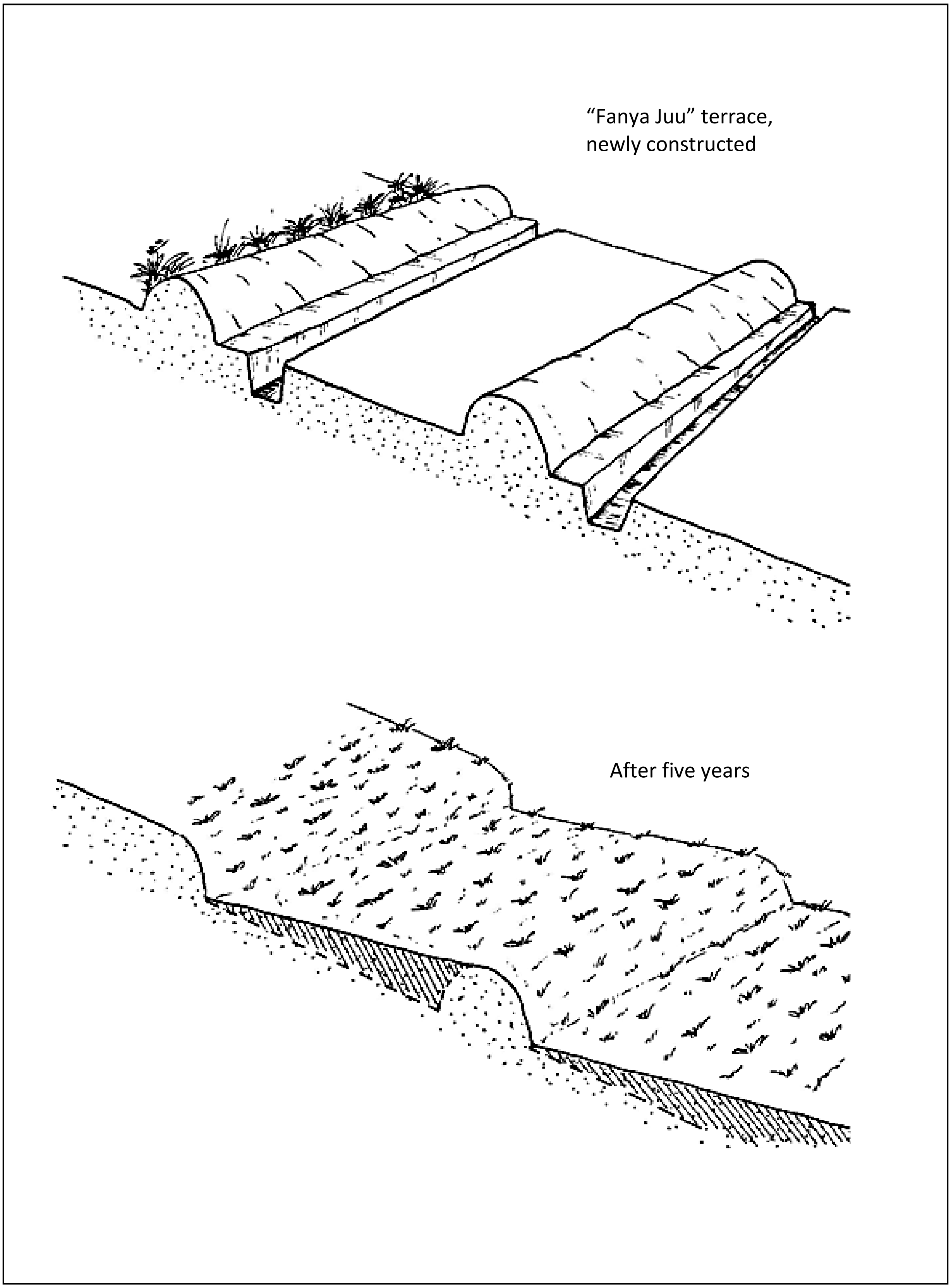


Figure 7-4: Progression of a Fanya Juu Terrace

Modified bench terraces (see Figure 7-5) can be used on steep and very steep slopes, for the planting of trees.

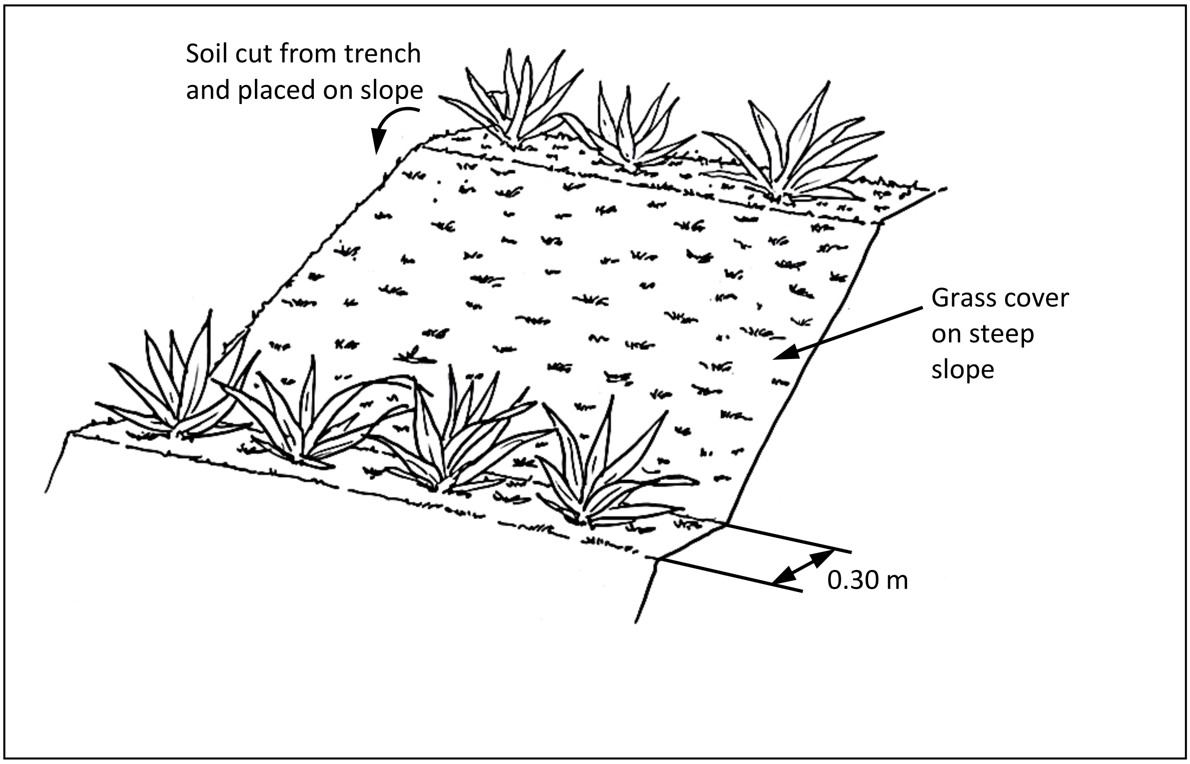


Figure 7-5: Modified Bench Terraces (35% < slope < 55%)

Mechanised terracing can be used on slopes not exceeding 12%. Two types are used in Kenya: the channel type and the ridge type. (See Figure 7-6) They will basically develop into bench terraces, as the channel fills up with sediment.

Dimensioning of Terraces: It is preferable to use a constant vertical interval of between 1.5 and 1.8 m (corresponding to the eye height of a person) in setting out terraces. For the variations which in some cases are applicable reference is made to Thomas D.B. ((ed.) 1997).

Terraces should not be longer than about 400m. However, if it is difficult to find a natural waterway or a non-erodible area to discharge water within that distance, it might be cheaper to make the terraces longer than to construct an artificial discharge channel.

Terraces should usually be sloped. Table 7-2 gives an indication of slopes recommended for terraces and cut-off drains. In dry areas it is preferable to make the terraces level to retain as much water as possible in-situ.

Table 7-2: Recommended Slopes for Terraces and Cut-off Drains

Soil Type	Recommended Slope (%)
Erosion resistant (Clay)	1
Normal	0.5
Erodible (silty, sandy)	0.25

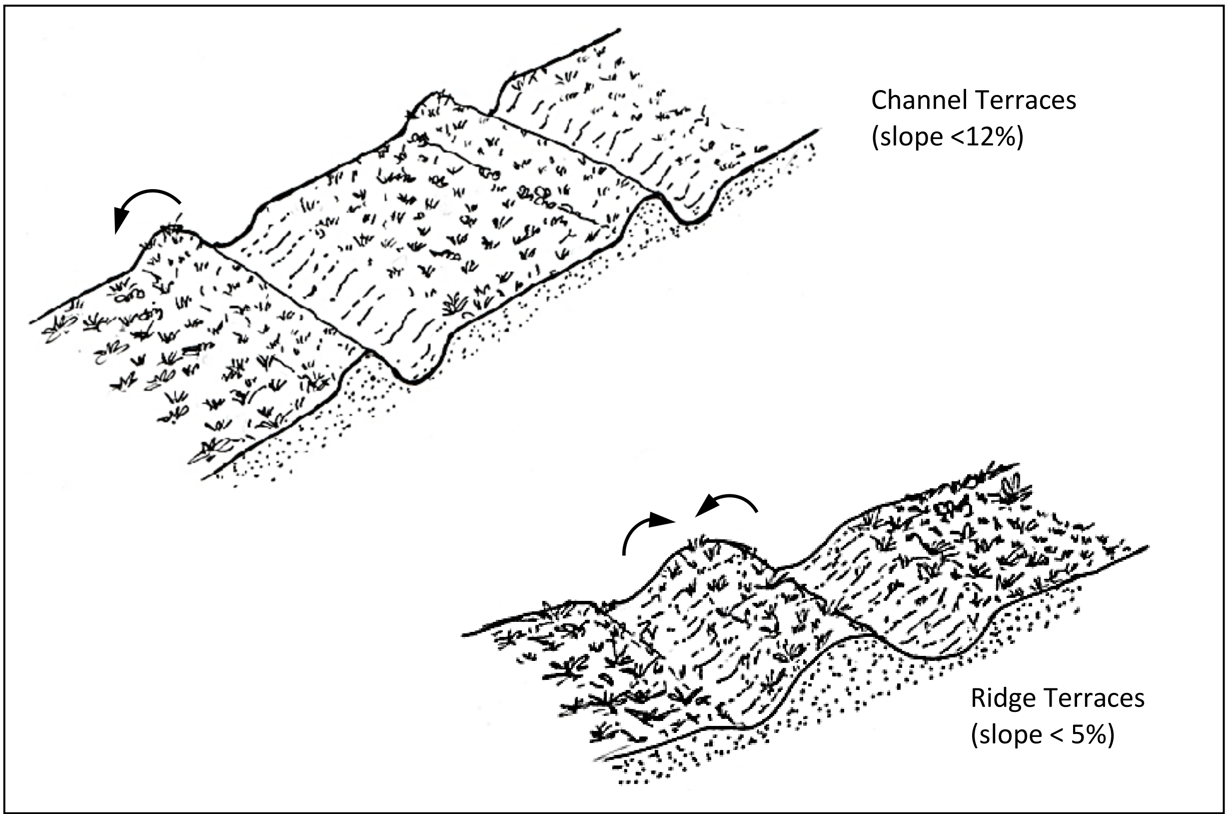


Figure 7-6: Mechanised Terracing

7.4.4 Use of Artificial Waterways and Cut-Off Drains

a) **Cut-off Drains/diversion ditches:** Preventing water from flowing down terraced slopes, or diverting large quantities of water from entering farms can be achieved by using cut-off drains. However, cut-off drains should only be used where there is evidence of large flows of water which cannot be stopped through normal terracing. Cut-off drains should be constructed only after terracing has been carried out.

An essential aspect when planning the construction of a cut-off drain is the location of the outlet point. Cut-off drains should not be constructed in locations where the water cannot be discharged safely.

Cut-off drains should further only be constructed where a sufficient level of community involvement can be obtained to guarantee regular maintenance (removal of silt from the channel etc.) by the farmers.

Approximate dimensions of manually constructed cut-off drains are shown in Figure 7-7. In Fanya Juu terracing, the top terrace should be constructed as a cut-off drain, with the soil being thrown down the slope so that the channel can carry as much water as possible. Mechanically dug cut-off drains have often a V-shaped cross-section and are somewhat larger than shown on Figure 7-7. Recommended slopes for cut-off drains are given in Table 7-2. Cut-off drains should not be longer than approximately 400m.

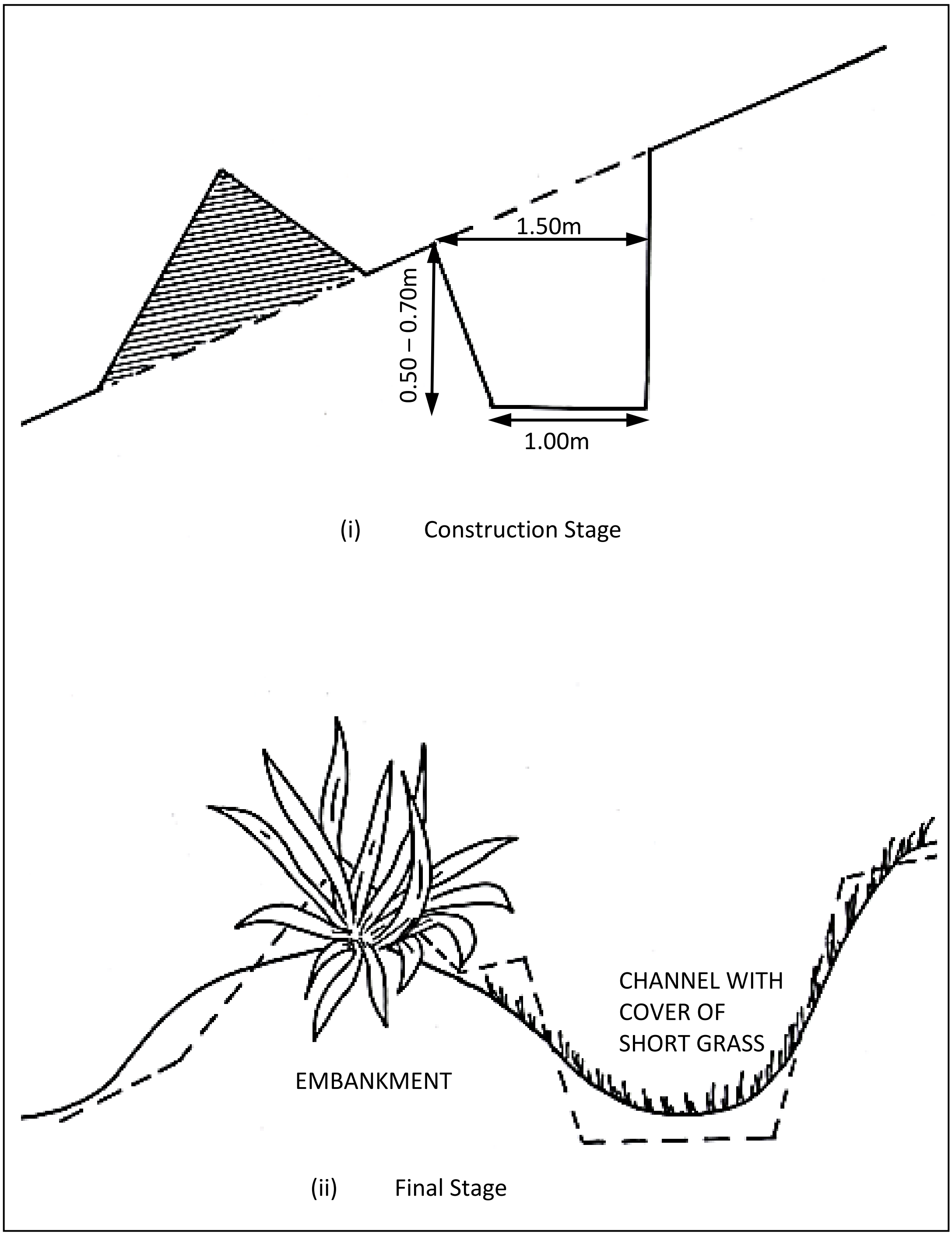


Figure 7-7: Cross Section of Cut-Off Drain

b) **Artificial Waterways:** The water from cut-off drains as well as from terraces should be discharged into natural watercourses (rivers) or onto non-erodible areas, such as rocky ground or permanent pasture with a good grass-cover. If such an outlet point cannot be found within a reasonable distance, an artificial waterway will have to be constructed to take the water down the slope (see Figure 7-8).

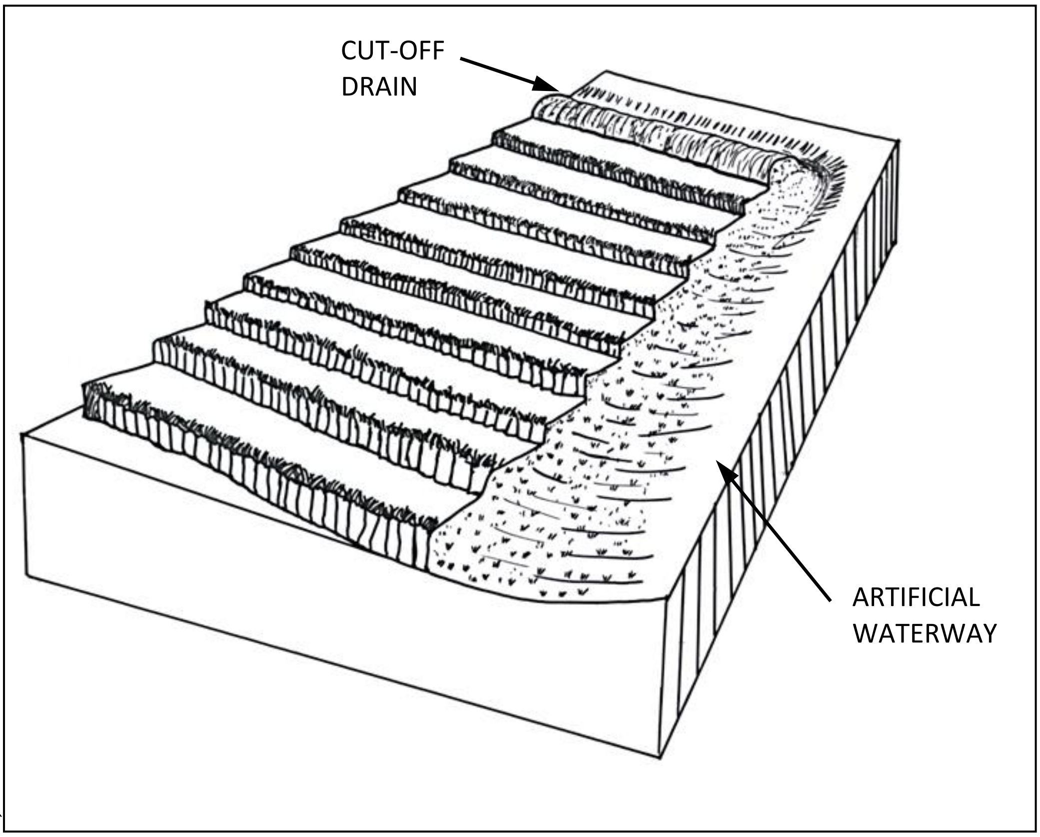


Figure 7-8: Discharging runoff along an artificial waterway

Such artificial waterways should be wide (at least 1.50 m), shallow (0.30 m deep) and should have a short grass cover in order to minimize erosion.

An appropriate method of constructing these waterways is shown on Figure 7-9. For further details of dimensions and recommended slopes, reference is made to Thomas D.B. ((ed.) 1997).

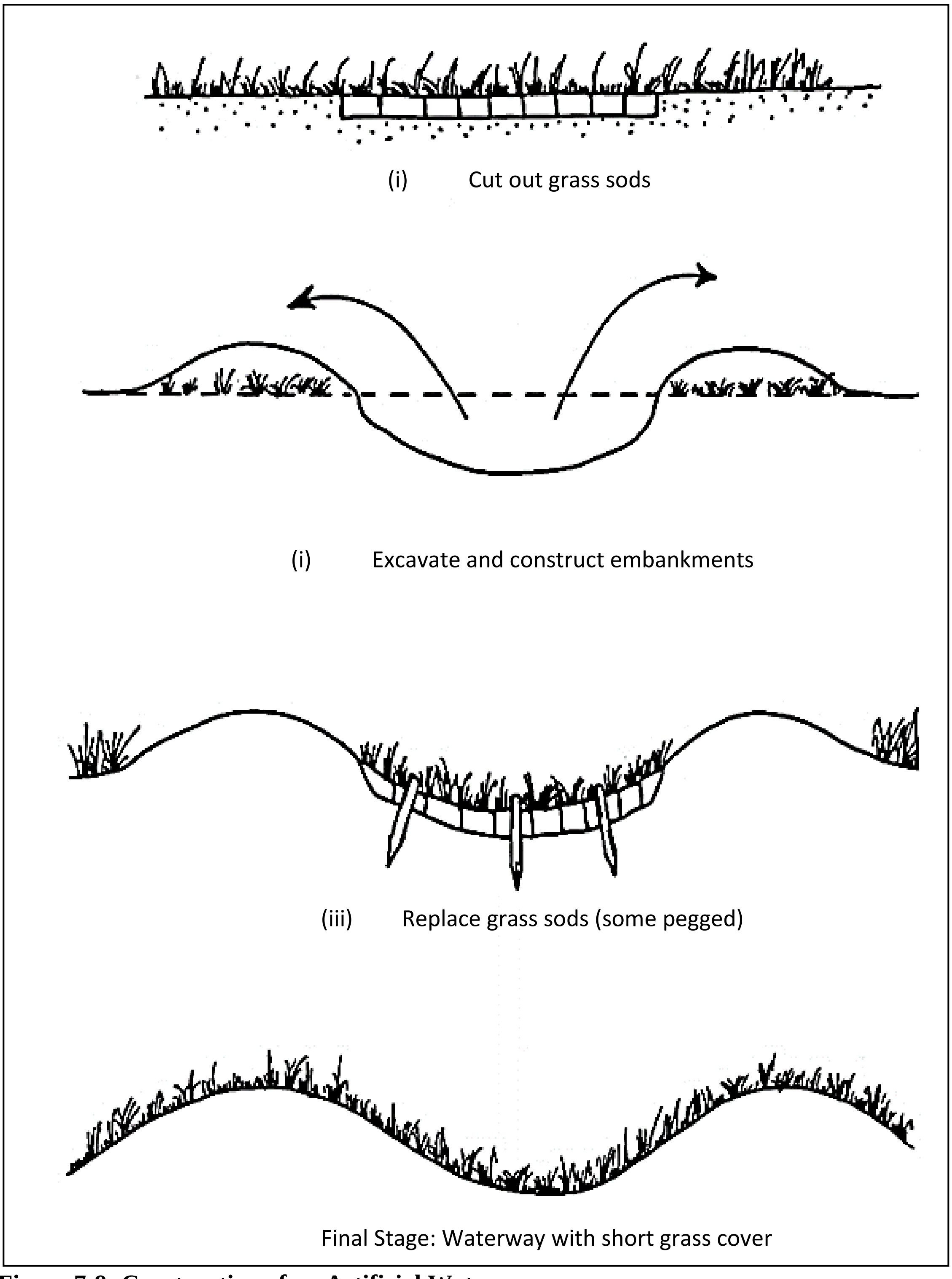
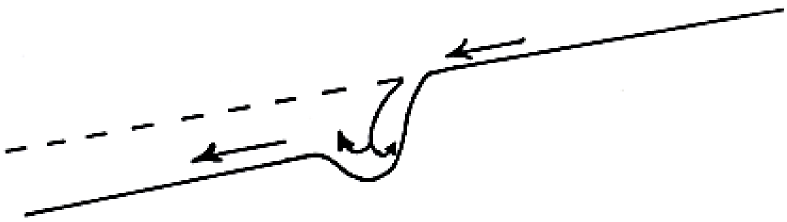


Figure 7-9: Construction of an Artificial Waterway

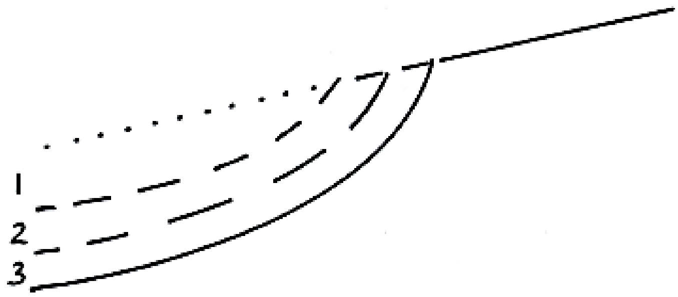
Artificial waterways and cut-off drains should not be constructed if proper maintenance is not guaranteed. Non maintained cut-off drains and artificial waterways will easily deteriorate into gullies, and are as such worse than nothing.

7.5 Control of Gully Erosion

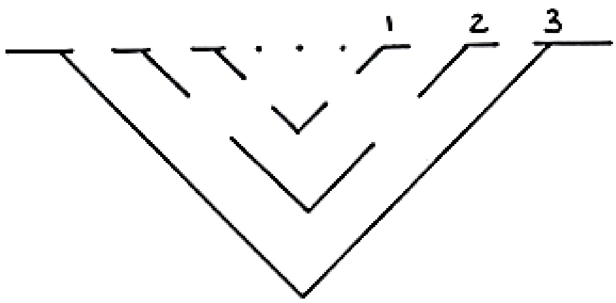
When rills have increased in size so that they cannot be levelled out by ploughing, they are called gullies. The mechanics of gully development are explained in Figure 7-10. First, at the head of the gully, erosion will cut back into the slope [(i)]. Thereafter, the flow over the floor of the gully will deepen it, until solid rock is reached [(ii)]. The gully will further be widened by soil which is rendered unstable by the deepening of the gully, eventually sliding into the gully [(iii)]. A V-shaped gully is indicative of a gully that is still forming and active erosion is taking place. A U-shaped gully indicates that the erosion surface is along the sides of the gully, rather than the floor or headwall.



(i) Erosion cuts back into slope



(ii) Flow deepens gully floor



(iii) Gully widens by sliding off of unstable walls

Figure 7-10: Mechanism of Gully Development

In its early stages, it is usually not difficult to stop gully erosion. Unfortunately gullies are usually left to develop until they cannot be returned to cultivated land. Small gullies (up to 0.50 m deep) can be filled with soil, brushwood, hay etc. Restoration of large gullies requires a great deal of time, effort and money. In terms of simple economics, the repair of gullies is seldom justified in semi-arid regions with soils of low agricultural value. However, an economic analysis should be undertaken before deciding whether the works are justified or not.

The main measures to contain gully erosion can be summarized as follows:

a. Diverting the water from the head of the gully by means of a cut-off drain, or a ridge of soil. If this is possible, no other measures are needed in the gully itself.

b. If diversion is not possible, then the velocity of the water needs to be reduced by means of scour checks and check dams. The head and floor of the gully need to be protected with erosion resistant materials. The type of measures used will differ according to the shape of the gully, and the availability of construction materials.

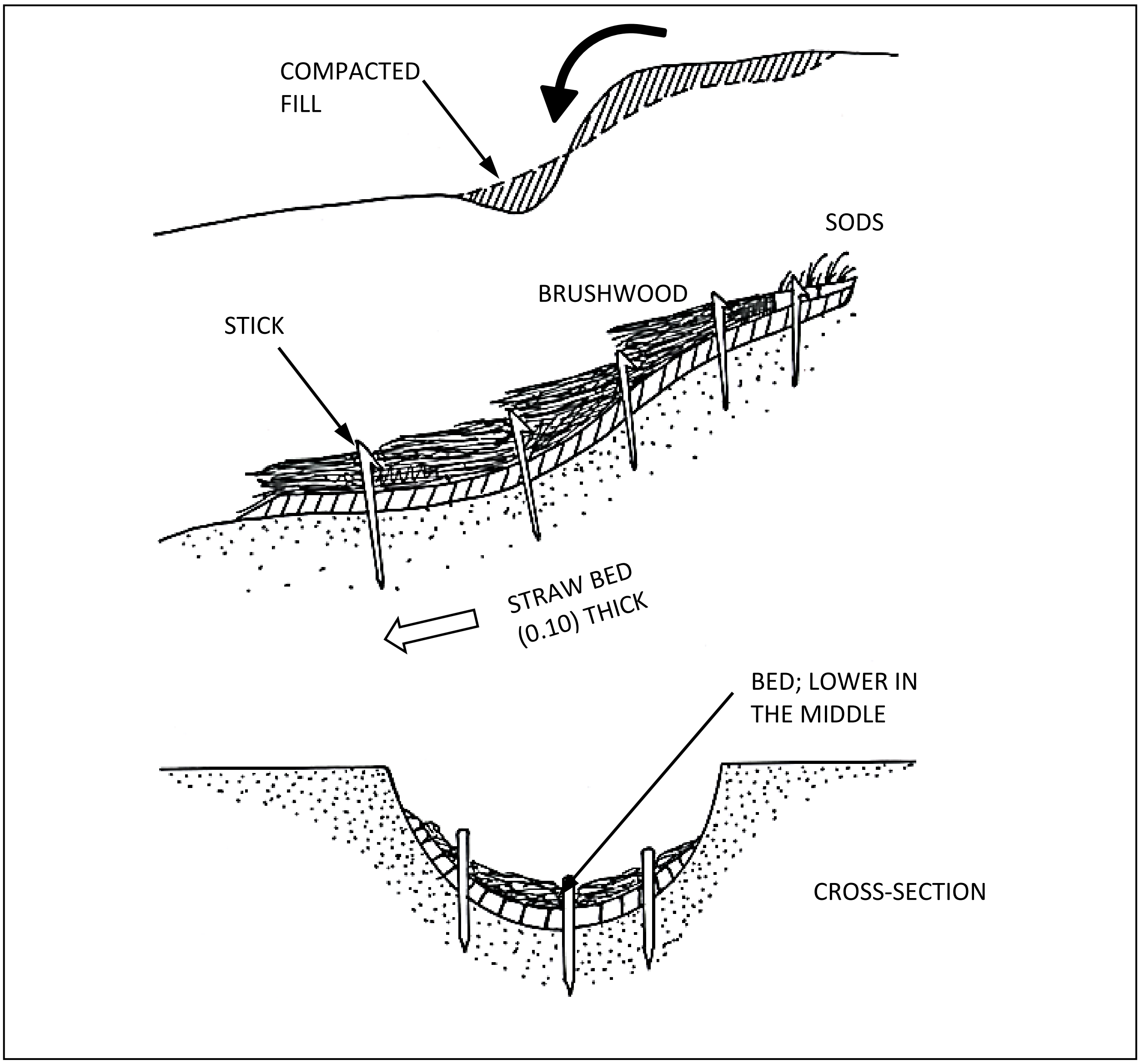


Figure 7-11: Gully Head Protection Using Wood

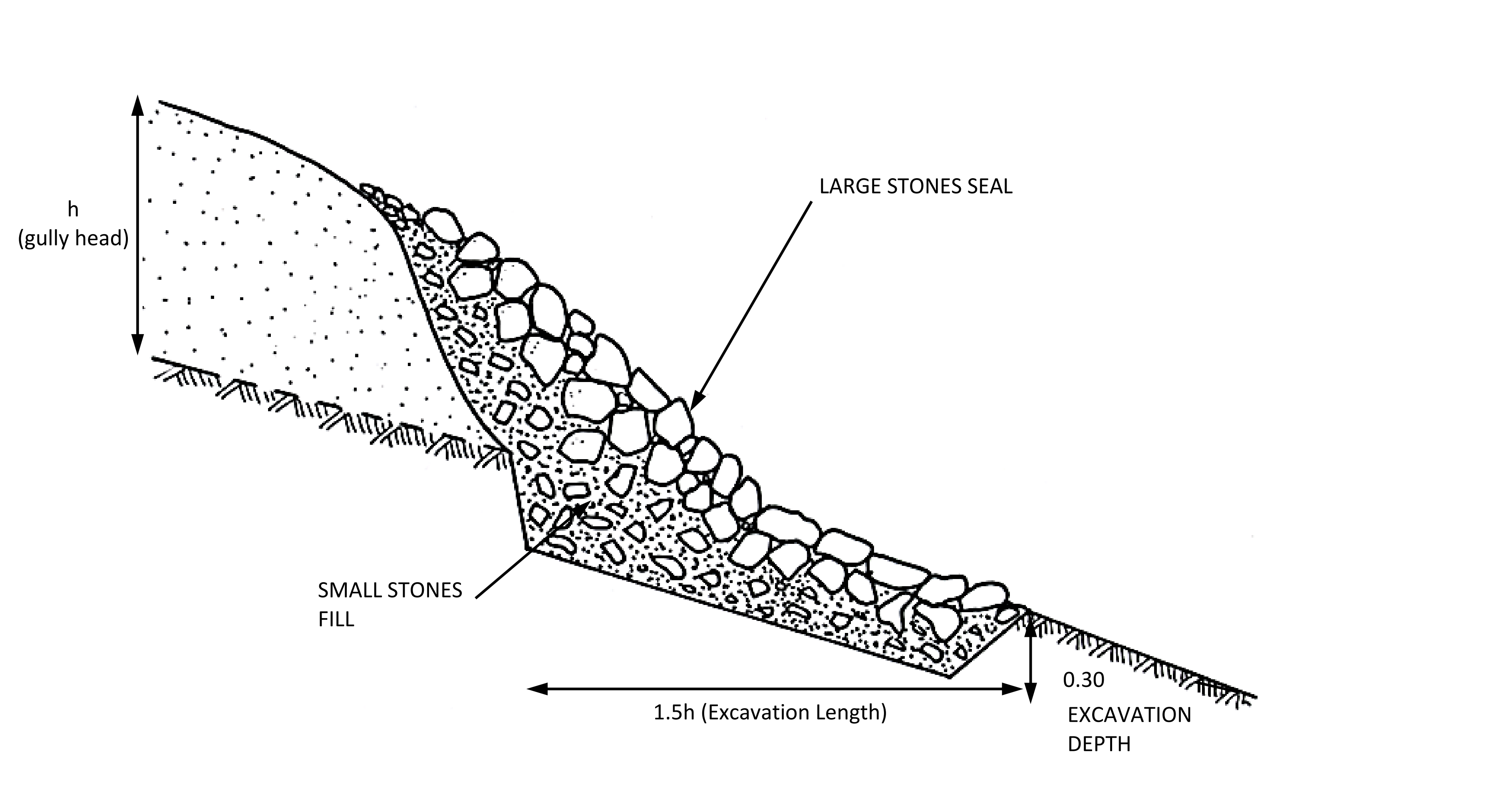


Figure 7-12: Gully Head Protection Using Stones (h<1m)

- **Head of gully:** Figure 7-11 to Figure 7-13 show adequate protection measures in various cases. In case only small stones are available, gabions will be used. Note that care should be taken when developing gabions to arrange stones inside gabion box so that the galvanising on the wire is not damaged and that the stones are stable in the event that the wire rusts and breaks.

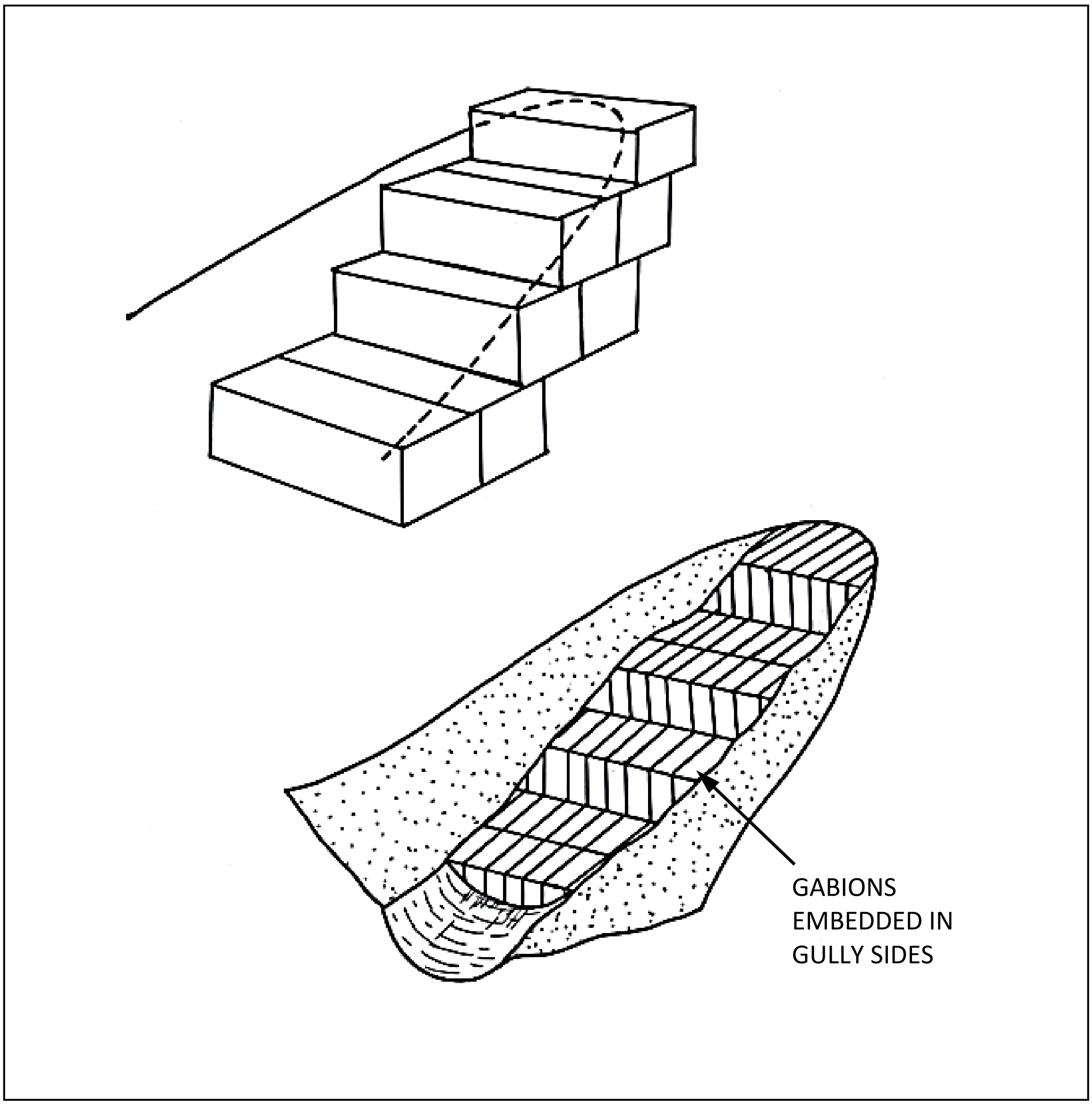


Figure 7-13: Gabion Gully Head Protection

- **Gully Floor:** Figure 7-14 shows how stone thresholds can be used to protect the floor of wide and shallow gullies. In case of steep and V-shaped gullies, check dams made using large stones or gabions as shown on Figure 7-15 and Figure 7-16 respectively can be used. Check dams should be lower than one metre and they must not block the gully valley.

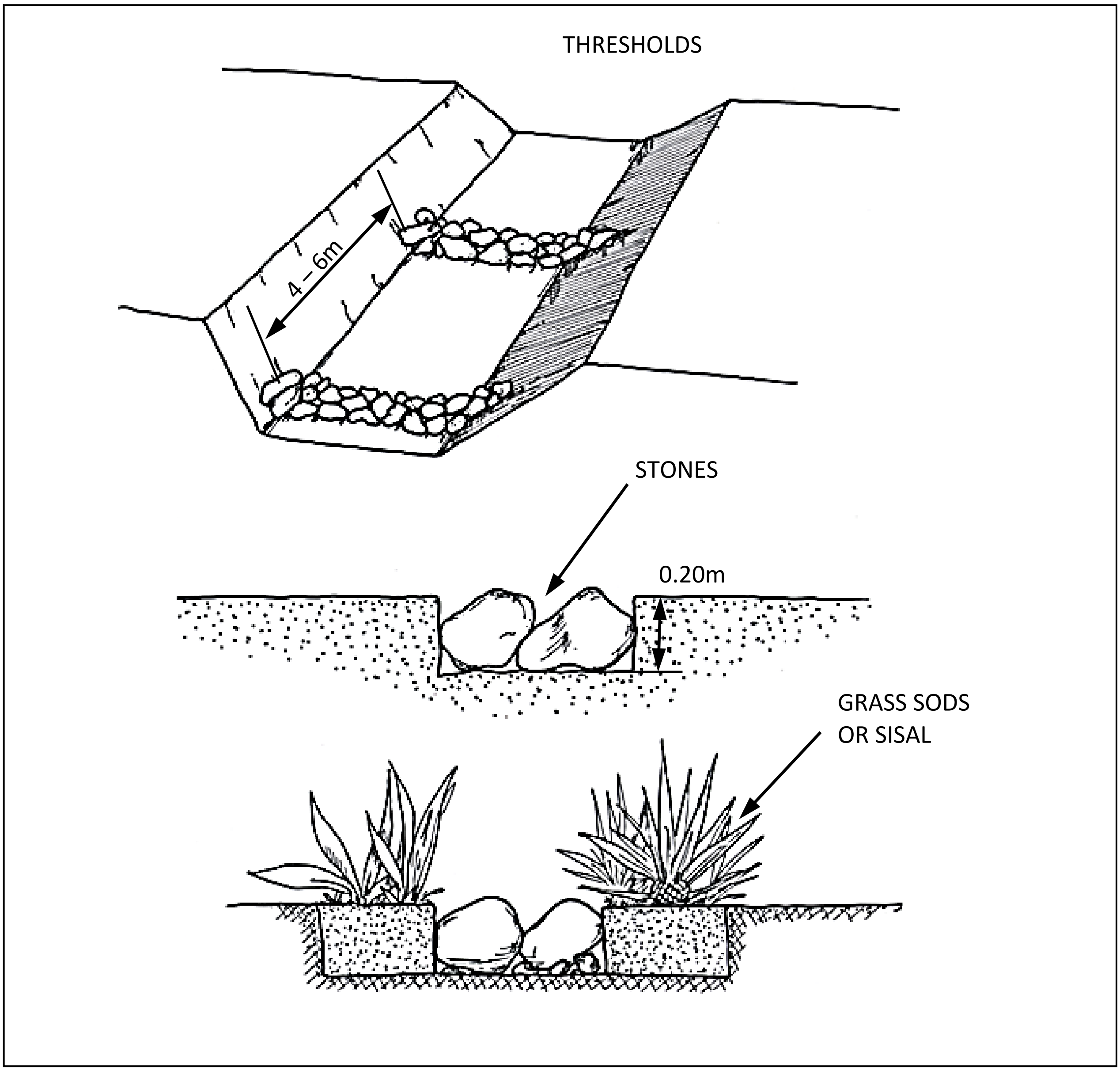


Figure 7-14: Stone and Grass Thresholds

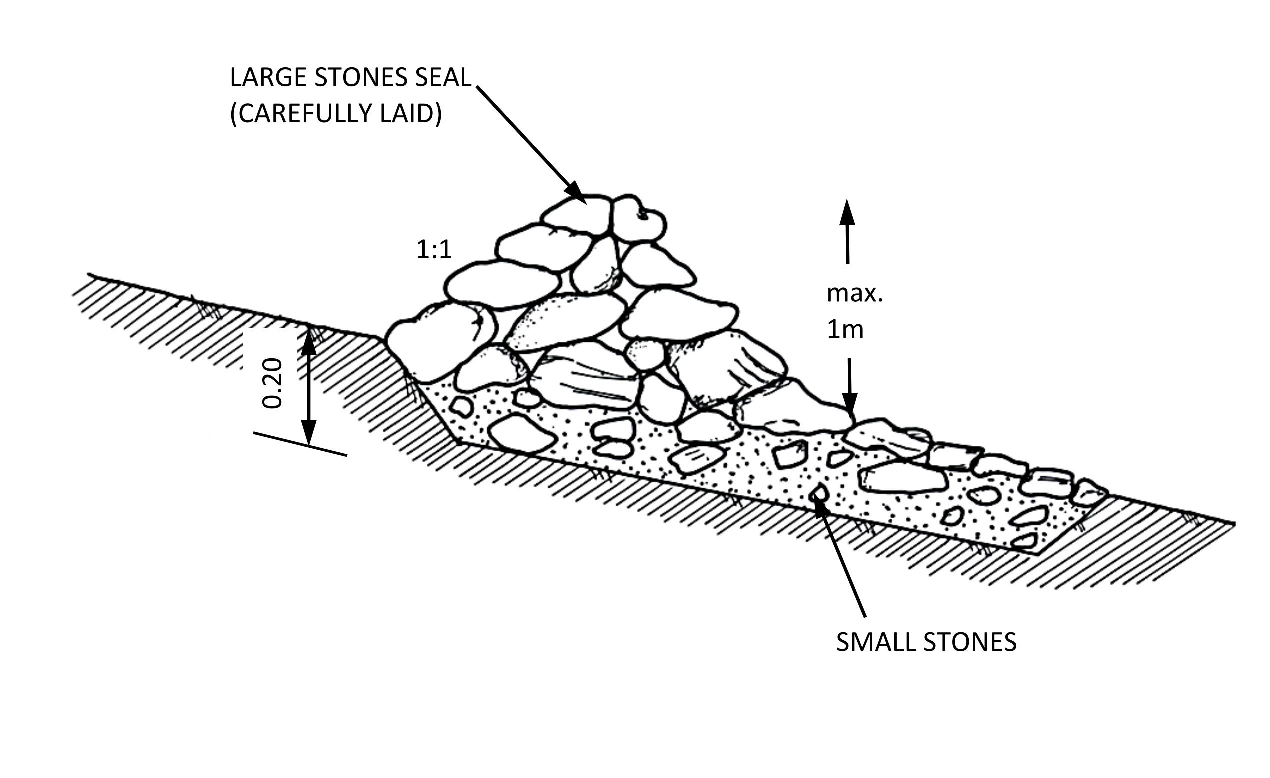


Figure 7-15: Cross Section of a Check Dam

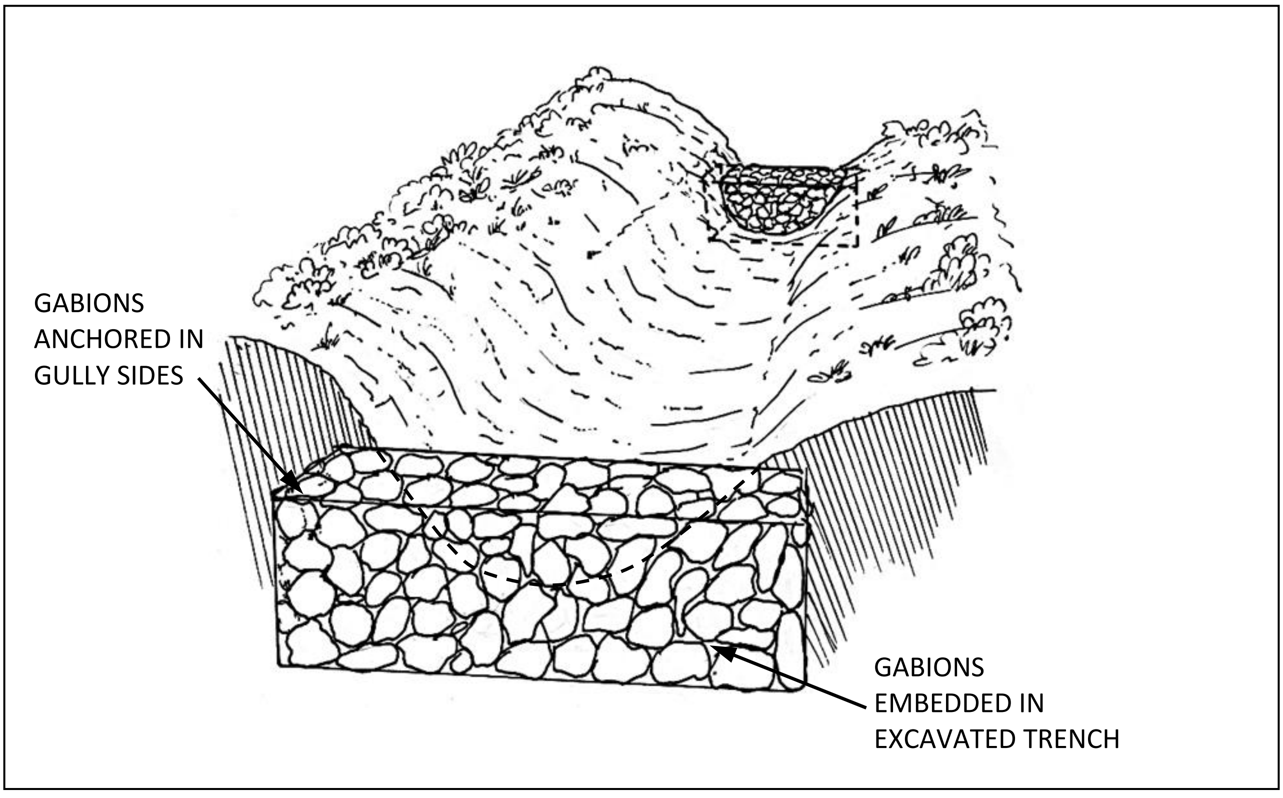


Figure 7-16: Gabion Check Dam

7.6 Protection of River-Banks

The quantity of sediment transported in a river depends largely on the rate of erosion of the catchment area. When sediment inflow into a river is reduced by the use of soil conservation methods in the catchment area, the river will tend to scour or cave its banks in order to regain its normal sediment load. Therefore, riverbank protection is necessary as part of a sediment control plan.

A river cannot erode above its highest water level (flood level). Erosion is most likely to occur in the stretches of the river where the water velocity is highest. In wide sections of the river (flood plains etc.) where low velocities occur, erosion protection measures are normally not required.

7.6.1 Riparian Area

Agricultural, environmental and water regulations specify the width of the riparian strip. In general it is a minimum of six metres, a maximum of 30 metres or otherwise half the width of the river (bank top to bank top) and is measured from the river bank edge. Riparian legislation addresses land use and not land ownership, as is frequently and mistakenly understood. The riparian legislation defines the width and which activities are not allowed within the riparian area, unless with government approval. The activities proscribed such as cultivation, development of permanent structures and latrines, grazing of livestock, etc., are aimed at ensuring proper ground cover and minimising the likelihood of human, agricultural or livestock pollution of the water body.

7.6.2 Riverbank Protection by Vegetation

Natural vegetation up to the flood level should be protected as much as possible. In cases where the natural vegetation has been removed by the streamflow, it should be replaced with permanent grass or trees.

Cultivated slopes above the flood level should be considered as ordinary slopes with regard to terracing (see Section 7.4.3)

7.6.3 Mechanical Riverbank Protection

In many cases vegetation alone will not provide sufficient protection against the scour of the water and mechanical protection will be required.

Gabions can be used to build stepped walls, while rip-rap (a cement grout can be brushed in if required) or Reno Mattresses are appropriate for protecting sloping banks.

Care must be taken to ensure that the intervention to protect the riverbank does not actually make the situation worse by disrupting the riverbanks. A “do no harm” approach should be taken and where mechanical measures cannot be done adequately to guarantee success, then alternatives of improving vegetation cover on the river banks should be considered.

7.7 Erosion Control on Grazing Land

7.7.1 Erosion on Grazing Land

Development of erosion follows a typical process. Through overgrazing and droughts bare spots will come into existence and increase in size. Perennial grasses are replaced by annual grasses and weeds. The bare soil is compacted by on-going trampling of cattle, thus increasing the run-off. Rain and rill erosion remove the topsoil. Gullies will start on slopes and develop by cutting back until the bedrock is reached, or alternatively up to the hillcrests. Later on the gullies will develop branches which will prevent normal cultivation and grazing.

7.7.2 Erosion Control Measures

Grazing control is the most important control measure on grazing land. Rotational grazing (including the closing of areas if required) and de-stocking where required are the most important components of grazing control.

For rehabilitation deep and loamy soils are best suited. Rehabilitation should only be attempted in areas with annual rainfall over 300 mm. Rehabilitation can start as follows:

a) Closing an area, and thereby allowing natural grasses to establish a new cover. To increase infiltration contour ploughing can be carried out. A spacing of three metres between the ploughing furrows should be maintained on land with some weeds and grass, while a one metre spacing is suitable for completely denuded land.
b) Closing and re-seeding, with suitable species of grasses after some land preparation. Usually scratch ploughing is needed. It is best to re-seed at the beginning of the rainy season. Perennial grasses should be used. As to recommended species, reference is made to Thomas D.B. (Eds. l997).

Finally, it must be emphasized that:

- Measures against erosion without grazing control are useless.
- Cut-off drains especially in sandy soils are useless without grazing control measures and establishment of grass cover around the gullies.

7.8 Control of Road Runoff

Nagle (1999) quoting sediment yield rates in Kenya by Dunne (1979) estimates that roads and trails may contribute 25 – 50% of total sediment yield, although they only constitute a small percentage of the land area (typically less than 2%). While these sediment yield estimates are based on basins with steep settled agricultural land, it is indicative of the scale of the problem which is generally given insufficient attention by road engineers and contractors and soil conservation practitioners. Essentially the runoff from road servitude and surrounding areas, if not properly controlled, can cause extensive damage to the roadway and to adjacent property.

This calls for the provision of (1) appropriate drainage structures on the road to reduce the concentration of and manage the runoff and (2) suitable erosion control structures to protect adjacent land where the road runoff is discharged.

Eriksson and Kidanu (2010) address this issue comprehensively in the Guidelines for Prevention and Control of Soil Erosion in Road Works. A brief summary of the key points is presented herein.

7.8.1 Genesis of the Problem

Roads and tracks are made by removing the natural vegetation and providing a compacted surface for traffic. The road surface itself is therefore designed to be impermeable and will generate runoff. The roadway is either elevated above or cut into the natural ground level and so either way, will intercept any flow paths that it crosses, thereby concentrating runoff into the roadway.

While the government road authorities are clearly responsible for the drainage within the road reserve, there is less clarity on who is responsible once the runoff leaves the road reserve. This ambiguity results in land owners having to deal with runoff problems, not of their making, that they perhaps do not have sufficient technical and/or financial resources to deal with or preventing road drainage onto their land thereby threatening the road.

7.8.2 Controlling Upland Runoff

The first step is to reduce the amount of runoff being collected by the roads through better soil and water conservation in the upland areas of the catchment, whether they are forested, agricultural or grazing lands. This may also include harvesting rainwater from roofs and paved areas into tanks and short-term retention ponds. In addition, cut-off drains can be constructed to intercept the runoff prior to reaching the road for safe disposal. The size, slope and surface cover of the drains need to be carefully designed to avoid creating a gully. See *"Guidelines for Prevention and Control of Soil Erosion in Road Works"* (Eriksson & Kidanu, August 2010) for details on sizing artificial waterways.

Insufficient attention to controlling upland runoff can result in more complicated and expensive structures in the road or lower catchment.

7.8.3 Road Drainage

Road drainage generally includes the following components:

1) **Side drains**. Reduction of erosion in the side drains along the edge of the road is achieved by controlling the water velocity. This can be accomplished by forming and lining the drain, and installing scour checks, check dams and/or drop structures.
2) **Mitre drains**. These are used to discharge accumulated runoff in the side drains into adjacent land. This is possible where the roadway is above or close to the natural ground level and impossible where the roadway cuts through an embankment. The interval of mitre drains can be as frequent as 20 metres in high rainfall, steeply sloped land. Mitre drains need to be cut at a slope of $\approx$5% and kept clear of sediments.
3) **Culverts**. This are constructed with suitable inlet and outlet boxes to pass runoff under the roadway from where it either discharges into the side drain on the lower side of the road or where it is directed away from the roadway to the river or natural water course. This may require a purposefully designed water way.
4) **Bridges**. Bridges are required to enable the roadway to pass above a natural water course. If the bridge does not have sufficient capacity to safely pass floods, then scouring of bridge abutments can occur; flood waters can look for alternative routes thereby creating the likelihood of erosion in other areas.

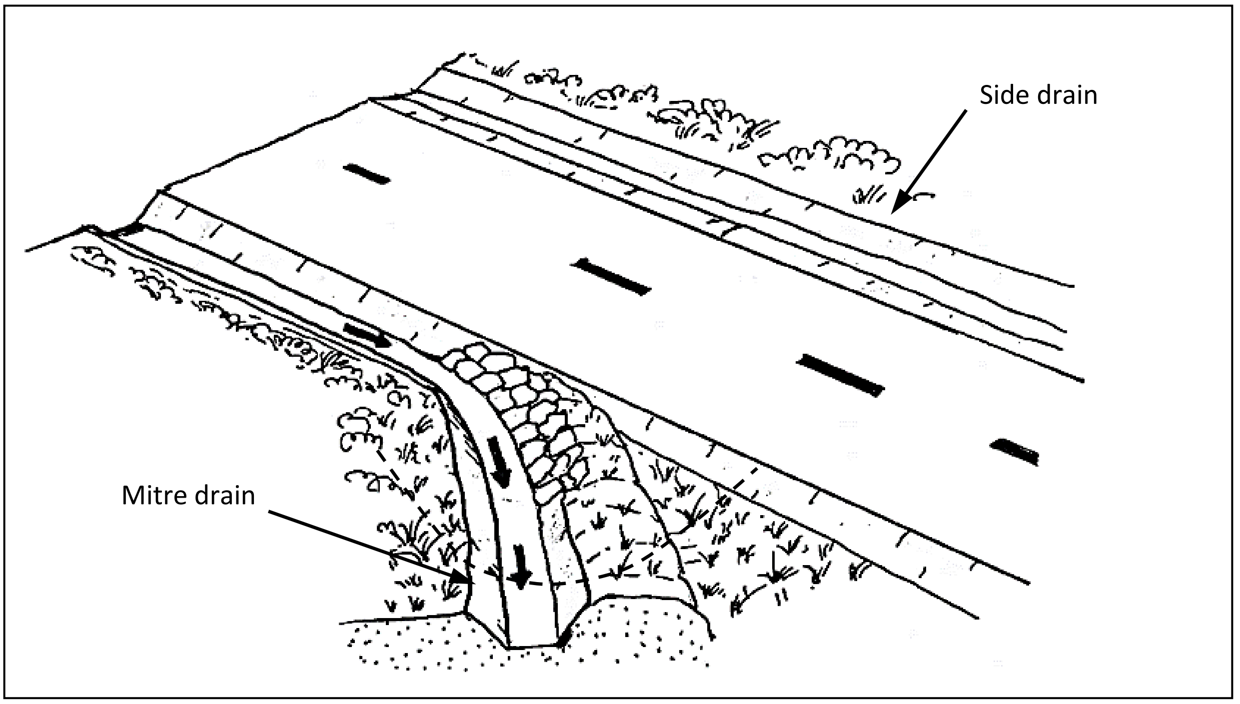


Figure 7-17: Example of a Mitre Drain and Side Drain

All of the road drainage structures require maintenance to remove sediment, and woody vegetation, frequent inspection (i.e. after each rainy season) and remedial work undertaken to arrest any scouring, gully formation, or degradation of the structures.

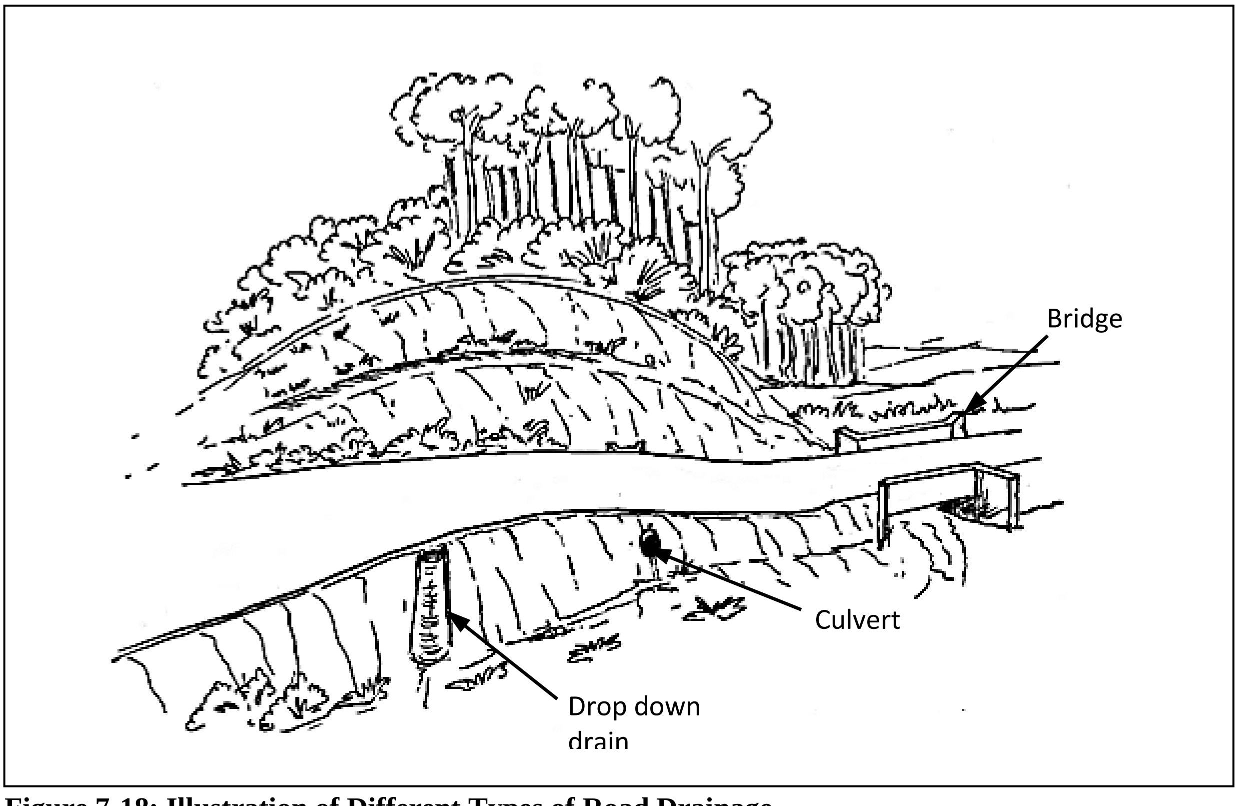


Figure 7-18: Illustration of Different Types of Road Drainage

7.8.4 Controlling Runoff in the Lower Catchment

The runoff that has been discharged from the roadway, through mitre drains or from culverts, should either be:

1) dispersed to soak into the adjacent ground;
2) channelled to an infiltration ditch;
3) channelled in a purpose built waterway to convey the water safely to the natural water course;
4) channelled to a water conservation structure (e.g. pan).

This frequently requires the design of an artificial waterway (e.g. vegetated or stone pitched channel) or outfall channel.

7.9 Stabilisation of Road Embankments

Where a road cuts through an embankment or is placed along the side of a hill, there is typically a steep bare slope on both sides of the road, either rising or falling away from the roadway. The road embankments are prone to erosion and should be stabilised through grassing on and above the slope or through the use of Reno Mats.

7.10 Control of Spillway Discharge

Although most of this chapter has concentrated on erosion control above the proposed storage structure, it is also necessary to consider the downstream erosion risks that the storage structure may generate. Spillways should be designed to minimize erosion and the use of gabions, concrete sills or other energy dissipating structures should be considered to ensure that the storage structure itself does not contribute to erosion within the catchment.

7.11 Financing Conservation Activities

Proper catchment management is essential to the performance and sustainability of water retention structures within the catchment. However, the activities involved are often time consuming and expensive.

Several options are currently available for financing catchment conservation activities. As earlier mentioned, the Water Services Trust Fund offers financial support to WRUAs for the implementation of their sub-catchment management plans.

Other private conservation organizations can also be approached to fund similar activities. Private companies are also a potential source of funding. Proposals can also be developed to incorporate catchment conservation into their corporate social responsibility (CSR) activities.

As many of the activities rely heavily on local labour, prices should be adjusted to reflect current labour costs.

CHAPTER 8

HYDROLOGICAL AND SEDIMENT ANALYSIS

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8 HYDROLOGICAL AND SEDIMENT ANALYSIS

This section presents the hydrological analysis required to support the design of a small dam, pan and other water storage structures. This chapter provides various methods to address the following issues with respect to a proposed project:

1. Is there sufficient inflow?
2. How much storage capacity is required?
3. What are the peak design flows that need to be passed safely by the structure?
4. What are the design flows for diversion or ancillary structures?
5. What are the environmental flow requirements?

8.1 Introduction

The level of hydrological analysis required will depend on the size and nature of the structure being proposed. It is helpful to know what the design criteria are for the proposed structure as this will direct the outputs required from the hydrological analysis.

8.2 The Hydrological Cycle

The hydrological cycle is a conceptual model that describes the continuous cycle of storage and movement of water on, above and below the Earth's surface.

Understanding the hydrological cycle provides insights into the processes involved in the movement of the earth's waters, and this knowledge can be utilised in planning, engineering and water resource management.

Figure 8-1 below illustrates the various stages of the hydrological cycle. The hydrologist's primary focus, in respect of this manual, is on the run-off component, as this provides an indication of the water available for storage.

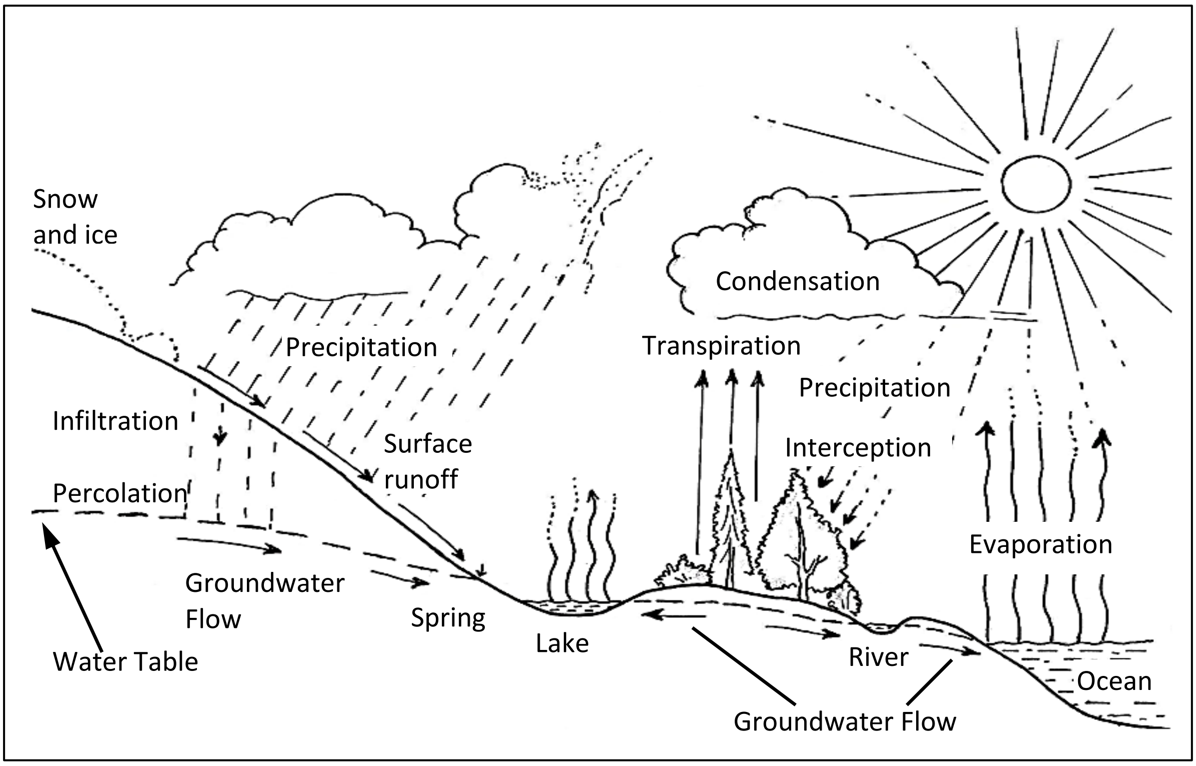

Figure 8-1: The Hydrological Cycle

8.3 Hydrological Design Criteria

Reference should be made to Chapter 2 to determine the Class of Dam based on the Water Resources Management Rules (2007) which also provide minimum design standards for storage structures of different classes as shown in Table 8-1. More conservative design criteria have been recommended herein to allow for various factors including:

- Catchment conditions are changing rapidly in some areas due to rapid urbanisation;
- Climate change predictions indicate rainfall of higher intensity;
- Risks associated with future settlement downstream of the structure.

Table 8-1: Return Period Criteria for Design Purposes

Class of Dam	Minimum Return Period for Design of Spillway (WRM Rules 2007)	Recommended Minimum Return Period for Design of Spillway	Recommended Minimum Return Period for Design of Diversion Works, if required
A (Low Risk)	1 in 50 years	1 in 100 years	1 in 5 years
B (Medium Risk)	1 in 100 years	1 in 100 – 500 years	1 in 10 years
C (High Risk)	1 in 500 years	1 in 1000 years	1 in 15 years

(Source: WRM Rules 2007)

Reference should be made to ICOLD guidelines for Class C dams. The Probable Maximum Flood (PMF), or the most severe flood event reasonably possible in the catchment should be considered during spillway design for Class C dams, but is generally not a requirement for Class A or B dams. However, a critical review of the potential downstream impacts associated with a dam failure should inform the final selection of the hydrological design criteria, even for Class A and B dams.

The design criteria for diversion works requires further discussion. The construction of diversion works can add a significant cost to the overall project cost. However, failure to provide adequate facilities for diversion of flow can impede construction or damage works already constructed.

One way to mitigate the cost of the diversion works is to schedule construction during the dry season or low flow period. In this case diversion works may not be necessary or can be reduced to accommodate minor flood events. In this case, the hydrologist may use the rainfall and flow data strictly from the expected construction period, using the criteria specified in Table 8-1 to determine the required capacity for the diversion works. It should be noted that, if the construction activities do not follow the stipulated construction plan, then any design based on assumptions regarding the construction schedule, will need to be reviewed and appropriate steps taken.

For storage structures other than dams for which design criteria are not specified in the WRM Rules (2007), the hydrological design criteria should be based on the following factors:

- The inflow to the structure should be of sufficient quantity, quality and reliability to justify the investment in the structure;

- The structure should be able to pass an extreme inflow event of a specified return period without damaging the structure or downstream environments. The return period to be used will be a function of the scale of investment and the risk posed to downstream environments. Therefore the return period for a sand dam or sub-surface dam may be a 1 in 50 year event although for an offline pan (not on a water course) a 1 in 10 year event may be appropriate.

8.4 Data Requirements

The quality of the hydrological analysis will depend in part on the availability of reliable data. The data user should expect to prove the reliability and consistency of any data sets he/she has obtained.

In general, the longer the record the better as the data set will reflect more extreme events and provide more confidence in the results. The computing power now commonly available means that data sets containing multiple records of more than 50 years can be analysed with ease. Table 8-2 provides a general listing of the data requirements and possible sources.

Table 8-2: Hydrological and Meteorological Data Requirements

Data Type	Detail	Comments
Rainfall	Location of rainfall stations within or neighbouring the catchment area and length of the record	Obtained from WRMA or KMS This facilitates the identification of which data sets should be pursued.
	Daily (24 hour) rainfall	Number of stations will depend on catchment size but at least 3 to 4 stations within or neighbouring the catchment area. Obtained from WRMA, KMS or from the station itself. Internet based rainfall records are also available, but check reliability and accuracy.
	Mean monthly rainfall	Derived from daily data or obtained from KMS.
	Mean annual rainfall	Derived from daily data or obtained from KMS.
	Annual 24 hour maximum	Derived from daily data or obtained from KMS.
	Rainfall-duration-frequency	Rainfall Frequency Atlas of Kenya (MoWD, 1978) NWMP (1992), NWMP 2030 or from Automatic Weather Stations (KMS, Syngenta, WRMA).
Evaporation	Mean monthly open water evaporation	Obtained from Chapter 3 this Manual or KMS or the FAO CropWat website.
Discharge	Location, reference number of river gauging stations and duration of the record	This facilitates the identification of which data sets should be pursued.
	Approved discharge rating equation(s) for selected RGS	WRMA.
	Daily discharge data	WRMA.
Catchment	Catchment boundaries	Derived from topographical map (obtained from Survey of Kenya) or calculated from a Digital Elevation Model (DEM). WRMA.
	Contours	Derived from topographical map or calculated from a Digital Elevation Model (DEM).
	River drainage network	WRMA.
	Land use map	FAO/Africover website.
	Vegetation cover	FAO/Africover website.
	Soil map	FAO/Africover website.
	Geological maps & Reports	Department of Mines & Geology.
	Cadastral maps	Provides information on land ownership. Survey of Kenya, Ministry of Lands.
	Seismic maps	Department of Mines & Geology.
Water Resource Allocations	Existing authorisations within the catchment	WRMA.

8.5 Data Availability and Reliability

It is generally considered good practice for a hydrologist or engineer who is using hydro-meteorological data to verify the location and on-site conditions for the data that is being used. This eliminates basic problems regarding the datasets.

The time-series data availability should be plotted as shown in Figure 8-2. The data availability chart provides a visual presentation of the available data. The length and consistency of the record can be seen and compared. This helps in the selection of the data for further analysis. Further investigation of the data is required to determine data gaps and to assess the usefulness of the dataset.

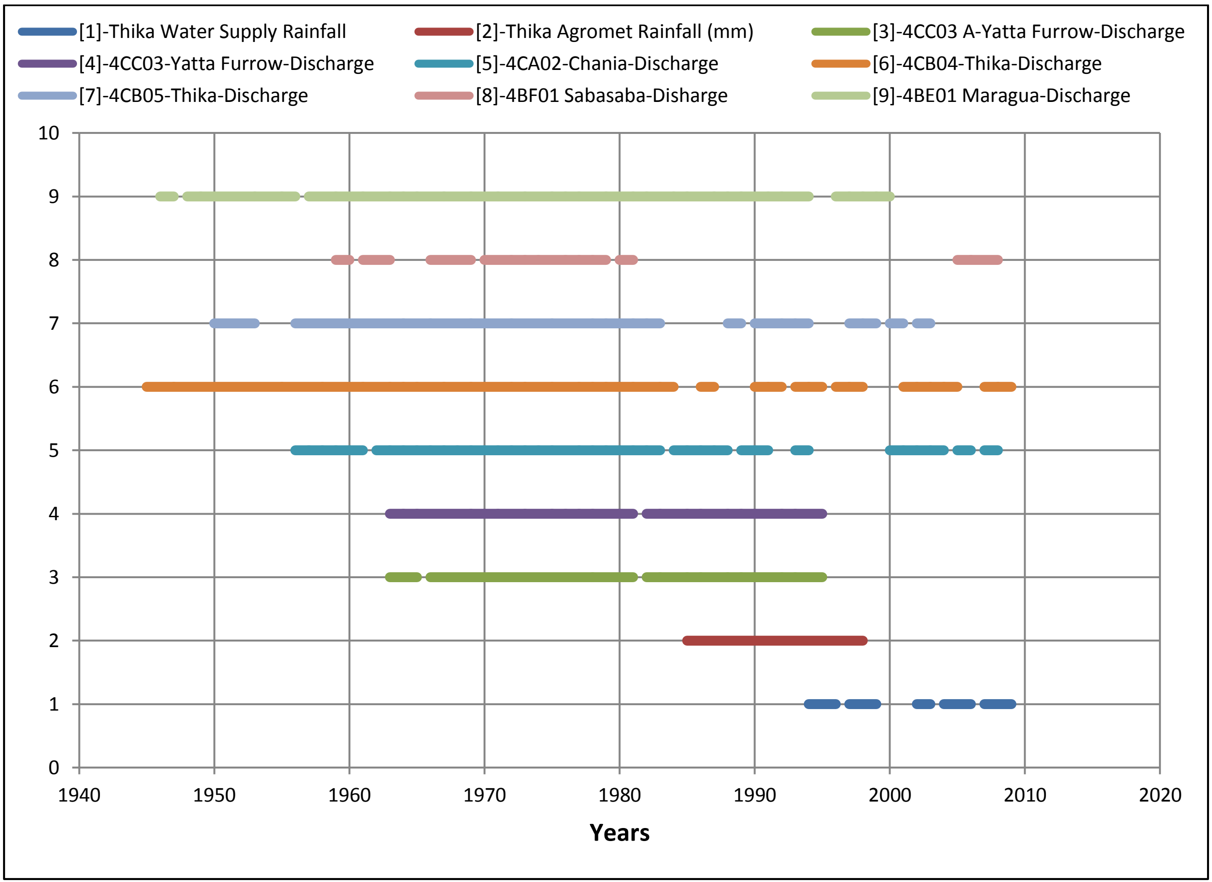

Figure 8-2: Sample Data Availability Chart

No analysis of any data sets should be undertaken until the data has been cleaned and there is confidence that the data provides an accurate record of the measured parameter.

The quickest method to check the reliability of the data is to use double mass plots as set out below:

1. Select two preferred rainfall stations;
2. Eliminate from both records any day in which either of the stations has a missing value;
3. Plot the resultant cumulative daily rainfall of each station on the X and Y axis;
4. Examine the plot. If the relationship between X and Y is reasonably constant (See Figure 8-3) the graph should plot as a straight line of uniform gradient. Change of gradients and undulations reflect an inconsistent relationship and may be explained by inaccurate data in one or both records;
5. Identify the general gradient and then eliminate inconsistent periods from both data sets;
6. Repeat the exercise with different combinations of rainfall stations, paying close attention to which combination provides the longest record of consistent data.

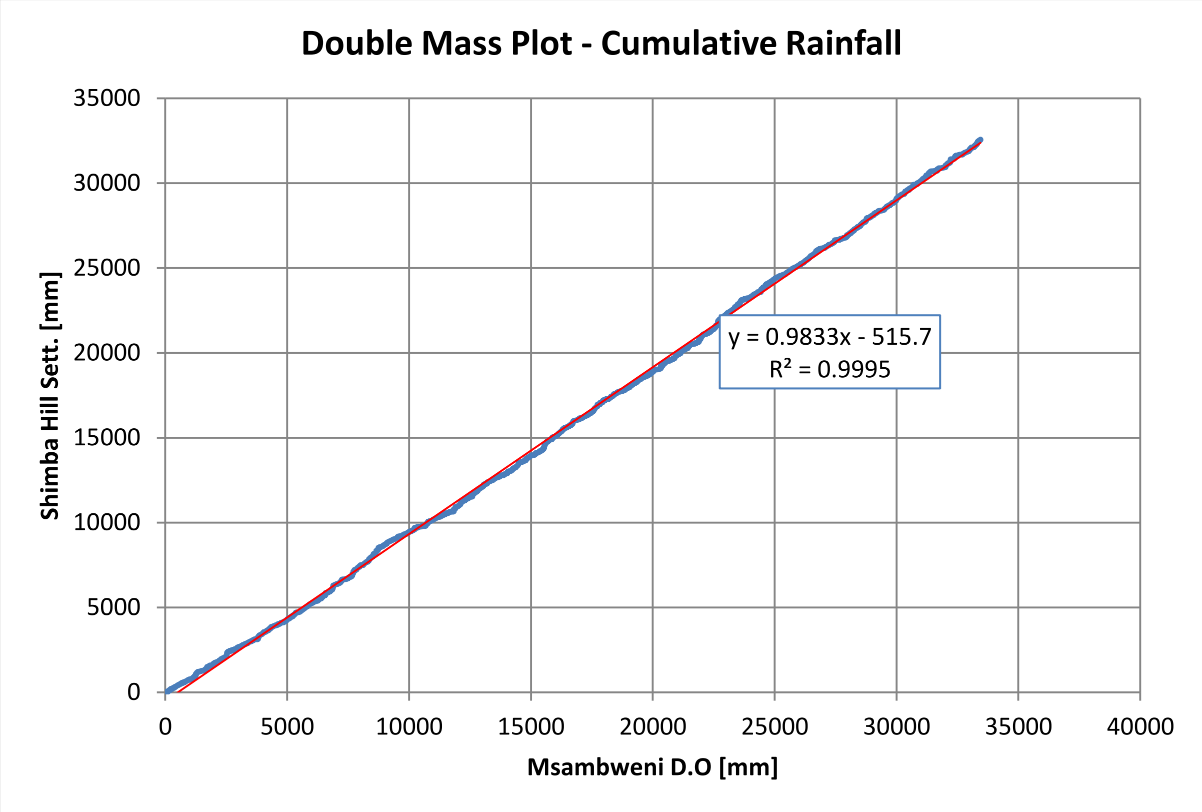

Figure 8-3: Sample of a Double Mass Plot of Two Reliable Rainfall Stations

Once at least one good quality rainfall record has been identified, the double mass methodology can be applied between the good rainfall record and the discharge time series. Again, periods of inconsistent data should be flagged in the discharge time series and removed from the data analysis. The result should be a line of fairly uniform gradient. Note that some variability can be expected due to natural variations in catchment conditions, rainfall location and intensity etc.

In the event that a non-uniform line is obtained, further investigations should be conducted on the discharge data. This will involve the following steps:

- Obtain water level time series from WRMA and review the general pattern, looking for anomalies, trends, extreme values outside the realm of possibility, etc.;
- Review the gauging data against the approved discharge rating equation to establish whether the rating equation matches the gauging data.

The final result of the consistency check should be a set of rainfall and discharge data for which there is confidence in its integrity.

8.6 Rainfall Analysis

Rainfall analysis should be conducted to provide three simple outputs:

1) Mean monthly rainfall. This is used to determine the longest and driest period and is used to estimate storage requirements and to select the best time to schedule construction activities. Median monthly rainfall, being less influenced by extremes, can also be used;
2) A time series of mean annual rainfall and the long term mean value;
3) Rainfall frequency.

It should be noted that the location of the rainfall station(s) should preferably be within the catchment area and upstream of the site under investigation. Appendix A provides a map of mean annual rainfall for the country.

8.6.1 Rainfall Frequency Analysis

Rainfall frequency analysis is an important component in the estimation of peak flows for specific return periods, especially where there are no stream flow records. A basic assumption is made that the return period for a storm corresponds to the return period for flows.

Furthermore, the duration of the storm should correspond to the time of concentration of the catchment (t_c). (See Section 8.10.2 for determination of time of concentration).

The Rainfall Frequency Atlas of Kenya (Ministry of Water Development, 1978) provides rainfall-duration-frequency (RDF) maps for the whole of Kenya for different combinations of storm duration and return periods as shown in Table 8-3. These maps are essential where the time of concentration is significantly less than 24 hours, which is the case for small catchments.

Table 8-3: Storm Duration-Frequency Combinations

	Return Period (Years)				
Duration	**5**	**10**	**25**	**50**	**100**
10 min	✓	✓	✓	✓	✓
30 min	✓	✓	✓	✓	✓
1 hr	✓	✓	✓	✓	✓
2 hr	✓	✓	✓	✓	✓
3 hr	✓	✓	✓	✓	✓
6 hr	✓	✓	✓	✓	✓
12 hr	✓	✓	✓	✓	✓
24 hr	✓	✓	✓	✓	✓

(Source: Rainfall Frequency Atlas of Kenya, 1978)

For larger catchments, the 24 hour rainfall frequency can be established based on the annual maximum 24 hour rainfall series. This time series can be analysed using extreme event distributions to obtain the estimated 24 hour rainfall for different return periods. An open source statistical software (e.g. Easyfit) can be used to fit different probability distributions to the annual maxima data series. Care should be taken to select probability distributions appropriate for rainfall analysis e.g.:

1) Gumbel – EV Type 1;
2) Frechet - EV Type II;
3) Weibul - EV Type III;
4) Log-Pearson Type III;
5) Log Normal Distribution;
6) Wakeby Distribution.

8.6.2 Catchment Rainfall

Rainfall is unlikely to occur equally over a catchment, especially for larger catchments. A single rainfall record has therefore to be adjusted by an area reduction factor (ARF) which is dependent on the size of the catchment as shown in Figure 8-6 and Figure B7.7 in NWMP 1992.

In the event that a number of reliable rainfall records are available for a particular catchment area, then a better estimate of the rainfall over the catchment on a storm, seasonal, or annual basis, can be made by using an area weighting factor as determined by the isohyetal, Thiesen polygon or a distance/gridded method.

8.7 Climate Analysis

The climate analysis is focused on mean monthly open water surface evaporation which is needed to estimate monthly evaporation losses from the dam or pan. See Section 3.3.10 for further details. A map of annual evapotranspiration is also provided in Appendix A.

8.8 Inflow Estimation

The aim is to obtain a fair estimate of the inflow to the storage structure. The method of estimation of the inflow to a reservoir will depend on the available data. A number of methods are set out below.

8.8.1 Estimation of Inflow without Discharge Data

1) Rough Estimate for Roof and Small Runoff Catchments (<5ha)

Monthly estimates of runoff volume from roof areas and small catchments (i.e. less than 5 ha) can be made using Equation 8-1.

Equation 8-1: $$V_m = \frac{C.A.R_m}{1000}$$

Where: V_m = runoff volume in month m [m^3]
C = runoff coefficient (Table 8-4)
A = catchment area [m^2]
R_m = rainfall in month m [mm]

Table 8-4: Runoff Coefficients

Surface Type	Runoff Coefficient (%)
Roof tiles, corrugated sheets, plastic sheets, concreted bitumen	80
Brick pavement	60
Compacted soil	50
Uncovered surface, flat terrain	30
Uncovered surface, slope 0 – 5%	40
Uncovered surface, slope 5 – 10%	50
Uncovered surface, slope >1%	> 50

2) Rough Estimate for Small Catchments (<10 Km^2)

Monthly and annual estimates of runoff are generally estimated to be between 10 – 20% of rainfall for the ASALs. Estimates of the runoff factor can be made from empirical data as shown in Table 8-5 for different catchments in East Africa or from values presented in NWMP 1992.

Table 8-5: Rainfall-Runoff Parameters for Small Catchments in Kenya

Catchments	Area (km^2)	Avg. Annual Rainfall (mm)	Elevation range (masl)	Predominant Land use	Predominant Soil Type	Annual Streamflow as % of rainfall	Runoff Coefficient K for daily rainfall[1]	Threshold T for daily rainfall (mm)
Laikipia[2]								
Mukogodo	2.21	335	1770-1890	Grass-bushland	Red clay loam	11	27.8	6.38
Ngenia	1.07	657	2110-2220	Small scale farming	Red clays	9.1	22.3	7.94
Sirima	3.59	791	1940-2070	Small scale farming/ bushland	Black clays	2.6	10.7	11.9
Kericho[3]								
IJC13 Sambret	7.02	2026	2000-2800	tea	Organic loams	37.8		
IJC14 Lagan	5.44	2129		indigenous forest	Organic loams	34.8		
Mbeya								
A	0.20	1658		smallholder	Organic loams	39.9		
C	0.163	1924		indigenous forest	Organic loams	28.1		
Kimakia								
10 C	0.65	2325	2000 – 3000 2440	Bamboo	Organic loams	45.6		
11 A	0.36	1997		Pines	Organic loams	41.6		
17 (Makiama)	0.37	2062		Grass	Organic loams	48.9		
Atumatak		734						
A	0.811	734	1345-1460	Grass/bush	Shallow, sandy	7.8 – 14.4		
B		759		Bush cleared, fenced		6.9 – 15.6		

[1] Runoff (mm) = K(P-T) where K is a runoff coefficient, P is rainfall (mm) and T is threshold (mm) for runoff to commence.
[2] Ondieki. C 1993. The Hydrological Characteristics of the Arid and Semi Arid Parts of Kenya. PhD Thesis. University of Nairobi.
[3] Blackie J.R, Edwards, K.A, Clarke R.T Hydrological Research in East Africa. *East African Agricultural and Forestry Journal*, Special Issue Vo. 43 1979

3) Multi-parameter Rainfall Runoff Models

There is a wide variety of rainfall runoff models from simple empirical models to complicated multi-parameter models that attempt to simulate hydrological processes. The decision to use multi-parameter models (such as NAM, SWAT, etc) should be made by experienced hydrologists. The use of complicated multi-parameter models requires rainfall and discharge data to calibrate and validate the model and can be data intensive.

8.8.2 Estimation of Inflow with Discharge Data

It is preferable to use discharge data from a reliable river gauging station near the proposed site. It is preferable to use naturalised flow data, although this is generally less significant for flood flow analysis and more significant for low flow analysis and inflow estimation. If the observed flow record is considered to be heavily influenced by abstractions, then use of historical data (i.e. prior to 1985) may reduce the influence of abstractions on the discharge data. The development of naturalised discharge time series would require an accurate time series of abstraction data for all the abstraction points in the catchment under investigation. However, historical discharge data may not adequately reflect current catchment conditions and catchment hydrological response. The hydrologist will need to make a value judgement regarding the most appropriate data to use for analysis.

Reliable discharge data from the study catchment or a similar catchment can be used to establish a discharge time series for the dam/reservoir site. It is reasonable to assume that the hydrological behaviour of two catchments are similar if the topography, rainfall, vegetation, soils, land use type and size are similar. Adjustments can be made for differences in size and rainfall using Equation 8-2.

Equation 8-2: $$Q_{A=\frac{P_A}{P_B}\times\frac{AREA_A}{AREA_B}} \times Q_b$$

Where: Q_A = discharge in catchment A (study catchment) [m^3/s]
Q_B = discharge in catchment B [m^3/s] based on discharge data
P_A = mean annual precipitation in catchment A (study catchment) [mm/yr]
B_B = mean annual precipitation in catchment B [mm/yr]
$AREA_A$ = area of catchment A [Km^2]
$AREA_B$ = area of catchment B [Km^2]

This approach also applies if Catchment A and B are on the same watercourse and are in reasonable proximity (within 20 – 30 km).

8.9 Flow Frequency Analysis

Once a reliable discharge time series for the dam/reservoir site has been established, the probability of equalling or exceeding a particular flow can be established. The flow duration curve (FDC) presents a graphical description of the relationship between the frequency of exceedence and flow, as shown in Figure 8-4, with a spread sheet percentile function applied to the entire daily discharge time series.

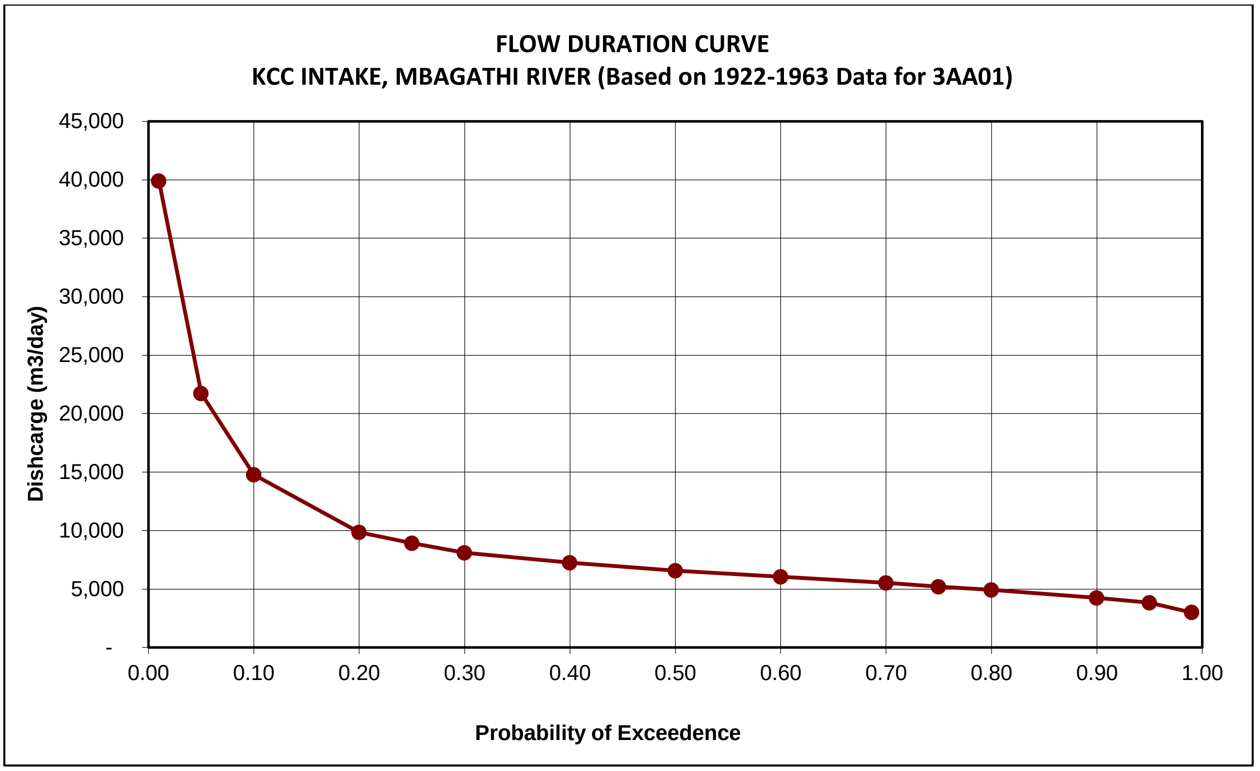

Figure 8-4: Example of a Flow Duration Curve

The FDC provides the hydrological design values for the design of a self regulating weir, a water supply intake and the environmental flows (Reserve) (See Section 8.14 for Environmental Flows).

8.10 Flood Frequency Analysis

The design of spillways, diversion works and storm water channels require an estimation of the peak discharge for a given return period. Insufficient spillway capacity causing overtopping of the embankment with subsequent erosion of the downstream slope is a major cause of embankment failure (breaching) for small earth dams. This indicates the importance of establishing a conservation estimate of the inflow design flood (IDF).

8.10.1 Initial Estimate of Design Flood Flows

The Design Manual for Water Supply (Ministry of Water and Irrigation, 2005) provides estimates (Table 8-6) of the peak discharge rate for a 1 in 100 year return period based on the catchment area. The figures presented in Table 8-6 are typically used as estimates in the initial stages of a project and can be confirmed during the detailed design process with other methods.

Table 8-6: 100 Year Return Period Discharge

Catchment Area (km^2)	Q_{100} ($m^3/s/km^2$)
< 1	15
1 - 5	12 - 10
5 - 25	3 - 6
25 - 100	3 - 2
100 - 1000	1 – 0.4
> 1000	< 0.3

8.10.2 Richard's Method

An empirical method well suited for Kenyan conditions is given by Richards which uses rainfall data and catchment conditions. The method is based on an empirical formula to calculate the "time of concentration" (Tc) of the catchment. This is the time it takes for the rain fallen on the furthest point of the catchment to reach the river at the point where the peak flow is to be estimated. Richard's method takes into account the rainfall pattern and intensity and the catchment characteristics determining its runoff: size, shape and slope, as well as soil and vegetation type (the latter two collated into a run-off factor Kr).

Table 8-7: Runoff Factors for Different Soil Types

Catchment Soil Type	K_r
Rocky and impermeable	0.80 to 1.00
Slightly permeable, bare	0.60 to 0.80
Slightly permeable, partly cultivated or covered with vegetation	0.40 to 0.60
Cultivated, absorbent soil	0.30 to 0.40
Sandy bare soil	0.20 to 0.30
Heavy forest	0.10 to 0.20

The formula for calculating the time of concentration is shown in Equation 8-3 which is resolved iteratively for different values of T_c until the equation balances.

Equation 8-3: $$\frac{T_c^3}{(T_c+1)} = \frac{C.L^2}{K_r.R.S.f(a)}$$

Where: T_c = time of concentration [hours]
L = longest path of the catchment [km]
C = a coefficient, function of (K_r.R) which can be obtained from Figure 8-5
K_r = runoff factor, which can be obtained from Table 8-7
R = rainfall coefficient, where $R = \frac{(t+1)}{t}F$
t = selected storm duration [hours]. This should approximate the time of concentration (T_c).
F = total rainfall [mm] = intensity [mm/hr] x storm duration (t) [hours], for the selected storm duration and frequency. Intensity is obtained from the rainfall intensity maps in Appendix A or from the Rainfall Frequency Atlas of Kenya (KMS).
S = the average slope of the catchment
f(a) = ratio of the average rainfall intensity (i) to the maximum rainfall intensity (I) over the catchment area, obtained from Figure 8-6.
a = the area of the catchment [km^2].

Once T_c has been found, a revised estimate of the average rainfall intensity for the catchment can be established using Equation 8-4.

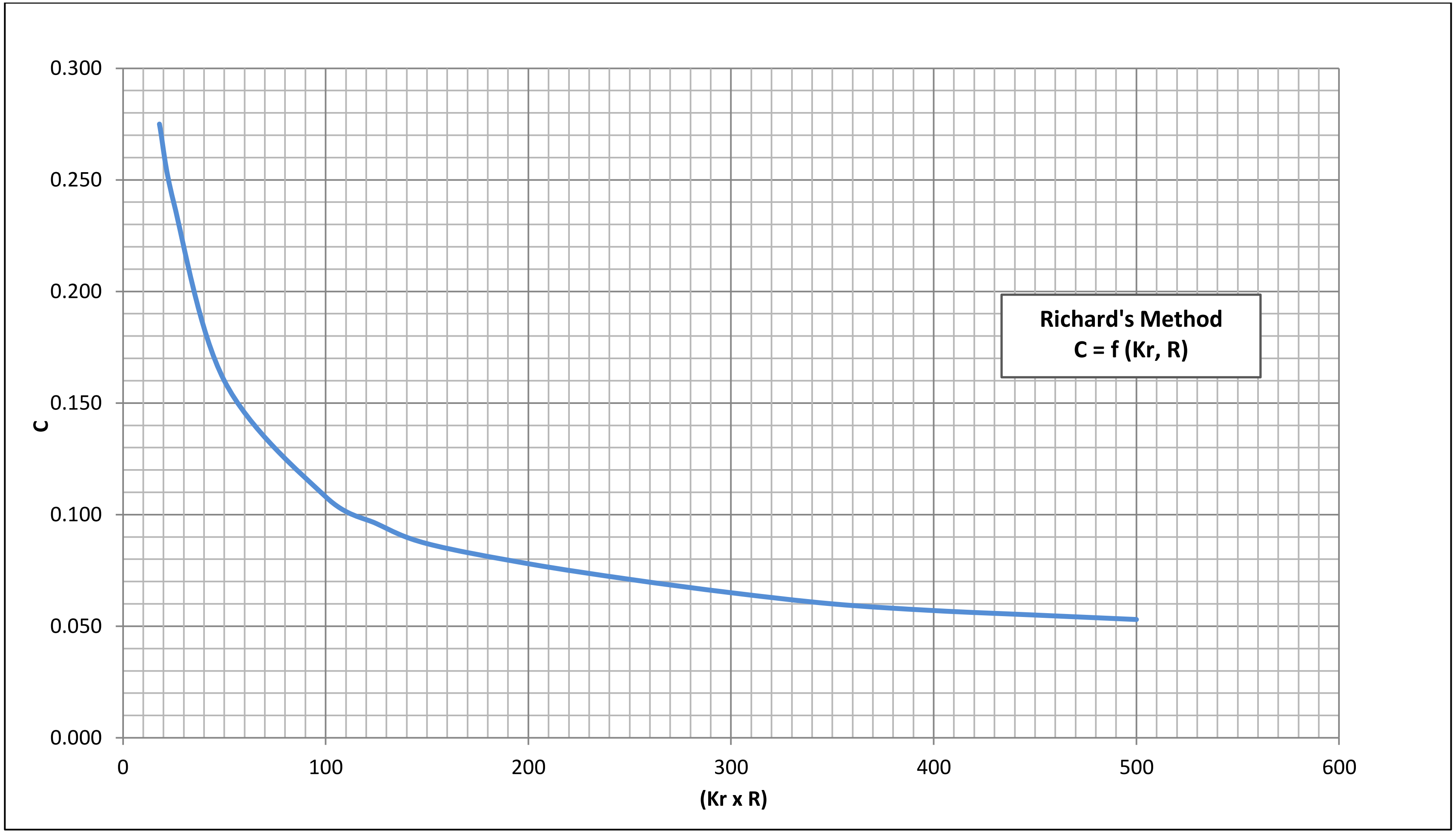

Figure 8-5: Coefficient C, Richard's Method

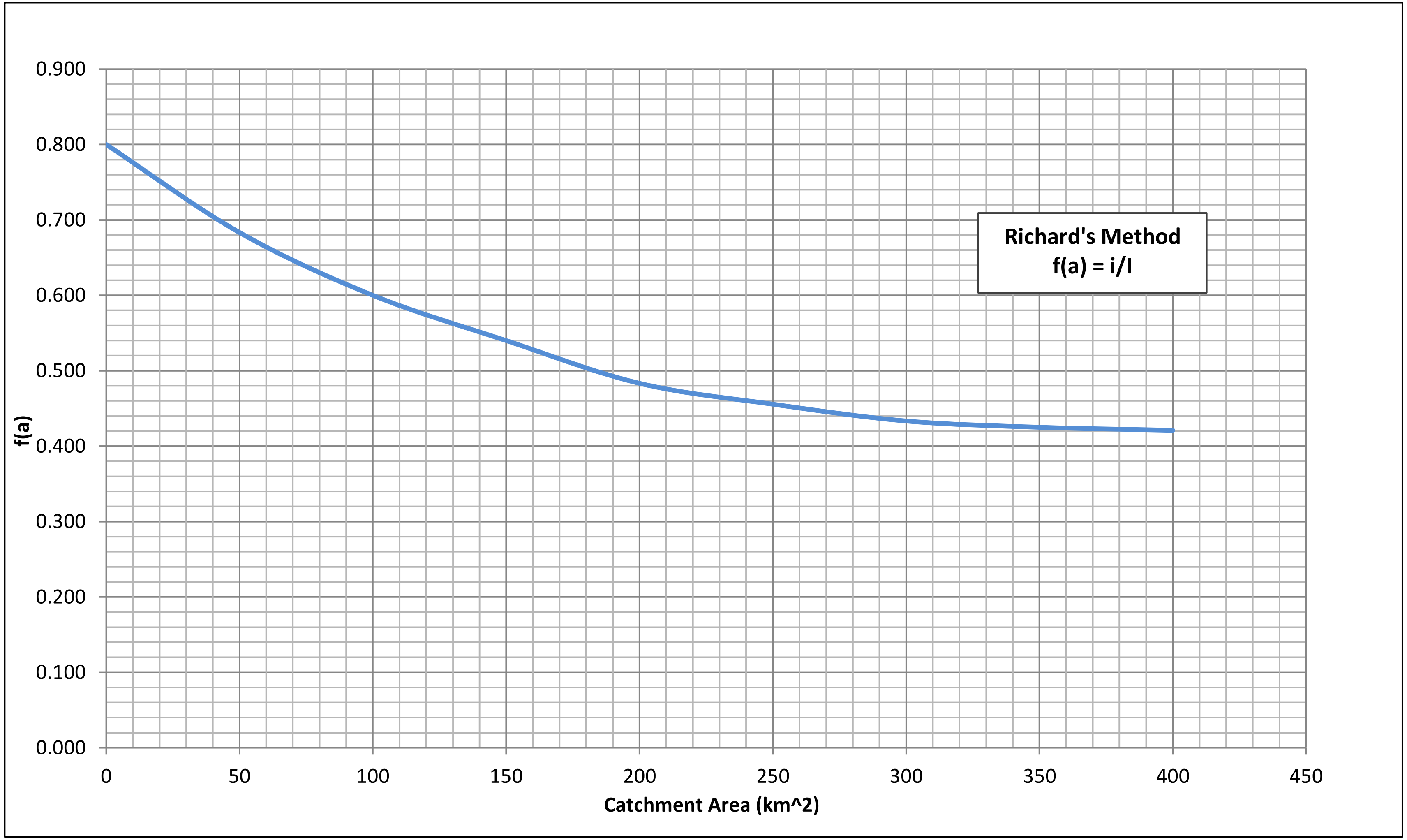

Figure 8-6: Rainfall Intensity Factor, Richard's Method

Equation 8-4 $\boldsymbol{i = I_{max} . f(a)}$

Where: i = average rainfall intensity [mm/hr] over the catchment
I_{max} = maximum rainfall intensity = $\frac{R}{Tc+1}$[mm/hr]
f(a) = ratio of the average rainfall intensity (i) to the maximum rainfall intensity (I_{max}) over the catchment area, obtained from Figure 8-6.

Finally the rational formula is used to calculate the expected maximum flow as shown in Equation 8-5.

Equation 8-5: $\boldsymbol{Q_p = \frac{K_r.i.a}{3.6}}$

Where: Q_p = peak flood flow [m^3/s] with return period equal to that of the selected storm and
i = average rainfall intensity [mm/hr] over the catchment
a = the area of the catchment [km^2].

8.10.3 TRRL and East African Flood Prediction Method

The TRRL and East African Flood Prediction Model as described in the TRRL Laboratory Report No. 623[4]. It can be used for catchments where Richards Method is not considered appropriate.

8.10.4 SCS Method

The US Soil Conservation Service (SCS) Rainfall/Runoff Model was developed in the USA and is widely used globally. The model uses rainfall data, catchment and soil conditions to estimate discharge.

The SCS Model assumes that there are two stages in modelling the response to rainfall:

1. Initial retention of rainfall.
2. Infiltration and runoff.

During Stage 1, there is no runoff. Once the “Initial Retention” storage is filled, Stage 2 commences. Both stages are functions of land use, soil infiltration capacity, depression storage, and antecedent soil moisture.

a) Initial Retention

The Initial Retention is computed by Equation 8-6 and Equation 8-7

Equation 8-6 $\boldsymbol{I_a = 0.2S}$

Equation 8-7 $\boldsymbol{S = 254\ (\frac{100}{CN} - 1)}$

Where: I_a = Initial Retention [mm]
S = Potential Maximum Retention [mm] calculated using Equation 8-7
CN = Runoff Curve Number as established from Table 8-9.

CN values are published for various land cover conditions, various antecedent soil conditions and for various hydrologic soil groups.

[4] Fiddes, Forsgate and Grigg 1974. The Prediction of Storm Rainfall in East Africa. TRRL Laboratory Report 623, Transport and Road Research Laboratory, Department of the Environment, UK.

b) Soil Group Classification

SCS developed a soil classification system that consists of four soil groups whose soil characteristics are provided in Table 8-8.

Table 8-8: SCS Method Hydrologic Soil Groups

Hydrologic Soil Group	Drainage characteristics
Group A	Deep sand, deep loess, aggregated silts
Group B	Shallow loess, sandy loam
Group C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay
Group D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils

(Source: McCuen, US SCS)

c) Curve Numbers for Different Cover Complexes and Hydrological Soil Groups

Table 8-9 provides curve numbers (for AMC II conditions) for different cover conditions and soil groups.

Table 8-9: Runoff Curve Number (CN) for AMC II

		Hydrologic Soil Group			
Land use	**Detail**	**A**	**B**	**C**	**D**
Cultivated land	Without conservation treatment	72	81	88	91
Cultivated land	With conservation treatment	62	71	78	81
Pasture or rangeland	Poor condition	68	79	86	89
Pasture or rangeland	Good condition	39	61	74	80
Meadow	Good condition	30	58	71	78
Wood or forest land	Thin stand, poor cover, no mulch	45	66	77	83
Wood or forest land	Good cover	25	55	70	77
Open spaces, lawns, parks	Good condition : grass cover >75% of areas	39	61	74	80
Open spaces, lawns, parks	Fair condition : grass cover 50 - 75% of areas	49	69	79	84
Commercial and business areas	≈ 85% impervious	89	92	94	95
Industrial areas	≈ 72% impervious	81	88	91	93
Roads	Paved	98	98	98	98
Roads	Gravel	76	85	89	91
Roads	Dirt	72	82	87	89

Fallow	Bare soil	77	86	91	94
Fallow	Crop residue	76	85	90	93
Fallow	Good	74	83	88	90
Row crops	Poor	72	81	88	91
Row crops	Good	67	78	85	89
Woods - Bush	Poor	57	73	82	86
Woods – Bush	Fair	43	65	76	82
Woods – Bush	Good	32	58	72	79

(Source: McCuen, US SCS)

In catchments with different cover conditions, it is possible to develop a Curve Number (CN) representative of the catchment based on an area weighted method in which the curve numbers for different parts of the catchment are established and the respective area determined.

d) Antecedent Soil Moisture Condition

The antecedent moisture factor has a significant effect on runoff and SCS developed three antecedent soil moisture conditions, as shown in Table 8-10. AMC II is typically applied.

Table 8-10: Antecedent Moisture Condition

Antecedent Moisture Condition (AMC)	Soil Condition
AMC I	Driest soil conditions (i.e. wilting point)
AMC II	Average conditions
AMC III	Wettest soil conditions (i.e. > field capacity)

e) Curve Number Estimation

Typical runoff curve numbers are tabulated in Table 8-8 for AMC II (average conditions). These can be adjusted according to the moisture conditions (Haith 1985). AMC III conditions would apply for periods of high rainfall or for flood analysis where the five day antecedent precipitation would be expected to exceed 53mm. AMC I would be applicable for dry conditions where five day antecedent precipitation is less than 35 mm.

Equation 8-8: $$CN(III) = \frac{CN(II)}{(0.4036+0.0059CN(II))}$$

Equation 8-9: $$CN(I) = \frac{CN(II)}{(02.334-0.01334CN(II))}$$

Where: **CN(I) = curve number under AMC(I) conditions**
CN(II) = curve number under AMC(II) conditions
CN(III) = curve number under AMC(III) conditions

f) Discharge Estimation

The runoff is computed by Equation 8-10:

Equation 8-10 $$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \times A \times 10^6$$

Where: Q = discharge [mm]
P = Storm catchment averaged rainfall [mm]
S = Potential Maximum Retention [mm] calculated using Equation 8-7
A = Catchment area [Km^2]

8.10.5 Flood Frequency Analysis Based on Discharge Data

Whenever possible, the determination of the expected peak flows should be based on actual and accurate stream flow data. When such data are available, a flood frequency analysis based on the annual maximum discharge series should be can be undertaken. Care should be taken to use the hydrological year (October – September), not the calendar year (January – December) to establish the annual maximum series. An open source statistical software (e.g. Easyfit) can be used to fit different probability distributions to the annual maxima data series. Care should be taken to select probability distributions appropriate for discharge analysis e.g.

a) Gumbel – EV Type ;
b) Frechet - EV Type II;
c) Weibul - EV Type III;
d) Log-Pearson Type III;
e) Log Normal Distribution;
f) Wakeby Distribution.

8.11 Probable Maximum Flood (PMF)

The analysis of probable maximum flood is not essential for small dams considered in this manual. However in the event that the small dam poses a significant risk to downstream populations, the PMF analysis may be required to provide additional information for the spillway design.

The estimation of the probably maximum flood requires an estimate of the probable maximum precipitation based on the annual maximum rainfall series. The Hershfield Formula[5] (Equation 8-11) can be used to calculate the PMP or values can be obtained from the isohyetal map of PMP in the NWMP (1992).

[5] Guide to Hydrological Practices, Fifth Edition 1994,WMO-No. 168, Data Acquisition and Processing, Analysis, Forecasting and Other Applications, Pages 416 and 417

Equation 8-11: $\mathbf{PMP = \overline{P} + K.Sp}$

Where: $PMP =$ Probable Maximum Precipitation
$\overline{P} =$ Mean of Maximum Annual Precipitation
$Sp =$ Standard Deviation
K = Number of Standard Deviations that must be added to the mean to obtain PMP
K is obtained from Figure 8-7.

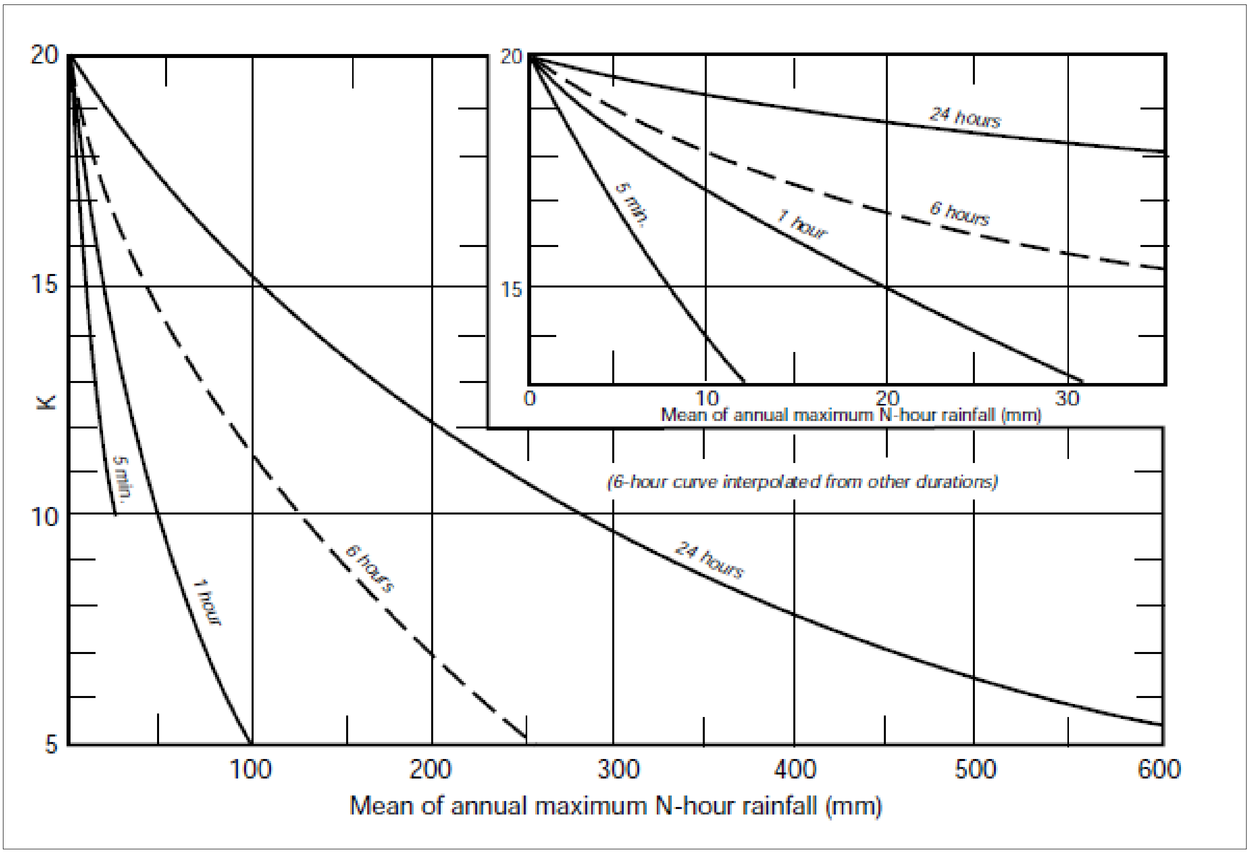

Figure 8-7: K as a Function of Rainfall Duration and Mean of Annual Maximum Series

The National Water Master Plan (MWI 1992) presents a relationship between PMF, PMP and Catchment Area[6].

[6] National Water Master Plan Sectoral Report (B) Hydrology, January 1992 by Japan International Cooperation Agency. Page BF-48, Figure B7.9

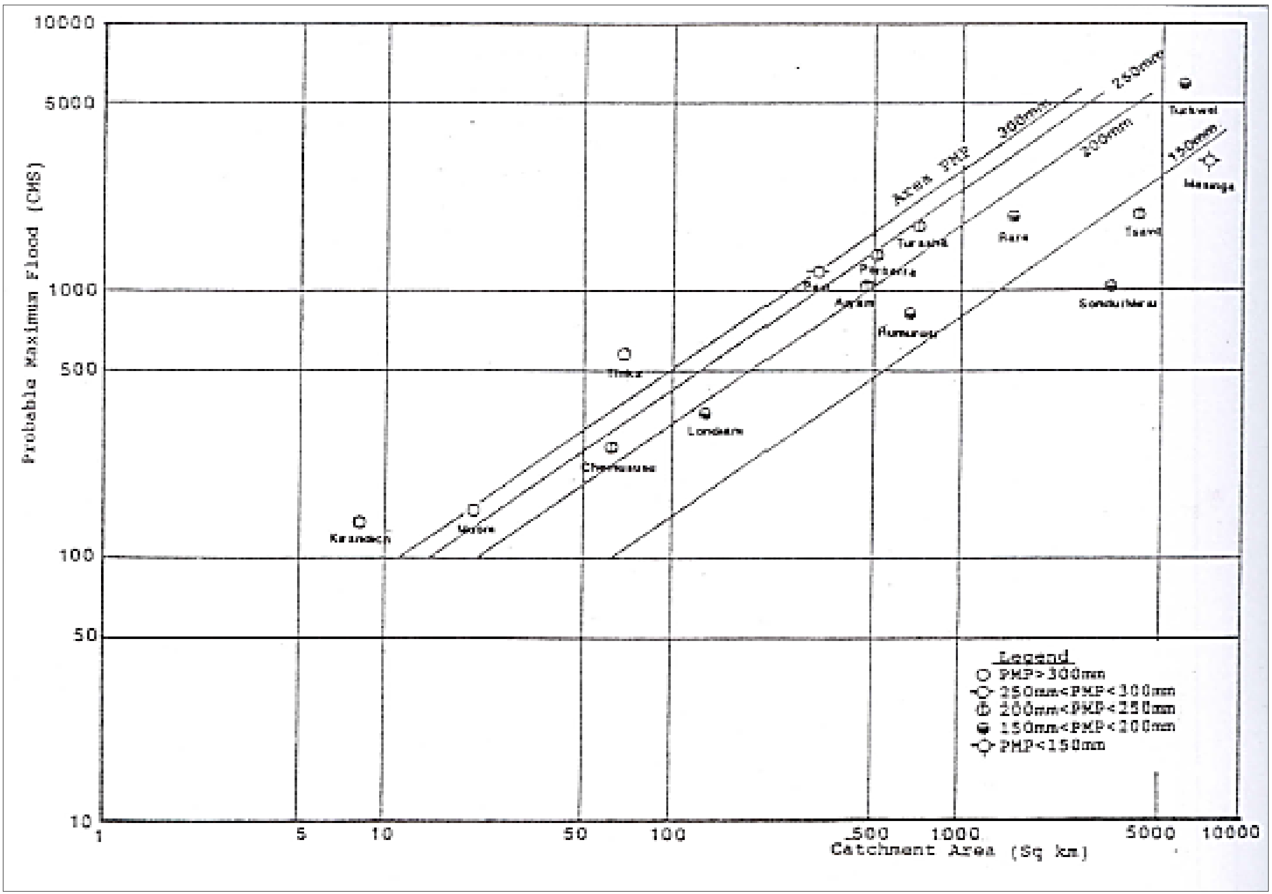

Figure 8-8: Relationship between PMF, PMP, and Catchment Area

8.12 Determining Storage Requirements

The total storage required is the summation of the active and dead storage requirements.

In general the active storage should be sufficient to meet the demand (with an acceptable level of reliability) and losses for the duration of the dry season until there is inflow in the rainy season to refill the storage.

In the semi-arid and arid areas, it cannot be ruled out that one of the two usual rainy seasons does not yield sufficient runoff to replenish the reservoir storage, and that consequently long periods will have to be catered for.

However, for most rivers in the rural areas, no stream flow data might be available and an educated guess towards the length of the dry period will have to be made. Table 8-11 presents estimates of the length of the dry season for different rainfall regimes from MWI (2005). Interviews with local residents will also provide good information regarding the length of the dry period.

Table 8-11: Estimated Length of Dry Period

Mean Annual Rainfall (mm)	Length of Dry Period (month)
1200-1000	5
1000-800	7
800-400	9
<400	11

In case no precipitation data from rainfall stations in the considered catchment area is available, comparison should be made with other similar catchments.

In the case of reservoirs for which the principal purpose is to provide water for nomadic populations and their livestock, it is not always desirable to create reservoirs where water is available on a year round basis. This might provoke a considerable concentration of people and livestock in the concerned area, in turn generating problems such as overgrazing and ecological degradation of the catchment area. The notion of “storage reliability" as outlined above is therefore not applicable in these cases and the storage structure should be designed to provide water for only part of the dry season.

8.12.1 Dry Season Storage

In such cases the graphical method outlined below can still be useful to evaluate the following:

1) Length of the dry period for which a given level of supply can be maintained;
2) Which level of supply can be maintained during a given dry period;
3) Assessing the relative importance of the evaporation losses compared to the water consumption.

The simplified graphical method for determining reservoir storage capacity is illustrated in Figure 8-9.

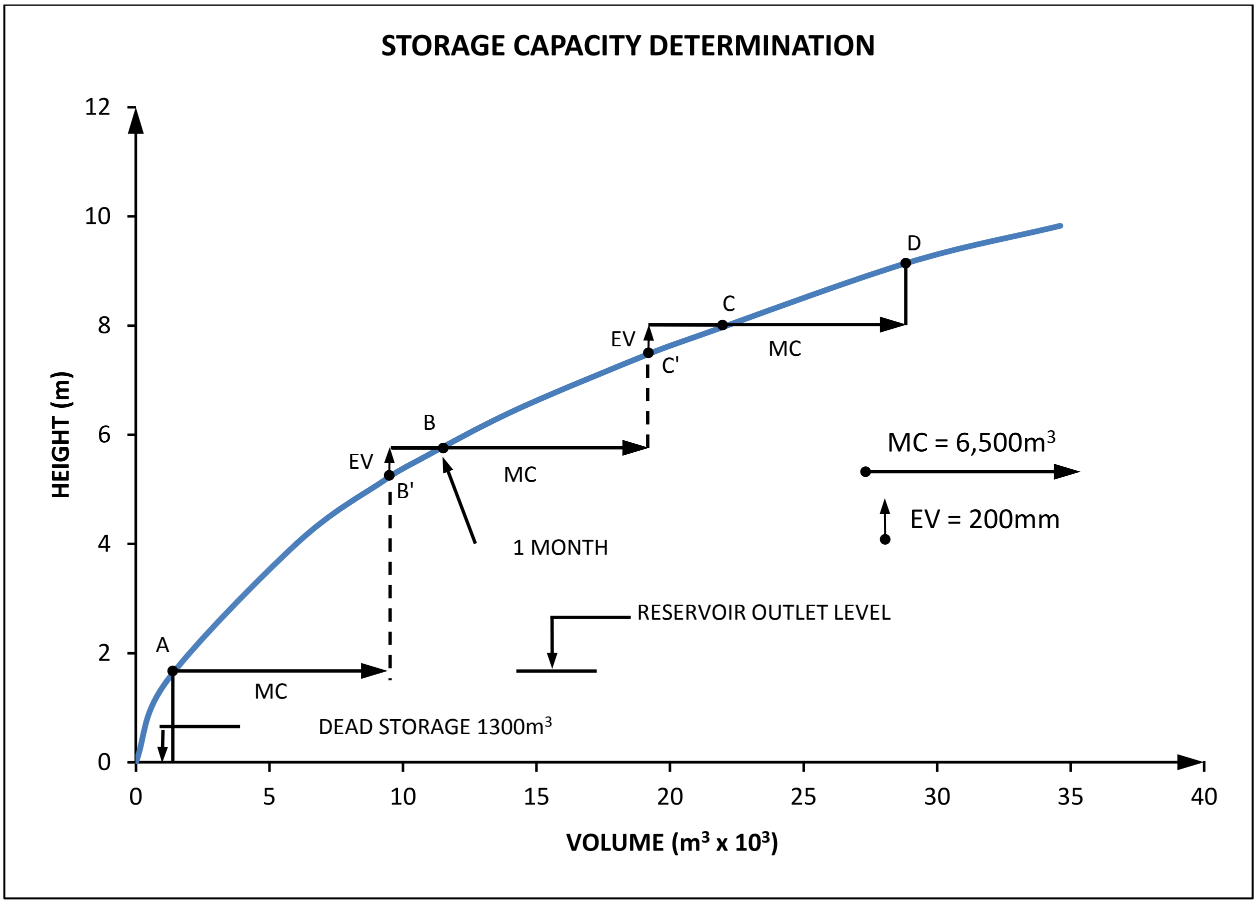

Figure 8-9: Storage Capacity Determination

The following input parameters are required:

1. A Height / Storage Capacity Curve of the reservoir;
2. Monthly water demand or anticipated monthly water consumption (MC);

3. The volume taken up by the "Dead Storage" in the reservoir. This is equivalent to the volume of water stored below the level of the lowest draw-off, and is usually very small for the type of dams and draw-off-systems considered;
4. Monthly flow (if any) in the river during the concerned dry period;
5. Monthly evaporation (EV) losses. Averages based on annual figures obtained from NWMP 1992 can be used as a general guideline;
6. The quantity of sediment expected to be retained by the reservoir during its design life period (See Section 8.13).

The graphical method proceeds as follows: (See Figure 8-9)

1) The level (or alternatively the storage volume) corresponding with the dead storage is set out from point O on the Height/Storage Capacity curve (Point A).

2) From point A, the monthly water consumption (MC) - where applicable reduced by the monthly inflow in the reservoir - is set out horizontally, and transported back to the curve (point B'). From point B', the monthly evaporation (EV) is set out vertically and transported back to the curve (Point B). Point B indicates the storage requirements for one month or for the last month of the dry period, excluding provisions for sediment storage.

3) From point B, the above procedure is repeated, until eventually the storage required to cater for the whole length of the dry period is established.

Since evaporation losses are expressed in millimetres of height, the volume of water lost through evaporation is directly related to the surface of the reservoir. The outlined method determines the evaporation losses at the beginning of each month, and therefore slightly over-estimates the losses. However, this compensates for a number of other minor water losses (infiltration, seepage etc.) which are not taken into account. It must be clear that from the point of view of evaporation losses, for a given storage capacity, deep reservoirs are preferable to shallow ones.

In addition to the storage requirements obtained as outlined above, it is common practice to provide some additional reservoir capacity to cater for sediment storage.

8.12.2 Mass Diagram Analysis

A quick check should be made to ensure that the sum of the average annual water demand (including evaporation and seepage losses) in cubic metres per day is less than the mean annual flow [m^3/day] as it is not possible to extract more water from the catchment than the estimated inflow.

The Rippl Curve (Loucks et al, 1981) provides a graphical method to establish the required storage based on average monthly values for demand, evaporation and seepage losses. The analysis is more easily done using the mean monthly inflows. The procedure is as follows:

1) Plot the cumulative mean monthly inflow volume for 24 months by repeating the 12 month average monthly inflow sequence;
2) Draw a straight line tangential to the cumulative inflow curve. This line should have a gradient equal to the combined average monthly evaporation, seepage and demand (m^3/month);
3) Establish the maximum difference between the cumulative mean inflow line and the cumulative demand line. This difference represents the active storage required in cubic metres.

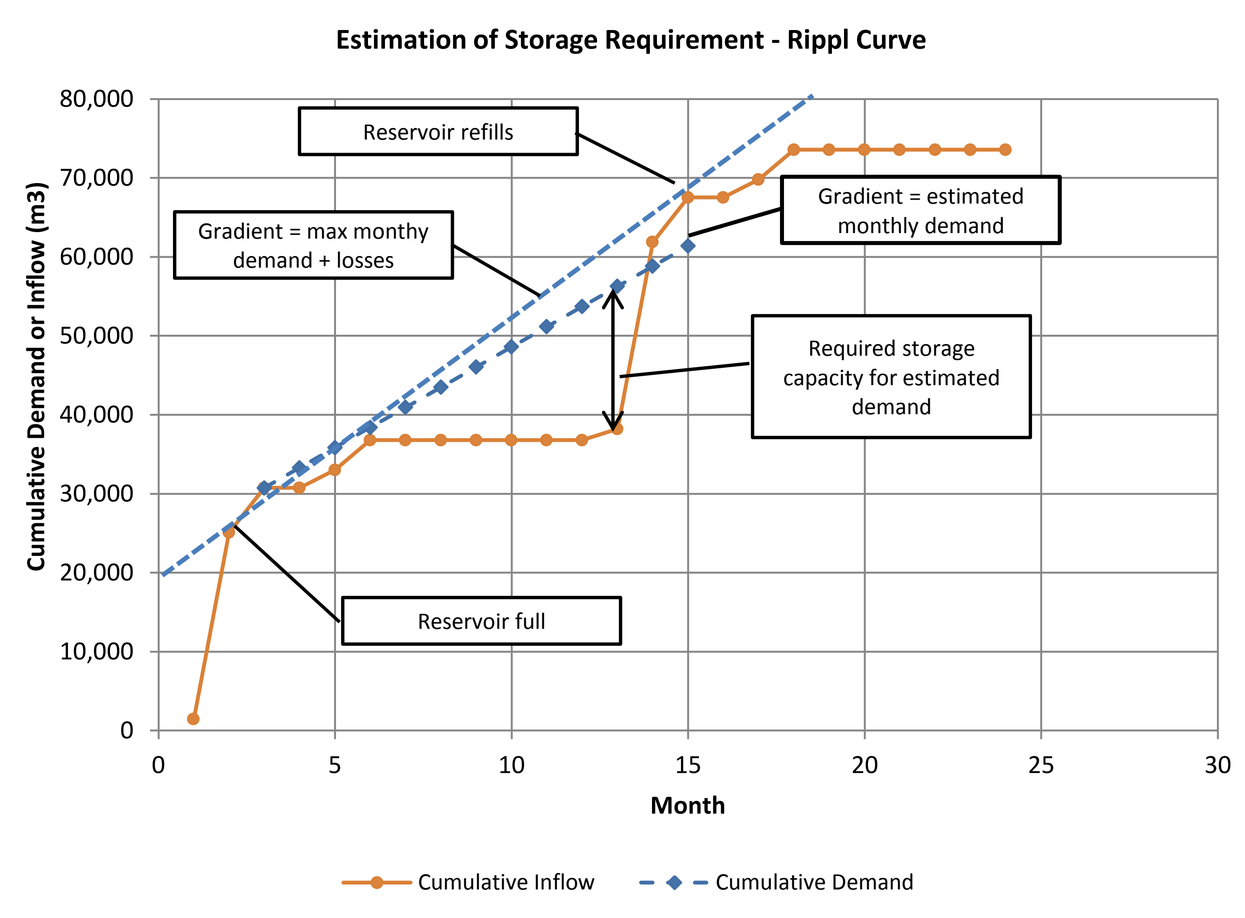

Figure 8-10: Mass Diagram Analysis – Rippl Curve

The method provides a quick way to ascertain the scale of storage that is required. The shortcoming with the Rippl curve methodology as presented above is that it uses mean monthly flow values and a uniform demand value.

If monthly demand values vary, then a modification is to plot the cumulative difference between monthly inflow and the combined monthly release (demand, evaporation and seepage).

8.12.3 Sequent Peak Analysis

The Sequential Peak Method (Loucks 1981) provides an alternative graphical method to establish the required storage. A quick check should be made to ensure that the sum of the average annual water demand (including evaporation and seepage losses) in cubic metres per day is less than the mean annual flow [m^3/day] as it is not possible to extract more water from the catchment than the estimated inflow.

Sections 8.8 and 3.3 provide the methodologies for estimating the inflow to the storage structure and the demand and losses (evaporation and seepage) respectively.

The analysis is more easily done using the mean monthly demand and flow values. A data set should be established for a 24 month period by repeating the sequence of mean monthly values. This takes care of the case where the critical sequence occurs across the end of the first 12 month period.

This method tracks the positive values of the storage required in any given time period which is the sum of the storage required at the end of the previous time period plus the demand (and losses) minus the inflow. This can be represented mathematically as shown in Equation 8-12.

Equation 8-12: $K_t = Max\,[(D_t + E_t + S_t - Q_t + K_{t-1}), 0]$

Where: K_t = storage required at the end of time period t [m^3]. K_0 is equal to zero
D_t = Demand in period t (inclusive of environmental releases) [m^3]
E_t = Evaporation losses in period t [m^3]. Note that this equals zero for a covered tank
S_t = Seepage losses in period t [m^3]. Equals zero for a covered tank and lined pan
Q_t = Inflow in period t [m^3]

The storage volume required (K_{max}) is the maximum value of K_t across the period of analysis.

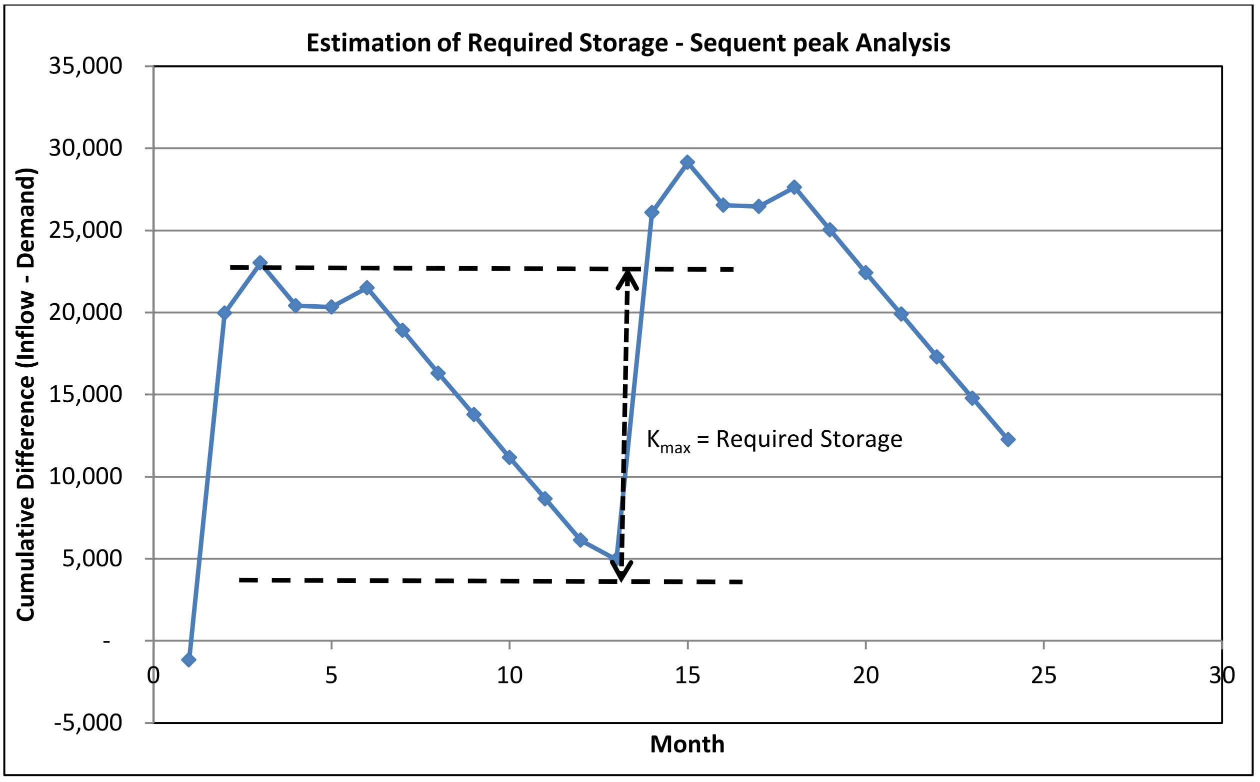

Figure 8-11: Example of Sequent Peak Method

Some consideration needs to be given to the values for evaporation losses (E_t) and seepage losses (S_t) in each time period because the value is actually dependent on the water depth in each given time period. Section 3.3.10 and 3.3.11 provide a method for estimating a maximum value for these parameters which can be used across all the time periods.

8.12.4 Reservoir Simulation

Computer simulation models, based on a daily or monthly time step, can be used to simulate the inflow, spills, demands and losses of a reservoir based on a simple water balance approach. A computer simulation model provides a useful tool to analyse the required storage, yield reliability and release rules. Various open source reservoir simulation models are available including HEC-ResSim.

The basic water balance equation for reservoir simulation as a daily time-step is shown in Equation 8-13.

Equation 8-13: $S_{t+1} = S_t + P_t + I_t - E_t - Seep_t - R_t - Spill_t$

Where: S_t = Storage on day t [m^3]. This is established from the HVA curve for the given depth on day t.

P_t= Precipitation on reservoir area on day t [m^3]. The area on day t is a function of the depth on day t which is determined from the HVA curve.
I_t = Inflow into reservoir on day t [m^3].
E_t= Evaporation from reservoir surface area on day t [m^3]. The area on day t is a function of the depth on day t which is determined from the HVA curve.
$Seep_t$ = Seepage from reservoir on day t [m^3].
R_t = Release from reservoir on day t [m^3]. This may include releases to meet environment flow requirements and to meet water demand.
$Spill_t$ = Spillway overflow on day t [m^3]. This is a function of the water depth above spillway crest and the hydraulic properties of the spillway control and channel.

It should be noted that the water balance equation can be modified appropriately for use with a monthly time-step.

A simulation model provides an easy tool for scenario analysis which may include:

- Establishing the expected length of the filling period;
- Reservoir performance in exceptionally dry or wet periods;
- Reservoir performance under different release rules.

The reservoir performance can be assessed against different indicators for different objectives which may include:

- Drawdown;
- Reliability of meeting specified releases;
- Spillway flows.

Use of a reservoir simulation model requires some consideration of the initial conditions. If the analyst aims to analyse the normal operating conditions of the reservoir, then it is advisable to start the simulation with the reservoir storage level at a level appropriate to the starting time period. For example, if the simulation starts at the end of the rainy season, then the storage level could reasonably be expected to be at the spillway level.

8.13 Sedimentation

Every river and water course carries some suspended sediment and moves larger solids along the stream bed as bed load. When sediment-laden water reaches a reservoir, the larger suspended particles and most of the bed load are deposited as a delta at the head of the reservoir (See Figure 8-12). Smaller particles remain in suspension longer and are eventually deposited farther down the reservoir or may even pass the dam with water discharged through the spillway.

The design of a reservoir should consider the need for and arrangements for sediment scour works. The first step is to estimate the extent of sediment inflow into the reservoir and the impact that may have on the functionality of the reservoir. If the sediment inflow is high, then a mechanism to remove the sediment, including the option of sediment scour works, can be considered. It should be noted that removal of sediment by means of a scour pipe is not straightforward because the heavy sediments lie at the head of the reservoir and sediments consolidate over time.

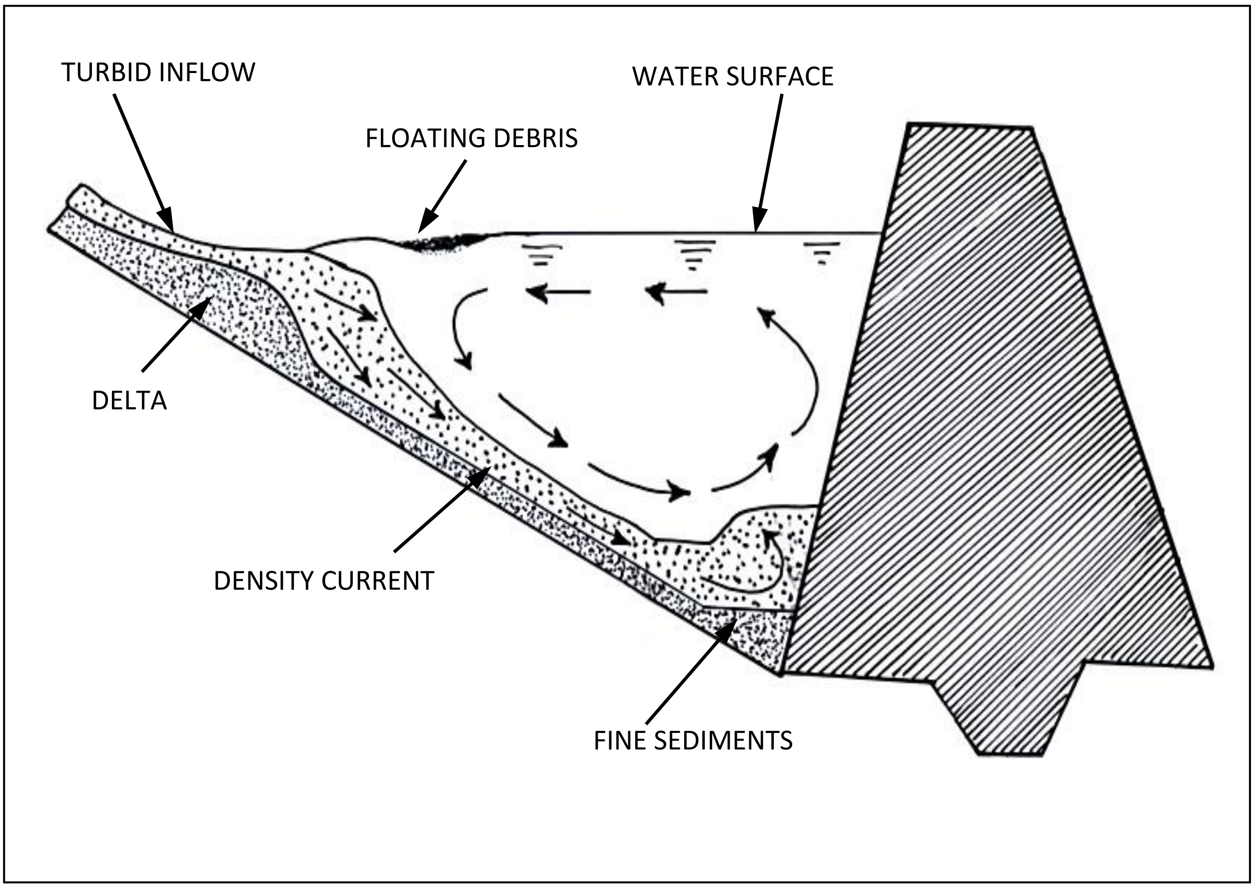

Figure 8-12: Sediment Accumulation in Reservoirs

The quantity of sediment transported in a river depends on many factors that include the rate of erosion within the catchment area, sediment deposition within the catchment, and flow conditions in the water course. The rate of erosion of a particular catchment may vary from flood to flood, with variations in rainfall intensity, soil condition, and vegetal development.

Especially in the arid and semi-arid areas sedimentation is generally of considerable significance in the design, construction and rehabilitation of small dams and pans in Kenya[7].

Some representative figures for the sediment yield of catchments in Kenya are given in Table 8-12. These figures can be considered as conservative values of sediment yield.

Table 8-12: Indicative Sediment Yields

Erosion Rate	**Sediment Yield (m^3/km^2/year)**
Low	500
Moderate	1000
Heavy	1500

Source: MWI 2005

The trap efficiency of a reservoir is defined as the percentage of in flowing sediment which is retained in a reservoir. The trap efficiency is a function of the ratio of reservoir capacity to total inflow[8]. Hence, the trap

[7] Specific weights of settled sediments seem to vary with the age of the deposit and the character of the sediment. Specific weights of sediments range from 7 to 15 kN/$m^3$with an average of about 10 kN/m^3 for fresh sediments and 13 kN/ m^3 for old sediments

efficiency of a reservoir decreases with age as the reservoir capacity is reduced by sediment accumulation. Figure 8-13 may be used to estimate the percentage of the annual sediment load which a reservoir will trap.

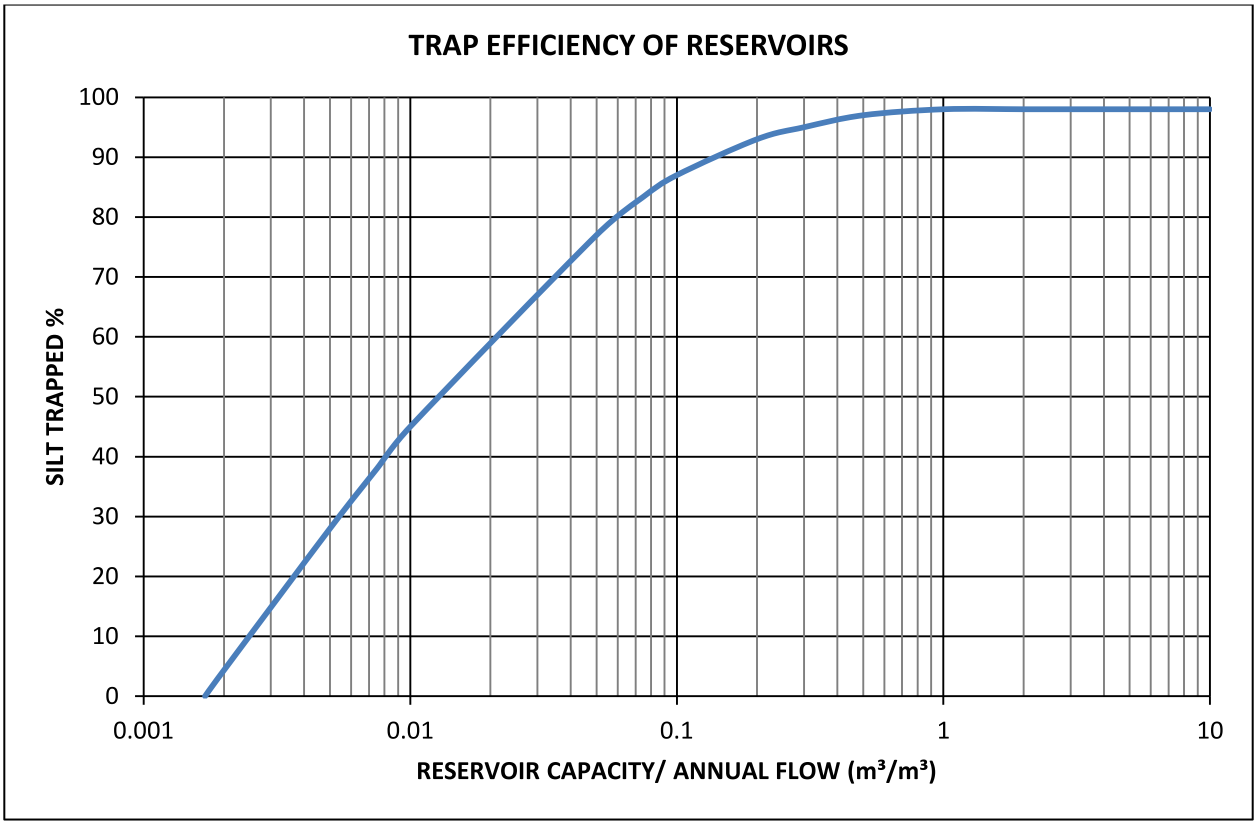

Figure 8-13: Trap Efficiency of Reservoirs

Sediment inflow to a reservoir may be reduced to a certain extent by the use of soil conservation methods within the catchment area as outlined in Chapter 7 (Erosion Control and Catchment Conservation).

In cases were heavy sedimentation is expected, and where no alternative dam location can be found, the possibilities of constructing an upstream silt trap should be examined. However, in order to fulfil their function properly silt traps require regular maintenance (emptying).

8.13.1 Dead Storage

The dead storage requirement is a function of the expected volume of sediments and the need for permanent water within the reservoir, for fish keeping or other ecological factors.

A reasonable estimate of the sediment volume that would accrue in the reservoir across the life span of the structure can be made using the details described above. This value should provide a guide to the extent of dead storage required. The dead storage level can be determined from the height-volume-area (HVA) curve for the dam site. This level will define the level of any offtake works.

[8] See Lindsley and Franzini (1979) pp. 159-164.

8.14 Environmental Flows

The Reserve is defined by the Water Act 2002 as the quantity and quality of water required:

- to satisfy basic human needs for all people who are or may be supplied from the water resource; and
- to protect aquatic ecosystems in order to secure ecologically sustainable development and use of the water resource.

The design of a dam or reservoir located on a water course will need to comply with the requirements of the Reserve as specified in the WRM Rules (2007). The Rules state that the flow shall exceed the Q_{95} or the flow that is exceeded 95% of the time, as defined by a naturalised flow duration curve. This value can be extracted from the FDC (See Section 8.9).

The environmental releases should be sufficient to meet human and ecological demands downstream. These may require a critical analysis of the release requirements with respect to water quality (physical, chemical and biological characteristics), and the quantity, pattern, timing, and reliability of in-stream flow downstream of the reservoir.

The water quality of reservoir releases, drawing water from the lower fraction of the reservoir, typically has low dissolved oxygen levels. Turbulence in the flow downstream of the reservoir will return the DO level to steady state conditions. This process can be assisted with structural features, such as energy dissipaters, to increase the turbulence and aeration of the releases.

The quantity, timing, and reliability of the releases can be addressed by establishing a schedule of release that reflects the natural pattern of flow. This can be partially achieved by establishing the Q_{95} for each month. This would provide a specific release for each month that reflects the natural pattern. Consequently the design of the scour pipe should be sufficient to release the maximum monthly Q_{95}.

Consideration should be given on how to ensure minimum downstream floods as this is part of the natural hydrological pattern. If the ratio of reservoir volume to mean annual flood volume is less than 0.5, then it can be expected that natural flood events will occur. For values greater than 0.5, the reservoir may eliminate or drastically reduce downstream floods which could affect recharge of river banks and flood plains, and sediment transport along the water course. It may therefore be necessary to periodically provide a high volume short duration release. The requirements for this type of release should be determined on a case by case basis in consultation with WRMA.

In the case of online reservoirs on an ephemeral water course, the timing and quantity of environmental releases requires some consideration. In order to be compliant with the WRM Rules (2007), there should be a release whenever there is inflow into the reservoir. The quantity of the release will depend on the water level in the reservoir. This schedule of release should be established in consultation with WRMA and the local WRUA.

A sample reservoir release rule is presented in Table 8-13 in which the maximum environmental release (QE_{max}) may be determined as the Q_{95} if discharge records exist, or flow sufficient to result in open water for at least 5 km downstream or such other distance as agreed by WRMA.

Table 8-13: Sample Reservoir Release Rule for Online Reservoir on Ephemeral Water Course

Status of Inflow to Reservoir	Status of Water Level in Reservoir	Environmental Release	Comment
Flow	< 50%	Equal to the inflow up to a maximum of 25% of QE_{max}	Scour pipe is set to release 25% of QE_{max}.
Flow	> 50%	QE_{max}	Scour pipe is set to release 100% of QE_{max}.
No Flow	< 50%	0	
No Flow	> 50%	25% of QE_{max}	Implies reservoir is increasing reliability of downstream flows

The analysis of the Reserve for an offline storage structure will focus on the design of the river intake works to ensure that this meets the Reserve flow requirements and compliance with the WRM Rules (2007) with respect to the quantity and timing of authorised abstractions.

It should be noted that the actual releases from a reservoir in any given month may be a function of two components:

1. Environmental flow requirements as determined from the discussion above;
2. Releases to meet authorised downstream abstractions.

CHAPTER 9

SITE SURVEYS AND INVESTIGATIONS

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9 SITE SURVEYS AND INVESTIGATIONS

Site survey work for small scale storage encompasses much more than just topographical survey work. It involves testing and sampling soils and available materials as well as undertaking preliminary subsurface investigations.

Other less technical site survey work can be carried out and is dealt with in other chapters. See Chapter 10 for Reconnaissance Survey Details and Chapter 6 for Resettlement Concerns.

There may be long periods between the initial survey work and the decision to proceed with a project. Different teams of people may be involved with the survey, the design and then the supervision of the project, so all survey work must be accurately recorded with sufficient details to allow anyone to make use of it at any stage in the project.

Depending on the size of the proposed reservoir, the initial survey team may consist of a design engineer, a surveyor, a hydrologist and a geotechnical engineer. Survey work may be carried out in stages with a preliminary survey to establish initial site conditions and a final survey to provide detailed information.

Additional information on site surveys and investigations can be found in *Small Dams and Weirs in Earth and Gabion Materials,* (Charman J. et al., 2001)*, Design and Construction of Small Earth Dams,* (Nelson K. D., 1985) or *Water from Ponds, Pans and Dams,* (Lindqvist A.K, 2005).

9.1 Topographical Survey of Storage Site

The topographical survey work output should be a set of drawings that provides all topographical information and details required to carry out the design of the dam, pan or water conservation structure. Historically grid based survey techniques were used. While still very effective, especially in areas with dense bush or tree cover, they are not entirely necessary with modern survey equipment.

The most important point is to make sure that any site survey has sufficient survey data to provide confidence in the contour maps produced.

A survey must extend a considerable distance above the expected crest level to allow for looking at overtopping scenarios and to allow for spillway planning and positioning decisions. In general, a survey that extends 5m in elevation above the proposed embankment crest level is desirable.

Surveys must extend a considerable distance downstream of the planned embankment to allow for the spillway return channel to be positioned correctly. In general, a minimum of 50m downstream of the proposed embankment alignment is permissible and at least 150m is desirable.

Surveys should also cover all expected borrow areas in sufficient details to allow for accurate borrow material calculations.

In the case of pan and lagoon surveys, it is often necessary to survey a considerable distance upstream (or uphill) from the proposed structure to ensure that an inflow channel that captures runoff can be constructed.

In the case of sand and sub-surface dam site surveys, survey data should not only include topographical information but should estimate subsurface conditions collected through the use of steel probes.

It is the responsibility of the design engineer to specify the limits of the survey work to be carried out.

9.1.1 Pre-Survey for Site Selection Purposes

In cases where a pre-survey has to be organised in order to select the most economical location of the dam from the topographical point of view, the following information should be obtained for each prospective dam site:

a. One cross-section of the valley at the location of the prospective dam axis. It is recommended that for this cross-section measurements be taken at intervals not exceeding five (5) metres.

b. A longitudinal profile of the river, with three (3) to five (5) additional cross-sections taken along the longitudinal profile. The number of cross-sections should be decided as a function of the length of the reservoir. The longitudinal profile must extend up to the expected water level and if possible should extend approximately 5m (vertical distance) above the expected water level.

During initial site visits, the GPS coordinates for the site should be recorded. The map datum used in the GPS should also be recorded. With this information, the site can be located on existing maps and within Google Earth.

If a detailed pre-survey is not possible, either a tape measure or an Abbney level and hand held EDM (preferably with an inclinometer) can be used to determine estimated embankment lengths and heights as well as longitudinal slopes along the valley.

The pre-topographical survey visit should also collect information relevant to an environmental scoping study (See Chapter 6 and Chapter 10 for more information). This can include information on stakeholders in the area, landowners likely to be affected by the project, assets within the project area, downstream infrastructure etc.

9.1.2 Final Survey of the Storage Site

The final survey of the storage site should allow the elaboration of contour maps as specified in Table 9-1.

Table 9-1: Specification for Topographical Maps

	RESERVOIR AREA	DAM & SPILLWAY AREA
SCALES	1/500 to 1/1000	1/250
CONTOUR INTERVALS	1.0m to 2.0m	0.5m to 1.0m

For the final survey, it is recommended that the following guidelines be adhered to in order to obtain all required information for the design of the dam:

a) The dam axis and the dam crest level shall be indicated by the designer. The highest (upper) contour line to be surveyed should be taken as 5m above the crest level of the embankment. The upper contour line for the site must close in the final survey drawings.
b) The survey of the dam and spillway area shall cover an area of 200 metres upstream and downstream of the dam axis, and should be extended for a distance of 15 metres from the upper contour. For this area a topographical map of 1/250 scale should be prepared with contour intervals not exceeding one metre. For the rest of the reservoir area, the scale of topographical maps can be 1/500 or 1/1000, while contour intervals should not exceed one (1) metre.
c) Four (or more) solid concrete bench marks shall be established by the survey team. Figure 9-1 shows details of a possible type of bench mark. Two of these bench marks will indicate the dam axis alignment. They should be located at a horizontal distance of at least 10 (ten) metres from the upper contour level.

d) If surveying using a grid system, an indication towards the grid (or cross-section) distances to be used for the final survey is given in Table 9-2. If surveying without establishing a grid, a higher point density should be established within the proposed embankment footprint and within the proposed spillway location. The density of points should be sufficient for an accurate contour map to be produced.

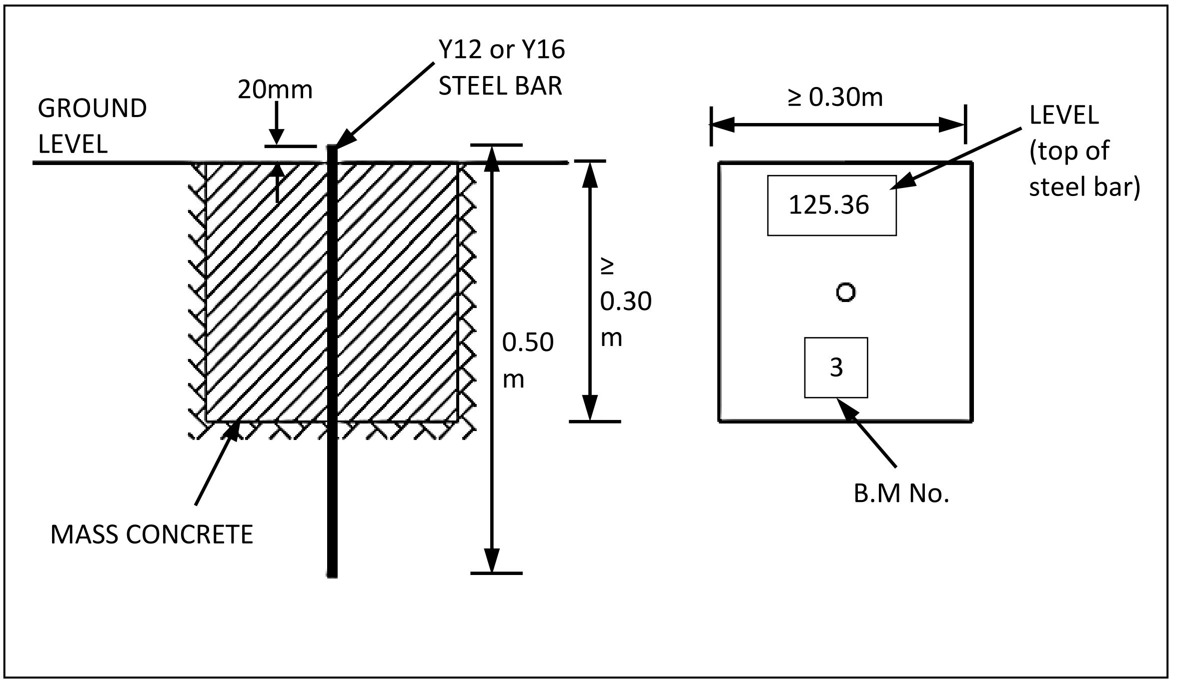

Figure 9-1: Survey Benchmark

Table 9-2: Suggested Grid Spacing for Survey Work

DISTANCE FROM DAM AXIS	GRID DISTANCE
<50m (up & downstream)	5m to 10m
50m to 150m (up & downstream)	25m
150m to 500m (upstream only)	50m
>500m (upstream only)	100m

e) Within the survey area covered, all man-made structures and cultivations should be indicated with their ownership. River crossings (bridges, pipelines, etc.) within 300 metres downstream of the dam axis or situated lower than 2 metres above the upper contour line on the upstream side of the reservoir, should also be indicated.

f) All surveys should be “closed out” to confirm the level of accuracy of the survey measurements. When this is not done, it should be noted and the reasons for not doing it should be clearly presented.

g) All contours below the high flood level must be closed contours (there must not be areas that will be flooded that have not been surveyed. It is often difficult to close high flood contours as it may require surveying up the watercourse a considerable distance past the normal water level.

Figure 9-2 shows a summary of the final survey guidelines for prospective dam sites for small earth dams.

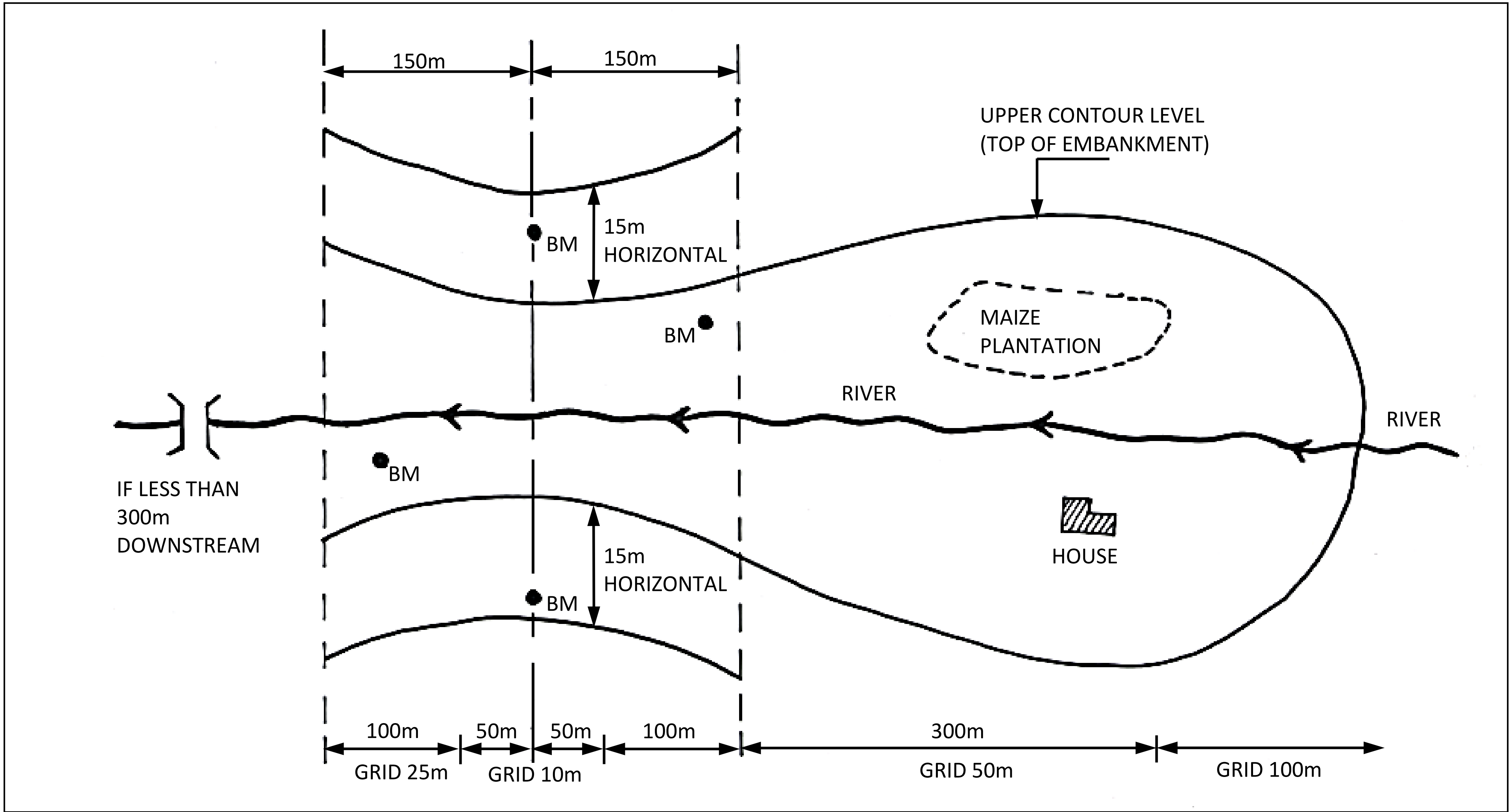

Figure 9-2: Summary of Final Survey Guidelines for a Dam Site

For pans and lagoons a 10m grid should be used in the areas where embankments will be placed. A 25 to 50 meter grid can be used in all other areas. The survey area **must** extend at least 25 meters horizontally beyond the final extent of any expected earthworks. The survey should extend far enough upstream to ensure that an inlet channel can be constructed and to provide information relevant to silt trap construction.

For sand dams and subsurface dams, the survey should be most dense (10m grid or finer) in the area where the wall will be constructed. It can be less dense in the upstream areas. It should capture the positions and relative heights of river banks (i.e. height above the stream bed) as this information is needed during the design process.

For rock catchments, the survey should cover the entire rock face with dense grids in any localized depressions. Rock catchment construction can often extend catchments through the use of training walls to direct water flow. It is important that the survey covers areas outside of the natural catchment area if training walls are to be used. When in doubt as to the required extents of the survey capture as much information as possible on the entire rock face, even if it means only a sparse (50 to 100m) grid can be used.

9.2 Survey Equipment and Software

A variety of modern survey equipment is available and, with careful choosing, can greatly simplify the survey tasks associated with small earth dam design and construction.

Table 9-3 gives a brief summary of types of survey equipment and their main uses. It is not an exhaustive list and other equipment may be used as determined by the surveyor and/or design engineer.

Table 9-3: Survey Equipment and Suggested Uses

Equipment	Main Uses	Notes
Abney Level	Initial site selection	Hand held, gives initial estimates for levels and slopes
Barometric Altimeter	Initial site selection	Can be accurate for relative changes in elevation and for checking handheld GPS elevations.
Handheld EDM (electronic distance measure)	Initial site selection and volume estimation	Available with slope readouts. Distance measure between 100m and 500m.
Handheld GPS	Initial site selection, locate site on contour map or in Google Earth. Can measure areas.	NOT SUITABLE FOR ACCURATE ELEVATION MEASUREMENT.
Dumpy levels	Mostly suitable for site layouts and construction supervision.	Can be used for site surveys on small (less than 100m x 100m sites)
Theodolites	Suitable for survey work for most sites	Requires lots of mathematical calculations
Total Stations	Suitable for survey work for all sites	Requires at least two benchmarks to ensure all measurements use same coordinate system.
Survey quality GPS	Suitable for survey work of sites. Not suitable for layout work	Requires only one benchmark. Not suitable for sites with dense tree cover.
Real Time Survey Quality GPS	Suitable for survey work of sites. Suitable for layout work	Requires only one benchmark. Can be used in sites with some tree cover.

Along with modern survey equipment, there is a variety of modern computer software to assist in survey tasks associated with the design and construction of dams, pans and other small water conservation structures.

Table 9-4 lists common software and suggested uses. Again, this is not an exhaustive list and additional software may be required for specific design work.

Table 9-4: Typical Software Used in Survey and Design of Small Dams

Software	Main Uses	Notes
AutoCAD Civil 3D	Survey data manipulation, contour generation, calculation of volumes (earth and storage)	Can be expensive, other similar software is available. Can be difficult to learn (PROGECAD….)
Microsoft EXCEL	Survey data manipulation. Repetitive calculation	Any spreadsheet software will work.
SURFER	Digital terrain modelling. Works well with excel based data sets and allows contour generation, volume calculations etc.	Fairly inexpensive and very easy to learn
ARC GIS	Larger scale mapping, catchment examinations	Can be expensive and difficult to learn.
Google Earth	Initial site examinations, catchment characteristics, relevant infrastructure	Requires accurate positioning data (from hand held GPS is more than sufficient)
HECRAS	Used for flood estimation and routing	Requires detailed survey data for river channels.

9.3 Height-Volume-Area Relationship Curve

The height-volume-area relationship ("HVA curve") or storage-area curve is developed from the survey data. The HVA curve provides an important site specific information on the relationship between height (or water depth), volume, and area. In addition to the layout map, the HVA curve is perhaps the most important output from the site survey work as the data it presents is used to evaluate different inflow, outflow and storage scenarios during the design work.

Convention dictates that the water depths or embankment heights are plotted on the vertical axis and the corresponding areas and volumes are plotted on the horizontal axis. This can be achieved by plotting the volumes on the primary horizontal axis (at the bottom of the graph) and the areas on the secondary horizontal axis (at the top of the graph) which is reversed as shown in Figure 9-3.

9.3.1 Developing the HVA Curve

Developing the HVA curve requires a site contour map drawn to scale. The contour map is the primary output from the site survey work and provides all required information for developing the HVA curve.

If the contour map is only available in a paper copy, the following procedure should be followed:

1. Make a copy of the map.
2. Lay out the centreline of the proposed embankment on the map.
3. Lay out an appropriately scaled grid on the map. The grid can be aligned with a compass bearing or with the proposed centreline or with any other relevant feature on the map. It is best to use a uniform grid spacing (10m x 10m or 20m x 20m….). The actual grid size is determined by the size of the proposed storage (for a large project, a 50m x 50m grid may be reasonable, for a small

project, a 5m x 5m grid may be sufficient). Once a grid is laid out, make several additional copies of the map. In general, for a 5m deep storage structure make 5 copies, for a 10 meter deep structure make 10 copies.

4. Starting with the lowest contour upstream of the proposed centre line, determine the “flooded” area between the contour and the centre line. This figure is the area that would be flooded if water filled the valley up to that specific contour level.
5. Continue working up one contour at a time, determining the flooded area for each contour. It is usually easiest to do this on a clean map copy and to determine areas by counting grid squares and then multiplying by the area of a single grid square. For example, with a 10m x 10m grid, if the flooded area took up 25.6 grid squares, the final flooded area would be 25.6 grid squares x 100m^2/grid square which is 2,560 square meters.
6. Establish a table as shown below. In this table, the contour at 2,134 was not closed and it was not possible to get a flooded area figure from the contour map.

Table 9-5: Typical Elevation and Area Table

Elevation(m)	Water Depth (m)	Flooded Area (m^2)
2,127.50	0	0
2,128.00	0.50	1,500
2,128.50	1.00	5,241
2,129.00	1.50	8,614
2,129.50	2.00	12,691
2,130.00	2.50	17,703
2,130.50	3.00	25,181
2,131.00	3.50	31,983
2,131.50	4.00	38,883
2,132.00	4.50	48,916
2,132.50	5.00	57,545
2,133.00	5.50	68,442
2,133.50	6.00	81,579
2,134.00	6.50	

If the contour map is available in a soft/CAD copy, then a similar procedure should be followed:

1. Draw the proposed centreline on the contour map.
2. Use the CAD programme to obtain area figures for the area between each contour and the centre line. The final result should be a table similar to Table 9-5 above.

Once the elevation and flooded area data has been established. The following steps need to be followed:

1. From Table 9-5 above, the estimated storage at each elevation can be calculated. This is done using the sectional storage between each contour and the one below it. Sectional storage is calculated using Equation 9-1.

Equation 9-1 $$\boldsymbol{V_{1-2} = ((A_1 + A_2)/2) * (E_2 - E_1)}$$

Where:
$V_{1\text{-}2}$ is the sectional storage capacity [m^3]
A_1 is lower contour flooded area [m^2]
A_2 is the upper contour flooded area [m^2]
E_2 is the upper contour elevation [m]
E_1 is the lower contour elevation [m]

The total storage below each contour is simply the sum of all the sectional storages below that contour level. Generally, the first or initial contour is assumed to have zero storage below it. For large flat sites (or for the bases of pans) this assumption should be checked and adjusted if necessary.

2. Table 9-6 shows volume figures based on the area data in Table 9-5. In the example below, the sectional storage at the 2,128m elevation was adjusted from 350 cubic meters (calculated) to 750 cubic meters (estimated) to reflect the ground conditions at the site. In the table the crest elevation of the proposed embankment is estimated to be 2,133.5m and the spillway sill elevation (Normal Water Level) is estimated to be at 2,132.0m. For the sake of this example, the High Flood Level (water level in the reservoir when spillway is flowing at design capacity) is estimated to be 2,132.5m.

Table 9-6: Typical HVA Data Table

Elevation (m)	Depth (m)	Flooded Area (m^2)	Storage Volume (m^3)	Cumulative Storage Volume (m^3)	Notes
2,127.50	0	0	0	0	Empty
2,128.00	0.50	1,500	750.00	750.00	
2,128.50	1.00	5,241	1,685.19	2,435.19	
2,129.00	1.50	8,614	3,463.76	5,898.96	
2,129.50	2.00	12,691	5,326.20	11,225.16	
2,130.00	2.50	17,703	7,598.49	18,823.65	
2,130.50	3.00	25,181	10,721.14	29,544.79	
2,131.00	3.50	31,983	14,291.05	43,835.83	
2,131.50	4.00	38,883	17,716.50	61,552.34	
2,132.00	4.50	48,916	21,949.85	83,502.18	Normal Water Level
2,132.50	5.00	57,545	26,615.29	110,117.47	High Flood Level
2,133.00	5.50	68,442	31,496.58	141,614.05	
2,133.50	6.00	81,579	37,505.13	179,119.18	Overtopping
2,134.00	6.50				No Area Data

3. The data from Table 9-6 can then be plotted as an HVA curve as shown in Figure 9-3. It is worth noting the dramatic increases in volume for small changes in elevation at the higher elevations. In this example, if the survey data had adequately covered up to the 2,134 elevation, the Normal Water Level could have been raised by 0.5m and the storage volume could have been increased by 30%.

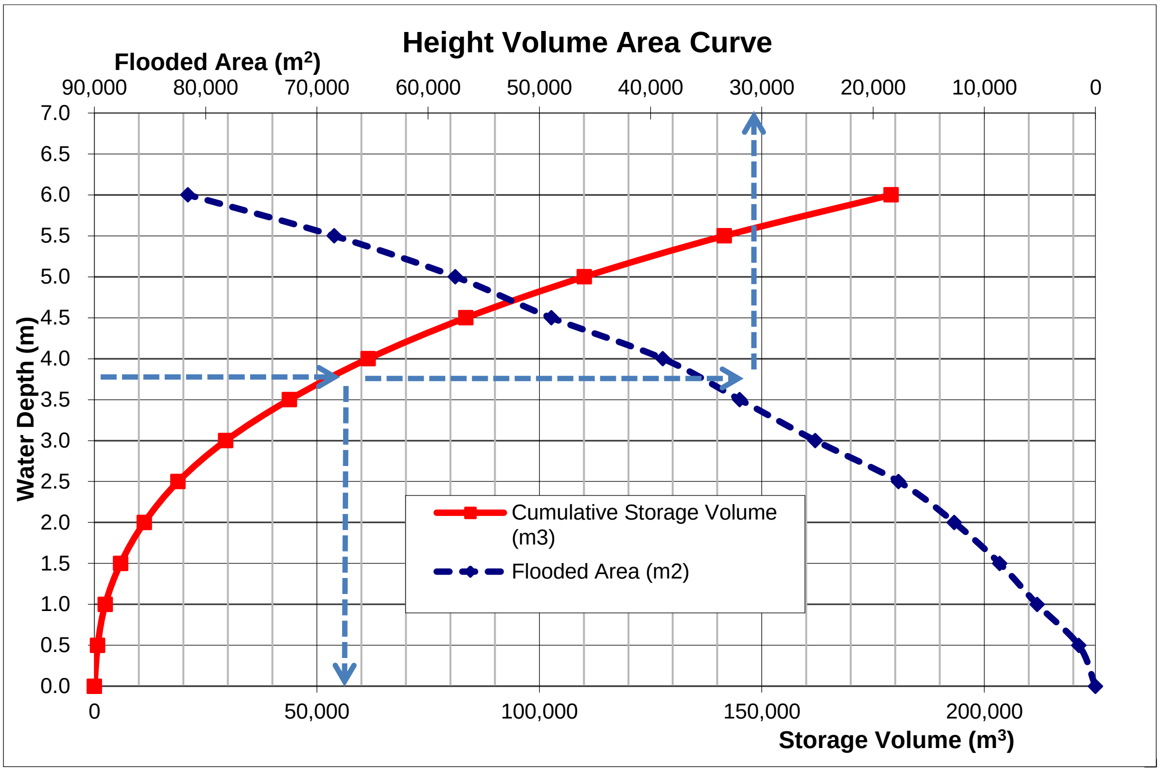

Figure 9-3: Height Volume Area Curve

As shown in Figure 9-3, a water depth of 3.75m would have a storage volume of 51,000 m^3 and a flooded area of 36,000 m^2. Alternatively, if 150,000 m^3 of storage is required, then the required water depth would be approximately 5.6m and the flooded area would be approximately 72,000 m^2.

9.3.2 Rough Estimate of Storage Capacity

During initial site visits it is often useful to establish an estimate of the site storage capacity before carrying out a detailed survey.

Stephens (1991) recommends the following equation for initial estimates of storage capacity.

Equation 9-2 $$Q = L \times T \times D/6$$

Where: Q is the storage capacity [m^3]
L is the length of the dam wall at the full supply level [m]
T is the throwback (or fetch) of the reservoir [m] and measured in an approximately straight line from the wall
D is the maximum water depth [m]

This figure is generally accurate to about 10% for the size and scale of dams covered in this document. L, T and D can all be estimated during a site visit with the use of an Abbney Level and an EDM.

9.4 Geotechnical Investigations

Geotechnical investigations will have to be carried out to establish the suitability of the foundation, abutments and spillway, any required foundation treatment, excavation volumes and slopes, and availability and characteristics of embankment materials. The investigations should cover classification, physical properties, location and extent of soil and rock strata, and occurrence and depth of groundwater within the foundation and reservoir area. Specifically the geotechnical investigations should yield information on:

1. Depth, nature and condition of the bedrock along the dam axis and spillway. Bedrock that is close to the surface can present significant construction and/or seepage problems;
2. Location of permeable layers along the dam axis which might cause excessive leakage or even piping failure under the dam;
3. Identification of impermeable material along the dam axis into which the cut-off will be keyed into;
4. Identification and assessment of suitable in-situ borrow material for dam construction;
5. Location and assessment of ground conditions within the foundation area for stability, compressibility, and seepage.

9.4.1 Site Investigations

It is generally advisable to carry out investigations into the geological and foundation conditions of a dam-site for a depth equal to the height of the proposed dam. However, in view of the limited hydraulic heads dealt with in this publication and considering the generally high degree of consolidation of the soils in Kenya, it will in most cases be sufficient to investigate up to a depth of 4.00 metres. For dams with higher risks, considerable downstream infrastructure or questionable foundation materials, more detailed investigation will be required.

A. Bedrock

Bedrock at shallow depths should be detected and mapped. The weathered parts under the dam foundation and along the base surface of the cut-off trench will need to be removed. As this is an expensive exercise, possibly requiring blasting (which can further fracture the rock), it is therefore important to carefully assess the nature of the bedrock and whether it will need to be removed.

Heavily fractured bedrock can cause serious leakage and can be difficult to seal. Excavation of the fractured portion of the bedrock can be difficult and expensive to remove.

However, bedrock along the spillway axis, particularly at the sill cross section, is desirable, as this means that the channel may be less prone to erosion.

B. Permeable Layers

All permeable layers, like gravel, murram, laterite and sand (old river-bed) under the dam and reservoir area should be detected as these can cause excessive seepage losses and potentially even cause dam failure. Mitigation measures (e.g. cut-off trench, clay layer, bentonite applications, synthetic lining etc.) can be included in the design if the presence of such permeable layers has been detected.

C. Foundation and Abutments

The foundation and abutment area requires ground conditions with stable soils. Ground conditions with soils of high organic content (e.g. peat), heavy swelling clays, saturated soils (swamps) or unconsolidated deposits are not suitable and the extent of unsuitable material should be mapped. A decision will need to be made as to whether the site is suitable, given these foundation conditions.

D. Access to Site

Access to site should be considered to ensure that the construction equipment can get to site. This may require an assessment of the roads and bridges that might be used by the construction equipment en route to the dam site itself.

E. Test Pits

Figure 9-4 shows a recommended type of test pit. Test pits should have a depth of at least 4.00 m, unless bedrock is struck before that depth. Test pits should be dug manually. Alternatively, the last two meters can be dug with the help of a hand auger. Test pits should be sized so that an engineer can enter the pit and examine the layers encountered. In general, a properly sized test pit will closely resemble the freshly excavated pit of a pit latrine.

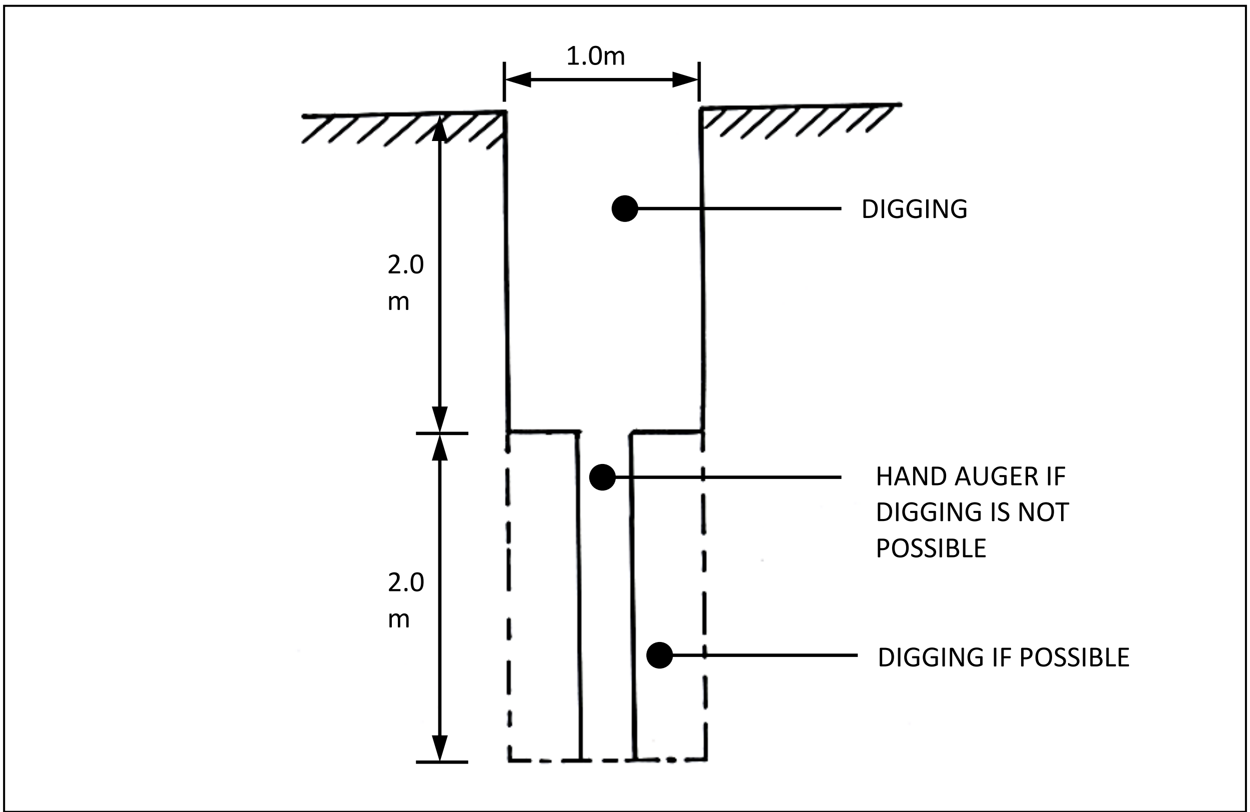

Figure 9-4: Test Pit Details

Figure 9-5 shows a tentative arrangement of test pits for a proposed dam. Along the dam axis test pits should be made at 10-15 metre intervals, and should be extended for approximately 30-50 metres horizontal from the expected upper contour level. Eight (8) more test pits should be made, four (4) upstream and four (4) downstream at 20-25 metre distances from the dam axis. The future centre-line of the spillway should be located as closely as possible and test pits should be sunk every 10-15 metres, in order to establish the position of the bedrock and to assess the suitability of the excavated material for construction purposes. Additional test pits should be located at any possible borrow areas and should be positioned to allow an initial estimate of available borrow volumes. To do this, a series of pits that establish the borrow area are needed. Their position and depths of suitable material can then be used for borrow volume estimations.

It is worth noting that Figure 9-5 shows 34 test pits which may seem excessive for a fairly small dam. Fewer pits can be used if there is confidence in the uniformity of the subsurface conditions. If this is the case, it should be noted in any survey reports. Final foundation details can be adjusted during core trench excavations. Test pits should be clearly numbered/named on the design drawings and when possible test pit logs should be digitally stored to allow easy access to the information they contain.

Uncovered test pits can present a danger to livestock, wildlife and humans. Test pits should be fenced off with brush or other materials when possible. Test pits should be back filled after examination if there is liable to be a significant period of time before construction begins.

Any test pits excavated in the reservoir area should be carefully backfilled with material which is watered and placed in shallow layers (150mm) and hand compacted such that the material has a density equal to the adjacent undisturbed soil.

Logs of the test pits should be established. Care should be taken to keep descriptions of the encountered materials as simple and understandable as possible. Figure 9-6 shows an example of a test pit log.

Samples of materials which will be submitted to the laboratory for testing (see Section 9.4.4) should be put in strong plastic bags, and marked with the (i) test pit number, (ii) depth at which it was taken, (iii) date at which it was taken. In general, samples of at least 20kg each are required for a full range of soil tests.

The results of the site investigations should allow the assessment of the suitability of the dam site with respect to the geology of the dam and reservoir area as outlined above. Reference is made to Section 9.6 where the basic geological requirements to be met are outlined.

Test pits on rocky sites should be carefully positioned to give as much information as possible with regard to both the soils and the rocky layers. In sites with large boulders, test pits that encounter boulders should be repositioned and re-dug.

F. Test Pits for Water Pans

The dimensions and purpose of test pits for water pans are exactly the same as test pits for dams. In the case of pans there is less concern with test pits under any embankments and more concern with test pits in the reservoir area. Test pits in the reservoir area should establish the types and quantities of soil to be excavated. If test pits expose pervious or porous layers, the depth to such layers should be noted so that the layers are not exposed during construction. For thin pervious layers, the depths to impervious layers below them should also be noted. For most small to medium sized pan projects, 15 to 25 test pits might be required to give a good feel for the subsurface conditions.

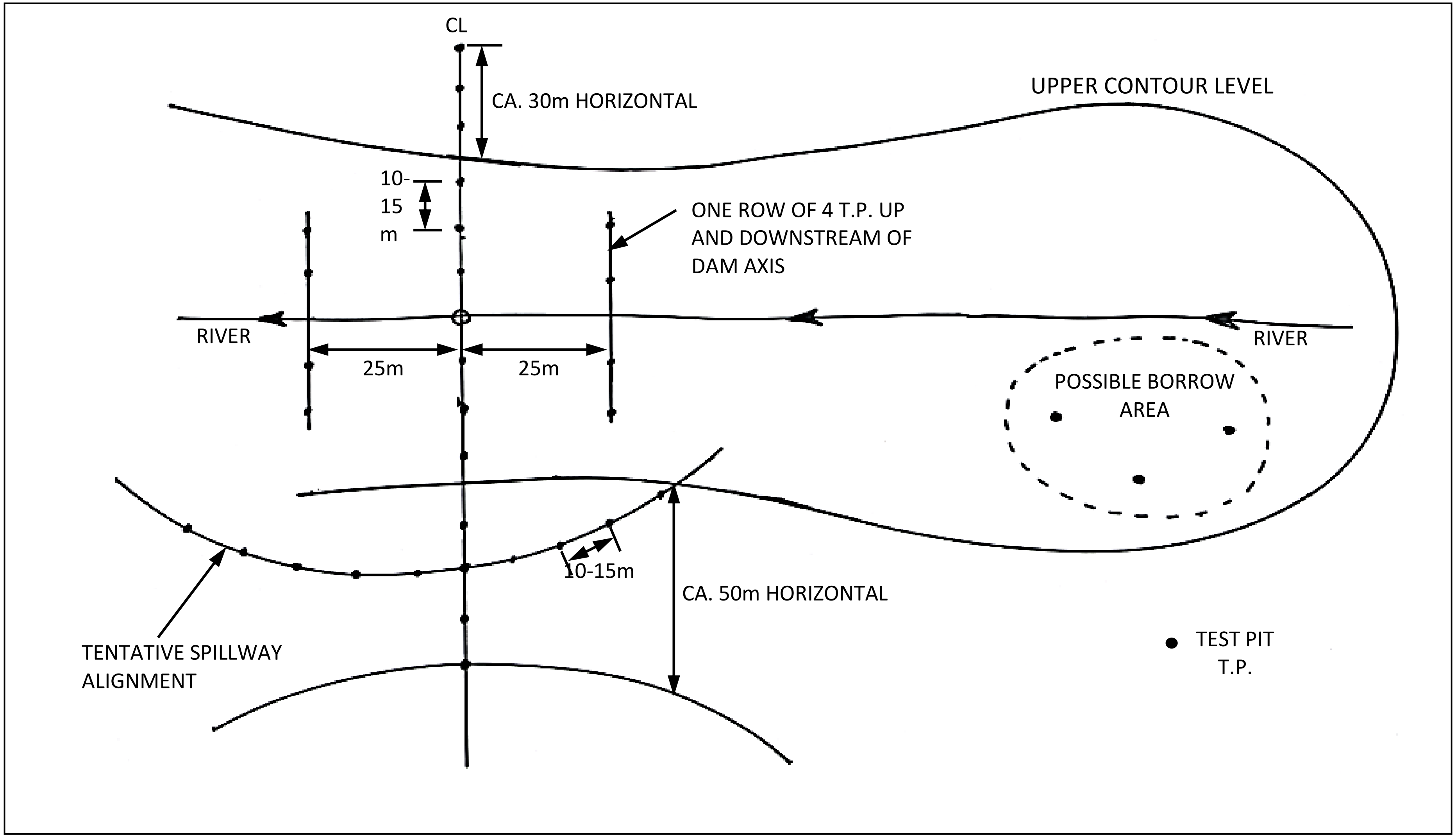

Figure 9-5: Site Investigations Test Pit Layout

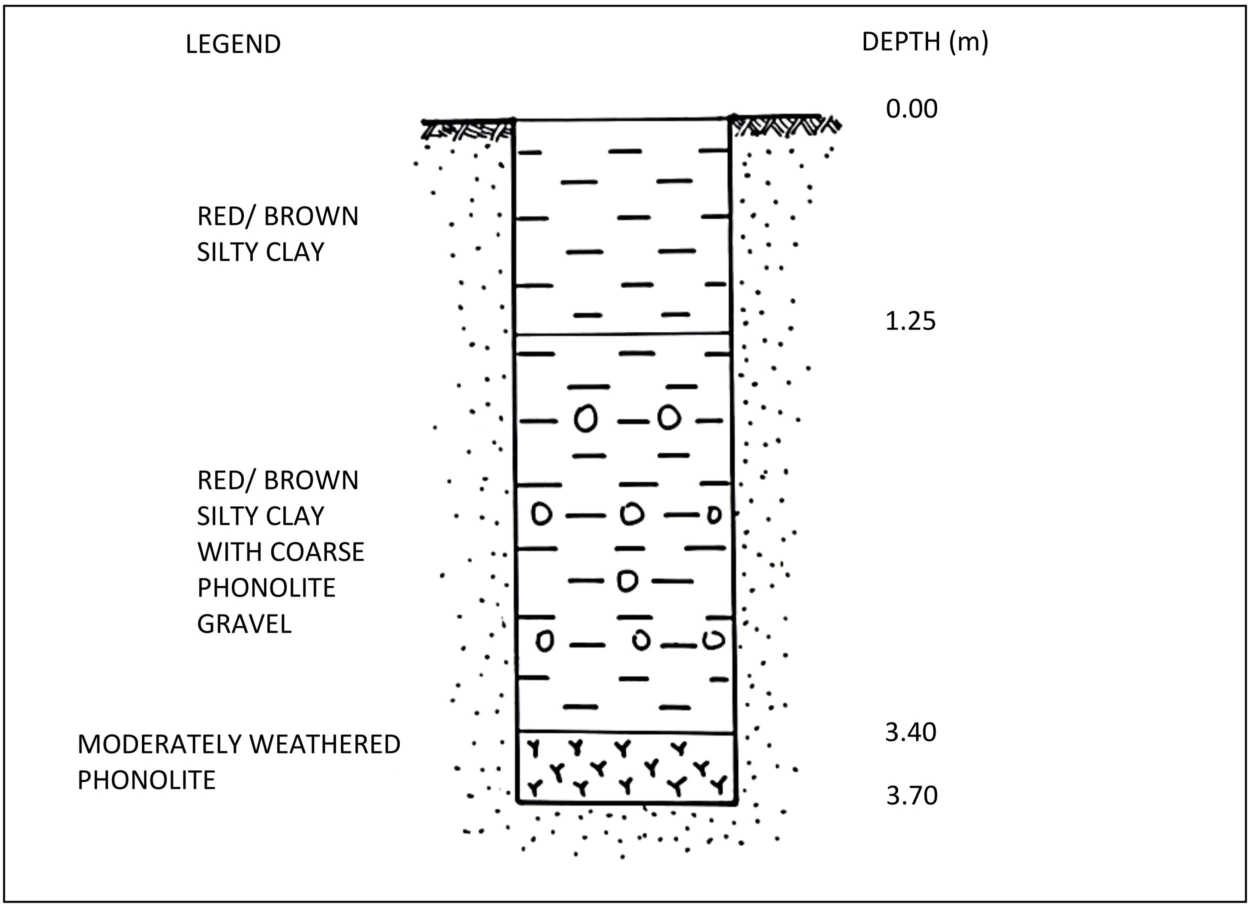

Figure 9-6: Test Pit Log

9.4.2 Availability of Construction Materials

Geotechnical investigations should establish the availability of construction materials.

A. Embankment Material

For reasons of economic feasibility, (see Chapter 11) suitable soil to be used as construction material should be available within a short distance from the dam site.

First the suitability of the material which will be excavated from the spillway should be investigated. Use of this material will mean a considerable economy on the construction cost. However, even when found suitable the quantity of material obtained from the spillway excavation may not be sufficient. Hence borrow areas for embankment fill material will have to be identified.

Borrow areas should preferably be located within the reservoir area, since removal of this material will increase the storage capacity. If this proves impossible, then borrow areas outside the reservoir should be identified. A few test pits (not less than four) should be dug in order to assess the suitability, depth, extent and homogeneity of the material. Initial borrow sources should be able to supply at least double the embankment fill volume. In general, it is not advisable to borrow clay material from the river bed just upstream of the embankment foundation as this can lead to exposure of pervious layers and lead to seepage problems.

Table 9-7: Soil Classification Chart (laboratory method)

CRITERIA FOR ASSIGNING GROUP SYMBOLS[a]				GROUP SYMBOL	GROUP NAME[b]
COURSE GRAINED SOILS More than 50% retained on No. 200 sieve (75mm micron)	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve (4.75mm micron)	CLEAN GRAVELS[c] ≤ 5% fines	Cu ≥ 4 and 1 ≤ Cc ≤ 3[e]	GW	Well graded gravel[f]
			Cu < 4 and/or 1 > Cc > 3[e]	GP	Poorly graded gravel[f]
		GRAVEL WITH FINES[c] More than 12% fines	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	SANDS 50% or more of coarse fraction passes No. 4 sieve (4.75mm micron)	CLEAN SANDS[d] Less than 5% fines	Cu ≥ 6 and 1 ≤ Cc ≤ 3[e]	SW	Well graded sand[i]
			Cu < 6 and/or 1 > Cc > 3[e]	SP	Poorly graded sand[i]
		SANDS WITH FINES[d] More than 12% fines	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
FINE GRAINED SOILS 50% or more pass sieve No. 200 (75 micron)	SILTS AND CLAYS Liquid limit less than 50	inorganic	PI > 7 and plots on or above line A[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below line A[j]	ML	Silt[k,l,m]
		organic	$\frac{Liquid\ limit - oven\ dried}{Liquid\ limit - not\ dried} < 0.75$	OL	Organic clay[k,l,m,n] Organic silt[k,l,m,o]
	SILTS AND CLAYS Liquid limit 50 or more	inorganic	PI plots on or above Line A	CH	Fat clay[k,l,m]
			PI plots below line A	MH	Elastic silt[k,l,m]
		organic	$\frac{Liquid\ limit - oven\ dried}{Liquid\ limit - not\ dried} < 0.75$	OH	Organic clay[k,l,m,p] Organic silt[k,l,m,q]
Highly organic soils		Primarily organic matter, dark in colour and organic odour		PT	Peat

Notes

a	Based on material passing No. 3 (75mm) sieve
b	If field sample contains cobbles and/or boulders, add "with cobbles and/or boulders"
c	Gravels with 5 – 12% fines require dual symbols
	GW-GM well graded gravel with silt
	GW-GC well graded gravel with clay
	GP-GM poorly graded gravel with silt
	GP-GC poorly graded gravel with clay
d	Sands with 5 – 12% fines require dual symbols
	SW-SM well graded sand with silt
	SW-SC well graded sand with clay
	SP-SM poorly graded sand l with silt
	SP-SC poorly graded sand with clay
e	$Cu = \frac{D_{60}}{D_{10}} Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
f	If soil contains ≥ 15% sand, add "with sand"
g	If fines classify as CL-ML, use dual symbol GC-GM, SC-SM
h	If fines are organic, add "with organic fines"
i	If soil contains ≥ 15% gravel, add "with gravel"
j	If the liquid limit and plasticity index plot in hatched area on plasticity chart, soil is CL-ML, silty clay
k	If soil contains 15 – 29% plus No. 200 add "with sand" or "with gravel" whichever is predominant
l	If soil contains ≥30% plus No. 200, predominantly sand, add "sandy"
m	If soil contains ≥30% plus No. 200, predominantly gravel, add "gravelly"
n	PI > 4 and plots on or above line "A" on Plasticity Chart
o	PI < 4 or plots below line "A" on Plasticity Chart
p	PI plots on or above line "A" on Plasticity Chart
q	PI plots below line "A" on Plasticity Chart

As outlined in Section 11.1, for dams over eight metres of height, where suitable soil is not available in sufficient quantities for constructing a homogeneous embankment, the construction of a zoned embankment can present a solution. In such cases, material excavated from the spillway could possibly be used as “random fill" for backfilling the shoulders of the embankment. It should however be noted that the design of zoned embankments should be carried out with due regard for the USBR filter criteria (United States Department of the Interior - Bureau of Reclamation, 1987).

B. Material for Drainage Blanket

The construction of a drainage blanket will require significant quantities for clean river sand and graded ballast. The source of the river sand should be determined and a sample taken for particle size analysis. In addition the source should be assessed to determine whether there are sufficient quantities and whether there are other competing interests for this sand (e.g. sand dams) that could cause a conflict or environmental degradation.

C. Material for Concrete Works

Most storage structures, although not all, require concrete to be used in various components of the structure (e.g. spillway sill, surround to draw-off pipe, ancillary structures, etc). Some consideration should be given to the availability of clean river sand and graded ballast in sufficient quantities for the proposed structures.

D. Water for Construction Work

Water is required for proper soil compaction, concrete works, and drinking water for the labour. Consideration should be given to the availability of water for the construction phase of the project as water trucking, if required, is an expensive undertaking. In general, during construction, an earth embankment will require a water volume equal to 30% of the embankment volume. Masonry and concrete structures will require less water but the importance of the water supply during construction cannot be overstated.

9.4.3 Soil Classification

The Unified Soil Classification describes different soil types by symbols as shown in Table 9-7.

A well graded soil implies that the soil has a fair proportion of all particle sizes. Conversely, poorly graded implies a soil with a significant proportion or excess of one soil type or particle size.

Silts and clays are further divided into those with low (L) and high (H) liquid limits. The Liquid Limit is the moisture content (water/dry weight soil %) at which the clay or silt becomes a slurry. A moisture content less than 50% denotes a low liquid limit. The liquid Limit is determined by the Atterberg Test (described in Section 9.4.4).

The Unified Soil Classification is presented in Table 9-7 together with the suitability of the soil for earth embankment dams shown in Table 9-8.

Table 9-8: Soil Suitability for Earth Embankments

Group Symbol	Description	Soil Suitability for Earth Embankments		
		Homogeneous Dam	Zoned Dam	
			Core	Shell
GW	Well graded gravels			High
GP	Poorly graded gravels			Good/Fair
GM	Silty gravels			Good
GC	Clayey gravels	High	High	
SW	Well graded sands			High
SP	Poorly graded sands			Fair
SM	Silty sands		Fair	
SC	Clayey sands	Good	Good/Fair	
ML	Inorganic silts with low liquid limits		Poor	
CL	Inorganic clay with low liquid limits, also known as a “lean clay”	Good/Fair	Good/Fair	
OL	Organic silts or clays with low liquid limits	Not suitable		
MH	Inorganic silts with high liquid limits		Poor	
CH	Inorganic clay with high liquid limits, also known as a “fat clay”	Fair	Fair	
OH	Organic silts or clays with high liquid limits	Not suitable		
Pt	Peat and highly organic soils	Not suitable		

The United States Department of Agriculture Texture Diagram (Figure 9-7) also provides a basis for classifying soil types based on the relative proportions of sand, silt and clay.

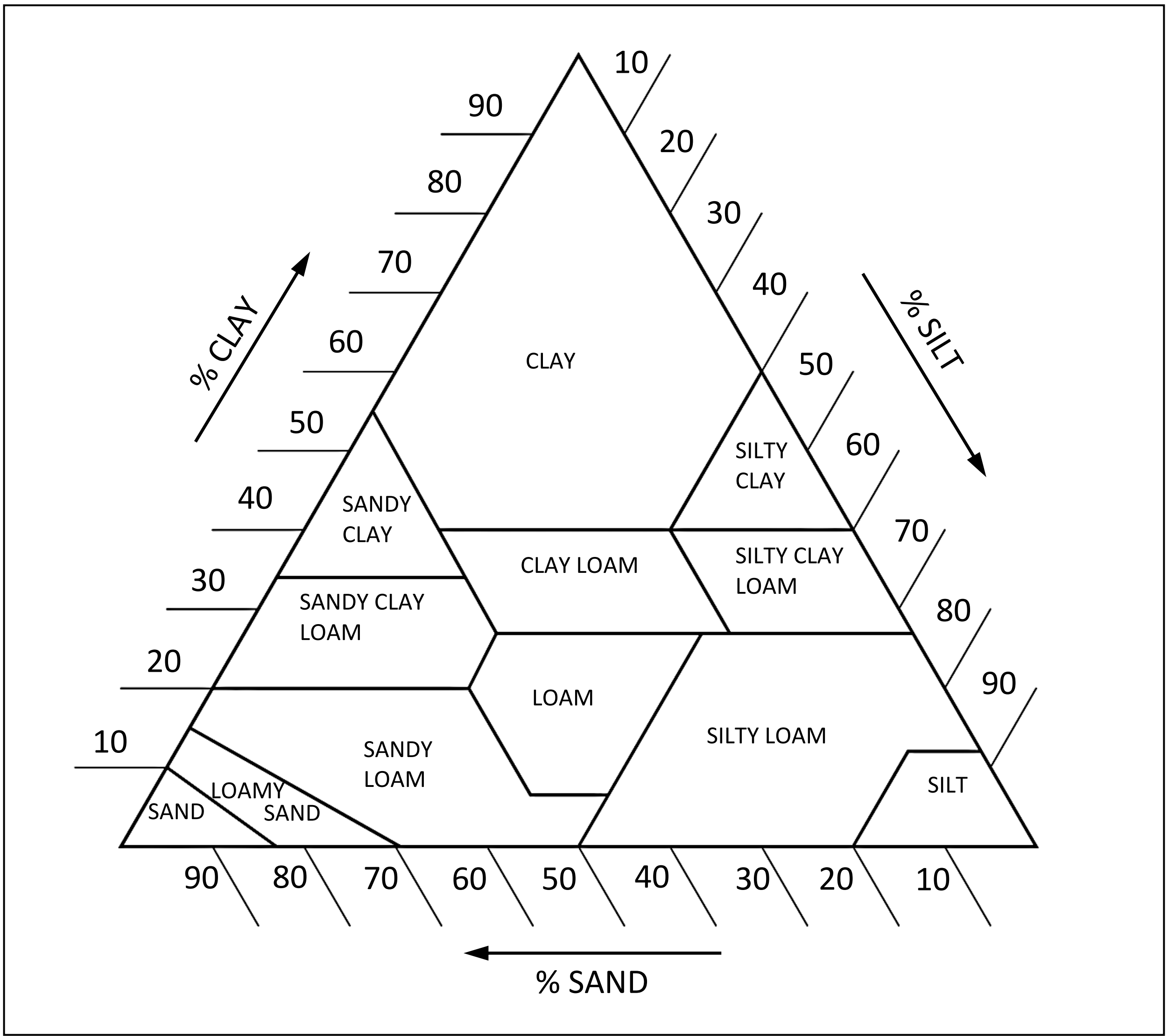

Figure 9-7: Soil Texture Class (USDA)

The USDA soil classes can be used for earth embankments as shown below.

Table 9-9: Texture Classes

Textural Class	% Sand	% Clay	Soil Suitability for Earth Embankments		
			Homogeneous Dam	Zoned Dam	
				Core	Shell
Sand	> 85%	-			
Loamy Sand	70 – 85%	-			✓
Sandy Loam	50 – 70%	< 20%			✓
Sandy Clay Loam	45 – 80%	20 – 35%	✓	✓	✓
Clay Loam	< 45%	25 – 40%	✓	✓	
Sandy Clay	45 – 65%	> 35%	✓	✓	
Clay	< 45%	> 40%		✓	

(Source: Stephens, T. 1991)

Generally silts are unsuitable as embankment fill material due to their inherent instability when wet.

The description of soil should provide the following information:

- Name and symbol;
- Percentage of gravel, sands and fines;
- Colour in moist condition;
- Perviousness or drainage properties;
- Plasticity characteristics.

9.4.4 Soil Tests

A. <u>Visual and Field Tests</u>: A number of simple tests which can easily be carried out in the field permit a preliminary evaluation of the prospective fill material's suitability.

1) With the naked eye a first classification of the available materials can be made. Heavy clays and soils containing an excessive percentage of sand can be identified and classified as unsuitable. Heavy clays are subject to considerable swelling, shrinking and cracking, while soils containing too much sand are in general too permeable.

   Soils in which most particles are visible to the naked eye are generally too sandy and should also be classified as unsuitable for homogeneous dam construction. Soils with hardly any particles visible to the naked eye (heavy clays) should be treated carefully and submitted to laboratory tests to assess their suitability for homogeneous dam construction.

   Consequently, soils with around half (50%) of all particles visible to the naked eye are generally suitable for homogeneous dam construction.

2) To establish the existence of cohesion, add roughly 10 % of water (volume) to a soil sample, mix, and proceed as follows:

   a. Soils which cannot be rolled into a ball without breaking up are either too sandy or have a highly irregular granular distribution. These soils should be avoided. Alluvial deposits of fine sand and silt may demonstrate this feature.
   b. Soils which can be rolled into relatively thin threads (diameter approximately 8 mm) without breaking up contain a high proportion of clay. Consequently suitable soils are those which can at least be rolled into a ball but no further than a thread with ±8 mm diameter and ±150 mm of length, before it starts breaking into pieces. A wet clay will also exhibit a shiny smooth surface when cut.
   c. Dry silts are similar in appearance and feel to wet clays but unlike clays which exhibit sticky, plastic like properties, a wet silt has a silky, smooth feeling. Silts do not make good embankment material unless mixed with other soil types (clays and sands).
   d. It should be confirmed that the soil is not rich in organic matter. Usually, organic soils can be distinguished from the inorganic by their characteristic odour and their dark-grey or black colour.
   e. The dilatancy or shaking test can indicate the degree of plasticity. The wetted soil is formed into a pat on the open palm of the hand. The hand is shaken horizontally. If water comes to the surface making it glossy, the sample should be squeezed between fingers and the gloss should disappear. A reaction that is rapid indicates lack of plasticity and no reaction indicates a highly plastic clay which is also unsuitable.

3) In situ permeability tests can be carried out to determine the permeability of soils at the dam site. A rough falling head permeability test is described below.

a. Dig or bore (with an auger) a suitable hole in the ground at the dam site. The hole should extend below the topsoil layer. If using an auger, a depth of 2 to 3 meters is desirable.
b. Measure the hole sufficiently to determine the wetted surface area when it is filled with water.
c. Fill with water and observe how quickly the water level drops. Keep track of elapsed time. Top up as needed to maintain the water level. Ideally this should be done over a period of several hours to ensure that the surrounding soil becomes saturated.
d. In the final 30 minutes, observe the drop in water level over a period of time and then top up with water and measure the volume used to top up.
e. A rough permeability can be calculated by taking the top up volume divided by the wetted area and then dividing the result by the time observed in step d. The result can be expressed in mm/s and should be in the range of 1×10^{-4} mm/s.
f. In the event of very permeable soils and community projects it is advisable to get the community to observe the in situ permeability testing so they fully understand the scale of possible seepage losses.

B. <u>Laboratory Tests:</u> The purposes of performing laboratory tests on soil samples in relation to design and construction of small dams are:

a. Assessment of suitability of foundation and construction material, and
b. Monitoring of compaction during the construction of the embankment.

The taking of “undisturbed" samples from test pits will in general require the services of specialised operators. Disturbed samples in plastic bags can be obtained as mentioned under Section 9.4.1. The performance of the following tests is highly recommended for the assessment of prospective construction materials. These tests can be performed at approved soil testing labs within the country. Services can be obtained from higher learning institutions, Ministry of Public Works, national research facilities (e.g. KARI, KALRO) and certified private soil testing laboratories: More information on specific tests can be found in British Standard BS1377 or in Eurocode EN1997.

1. **<u>Particle Size Analysis:</u>** This test is used to determine the type of soil. The test is carried out by means of sieving (and for the finer fractions by sedimentation). Basically, the soil sample is passed through a series of standard test sieves having successively smaller mesh sizes. The results of this test are represented as a curve on a semi-logarithmic plot, the ordinates being the percentage by weight of particles smaller than the size given by the abscissa (Figure 9-8). Soils suitable as construction materials for homogeneous small dams are generally represented by a smooth concave distribution curve.

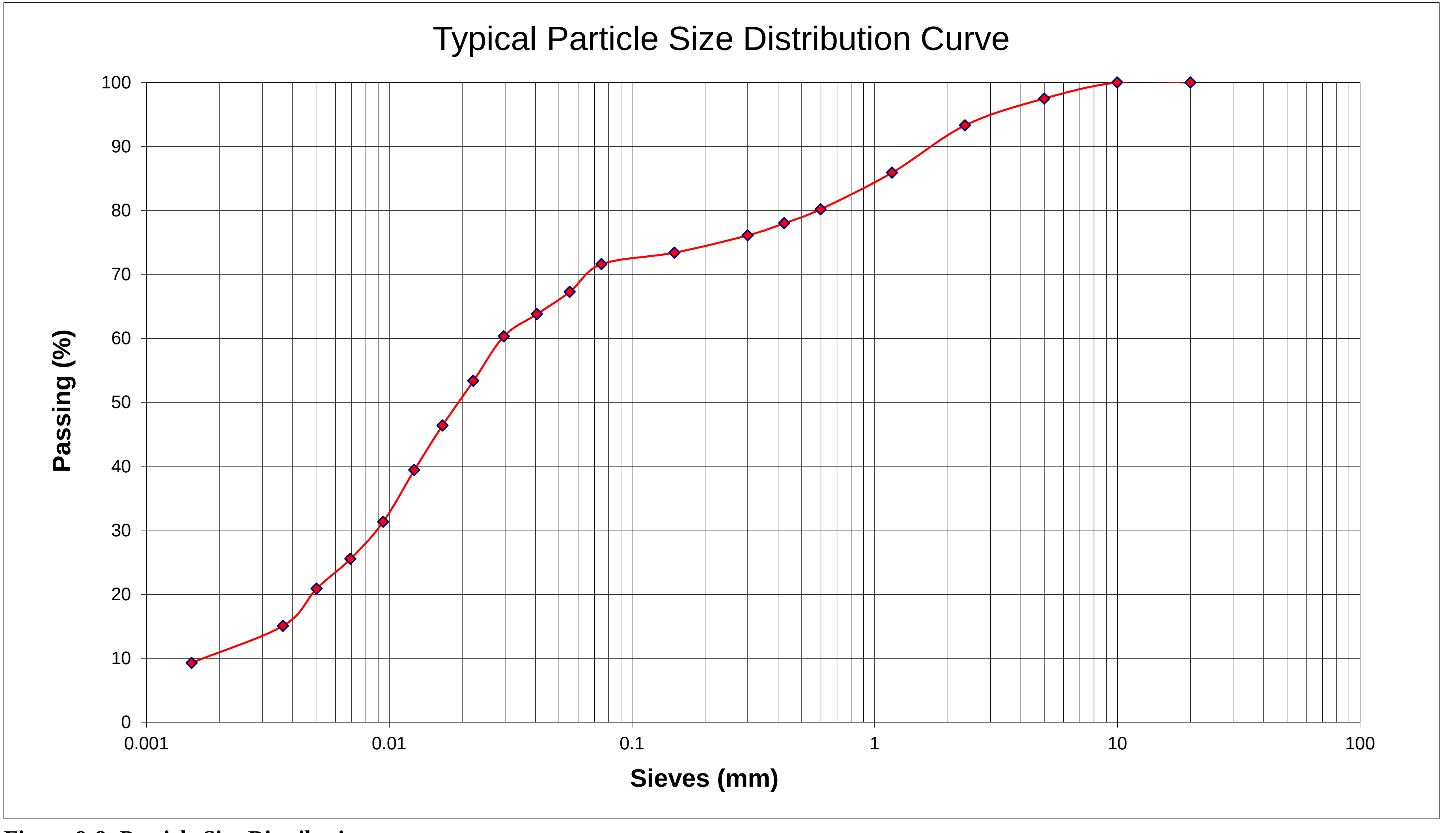

Figure 9-8: Particle Size Distribution

2. **Atterberg Limits:** Depending on its water content a soil may exist in the liquid, plastic, semi-solid or solid state. The water contents at which the transitions between states occur vary from soil to soil. Most fine-grained soils exist naturally in the plastic state. The upper and lower limits of the range of water content over which a soil exhibits plastic behaviour are defined as the liquid limit (W_L) and plastic limit (W_P) respectively. The water content range itself is defined as the plasticity index ($I_P = W_L – W_P$). The transition between the semi-solid and solid states occurs at the shrinkage limit, defined as the water content at which the volume of soil reaches its lowest value as it dries out.

   It has been observed that many properties of silts and clays, can be correlated with the Atterberg limits by means of the plasticity chart (Figure 9-9).In general soils that fall above the "Line A" in Figure 9-9 and have a liquid limit (W_L) above 30% are suitable:

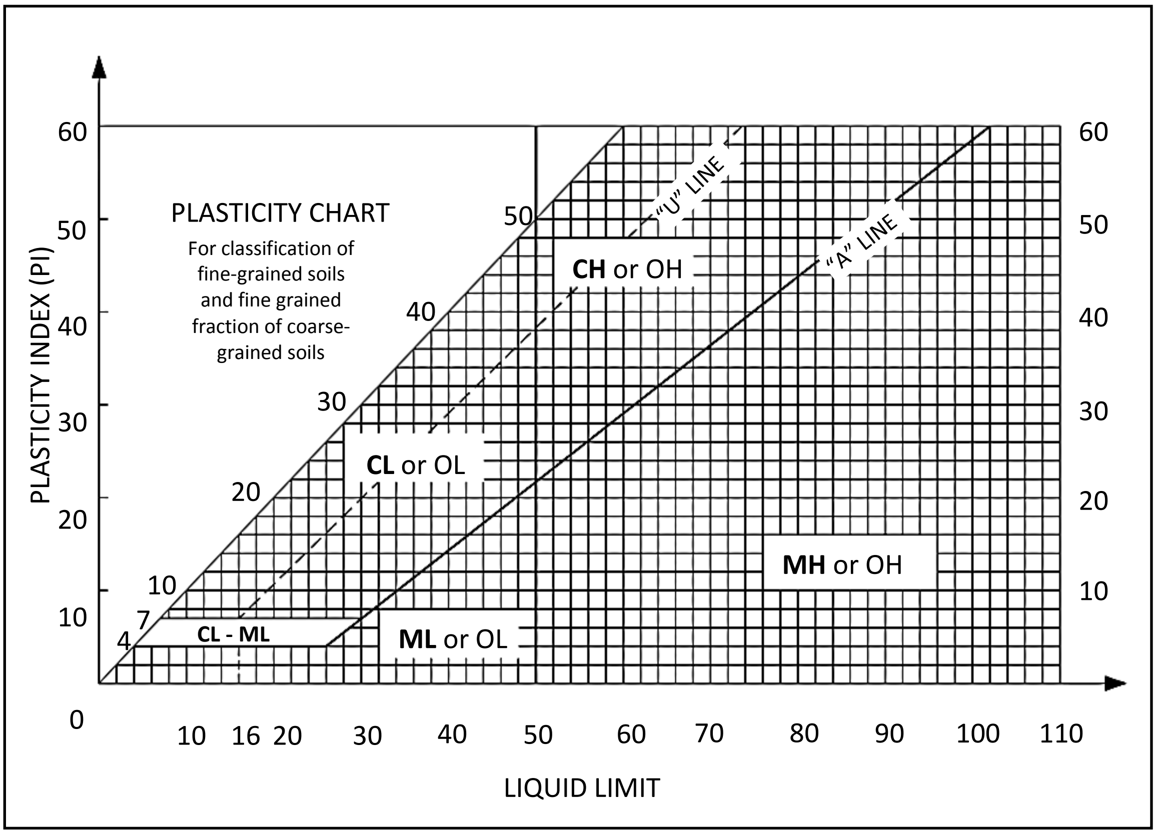

Figure 9-9: Plasticity Chart

4) **Proctor Compaction Test:** This test will determine the optimum water content, and the corresponding maximum dry density of the soil which can be obtained for a particular compactive effort. Figure 9-10 shows a typical (dry density/water content) relationship.

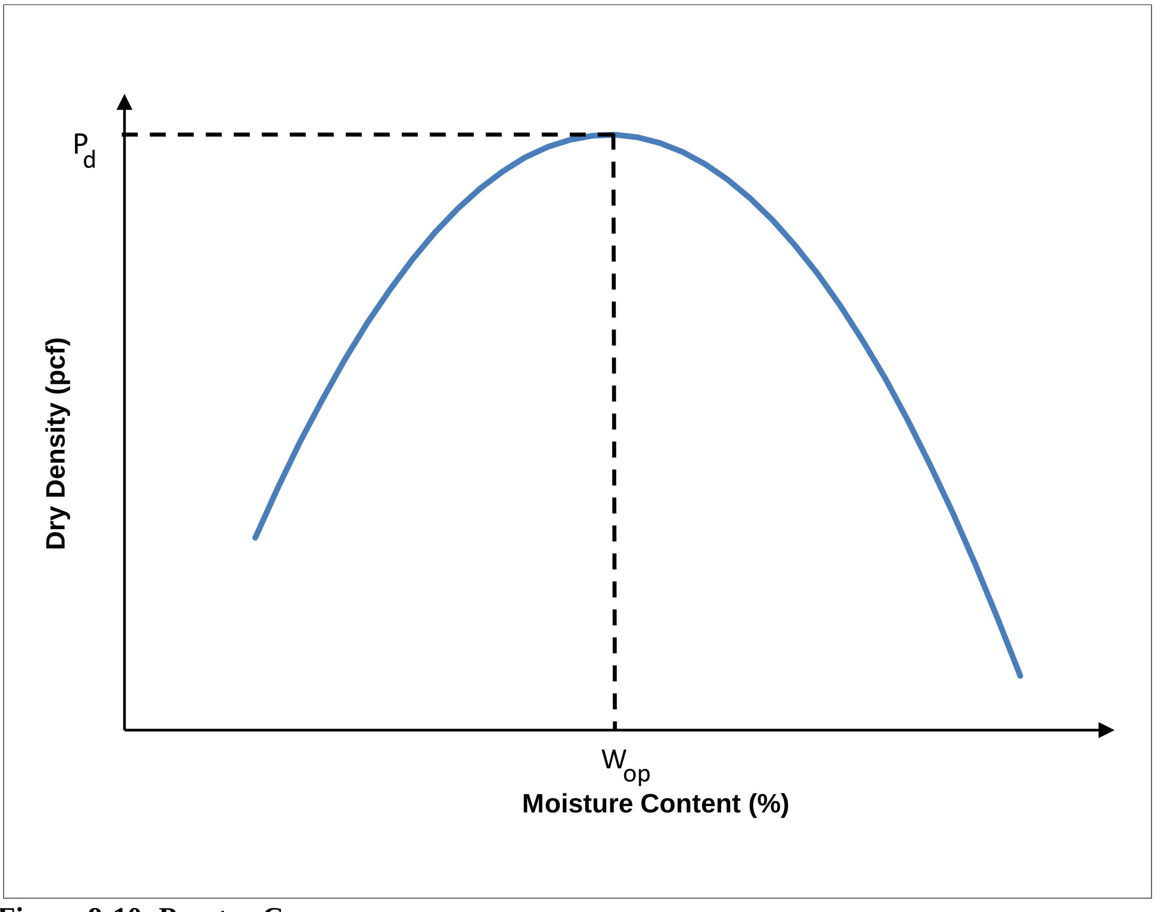

Figure 9-10: Proctor Curve

A minimum construction requirement of 95% to 100% of the maximum dry density obtained in the Proctor test is usually specified.

Test procedures concerning the above laboratory soil tests are described by Craig (Knappett & Craig, 2012) pp. 26-29. The tests can be performed on disturbed samples.

5) **Dispersivity:** Soils which are to be used as construction material should systematically be tested for dispersivity. The use of dispersive clays as construction material has caused many failures of small dams in Kenya and elsewhere. Dispersivity can be defined as post-construction de-flocculation of clay, which leads to a complete loss of tensile strength. Dispersive characteristics of soils are generally closely related to the exchangeable sodium percentage (ESP). If the ESP value is higher than 8 %, the soil may be dispersive. An ESP value of 5 % or less can be considered safe. In cases of doubt a crumb test can be performed: in this test a cube of soil is placed in a dish of de-ionised water, and the tendency of colloidal-size particles to de-flocculate and go into suspension is closely observed.

   For higher risk structures (taller walls, downstream infrastructure, etc) the performance of more advanced tests to determine the shear strength parameters (cohesion; friction angle) of the prospective construction material should be considered. These tests are necessary for any sort of foundation and slope stability investigations. Reference is made to Craig (1983) for any inquiries concerning these tests.

9.4.5 Typical Soil Properties

Typical soil properties are presented in Table 9-10 which provides a guide to cross check laboratory results.

Table 9-10: Average Soil Properties for Different Soil Types

Soil Group Symbol	MDD (Kg/m^3)	OMC (%)	Cohesion (Kg/m^2)	Tan ϕ	Permeability (cm/sec)
GC	> 1840	<15	N/A	> 0.60	10^{-6} to 10^{-8}
GM	> 1830	<15	N/A	>0.67	10^{-3} to 10^{-6}
SM	1830 ± 16	15 ± 0.4	500 ± 500	0.58 ± 0.07	10^{-3} to 10^{-6}
SC	1840 ± 16	15 ± 0.4	1100 ± 600	0.6 ± 0.07	10^{-6} to 10^{-8}
ML	1650 ± 16	19 ± 0.7	900 ± NA	0.62 ± 0.04	10^{-3} to 10^{-6}
CL	1730 ± 16	17 ± 0.03	1200 ± 200	0.54 ± 0.04	10^{-3} to 10^{-6}
CH	1510 ± 32	25 ± 1.2	1300 ± 600	0.35 ± 0.09	10^{-6} to 10^{-8}
MH	1310 ± 64	36 ± 3.2	2000 ± 900	0.47 ± 0.05	10^{-4} to 10^{-6}

CHAPTER 10

RECONNAISSANCE SURVEY

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10 RECONNAISSANCE SURVEY

The reconnaissance survey is an important component of the project assessment. This activity compliments the project planning exercise and should be conducted properly to ensure as much information pertinent to the project is identified and collected (See also Chapter 3 - Project Planning and Management). Preliminary ESIA screening and scoping can also be conducted during this stage.

A reconnaissance survey should take place early in the project planning. It will help to:

- determine the suitability of a storage project to address water supply problems;
- determine the best embankment or reservoir options;
- establish catchment size and initial water availability and historical flood marks;
- establish other areas of concern.

10.1 Desk Study

This will involve assessment of primary and secondary sources of information relevant to the proposed survey area, including maps (topography, satellite, vegetation, soils, etc.) and reports from similar works done. The desk study must provide preliminary insight on the suitability of the proposed site, before proceeding to site. A second desk study will almost always be needed after the site visit as well.

10.2 Site Identification

Site identification can involve many different criteria depending on the type of storage to be constructed. The following section applies mainly to earth embankments but can also be applied to other types of storage as well. It cannot be stressed enough that an accurate GPS position for the proposed site should be recorded. The position and map datum that the GPS uses can then be used to locate the site accurately on existing contour maps, Google earth and other software/mapping programs.

The general considerations that must be considered for all storage sites include:

- Location of final water use;
- Size of catchment and available inflows;
- Site topography;
- Geology of the dam and reservoir areas;
- Availability of construction materials;
- Spillway location considerations;
- Sedimentation;
- Evaporation;
- Settlement in the area and potential settlement or resettlement requirements;
- Risks to downstream inhabitants;
- Other storage structures in the area;
- Land ownership.

10.3 Tools and Equipment for Reconnaissance Survey

Table 10-1 suggests tools and equipment that the reconnaissance team can use during the initial visit. Other more basic field items that could assist the team should be considered. These include gumboots, umbrellas, hats, sunscreen etc. Spare batteries for all equipment should be carried. If the field visit is in a remote area, battery charging equipment should be carried for computers and other equipment.

Table 10-1: Tools and Equipment for Reconnaissance Survey

Tool/ Equipment	**Function**
Camera	Capturing still images of survey area
GPS	Collecting geospatial information; elevations, location etc. A variety of handheld GPS units are available in Kenya. They can be set to a variety of reference datums and units. They can record positions very accurately but are less reliable for elevations. In general if a handheld GPS is showing a positional accuracy of ±3m the elevation accuracy is ±30m. More advanced GPS units can measure areas and distances very accurately.
EDM	Collecting distance measurements. Modern EDMs can measure distances up to about 1,000m without the need for specialized survey reflectors. Typical high end EDMs come with an inclinometer and can measure angles of inclination (usually within a ±30 degree range). EDMs with inclinometers can be used to calculate vertical heights with the use of basic trigonometry.
Abney Level	Measuring angles of inclination/ slopes between points
Sample Bags and Labels	Collecting and identifying soil samples
Maps	Identifying locations, features etc. within survey area. In general 1:50,000 topographic maps for most areas in Kenya are available. These are a reasonable scale for determining catchment areas but are not detailed enough for any sort of storage estimations. In Northern Kenya, only 1:100,000 maps may be available. For large projects and large catchments areas 1:250,000 maps of the entire country are available.
Spade, hoe, machete (“*panga*”)	Collecting soil samples
Water Bottle	Collecting water samples for analysis and conducting field soil tests.

The list above is by no means exhaustive and additional more specialized equipment may be required.

10.4 Selection of Appropriate Type of Structure

For reservoirs located along water courses, the main options and considerations for storage structures are summarised in Table 10-2 below:

Table 10-2: Considerations for Structures on Water Courses

Storage Structure	Considerations
Earth Embankment	Valley side slopes (must be gentle enough to allow machinery access). Impermeable foundations capable of supporting the embankment. Availability of borrow material. Suitable spillway locations away from embankment (i.e. gentle side slopes on at least one side of the valley where a spillway can be excavated).
Mass Gravity Walls	Type of foundation materials (usually require firm rock foundations and rocky valley side slopes). Availability of construction materials. Suitable spillway locations along valley side slopes. Relatively low water depths (generally less than 5m for small structures).
Sand Dams	As with mass gravity walls above but in a valley with large sand/sediment loads, construction will be carried out over several rainy seasons. Suitable solid and impermeable banks into which the wall can be anchored.
Subsurface Dams	Wide, permeable valleys with significant subsurface flow or sub surface water holding capacity. Suitable subsurface rock surface or solid impermeable foundation on which the wall can be constructed.

For reservoirs located outside of water courses, the main options and considerations for storage structures are shown in Table 10-3:

Table 10-3: Considerations for Structures Not on Water Courses

Storage Structure	Considerations
Pans	Flat or gently sloping (max 3%, best 1%) ground, may have an existing natural depression. Deep soils that can be excavated and the ability to direct surface runoff into the structure.
Lagoon	Sloping ground (up to 10%) Suitable borrow material within the reservoir area for construction of embankment walls on the low sides of the site. Ability to direct surface runoff into the structure.
Rock Catchment	Suitable rock face for collecting runoff (gently sloping with topography that allows runoff capture with low training walls). Suitable area for tank or gravity wall construction to store water.

10.5 Location of Final Water Use

In principle, the reservoir site should be located reasonably close to the area where the stored water will be used as this reduces the cost of conveying the water to the supply area. In addition if the topography allows, the option of a gravity supply from the reservoir to the supply area should be considered.

10.6 Size of the Catchment Area and Available Inflows

The size of the catchment has a bearing on (i) inflow into the site, (ii) size of spillway and required freeboard and (iii) rate of sedimentation. Consequently it is not economic to put a small dam on a large catchment. Furthermore a catchment area that is too small for the proposed storage structure may result in insufficient inflow.

Catchment size and expected inflows are discussed in detail in Chapter 8 on hydrology.

For a quick estimation of inflows, Equation 10-1 can be used.

Equation 10-1 $$\boldsymbol{Q = C \times A \times R \times 1000}$$

Where:
Q is the annual inflow [m^3/year]
C is runoff coefficient. A value of 5% (dry year) to 20% (normal) can be used
A is the catchment area [km^2]
R is the annual rainfall [mm]

10.7 Topography of Dam Site

Ideally, a dam site should be located in a narrow part of the river, just downstream of a relatively wide stretch (See Figure 10-1). The dam should be located in a stretch of the river which has a flat longitudinal slope. This will generally allow the best storage option without excessively tall embankments. Although a variety of alignments can be tried, the best alignment is usually perpendicular to the watercourse. If the axis cannot be positioned at the narrowest point on the river course, then the next best options are on the upstream part of the constriction (i.e. as the valley narrows).

An important factor in assessing the economic suitability of a prospective earth dam site is the storage ratio (water storage volume / earth fill volume). Sites where the storage ratio is below 3 (poor) to 5 (moderate) should, in fact, not be considered. A good site should offer a storage capacity / earth fill ratio of at least 8 (fair) to 15 (good).

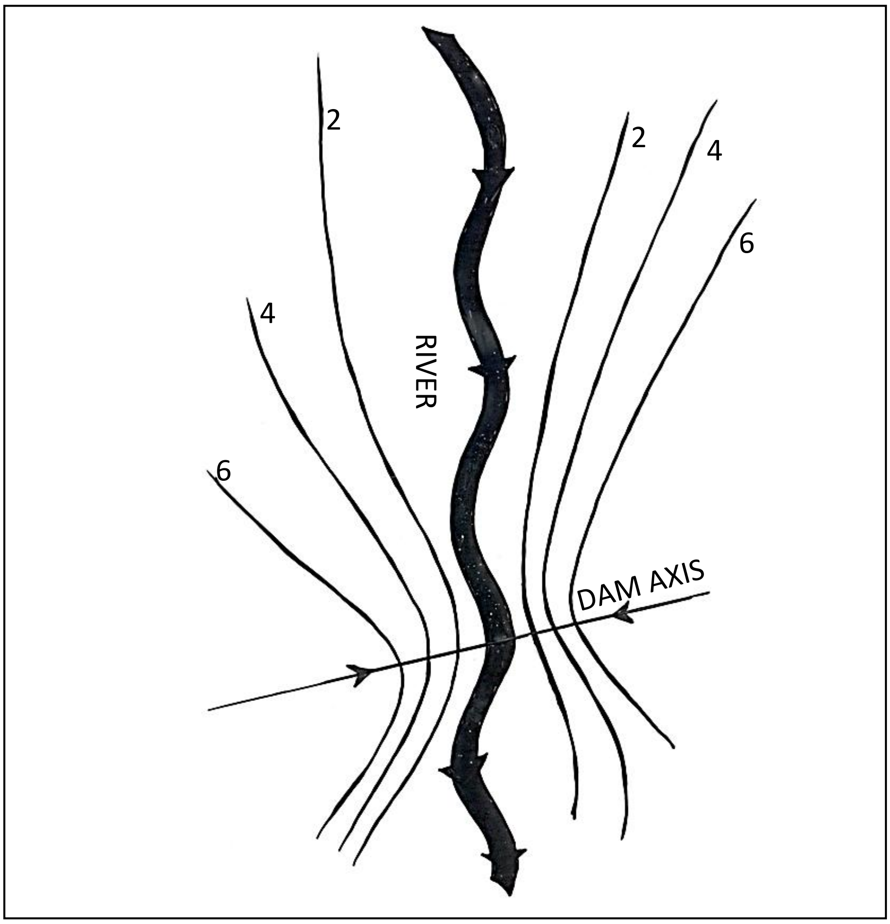

Figure 10-1: Topographic Location of Dam Axis

10.8 Topography of Pan Sites

In general pans are best suited to sites that are flat or very gently sloping (1% to 3% slopes work best). For rectangular pans, the long side of the pan should be positioned parallel to the contour lines. This will result in the minimum earthworks required for any embankments and will ensure that the pan can capture surface runoff.

10.9 Geology of Dam and Reservoir Area

Two basic requirements are to be considered concerning the geology of the dam and reservoir area.

10.9.1 Embankment Foundation

At the location of the embankment, the foundation material should be firm enough to carry the weight of the dam wall. Care should be taken in detecting the presence of any sand layers in the foundation area, since this can cause excessive seepage. In general, for earth embankments less than five metres in height, the risk of embankment failure through piping - washing out of small soil particles because of uplift pressure in the downstream area of the embankment - is not very serious, due to low water heads.

10.9.2 Water-tightness of the Reservoir

In sandy areas, heavy losses of water from the reservoir through infiltration and/or leakage (on top of the evaporation losses) will render the construction of the dam unviable. Therefore, in such areas, construction of surface water storage facilities without lining should, as a rule, not be recommended.

Technical solutions for excessive water losses through infiltration can be found and include clay lining, soil additives (bentonite or similar) or lining with HDPE or LDPE lining materials. In general, the use of lining materials only makes economic sense where the stored water is being used for commercial production.

10.10 Availability of Construction Materials

Suitable construction material (soil) should be available in sufficient quantities within a short distance from the dam site, preferably within the reservoir area since this will increase the storage capacity. Lack of suitable construction material within an acceptable distance from the dam site can render the construction of a dam at an otherwise good site completely uneconomical.

Construction material considerations should not be limited to suitable earth embankment materials but should also examine availability of water, sand, ballast and hardcore.

Water will be needed for proper compaction. Generally a water volume of 25 to 30 percent of the embankment volume will be needed.

If heavy equipment is to be used, both access roads and availability of fuel and oil must be considered.

10.11 Site Accessibility for Construction

Issues for consideration would include availability and loading capacity of river crossings, height restrictions, and road orientation in the case of low loaders.

10.12 Possibilities for Spillway Location

For the type and size of dams under consideration, it is usually not economically viable to incorporate large concrete structures (such as spillway channels, culverts, earth retaining walls etc.) into the design. In order to avoid the need for such structures, the following should be kept in mind, while assessing prospective dam sites:

(i) The spillway should be kept away from the embankment in order to minimise the need for protection structures (retaining walls);
(ii) Steep valleys should be avoided, since they will either require a concrete sill (due to high water velocity), or excessive excavation (the length of the spillway will have to be increased) in case an earth channel spillway is used.

With regard to spillway placement, the preferred sites are narrow valley stretches with relatively steep sides up to the required water level, above which at least one of the valley embankments flattens considerably (Figure 10-2).

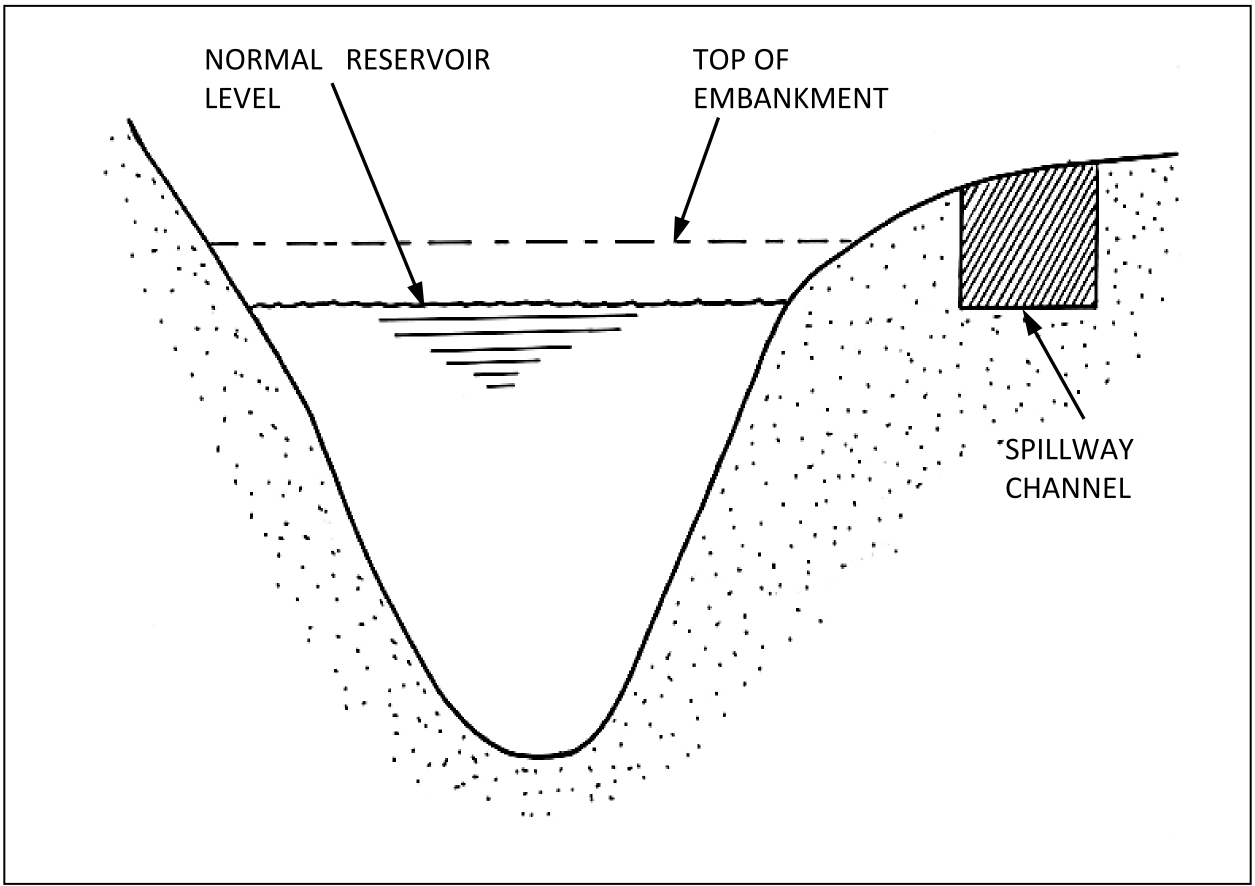

Figure 10-2: Spillway Location Possibilities

10.13 Sedimentation

Sedimentation is discussed in Chapter 8 (Hydrological and Sediment Analysis). In general, sedimentation from large catchments can be very problematic in Kenya. The size of catchment, land use in the catchment and existing evidence of erosion or large sediment loads should be noted.

Often, offline storage (storage constructed off the main river or *lagga* channel) is preferable from a sedimentation viewpoint. In such cases, inlet channels/structures can be constructed to allow water inflow from the main catchment when sediment conditions are the most favourable (toward the end of the rainy season).

10.14 Evaporation

Evaporation is discussed in Section 3.3.10 and Chapter 8 (Hydrology) and should be considered during the reconnaissance survey. Evaporation losses from reservoirs with large surface areas can be excessive. In general, shallow reservoirs with large surface areas should be avoided in areas with high evaporation.

10.15 Land Ownership

Land ownership and sample agreements for storage construction are discussed in detail in Section 4.5.
Ownership of the dam and reservoir area (or other storage structure) should be thoroughly sorted out as soon as a suitable construction site has been selected. Cost of land compensation and/or resettlement of people might be forbidding or might upset the economic viability of the project.

In the case of a dam, enough land should be set aside, not only to cover the dam and reservoir area, but also for the spillway channel and other ancillary structures, as well as for possible borrow areas outside the reservoir. Sufficient allocation of land for other water conservation structures should also be considered.
The social impact on the affected landowners and communities should also be taken into account for each considered dam site, and sites where the consequences of resettlement or loss of valuable (fertile) land are considered important should be avoided.

As early as possible in the planning and design process, land issues should be addressed and resolved by the community with the possible assistance of the County Lands Office.

For smaller, more commercially oriented storage, it is important to note that rivers and laggas often form property boundaries and it can be difficult to get agreement from property owners on both sides of the stream or *lagga*.

10.16 Risk Assessment

Potential risks due to the proposed project should be identified and assessed. Fatal flaws, or issues that can cause unanticipated problems should be pre-empted and possible solutions identified. At this stage of the project, the risk assessment will mainly examine what sort of “deal breakers” exist and how likely they are to become serious issues.

10.17 Desk Study (Part II)

At the conclusion of the reconnaissance visit, it is usually beneficial to undertake a second desk study. Maps and reference materials that were mentioned during the visit can be located and used for additional planning. Consultation with other actors can be undertaken, including the local county water office if appropriate.

10.18 Reconnaissance Visit Reports

A brief report on the reconnaissance visit is recommended. It should cover all of the topics listed above and it should make a recommendation as to whether the proposed site should or should not be considered for storage development. If the site should be considered for storage development, the report should highlight any emerging issues that require further investigation.

A proposed reconnaissance visit report outline is provided in Chapter 19.

CHAPTER 11

FEASIBILITY STUDY

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11 FEASIBILITY STUDY

A feasibility study for small scale storage looks at most of the same issues that the full design addresses. The main difference between the feasibility and design reports are that the feasibility report looks at a broader selection of options and does not address the same level of detail that the design report covers once a recommended option has been agreed upon.

In general, the field work required for a feasibility report is the same fieldwork required for a full design. This includes a complete survey of the site as well as visits throughout the catchment and downstream areas.

The following sections describe the steps that should be taken when conducting a feasibility study for a proposed site. They are laid out to match as closely as possible the feasibility study reporting format set out in Chapter 19.

11.1 Executive Summary

This should be a brief (one page is normally sufficient) description of the project. It must state the background and purpose of the project. It should summarize the expected costs, the beneficiaries and the technical details of the project. It should present any viable alternatives and lay out the way forward.

11.2 Background and Purpose of the Proposed Storage Structure

The proposed project should be fully described in the report.

11.2.1 Stakeholders

An assessment should be made of the project stakeholders and contact details of relevant people (owners, managers, caretakers, local WRUA, etc) should be obtained.

11.2.2 Purpose of the Project

The feasibility report should clearly state the purpose of the project (e.g. to store 20,000 cubic meters of water in a pan to increase the time when grazing can occur in the sub-location). The purpose must be attainable by the proposed project.

11.2.3 Location

The location of the proposed project should be correctly identified and detailed. This includes land registration details where available, ownership details, GPS coordinates and datum. This information will be used to develop the relevant layout maps. Ideally, at least two maps should be produced. One should show the proposed storage location and the extents of the catchment area and the other should show the site layout that shows possible dam alignments and flooded areas.

Details of the county, sub-county, location and sub location should be captured, as well as a summary of how to access the site. This information will be included in the feasibility report.

11.2.4 Details of Site Visits

The report should show the time frame for site visits, identify who went to site and identify who was contacted in the project location.

11.3 Analysis of Alternative Options to Meet Project Objectives

This should briefly touch on what other alternatives exist that might also meet the project objectives. It does not have to examine the alternatives in detail but it must give sufficient details to explain why they are not being considered as a way forward.

11.4 Analysis of Water Demand

The study should look at the expected water requirements. Expected water use figures for a variety of design options can be found in Section 3.3 of this document.

For specialized water use or requirements, the details should be clearly captured and documented in the feasibility report.

11.4.1 Offtakes and Other Structures

This will involve the identification of possible sites for offtakes, community water points, cattle troughs, pump houses, fencing and other associated structures. These points can be mapped out and detailed on the layout map submitted in the feasibility report.

This section is mainly to help finalize cost estimates. Final details and drawings for offtakes and structures will be concluded in the design process.

11.5 Site Investigations

Details should be given on the site topography and on the soils and geotechnical information. If additional information is required for the design it should be identified here.

11.5.1 Impoundment Area Details

A full survey of the impoundment area will be required. This will also include identification and marking out of the possible locations of dam components (spillway, embankment, etc…). A contour map of suitable scale should be produced. Various dam alignments and heights can then be considered to produce a selection of possible storage options.

Survey data and contour maps for each option should be included in the report. Section 9.1 provides details on the intensity of survey points and survey beacons.

If relevant, the site maximum site storage should also be presented.

11.5.2 Results of Geotechnical Investigations

The results of any geotechnical investigations should be presented. At a minimum these should include test pit details and borrow material details, plus the laboratory analysis of the soils sampled from the test pits.

This section should also identify any further investigations that may be needed as part of the final design.

11.5.3 Recommended Storage Structure

Details of the recommended storage structure should be given along with justifications for choices.

11.6 Environmental and Social Considerations

Any emerging environmental considerations should be identified and examined during this phase. Essentially, the feasibility study provides an opportunity to scope out the work involved in conducting the Environmental and Social Impact Assessment, as described in Chapter 6.

In arid areas, special attention should be paid to grazing concerns and the potential for human/wildlife conflict.

In more built up areas, the risk of failure of the embankment on downstream settlements and/or developments should be examined.

Any resettlement or other legal issues that may arise as a result of the project should be identified at this point.

11.7 Hydrological Analysis

The full hydrological analysis for the project should be carried out during the feasibility stage. Refer to Chapter 8 for details.

11.7.1 Inflow Estimation

Catchment details should be determined. They should include catchment area, soil types, vegetative cover and catchment condition. Catchment elevations and flow path elevation and length details should be determined.

Inflows for the dam should be estimated. This can be done based on general annual rainfall averages, on monthly or daily stream flow or on monthly or daily rainfall data. See Section 8.8 for further details.

All data (and their sources) should be recorded in the report.

11.7.2 Reservoir Simulation or Modelling

Reservoir simulation can be carried out based on the storage options, the expected water use, the estimated inflow estimations, as well as the evaporation and seepage estimates. This is discussed in Chapter 8 and can be done graphically, via spreadsheet calculations or via commercial software.

The result of the reservoir simulation is to examine a selection of storage options to determine which option best fits the inflows and water use.

11.7.3 Estimated Spillway Sizes and Inflow Design Flood

In order to produce a cost estimate as part of the feasibility study, a spillway design must be carried out. At this stage, it is sufficient to base the spillway design on an estimated flood based solely on the catchment area. Table 8-6 provides general guidelines for design flood estimation based on catchment area.

Once a design flood has been estimated, initial estimates of spillway details (width, depth, alignment, etc) can be calculated and presented in the report. Table 11-1 gives preliminary estimates for required spillway widths for a variety of design floods and approach heights. The widths have been calculated based on the broad crested weir formula.

Table 11-1: Spillway Widths for Various Design Floods and Approach Heights

Design Flood (m^3/s)	Required Width for 0.5m Approach Depth (m)	Required Width for 1m Approach Depth (m)	Required Width for 1.5m Approach Depth (m)	Required Width for 2.0m Approach Depth (m)
5	9	N/A	N/A	N/A
10	17	N/A	N/A	N/A
15	25	9	N/A	N/A
20	34	12	N/A	N/A
25	42	15	N/A	N/A
30	50	18	10	N/A
40	67	24	13	N/A
50	84	30	16	11
75	126	44	24	16
100	N/A	59	32	21
150	N/A	89	48	32
200	N/A	119	64	42
250	N/A	147	80	52
300	N/A	177	96	63

11.7.4 Sediment Inflow

Sediment inflow should be estimated as laid out in Section 8.13. Sediment inflow will affect the life span of the dam. The NWMP 2030 suggests the following design criteria for water supply infrastructure:

- Rural = 20 yrs
- Urban = 50 yrs

11.8 Identification of Design Issues

Identification of design issues that will be addressed during the full design should be mentioned in the feasibility report. In general, they will involve revising spillway dimensions, finalizing offtake arrangements, determining details on ancillary structures, and additional modelling that might be necessary for larger projects, etc.

11.9 Construction Plan

A project construction plan should be developed. It should present tentative time frames for design work, any necessary social interventions, NEMA and WRMA approvals, actual construction period and a tentative date for completion of works.

A critical path analysis can be included to emphasize the key steps that must be achieved.

11.10 Cost Estimate

The feasibility study must present a cost estimate for the project. The cost estimated should be based on expected construction costs and calculated from a preliminary bill of quantities for the project.

In general, in 2014, construction costs of small scale water storage in Kenya ranged from Ksh. 50 per cubic meter to over Ksh. 500 per cubic meter of water stored. The lowest costs are usually seen in large reservoirs (500,000 to 1,000,000 m^3 of storage) while the highest costs are usually seen in smaller reservoirs. (5,000 to 20,000 m^3 of storage). These figures are based on construction experience in Kenya over the last 10 years and compare well with the NWMP 2030 estimate of Ksh. 333 per cubic meter for small dams.

11.11 Economic and Financial Considerations

The feasibility study should look at the expected cost of the project and compare it to alternative options and average costs in Kenya. A cost per beneficiary can be established and compared against the alternative options. Cost per beneficiary for a variety of alternative options can be established from the Project Unit Costing, WSTF, and (June 2011) or from the NWMP, 2030. A cost/benefit analysis should be conducted even if the scale of the project and the detail at this stage does not warrant a detailed analysis. However the total project cost, covering design, construction, supervision, and all environmental and social mitigation measures can be estimated. This may not reflect the true cost of all the impacts but assumptions regarding the estimation process can be described.

The benefits of the project should be described and estimated and compared to the costs. The intention is to avoid making investments in projects that cannot be justified due to the cost. The willingness and ability of project beneficiaries to pay (or assist in paying) for the proposed project should be stated.

11.12 Project Financing

The feasibility report should identify possible financing sources and raise any financial issues that may affect the project.

11.13 Analysis of Risks and Proposed Mitigation Measures

Results of any risk analysis work should be included. If mitigation measures have been identified, then details should be provided. If no mitigation measures have been finalized, then the report should describe in detail what work still needs to be carried out to ensure that the project can move forward.

11.14 Conclusions

The conclusions should address:

- Legal, Social and Environmental feasibility;
- Technical feasibility;
- Financial details/feasibility;
- Economic Feasibility.

They should be clearly stated in the executive summary.

11.15 Recommendations

The recommendations should summarize:

- Measures required to enable the project to meet its objectives;
- Way forward.

They should be clearly stated in the executive summary.

11.16 Annexes to the Feasibility Report

There is often a long period between the initial feasibility report and the implementation of the project. Social, environmental and legal issues must be dealt with, financing must be secured, and other government approvals must be obtained. In order to keep continuity between the feasibility work and any future design work, it is strongly recommended that the feasibility report has annexes with site photos and all relevant maps. These can be extremely important for refreshing memories and for introducing new team members to the project.

CHAPTER 12

DESIGN OF EARTHFILL EMBANKMENT DAMS

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12 DESIGN OF EARTHFILL EMBANKMENT DAMS

This section deals with the design aspects of earthfill embankments. Where the site conditions and soil test results are favourable a safe and economical design for structures within the feasibility limitations set out in Chapter 10 can be achieved through careful application of the requirements specified below. In situations where the standard design requirements cannot be met or where the suitability of site or material conditions is uncertain, then specialised investigations and analysis and the services of a government approved dam design engineer, hydrologist and geologist/geotechnical expert will be required.

Any embankment dam must meet design requirements for stability under all conditions of construction and operation, and imperviousness, both through and beneath the embankment. This chapter examines the design required for earth-fill embankment dams which are widely used in Kenya. Rockfill dams are not discussed due to their limited utilisation in Kenya, primarily due to the difficulty in providing a robust impermeable membrane over a rockfill embankment.

12.1 Types of Embankment Dams

Earthfill embankment dams in Kenya are generally homogeneous or zoned embankments with a drainage blanket for internal seepage control for structures greater than 5 metres in height as shown in Figure 12-1 and Figure 12-2. The choice of whether to use a homogeneous or zoned embankment will be a function of the availability of suitable materials. Where there are limited quantities of impervious material, more pervious material can be placed on either side of the core creating a shell. The most economical type of dam will usually be the one for which materials can be found within the site or a reasonable haul distance.

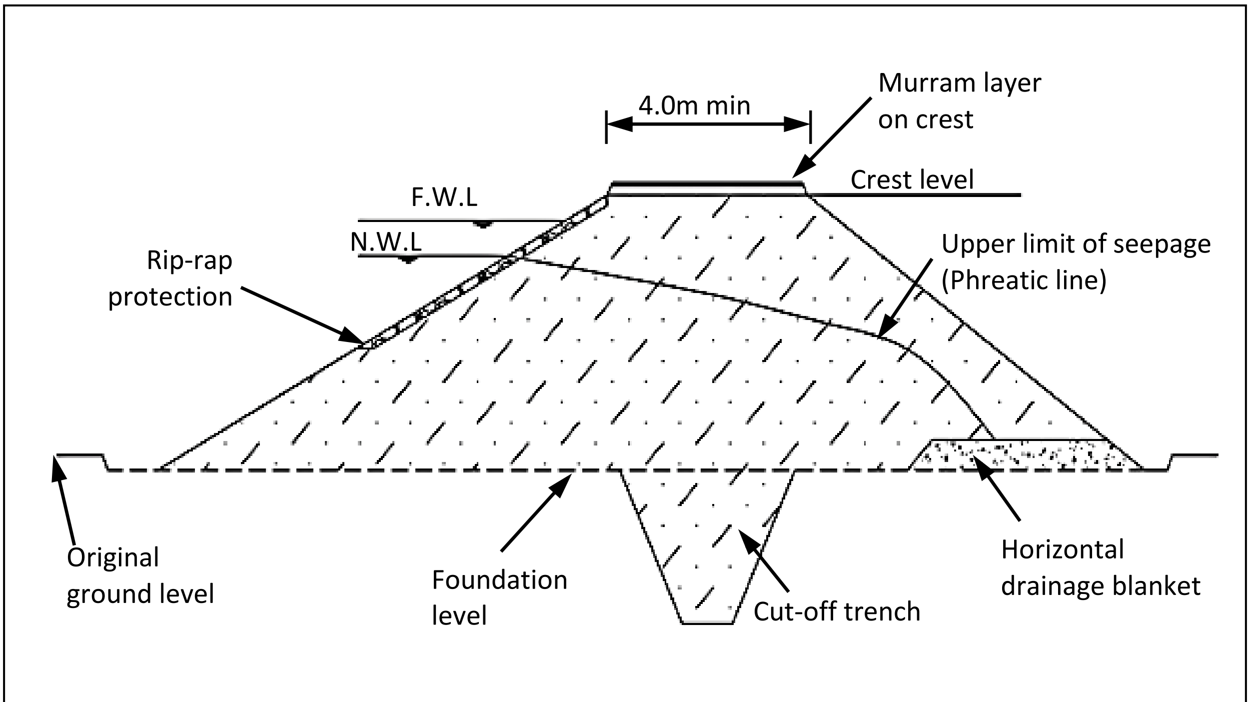

Figure 12-1: Homogenous Earthfill Dam with Drainage Blanket

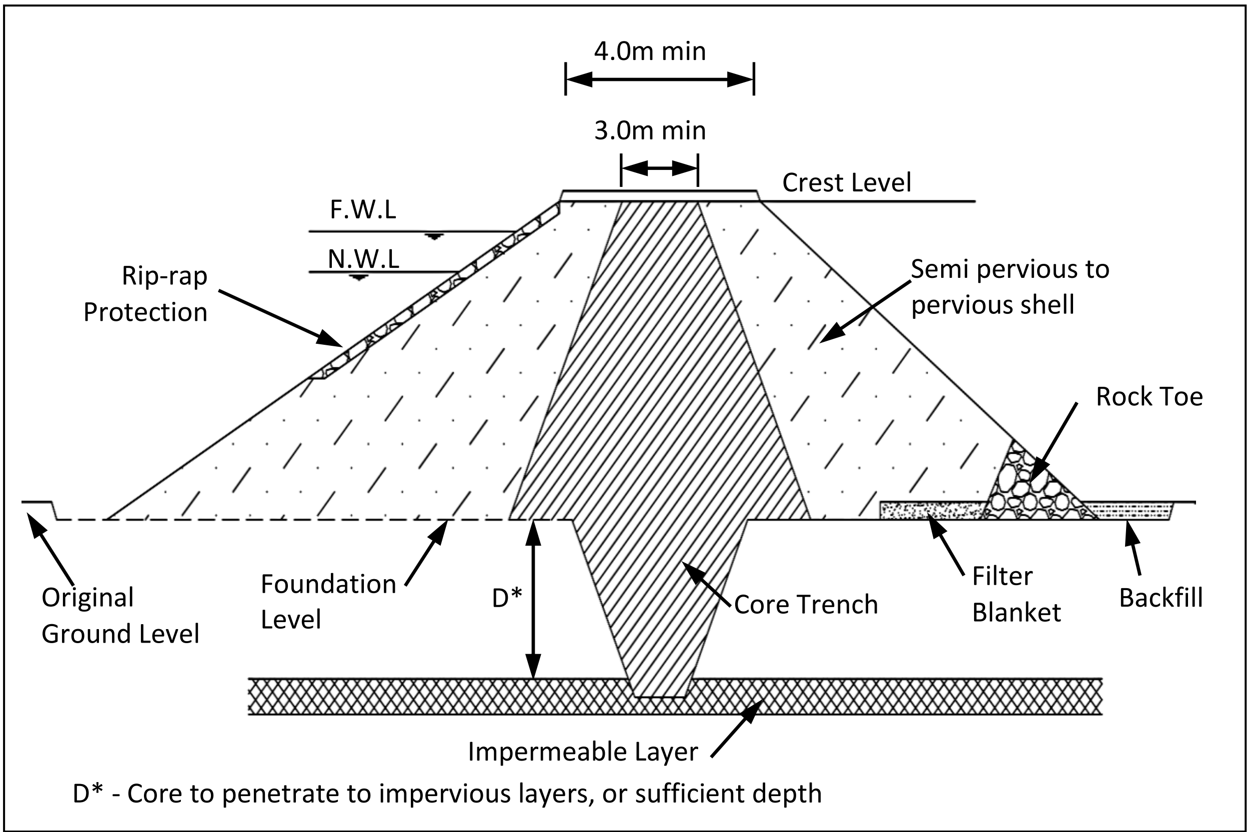

Figure 12-2: Zoned Earthfill Dam

12.2 General Guidelines for the Design of Embankments

12.2.1 Design Criteria

The basic requirements for the design of an embankment dam are to ensure:

i) **Safety against overtopping**. This is a function of the spillway capacity and freeboard;
ii) **Stability**. The stability of the slopes should be considered for the case of construction, steady state and rapid drawdown. Acceptable values for upstream and downstream slopes are provided in Table 12-3;
iii) **Safety against internal erosion**. The selection of material for the downstream shell and the design of internal drainage blanket and toe drain address this aspect;
iv) **Functional performance in terms of excessive seepage**. The design of the cut-off and impervious core address this aspect.

12.2.2 Dam Axis

The location of the dam axis should be chosen in such a way that the amount of fill required for the embankment is minimal. Usually the most appropriate location will be indicated by a narrowing of the contour intervals on the topographical map (see also Section 10.7).

The dam axis should normally be designed straight, unless special topographical features impose a curved axis.

Consideration should be given to the stability of the abutments and to avoid abrupt topographic discontinuities which can lead to differential settlement and shear cracks in the embankment.

12.2.3 Height of Embankment

The height of the embankment should be determined in accordance with the water depth calculated in Section 8.12 (Determination of the Required Storage Capacity) and then increased by the required gross freeboard (GF) which is a function of the width of the spillway; the wider the spillway, the lower the gross freeboard. This means that the final embankment height should be established through an iterative process which considers the cost of spillway excavation and the cost of embankment construction as the cost of the intake and other structures is constant irrespective of the height of the embankment.

An extra allowance or camber should be provided along the crest of earthfill dams, to ensure that the freeboard will not be diminished by post-construction settlement of the dam and the foundation. For small earthfill dams on relatively non-compressible foundations, a camber of about 2% of the embankment height (with a minimum of 0.20m) should be provided. Linear equations should be used to vary the amount of camber, and make it roughly proportional to the height of the embankment.

Figure 12-3 and Figure 12-4 show a diagrammatic cross-section and lay-out plan respectively of a small earth embankment.

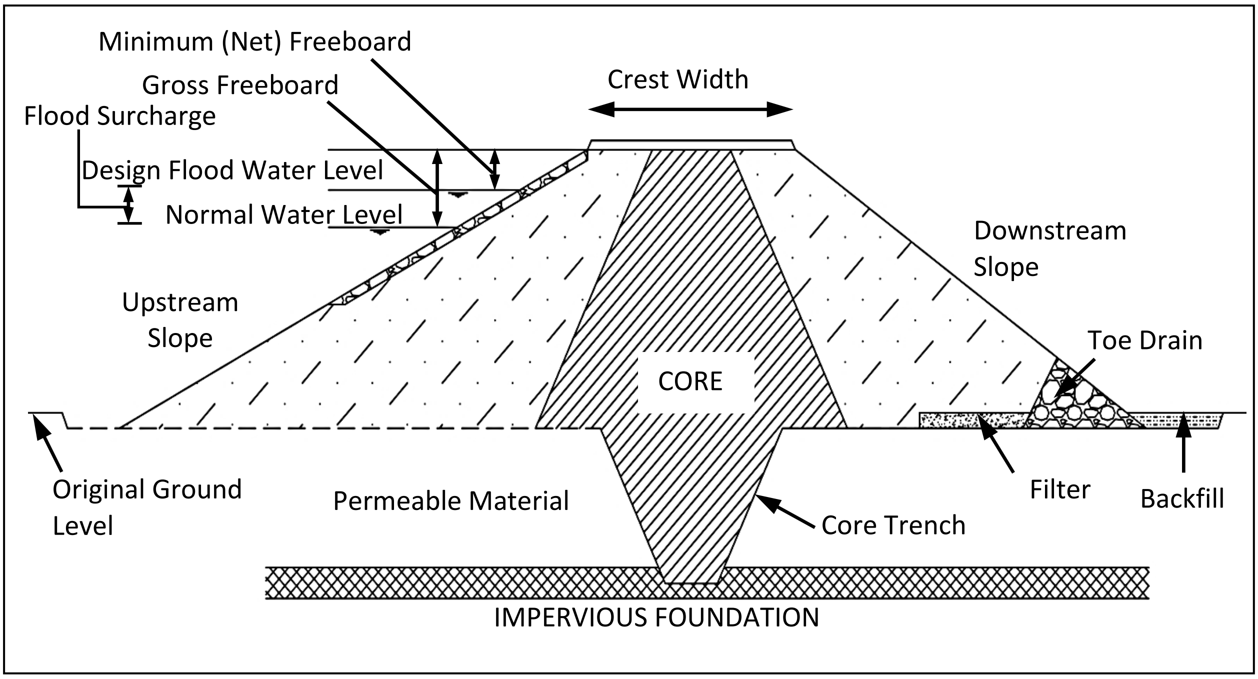

Figure 12-3: Cross Section of a Small Earth Dam

It should be noted that dam heights less than five metres be carefully considered as the freeboard is usually 1.00 – 1.50 metres and evaporation in arid areas is above 2.00 metres with the result that the effective storage available for use is less than is justified by the cost of the project.

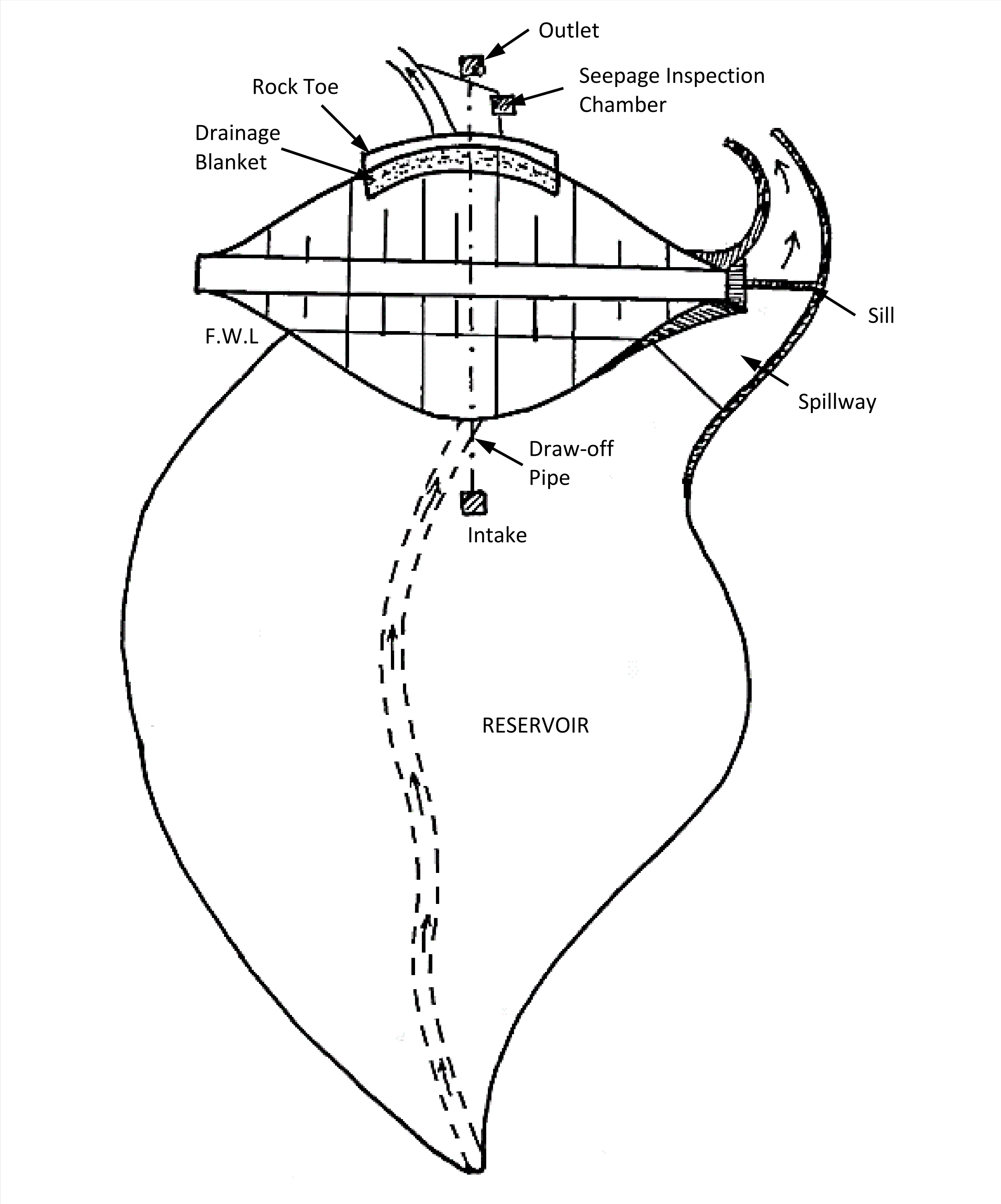

Figure 12-4: Layout Plan of a Small Earth Dam

12.2.4 Dam Freeboard

The embankment crest must be sufficiently higher than the maximum design water level in the dam to prevent any overtopping, including the possibility of waves washing up the embankment. The critical condition is when the inflow design flood (IDF) (for the design return period) is passing through the spillway. The net freeboard (NF) is defined as the minimum freeboard that occurs when the spillway is flowing at its maximum design flood capacity. The gross freeboard (GF) is therefore the minimum freeboard (NF) plus the water depth in the reservoir (h_A) above the spillway crest when the IDF is passing.

Equation 12-1 $$GF = h_A + NF$$

Where: GF = Gross Freeboard (m)
h_A = water level in reservoir when spillway is passing inflow design flood (m)
NF = net or minimum freeboard (Table 12-1)

The WRM Rules (2007) specify a minimum freeboard of 0.6m for Class A dams and 1.0m for Class B and C dams, unless otherwise specified by the WRMA. MoWI (2005) provides a relationship between fetch and minimum freeboard which has been summarised in Table 12-1. The more conservative value should be used. The water depth (h_A) is established from the spillway design described in Section 12.3.

Table 12-1: Fetch and Minimum Freeboard

Fetch (Km)	Minimum Freeboard (m)
0 – 0.10	0.80
0.10 – 0.50	1.00
0.50 – 1.0	1.10
1.0 – 3.0	1.30
3.0 – 5.0	1.60
> 5.0	Reference should be made to publications for the required minimum freeboard

12.2.5 Crest Width

The main criteria for crest width is related to construction and post-construction use of the crest, rather than slope stability. The crest width (CW) should therefore comply with **Table 12-2**.
Consideration should be given to the camber and surface dressing of the crest. The crest should be sloped at 1% to shed rainwater. The crest should be dressed with a minimum of 200 mm of compacted murram or gravel. This provides a hardwearing surface that can handle periodic light traffic and is less likely to erode.

If the crest will not be used by traffic, it can be grassed which requires at least 200mm of top soil, lightly compacted, into which grass splits are planted. The grass species that are suitable are creeping (stoloniferous) grasses which cover the ground closely. These species include Kikuyu, Signal (*Brachiaria humidicola*), Bahia (*Paspalum notatum*), and Star (*Cynodon spp*) grass. Grass species that form tuffs should be avoided.

Table 12-2: Crest Widths

Depth of Water (m)	Minimum Crest Width (m)	Comments
0 – 3.0	3.00	Note: minimum width for machinery access is 4.00 metres. A comfortable roadway width is 6 metres
3.1 – 5.0	4.00	
Greater than 5.0	5.00	

12.2.6 Impervious Core for Zone Embankments

For dams of 5 - 15 metres high, where suitable soil is not available in sufficient quantities for constructing a homogeneous embankment, the construction of a "zoned" embankment can present a solution. In such cases a core of impervious material (generally clay) is incorporated in the embankment (See Figure 12-2), while more pervious fill material (a soil containing more sand than would normally be admissible) can be utilised for backfilling the shoulders of the embankment. The more pervious material on the downstream shoulder serves to lower the phreatic line to keep it within the embankment. A more granular material on the upstream also helps to reduce the uplift pressure under the embankment.

In the case of a zoned embankment, the impervious core should by designed with upstream and downstream slopes of 1.5:1 and should constitute at least 30% of the cross sectional area. The impervious core should penetrate through the cutoff trench to the impervious foundation layer. The top of the impervious core should exceed the flood water level.

12.2.7 Embankment Slopes

Embankment slope stability usually considers three critical conditions, namely:

1. Sudden drawdown. This is a post-construction condition that assumes that the reservoir water level has dropped but the upstream face remains saturated;
2. Sudden post-construction drawdown.
3. Steady state. This assumes that the water level is at full supply level;

Embankment slope stability[1] depends on the type of fill material used and on the height of the embankment. Analysis of the slope stability for different embankment heights and fill material has informed the recommended steepest slopes given in Table 12-3 for well compacted material.

Table 12-3: Recommended Slopes for Earth Embankments

Embankment Height	Fill Material Type	Casing Slopes (H : V)	
		Upstream	Downstream
< 5m	Well distributed granular/clay mix (GC, SC, CL, CH)	2.5 : 1	2.0 : 1
5 m to 10m	Well distributed granular/clay mix (GC, SC, CL)	2.5 : 1	2.5 : 1
10 to 15 m	Well distributed granular/clay mix (GC, SC, CL)	3.0 : 1	2.5 : 1

Incorporation of a clay-core does not affect the slopes of the embankment. However, in cases where particularly bad foundation conditions occur, it is advisable to select flatter slopes.

12.2.8 Embankment Foundation

The complete foundation area of the dam should systematically be cleared of all vegetation and topsoil containing organic matter including the removal of all logs, tree stumps, and unconsolidated material. Sand and/or silt from the river bed will also need to be cleared.

Adequate measures should be taken to eliminate steep slopes from the foundation area. No slopes steeper than 25% can be tolerated without special considerations. Steep slopes can create sliding planes through unequal embankment settlement against the original ground, thus creating seepage paths for water. Where steep river banks are encountered, these should be smoothed out as part of the foundation preparation work.

[1]In cases where particularly heavy clays (which are in principle unsuitable for embankment construction) will nevertheless be utilized for the construction of the embankment, an upstream slope of 1(h):3(v) can be adopted for embankments less than 5 m.

12.2.9 Core Trench

A core or cut-off trench is generally used to prevent seepage under the dam, by cutting off seepage paths through underlying pervious layers (see Figure 12-3). The core trench should extend up the abutments to the height of the normal water level.

The core trench should penetrate into impervious material by a minimum depth of not less than 1.00 metre. MoWI (2005) recommends a total core depth of $^1/_3$ to ½ embankment height for economical reasons. However the project engineer should make a conscious decision regarding the depth of the core trench. The final excavation depth of the core trench will be determined once the core trench is fully exposed. However the estimated depth can be established based on test pit details. Note the depth of the core trench is generally not uniform because of additional excavation required at the intersection with the water course to ensure the impervious layer is fully penetrated. Alternative alignments should be considered if seepage under the embankment cannot be controlled.

The bottom width of the core trench is determined by the excavation width of the machines which are to be utilized (usually 1.5 times the machine width is normally accepted) with a minimum width of 3 metres.

The side slopes of the core trench should be a minimum of 1:1 or flatter in overburden or 1(h): 2(v) in soft/hard rock to provide sufficient contact between the core material and the undisturbed material and to reduce the likelihood of differential settlement causing tension cracks which create seepage flow paths. In addition, sufficient side slope enables proper compaction right up to the edge of the core-trench.

12.2.10 Grout Curtain

Dams placed on fractured rock may require treatment such as a grout curtain to minimise seepage below the embankment. The associated investigations, design and construction of a grout are not covered in this document. The reader is referred to other reference material for information on the investigations, design and construction of grout curtains and the services of a qualified geotechnical engineer will be required.

12.2.11 Filter Blanket and Toe Drain

A horizontal filter blanket and toe drain (see Figure 12-3 and Figure 12-4) are important for seepage control to capture the phreatic line within the downstream embankment and to relieve uplift pressures on the downstream side of the embankment. Filter blankets and toe drains are normally only used for dams exceeding 5 (five) meters in height.

The filter blanket must satisfy three design requirements:

1. The filter material acts as a filter to prevents ingress of the embankment material into the filter;
2. The filter material acts as a drain and should be sufficiently porous to alleviate seepage uplift forces and to drawdown the phreatic line;
3. The filter should have sufficient capacity to convey the total seepage from both the foundation and embankment.

A horizontal drainage blanket is usually composed of clean river sand free from any organic matter. Ideally the filter blanket should consist of graded material (usually sand and graded ballast) and a toe drain which satisfies the recommended USBR filter criteria (United States Department of the Interior - Bureau of Reclamation, 1987). These criteria are:

Equation 12-2: $\frac{D_{15}F}{D_{15}B} \geq 5$

Equation 12-3: $\frac{D_{15}F}{D_{85}B} \leq 5$

Equation 12-4: $\frac{D_{15}F}{PD_{max}} \geq 2$

Where: D_{15} = particle diameter for which 15% of the soil is smaller [mm]
(Note: filter material should not contain more than 5% of material smaller than 0.074 mm (No. 200 sieve))
D_{85} = particle diameter for which 85% of the soil is smaller [mm]
F = filter material
B = base material
PD_{max} = maximum opening (holes) in the pipe drain [mm]

(Note: DF_{15}, DF_{85}, DB_{15} and DB_{85} are determined from the particle size analysis described in Section 9.4)

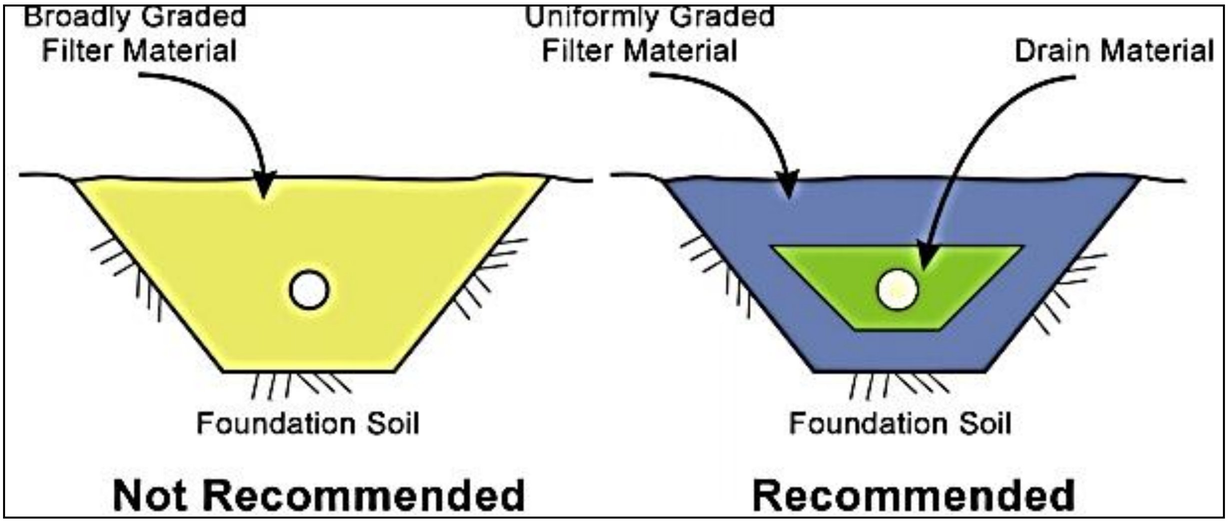

Figure 12-5: Filter Drain

Synthetic geotextile membranes have been used as an interface to assist in meeting the particle size requirements for a graded filter. However, FEMA (2011)[2] does not recommend the use of synthetic geotextile membranes where the membrane would be buried and its failure could compromise dam safety.

The thickness of the drainage blanket should not be less than 1.00 meter, while a width of at least 5.00 meters is recommended. The drainage blanket should be extended up to an elevation of 4 to 5 meters below the embankment crest. The position of the drainage blanket within the embankment cross section should be such that there is at least one metre of material on top of the blanket at the downstream point (as shown in Figure 12-3). The drainage blanket is typically placed horizontally on the foundation surface. However, alternative geometry, alignments and placements should be considered where there is concern regarding potential seepage paths beneath the core trench or within the abutments.

A toe drain consisting of graded ballast placed against or below the drainage blanket can be used to increase drainage capacity. A perforated pipe (e.g. perforated corrugated HDPE pipe), acting as a collector drain, is placed within the toe drain to convey seepage water away from the embankment. The collector drain should pass through a chamber where seepage flow rates can be observed and monitored. Care should be taken during construction to avoid crushing the pipe drain.

[2] FEMA 2011 Filters for Embankment Dams; Best Practices for Design and Construction. Federal Emergency Management Agency, USA

12.2.12 Rock Toe

A rock toe, composed of variable size rip-rap (25 – 250mm) can be placed along the groin of the embankment to protect the embankment against erosion. The rock toe can be 1 metre in height and placed at a 1.5 (h): 1(v) slope. The rock toe is distinguished from the toe drain which has graded material designed to convey seepage water away from the embankment.

12.2.13 Upstream Slope Protection

In cases of long reservoirs (fetch > 500m) a protective layer of hand placed rip-rap (rubble stone/hardcore) should be placed on the areas of the upstream embankment slope which are likely to be affected by the wave action. This zone is usually 0.6 metres above the normal water level to 2/3 of the water height. The thickness of the rip-rap layer should not be less than 0.30 metres. A gravel blanket (min 150mm) will normally be provided under the rip-rap layer. The bottom toe of the rip-rap layer needs to be keyed into the embankment face to prevent gradual movement of the rip-rap down the slope. This can be achieved by the construction of a step or inset at the appropriate height along the embankment face. The space between the top line of the rip-rap and the crest can be grassed to reduce erosion.

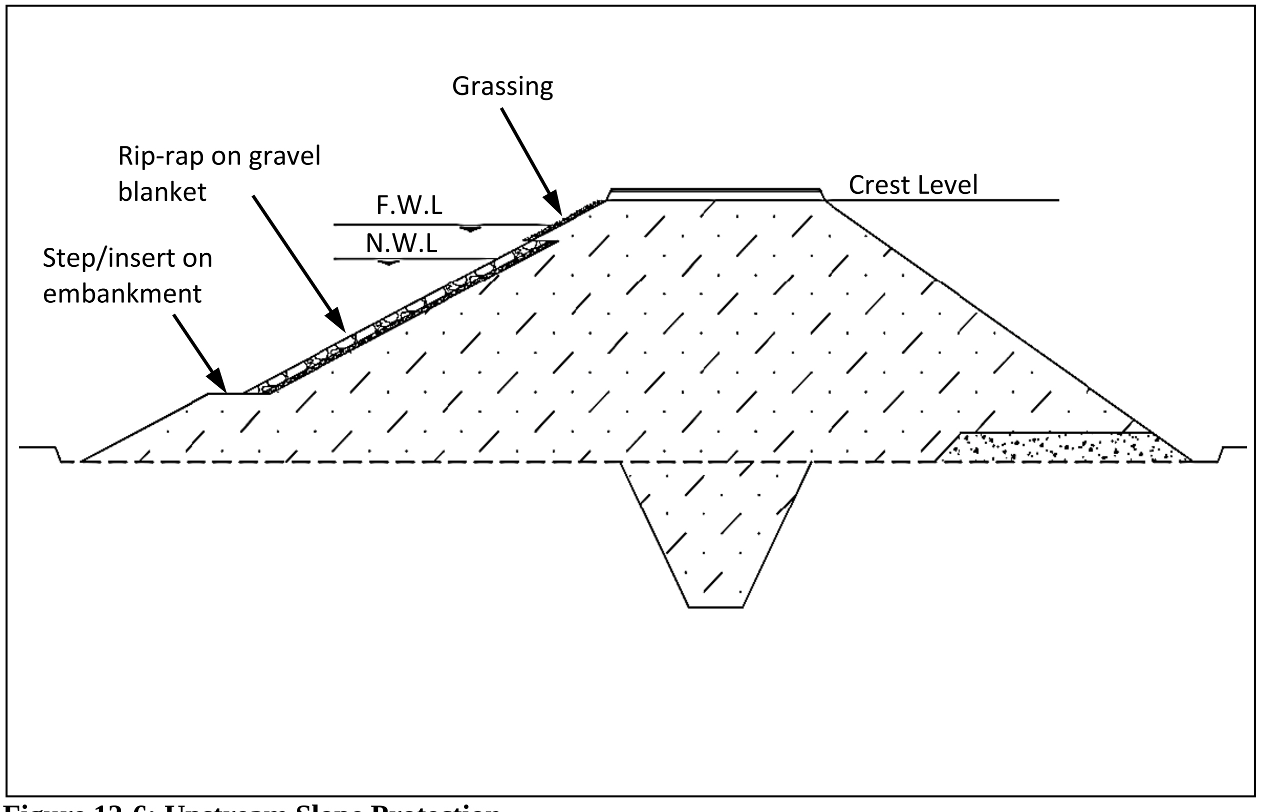

Figure 12-6: Upstream Slope Protection

12.2.14 Downstream Slope Protection

The best form of erosion protection for the downstream face is a good cover of creeping grass (e.g. Kikuyu, Signal (*Brachiaria humidicola*), Bahia (*Paspalum notatum*), and Star (*Cynodon spp*) grass). Grass species that form tuffs should be avoided. In order to obtain a good grass cover, a layer of top soil (200 – 400 mm thick) is placed on the downstream face. This requires particular attention during construction to schedule stockpiling of top soil and inclusion of the top soil along the downstream face during the construction process.

12.3 Design of Spillway Structures

The function of the spillway is to discharge the normal and flood flows safely around the embankment and back to the water course without compromising the long term functionality and integrity of the dam.

12.3.1 Location and Type of Spillways

The common type of spillway used with earth embankments is a side channel spillway, excavated in earth or rock next to the embankment. The incorporation of relatively large concrete structures as spillways for small earth dams is difficult to justify on economical grounds.

The basic factors to be taken into account when choosing a spillway location are:

i. The spillway should be kept away from the embankment in order to avoid the need for concrete protection structures, and
ii. Excessively steep valleys should also be avoided, in order to prevent erosion problems in the spillway channel and to reduce excavation volumes.

Consequently spillways are usually located on the side of the embankment where the valley slopes are flattest. In the case of large discharges to be catered for, the possibility of constructing two spillways - one on either side of the embankment- can be considered; the quantity of excavation required usually being the decisive factor. In cases where the topography of the site favours such a solution the possibility of discharging the flood waters into a valley other than the original river valley can also be considered. This could however have adverse effects on eventual water users downstream of the dam and on the flow regime of the other river.

Because of the cost of rock blasting, extensive excavation in rock should be avoided, but the location of the spillway channel on a relatively horizontal layer of bedrock is wherever possible a handsome solution to all erosion problems in the spillway channel. Problems with spillway channel erosion prohibit the construction of spillways on backfilled soil. Spillways should always be excavated in original material.

It is always preferable to let spillway channels discharge on bedrock. Where this is not possible, it is advisable to protect the river-bed from scouring at the location of the spillway discharge. Lining with reno-mattresses, gabions or pitched stone is usually appropriate.

Only side channel spillways excavated in earth or rock will be considered. For all other types of spillways, reference is made to the United States Department of the Interior - Bureau of Reclamation, 1987. A site may require a side spillway on both sides of the embankment.

The side spillway normally consists of three parts: Inflow Section, Control and Outflow Channel (see Figure 12-7) and Drawing Type III in Appendix B.

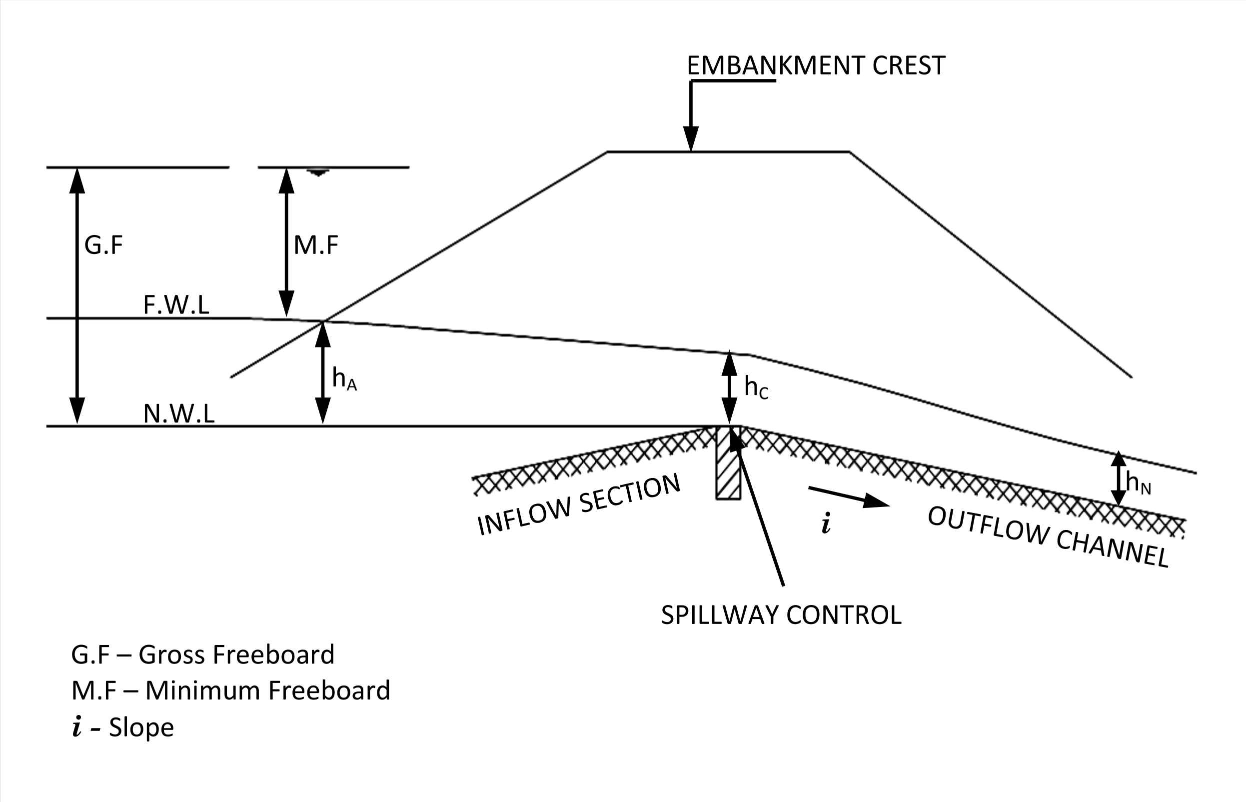

Figure 12-7: Spillway Design

12.3.2 Control Section

The normal water level in the reservoir is controlled by the height, length (i.e. width of spillway channel) and geometry of the spillway sill. The sill level is controlled by a reinforced concrete sill (minimum width 300 mm), thus preventing lowering of the crest level by erosion. This sill is usually aligned with the dam axis. The depth of the sill (minimum 1.00 m) below ground level should be determined by the engineer to minimise seepage underneath the sill. Where the sill is proud of the spillway bed and there is a risk of erosion and undercutting of the sill, a 150 mm thick reinforced concrete apron should be placed downstream of the sill. The width of the control section should be a minimum of 10 m unless a detailed analysis justifies otherwise.

Consideration should be given to the likelihood of erosion along the spillway floor and side slopes, particularly in the control section. Grouted masonry can be laid along the floor and side slopes to protect against erosion where the spillway is cut into soil and where a good grass cover cannot be guaranteed.

12.3.3 Inflow Section

The inflow section leads the flood water to the control section. Usually, it slopes moderately (maximum 1 %) upwards to the sill. The cross-section is usually narrowed down gradually towards the sill. Care should be taken that the water flowing to the control section remains far enough from the earth embankment to minimise the risk of erosion of the embankment face.

12.3.4 Outflow Channel

The outflow channel discharges the flood water back into the riverbed at acceptable velocities that do not cause erosion. For spillways excavated in undisturbed earth, a maximum velocity of 2.5 m/s is usually acceptable under Kenyan conditions. Control of the outflow channel water velocity is usually achieved through adequate slope selection. Otherwise lining of the channel (or parts thereof) with rip-rap will be required. In such cases velocities up to 6-7 m/s can be accepted. In case of unacceptably long outflow channels, the possibility of incorporating a gabion or concrete drop structure can offer a solution. The Manning formula (Equation 12-5) can be used to establish the velocity in the outflow channel for different gradients, widths and channel roughness.

Recommended values for outflow channel slopes are presented in Table 12-4.

Table 12-4: Recommended Values for Outflow Channel Slopes

Type of Soil	Slope (%)
Earth	<0.5
Murram	0.5 – 1
Hard rock	1-2

Consideration should be given to the velocity of the water as it re-enters the water course as this can create unwanted erosion of the river bank. There are various options to reduce the speed of flow including changing the slope to induce a hydraulic jump, creating a stilling basin or placing chute blocks in the line of flow. Reference should be made to detailed design documents where energy dissipaters are required. Energy dissipaters should not impede flow through the control section.

12.3.5 Training Walls for Control and Outflow Channel

Training walls are required if the control and outflow channels are cut into soil to prevent erosion of the channel side walls. The height of the training wall should exceed the flood water level. A masonry wall, anchored on a secure footing, with appropriate buttresses, is acceptable for heights less than 2 m. A reinforced concrete retaining wall is required for wall heights above 2 m. Well constructed gabions are feasible where wall heights are less than 1m. The services of an engineer should be engaged to establish the full design for a reinforced concrete retaining wall.

12.3.6 Return to Water Course

The point at which spillway flows join the water course should be examined and protected against erosion that may occur if high velocity flows are expected. Maintaining well vegetated river banks, or placing well constructed gabion boxes, are options to minimise river bank erosion. See also the discussion on options for reducing flow velocities in Section 12.3.4.

12.3.7 Determining the Height, Width and Slopes of the Spillway

The design of the spillway determines the water level in the reservoir or approach height (h_A). There are two conditions that can apply:

1) Flow in the outflow section is supercritical and the spillway sill acts as a broad crested weir and therefore the sill controls the approach height;
2) Flow in the outflow section is subcritical and the depth of flow in the outflow channel controls the approach height.

It is therefore important to determine which condition applies or to design the spillway width and slopes so that the selected condition applies.

Typically the design of the spillway aims to ensure that the first condition applies i.e. the spillway sill controls the flow and level of water in the reservoir. This is achieved by ensuring that:

i. The capacity of the outflow section exceeds the capacity at the control section;

ii. The flow condition in the outflow channel is supercritical. For channels excavated in soil (n = 0.025) this implies a gradient of more than 0.75% (0.0075);
iii. The depth of water in the outflow channel is less than the depth of flow over the sill.

When these conditions are met, the sill in the control section will act as a broad crested weir.

Flow characteristics in the outflow channel will correspond to the Manning equation as shown in Equation 12-5.

Equation 12-5: $\mathbf{v} = \frac{1}{n} R^{2/3} . i^{1/2}$

Where: v = water velocity [m/s]
n = channel roughness factor equal to 0.025 for earth channels
(See Table 12-5 for appropriate Manning n values))
R = the hydraulic radius of the channel [m]
= [channel cross sectional area]/[wetted perimeter]
i = the channel slope [m/m]

Table 12-5: Manning n Values for Typical Spillway Channel Material

Type of channel and material	Minimum n value	Normal n value	Maximum n value
Concrete lined	0.015	0.017	0.020
Masonry line with cemented rubble	0.017	0.025	0.030
Straight, uniform channel excavated in clean earth	0.018	0.022	0.025
Straight, uniform, earth channel with short grass, few weeds	0.022	0.027	0.033
Straight, uniform, earth channel not maintained with dense weeds	0.050	0.080	0.120
Rock cut – smooth and uniform	0.025	0.035	0.040
Rock cut – jagged and irregular	0.035	0.040	0.050

(Source: Chow (1959))

The water depth corresponding with the Manning equation (the " normal depth" h_N which will occur at sufficient distance downstream from the sill) may be determined by writing the Manning equation in terms of discharge as shown in Equation 12-6.

Equation 12-6: $\mathbf{Q} = \frac{1}{n} A . R^{2/3} . i^{1/2}$

Where: Q = design flow for the specified return period [m^3/s]
A = channel cross-section [m^2]
and substituting for A and R expressions involving h and other necessary dimensions of the channel cross section. The resulting equation can then be solved by trial and error to determine h_N.

When supercritical flow occurs in the outflow channel, the sill (control) will basically play the role of a weir, and the water depth over the sill will be equal to the critical depth h_C. The depth of approach h_A, will then be 1.5 times the critical depth as shown in Equation 12-7. Table 12-6 presents values of approach depth for a range of unit discharge values (q).

Equation 12-7 $$\boldsymbol{h_A = \frac{3}{2}.h_c = \frac{3}{2}.\sqrt[3]{\frac{q^2}{g}}}$$

Where: q = discharge per unit width of the spillway sill [m^3/s/m or m^2/s] = Q/L
L = length of sill [m]
g = 9.81 [m/s^2]

Table 12-6: Values of q and h_a

q [m^3/s/m]	**h_A** **[m]**	**h_c** **[m]**
0.25	0.28	0.19
0.50	0.44	0.29
1.00	0.70	0.47
1.50	0.92	0.61
2.00	1.11	0.74
2.50	1.29	0.86
3.00	1.46	0.97
3.50	1.62	1.08
4.00	1.77	1.18
4.50	1.91	1.27
5.00	2.05	1.37
5.50	2.18	1.46
6.00	2.31	1.54
6.50	2.44	1.63
7.00	2.56	1.71
7.50	2.68	1.79
8.00	2.80	1.87
8.50	2.92	1.95
9.00	3.03	2.02
9.50	3.14	2.10
10.00	3.25	2.17

For supercritical flow conditions, the normal depth of flow in the outflow channel (h_N) is smaller than the critical depth (h_C).

If h_N is greater than h_C the flow in the outflow channel is subcritical, and the depth of approach h_A will depend of the water velocity and height in the outflow as in Equation 12-8.

Equation 12-8: $$\boldsymbol{h_A = h_1 + \frac{5}{4}.\frac{V_1^2}{2g}}$$

Where: h_1 = depth of flow in the outflow section [m]
v_1 = velocity of flow in the outflow section [m/s]

The procedure for spillway design is an iterative process that can follow the sequence described below:

For the inflow design flood (Q), use:

1) Equation 12-7 to test different values of sill length (L) to determine an acceptable approach height (h_A), noting Equation 12-1 that determines the gross freeboard (GF) and the final embankment crest elevation;
2) Select trial widths and slopes for the outflow section and use Equation 12-6 to establish the depth of flow (h_N);
3) Check that flow conditions in the outflow channel are supercritical and that the flow depth in the outflow section (h_N) is less than the flow depth over the sill (h_c);
4) Check that flow velocities are acceptable (less than 2.5 m/s for earth channels, less than 6 m/s for rock lined channels).

12.3.8 Construction Details

Basic construction details for earth channel side spillways are outlined in Appendix B (Type Drawing II - Spillway for Small Earth Dam).

The spillway alignment is usually curved to keep the spillway away from the embankment.

The side slopes of an earth channel spillway should be decided as a function of the material in which the spillway is excavated. Side slopes of 1:1 (for shallow spillways excavated in firm material) to 3:1 (for deep spillways excavated in soft soil) are possible. Spillways should always be excavated in original undisturbed material.

Concrete sills can be constructed at various locations in the outflow channel. Their essential function is to fix the spillway level and act as an erosion barrier. At the spillway crest, the construction of a concrete sill is imperative. Concrete sills should also be constructed where changes of the slope in the outflow channel occur.

Where soft materials or excessive velocities occur in parts of the spillway, a lining with angular rip-rap (made of solid rock and least 0.30m thick) can provide a solution. This rip-rap layer should be provided with an underlying gravel layer, and should be compacted.

Wherever run-off water from the valley slopes is expected to flow into the spillway channel in substantial quantities, the construction of a spillway protection trench (cut-off drain) is recommended. Construction details for this trench are given in Appendix B -Type Drawing II.

12.3.9 Trickle Spillway

A trickle spillway is required where there is likely to be a fairly continuous flow over the spillway. A continuous flow can cause steady erosion along the spillway bed leading to rills and potentially gullies that can threaten the integrity of the dam.

The options for a trickle spillway are:

1) Low section in the spillway sill that directs flow into a lined channel (masonry, concrete, etc). This low section of the sill and the lined channel should be placed along the outside edge of the spillway to keep any risk of erosion or seepage away from the embankment;
2) Pipe (GI, PVC, HDPE) at normal water level, placed through the concrete sill and buried along the outside edge of the spillway. The pipe is vulnerable to being washed out by the flood flows and so should be properly buried, anchored and protected;
3) If the flows warrant, then a culvert can be used to convey continuous discharges along the outside edge of the spillway.

In the event that the normal flows exceed the options above, then other options (e.g. concrete lined spillway, drop-inlet spillway) may be required and the reader is advised to refer to other publications on the topic.

12.4 Design of Draw-Off Works

Due to the risk of pollution of the reservoir from human or livestock contamination, it is preferable to provide a draw off system that delivers water below the dam. However, a draw off system can create a seepage flow path that can compromise the integrity of the embankment unless designed and constructed properly. Consequently the additional cost and construction complications may outweigh the benefits of a draw-off system through the embankment for a small dam.

The design of the draw off system should consider:

- Peak flow requirements to satisfy water demand;
- Variable water level in the reservoir;
- Risk of debris and blockages in the pipe;
- Minimum flow velocities (0.6 m/s) and minimum size of pipe (50 mm dia.) to ensure these are self-cleaning;
- Need to regulate the discharge in the drawoff pipe.

If the draw-off system is also being used to release compensation flows for downstream water rights, then this requirement should also be factored in to the design of the draw-off system.

A typical draw-off system consists of an intake at the bottom of the reservoir with a draw-off pipe passing through the embankment or foundation. This pipeline is then connected to a pump house or valve chamber (Figure 12-8) from where water will usually be provided to the distribution system and consumer points. Provision for compensation flows to safeguard downstream water-rights can also be made from this structure.

If the draw-off works are intended to pass normal river flows then a drop inlet concrete structure may be more appropriate with concrete culvert of sufficient capacity to convey the required flows. Reference should be made to alternative detailed design documents for the design of a concrete drop inlet structure and concrete culverts.

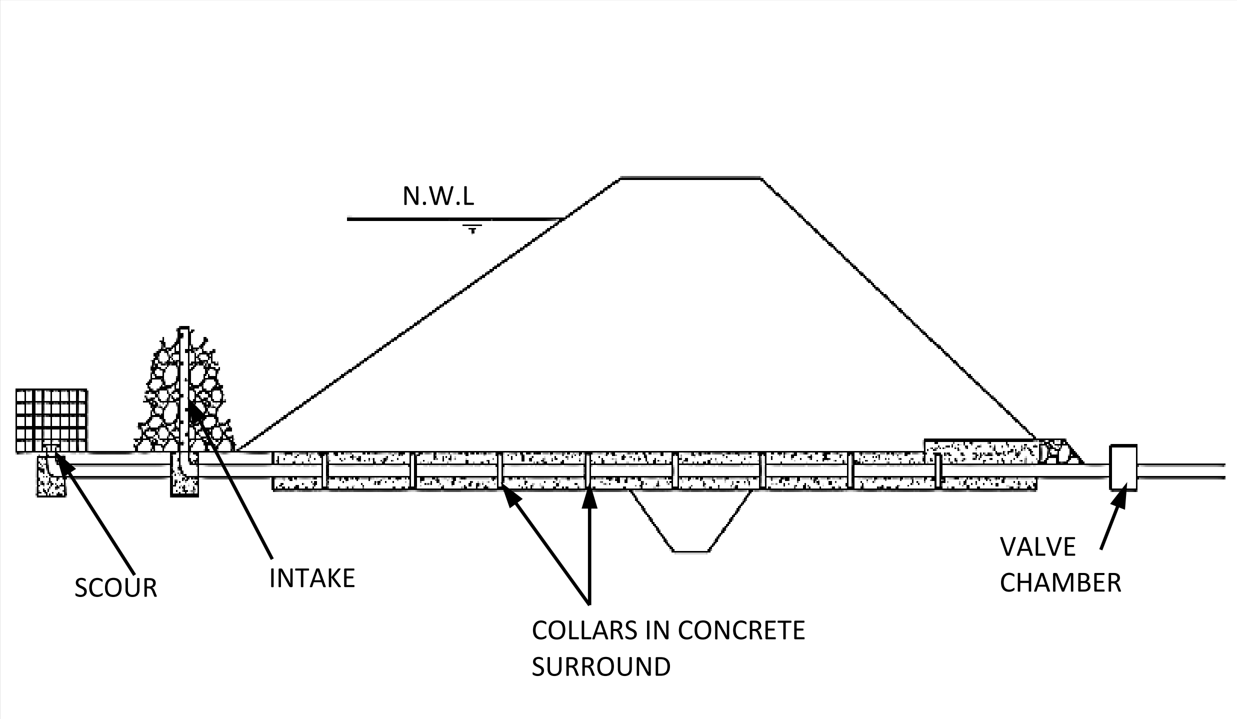

Figure 12-8: Typical Outlet Works for a Small Earth Dam

Type Drawings III, IV and V given in Appendix B show construction details for intake, draw-off pipe, valve chamber (III) as well as cattle trough (IV) and communal water point (V) for small earth dams.

12.4.1 Intake Structure

The intake structure will generally consist of a concrete anchor block supporting a vertical perforated galvanised steel pipe. Plastic pipes should not be used since they tend to degrade in the sun during periods when low water levels occur. The pipe upstand can be surrounded by a protective steel structure or a cone of rubble stone and large diameter gravel which serves to protect the upstand from debris, livestock, wildlife and vandalism (See Type Drawing III).

The pipe diameter is typically 100mm in order to decrease the risk of the pipe getting blocked. The perforations on the upstand, starting above the expected silt level, should be at least 12mm diameter and should constitute at least 10% of the surface area of the pipe. The flanged joint for the upstand should be above the concrete anchor block. This means that the upstand can be replaced if needed without damaging the anchor block. However, this introduces a risk of vandalism or theft of the upstand when the reservoir is dry.

In the event that a rough filtration system is desired to improve water quality for public use, it is possible to lay a 30 metre long perforated pipe at an inclined slope (1%) within a graded filter as shown in Figure 12-9. In general, treatment facilities, if needed, should be provided on the downstream side of the embankment where routine maintenance of the treatment works can be undertaken.

12.4.2 Draw-Off Pipe

The draw-off pipe(s) should have a minimum diameter of 100mm, in order to decrease the risk of the pipe getting blocked by debris or silt. The pipe can be galvanised iron, uPVC (Grade E), or HDPE.

As the draw-off pipe forms a preferential seepage path, it should be situated on firm ground preferably below the foundation level of the embankment. Anti-seep collars should be provided at regular intervals (e.g. one per every six metre pipe length) so that the length of the potential flow path is increased to at least 115% the length of the pipe.

There is a risk that the pipe can be damaged by the construction activities as the embankment is being built. Consequently, the trench for the draw-off pipe should be at least one metre below the construction working surface. In order to minimise the risk of damage to the pipe, the pipe itself should be surrounded with concrete.

Where the pipe is not being placed in a concrete surround, a compacted bentonite/soil mix (50 Kg bentonite to one cubic metre of soil) is recommended along the entire trench to minimise the chance of seepage.

It is not recommended to put a control valve on the upstream side of the draw-off pipe. In general accessing the valve for regular maintenance is not possible.

Type Drawing III shows a typical arrangement for the draw-off pipe and intake.

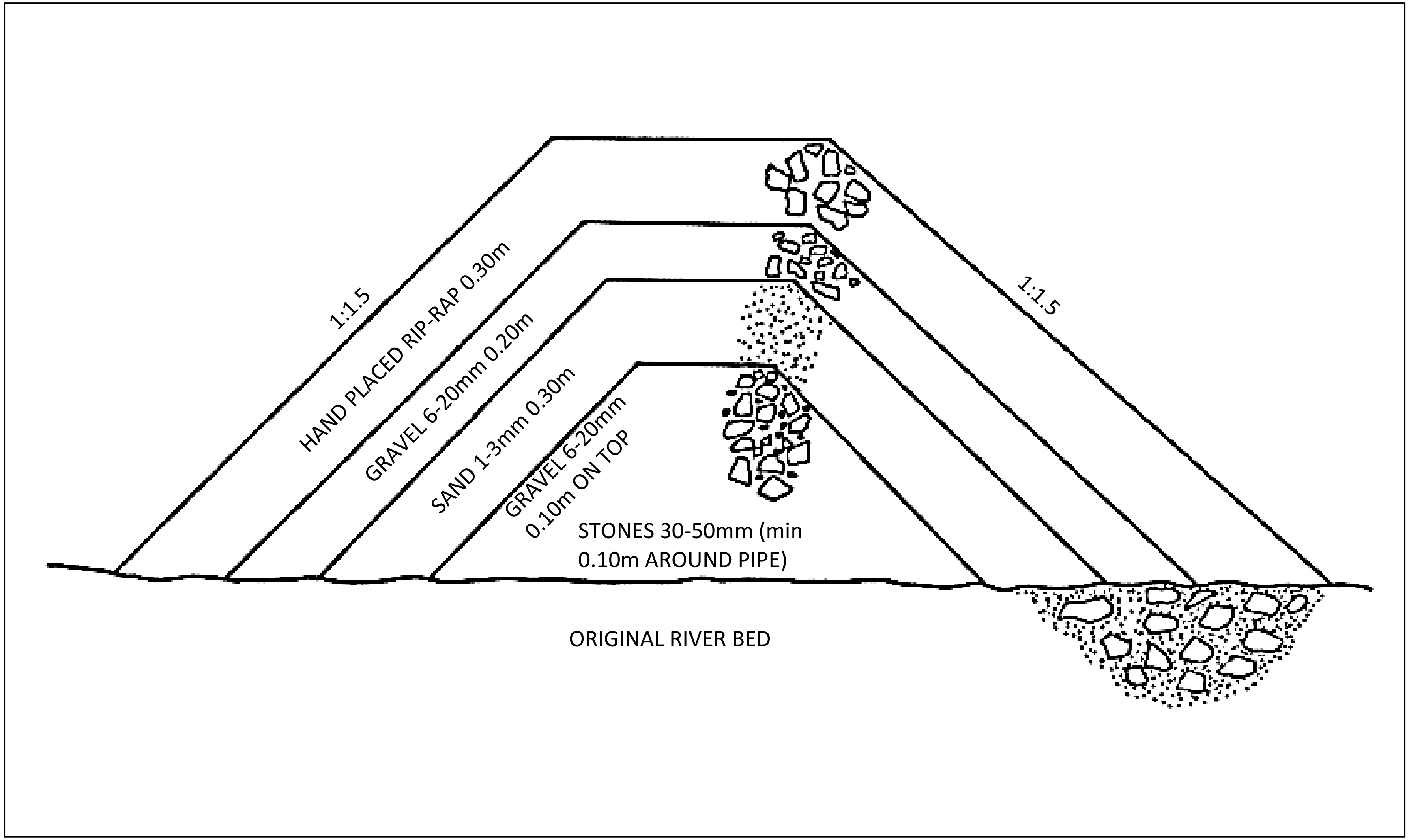

Figure 12-9: Graded Filter for Public Water Point Intake

12.4.3 Outlet Works

The outlet works usually consist of a pump house or valve chamber at or below the downstream toe of the embankment where fittings are placed on the draw-off pipe for purposes of controlling and directing the flow. The arrangement of pipe, tees and valves should allow water to be directed to the consumer points and allows flushing of the draw-off pipe to remove any sediments. The pipe and fittings should be securely anchored to ensure that the action of opening and closing the valves does not result in any movement of the pipe.

12.5 Design of Scour or Compensation Flow Arrangements

Scour is the flushing of sediments from the reservoir area. In general for the type of dams under consideration, a scour outlet to allow flushing of sediments from the reservoir is not foreseen. Sediments tend to settle and then consolidate and so the effectiveness of a pipe to draw out the sediments is questionable. Other forms of de-silting a reservoir should be pursued.

A compensation flow pipe can be included in the design. This is essentially a second draw-off pipe albeit for a different purpose and should therefore follow all the design requirements of the draw-off pipe. However, the inlet structure can be a bell-mouth pipe inlet surrounded by a steel cage (to prevent ingress of large debris). The compensation flow pipe should also emerge into a pump house or valve chamber in which control fittings are placed.

12.6 Long Term Monitoring of Embankment

The long term behaviour of an embankment should be monitored for early detection of any problems. Options are outlined in Table 12-7. These will require ancillary structures to be placed at the end of the construction period.

Table 12-7: Options for Long Term Monitoring of Embankment

Aspect to be monitored	Options for Monitoring
Settlement	Bench marks placed at end of construction along crest. These are surveyed periodically with reference to the site datum to detect changes in elevation of the crest.
Alignment	Bench marks along crest aligned in a perfect straight line. Periodic checking of alignment of bench marks will help detect any shift in embankment alignment.
Seepage	Seepage through the embankment should be captured in the filter drain and conveyed via the drain pipe away from the embankment. The discharge and turbidity of the seepage water should be monitored. This can be achieved by placing a v-notch weir or flow meter on the seepage water.

The placement of piezometers to detect the phreatic level within the embankment is not generally expected for small earthfill dams. However, close attention should be given to the point of emergence of seepage water, if any, on the downstream face, and remedial measures taken to contain the phreatic line within the embankment and directed to the drainage blanket.

12.7 BoQs, Specifications and Reporting

A design report format is provided in Chapter 19. Sample BoQs and specifications are also available on the complementary website.

Construction techniques can vary from contractor to contractor and it is important to have close collaboration between the dam owner, the contractor and the construction supervisor to make sure that the final product meets or exceeds the design specifications and requirements.

12.8 Construction

12.8.1 Construction Team

Assuming mechanized construction, a typical construction team will consist of a foreman, several drivers and a selection of manual labourers. A site engineer or construction supervisor will also be present.

The foreman's role is to oversee the workers, to plan the construction activities and to ensure that materials (including fuel, water, etc) are available as needed for the construction activities. The foreman and the site engineer/construction supervisor will always need to work closely together.

Drivers for the various machines will be needed. Drivers should have suitable experience with their machinery and with similar projects.

Manual labour will always be required in construction of earth embankments. Labour for removal of stones, roots and organic materials are needed throughout construction. Manual labour is essential for piping tasks and for concrete work on pipe surrounds and spillway sills.

The site engineer will be responsible for checking or setting out the initial layout out as needed and for carrying out all supervision activities.

12.8.2 Personal Protective Equipment

Occupational Safety and Health Act, No. 15 of 2007 and revised in 2010, provides for the safety, health and welfare of workers and all persons lawfully present at workplaces, which includes construction sites. The Act also requires that in workplaces where employees are exposed to wet or to any injurious or offensive substances, the employers must provide and maintain clothing and appliances that are adequate, effective and suitably protective. Such equipment includes:

- Helmets/ hard hats;
- Gloves;
- Reflector jackets;
- Goggles;
- Sound mufflers;
- Hard-nosed boots.

12.8.3 Schedule of Works

As dam construction must normally be scheduled with regard to expected rains, it is important to develop an accurate and attainable schedule of works before construction begins. Figure 12-10 shows an elaborate example of a bar chart construction schedule for the construction of a new dam. A construction plan should be prepared at the start of construction and used to evaluate progress. If construction work does not follow the planned schedule it should be noted why this is so.

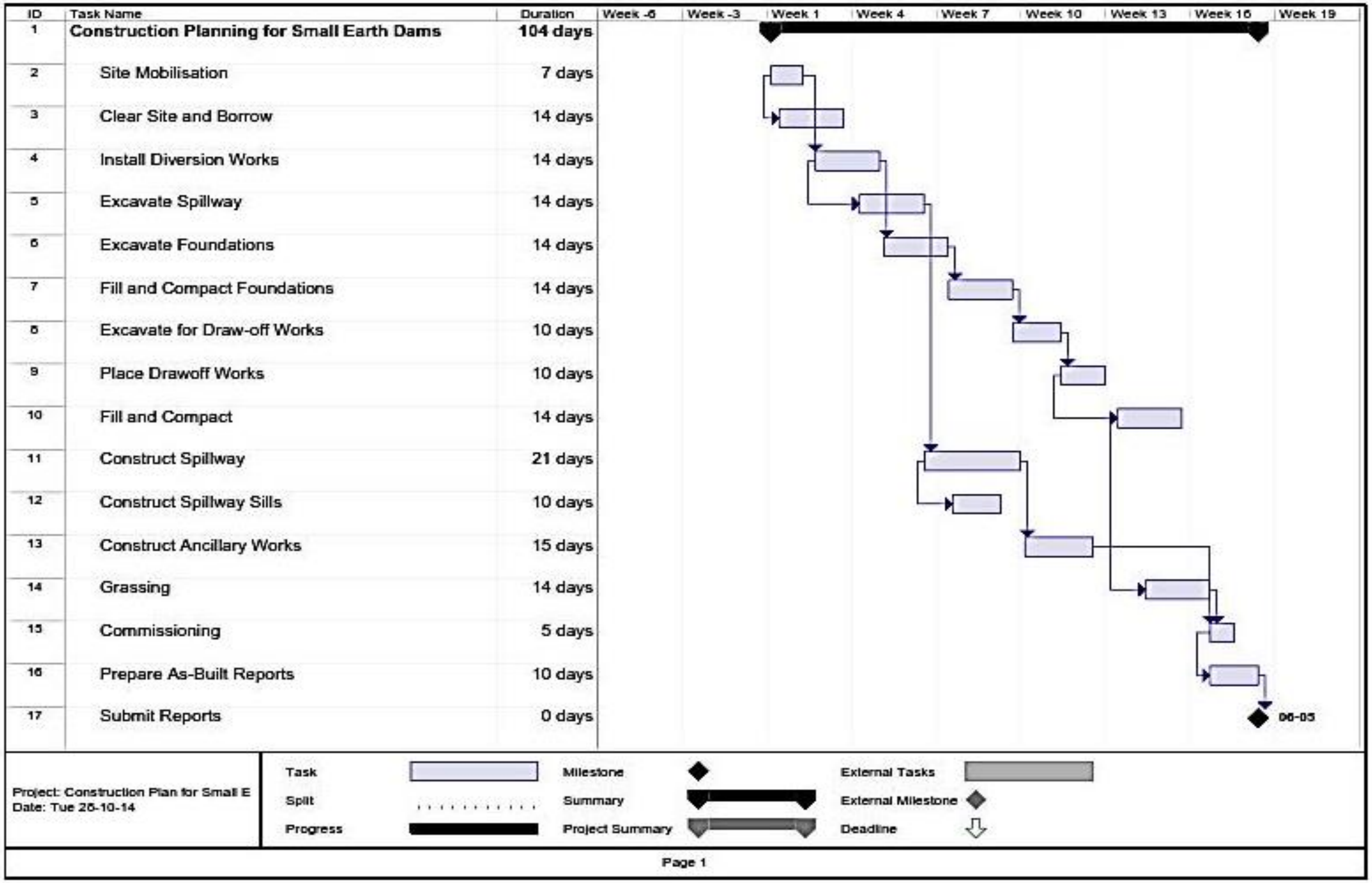

Figure 12-10: Construction Schedule for Small Earth Dam

12.8.4 Embankment Work

The main items which should receive attention during the construction of the embankment are the foundation and the compaction of the fill.

a Setting Out: With the help of the appropriate bench marks, the centre-line of the embankment, embankment area, spillway area and the F.W.L. (delimitating the maximum impounded area) will be demarcated. During the construction of the embankment, width and height of the embankment will be set out at least every metre, preferably using wooden pegs with fill levels indicated by painting.

Calculations for setting out are shown below. Equation 12-9 shows the typical information needed for setting out embankment toes.

Once the centreline has been established, the most important layout work is to locate the toes of the embankment. The toe position is determined by the elevation of the working surface and can be calculated as a distance from the centreline.

Equation 12-9 $$\boldsymbol{D = 0.5\,CW + S \times (CH - GE)}$$

Where: D is the horizontal distance from the centreline to the toe in m;
CW is the crest width in m;
S is the embankment slope in m/m (i.e. 3 for a 3H to 1V slope);
CH is the final crest elevation in m;
GE is the ground elevation in m.

To layout, the ground elevation is measured at the approximate toe position and the distance D is calculated. This is then measured off and the ground elevation is taken again. D is recalculated and re-measured and the ground elevation is taken again. This is repeated until the measured distance D agrees with the calculated distance D for the elevation at the toe peg.

b. Foundation Preparation: The whole area of the embankment should be cleared of loose rocks, trees, tree roots and other vegetation and should be" stripped" of top-soil up to a depth not less than 0.30 m below the natural ground level. The resulting spoil can be stock-piled for re-use as topsoil for covering the down-stream slope of the embankment before grassing.

The reservoir area should be cleared of all vegetation below full supply level. Grass may be left in place, but bushes, trees and tree roots must be completely removed. The borrow areas should be stripped in the same way as the embankment area.

c. Core Trench and Embankment Construction: During excavation of the core trench the information obtained from the test pits should be checked. If required, at this stage, the depth of the core trench can be modified in accordance with the findings during the excavation.

For the actual construction of the embankment it is preferable to use earth moving equipment and heavy mechanical compactors, since it is extremely difficult to achieve acceptable standards of compaction by labour intensive methods. Furthermore, it is worth noting that manual labour can achieve about 2 rn^3 of fill per day per person, which means that 10,000 man-days are required for the construction of an embankment of 20,000 m^3. A typical mechanized operation can normally place and compact between 300 to 800 cubic meters of material per day which would then take between 25 to 67 days.

The actual embankment construction should be carried out by spreading soil in 0.15 m to 0.20 m layers, watering in order to approach the optimum water contents and compacting to achieve the specified density. Scarifying should take place at least once every morning, in order to assure proper adherence between new layers and layers from the previous day.

Note that failure to work at Optimum Moisture Content (OMC) will not deliver the Maximum Dry Density (MDD).

Depending on the compaction equipment being used, slightly thicker layers (up to 0.3m) can be placed but care must be taken to ensure that any water applied to the spread material penetrates throughout the entire layer and that the required density is achieved throughout the layer.

Some contractors also prefer to water the material during borrow excavation so that once it is spread it is already at a moisture content that will allow proper compaction. This method ensures a more uniform moisture content throughout the layer.

d. Compaction Control: Some form of compaction control during the construction should always take place. A minimum compaction of 95 % of the maximum dry density of the BS Proctor test (2.5 kg rammer) is required. Proper field density tests can unfortunately only be carried out if experienced laboratory personnel and equipment are available. If this is not the case, a rudimentary method of compaction control is to try to re-excavate a fill by hand: in case excavation of the fill is only possible by means of a hoe (jembe) an acceptable degree of compaction has been achieved. In cases where fairly easy excavation by shovel is possible, the compaction is insufficient.

e. Construction Period: It is always preferable to complete construction of a dam during one dry season. A partially filled embankment without adequate temporary flood diversion works has the risk being washed away by the floods. In case it is impossible to avoid the rainy season during the construction period, it is advisable to design and construct appropriate flood diversion works.

f. River Diversion: The simplest way to provide a river diversion is by conveying the water through one or two steel pipes which will after completion of the works be incorporated in the draw-off system. The diameter and the number of pipes to be used depends largely on the size of the catchment area and the expected run-off during the construction period. Generally, one or two pipes of 600 mm diameter should be sufficient. The pipe(s) should be placed below the foundation level of the embankment, and should be surrounded by concrete. Cut-off collars should also be provided at regular intervals. It is recommended to provide nominal reinforcement (e.g. Yl0@200 mm both ways) for the pipe surround and the cut-off collars. A small coffer dam upstream of the main works area should also be constructed.

After completion of the embankment works, one of the diversion pipes can be incorporated in the draw-off system, while the other can simply be closed with a flanged plate or alternatively incorporated into the scour outlet/compensation flow pipe.

g. Finishing Works: Whenever possible, selected, mechanically compacted “murram" should be used to cap the embankment crest. The downstream slope should be covered with a layer of top-soil (selected from the “stripping" spoil) and grass planted. “Kikuyu-grass" is recommended. To ensure a good finish, slopes should be trimmed off by hand. Construction of rip-rap layers should take place under the supervision of an experienced operator.

Contributions in labour towards finishing works on the embankment should normally be provided by the community which will benefit from the water. It is however important, especially where rip-rap placing is concerned, that these activities be closely monitored by an experienced supervisor.

12.8.5 Other Works

a. Spillway Construction: New spillways should be cut in undisturbed ground. Sills should be made from Class 25 mass concrete, vibrated where possible. Rip-rap protection (lining) in the spillway should be compacted (by passing over the layer with a dozer) after placing.

Good quality soil from the spillway excavation can be used for the embankment fill.

b. Ancillary Structures: The most important issue here is the compaction around the draw-off pipe. As indicated in Type Drawing III, the draw-off, its concrete surround and the cut-off collars (Class 25 mass concrete), should be placed in a trench re-excavated in the fill. This way, no form work is required. After placing of the concrete however, careful re-fill and hand compaction of the trench is required.

Fencing of the dam and reservoir area and construction of cattle troughs and water points, including the provision of the required building materials should be the responsibility of the beneficiaries of the scheme, as part of their contribution to the project. These relatively simple and inexpensive tasks are nevertheless essential for the long term success of the scheme, and it should be ascertained that they are undertaken.

12.9 Equipment

A wide variety of construction equipment is available in Kenya and it is very difficult to specify specific equipment for general tasks. For example, a wheeled shovel used to be considered essential for loading tippers. Nowadays, the task can also be carried out by a tracked excavator. It is perhaps best to look at the specific tasks involved and suggest appropriate equipment. Table 12-8 provides examples of equipment used and their functions. Detailed descriptions are given in the preceding sections. Photographs of the same can also be accessed on the website.

Table 12-8: Summary of Typical Construction Equipment

Equipment/Machinery	Function
Bulldozer	Site clearing, excavation, trimming
Excavator	Borrow excavation, loading tippers
Dam scoops	Borrow excavation and placement
Tippers	Earth movement from borrow to site
Grader	Levelling, trimming placed construction material
Sheepfoot Roller	Compaction of levelled material
Bowser	Applying water to material
Harrow	Turning material to ensure proper mixing with water
Mixer	Mixing concrete
Vibrator	Consolidate fresh concrete by releasing trapped air
Tractor	Can be adapted to serve different purposes, e.g. water supply, compaction

12.9.1 Site Clearing

Site clearing is best carried out with a tracked bulldozer. Both vegetation and topsoil should be removed from within the footprint of the dam embankment. This can be done with a grader as well.

12.9.2 Core Trench Excavation

Core trench excavation can be done fairly efficiently with a bulldozer. For long core trenches, an excavator may be required to remove material from within the trench.

12.9.3 Borrow Excavation

Borrow excavation can be done with a wide variety of equipment. Excavators can excavate and load tippers very efficiently. Alternatively a bulldozer can rip and stockpile borrow and then a wheeled or tracked shovel can be used to load tippers. Dam scoops can also be used effectively.

Water can be added to borrow material at the borrow area or after placement on the working face. Typically a bowser and some hosepipes are required.

12.9.4 Placement

In general, material is placed on the working face using tippers and then spread with a grader, bulldozer or tracked shovel.

In some cases, material can be pushed from the borrow area on to the working surface with a bulldozer. This is most common in pans where the pushing distance is less than 60m.

Smaller construction site dumpers or tractors with tipping trailers can also be used.

Once spread, water can be added as needed to get the proper moisture content. It may be necessary to mix in the water with a harrow or similar machine in order to ensure uniform water distribution.

Compaction testing results also give water content information and after a few days of work and experience there should be good consistency in water application and water content.

12.9.5 Compaction

For compacting the type of soils used for embankment fills, the use of a sheepfoot roller is recommended. With this type of roller (either towed or self-propelled) the action of the feet causes significant mixing of the soil, thus improving its homogeneity, and will break up lumps of stiff material. Due to the penetration of the feet, excellent bonding is obtained between successive soil layers, which is an important requirement for water-retaining earthwork. Sheepfoot rollers are most suitable for compacting soils at water contents slightly lower than the optimum soil moisture content.

Many self-propelled, vibrating sheepfoot rollers are available in Kenya. Towed rollers can be used but may require either more passes or thinner layers to get proper compaction. When possible vibrating rollers are preferred.

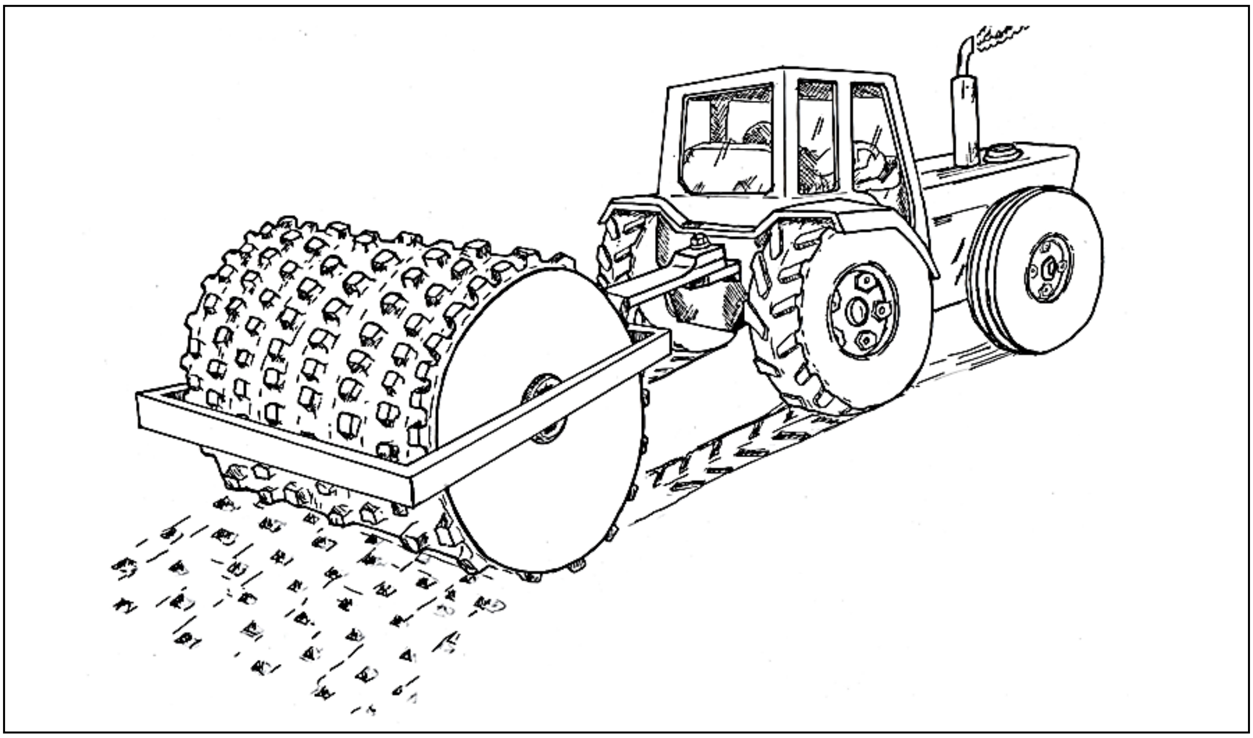

Figure 12-11: A Tractor Drawn Sheepfoot Roller

Flat rollers can be used but they require scarification of the working surface to ensure bonding between layers.

12.9.6 Slope Trimming

Slope trimming can be done with graders or with bulldozers. Excavators can also be used for trimming slopes.

12.9.7 Concrete Works

In general, concrete works will require a mixer and poker vibrator. Hand mixed concrete and non-vibrated concrete should be discouraged.

12.10 Construction Supervision

The level of construction supervision required is dependent on a variety of factors including the size of the reservoir, the risk to downstream users, the competence of the contractor, and the payment structure adopted.

In general construction supervision will ensure a better finished reservoir.

The main construction supervision tasks can be broken down as:

- Site layouts;
- Confirming quality of materials and workmanship;
- Compaction testing;
- Pressure testing pipes;
- Compaction testing;
- Adapting design based on unforeseen site conditions;
- Recording design changes;
- Calculating quantities;
- Preparing payment certificates;
- Reporting to WRMA and the client as needed.

12.10.1 Site Layouts

Site layout is perhaps the most important part of a water storage project. It must be done with sufficient accuracy to ensure that the reservoir positioning is appropriate to the actual site. In the case of earth dams, the site layouts activity will continue throughout the embankment construction and will ensure that embankment slopes are built as per the design.

12.10.2 Compaction Testing

Compaction testing should be carried out throughout the construction of earth dams. Several samples should be taken for each layer. Conventional sampling and drying in an oven takes 24 hours, so there needs to be planning with regards to working areas and sample taking in order to ensure that layers and areas with insufficient compaction are not covered over before test results are ready.

Compaction testing requires a set of moulds, a drying oven, a scale (capable of reading to 0.1gm) and several graduated cylinders.

The actual calculations are easily done in a spreadsheet and results can be filed both hardcopy and electronically in case they are needed at a later date.

12.10.3 Pressure Testing Pipes

Pressure testing of pipes MUST be carried out before pipes are encased in concrete or buried. Pressure testing details are given in the sample specifications.

12.10.4 Recording Design Changes

Most projects will have some design changes in order to accommodate site conditions. These should be handled by the project engineer to ensure that they do not compromise the original design of the dam.

12.10.5 Adapting Design Based on Unforeseen Circumstances

In some cases, significant design changes will be needed due to unforeseen circumstances and it will be necessary to actually redesign portions of the project. This re-design work should be carried out by the project engineer.

12.10.6 Calculating Quantities

A large part of the construction supervision process involves calculating and recording quantities of materials and work. This is necessary to calculate payments due to the contractor, to minimize the chance of disputes and to ensure that the design figures are accurate.

12.10.7 Preparing Payment Certificates

Most water storage construction projects span several months and payments are usually based on actual work completed. Payment certificates must be prepared to detail amounts due to the contractor and to provide accurate records in case of disputes.

12.10.8 Environmental Protection

The main environmental issues during construction are generally dust, noise and pollution from oil or fuel spills. These can be dealt with on a case by case basis. In addition, worker safety due to site traffic should be given proper consideration and traffic flow should be controlled to minimise the likelihood of accidents.

Where there is water flow through the site and downstream water users, care must be taken to ensure that the water quality of the flow is not affected by the construction activities. This typically involves diverting flow around the site and may require pipes or culverts at road/equipment crossings.

12.10.9 Defects Liability Period

For contractual projects, the defects liability period is a duration specified in the contract, usually six months after practical completion, when the contractor is required to make good defects that may arise. A certain percentage of the contractor’s final pay is also retained during this period, such that in the event that the contractor does not honour this requirement, then the project owner may use this amount to rectify any defects that have occurred.

12.11 Operation and Maintenance

Operation and maintenance covers the range of tasks, some of which are routine, that enable the dam to provide the expected benefits over the life span of the dam.

Maintenance of small dams and reservoirs is simple and inexpensive but is nevertheless essential since unattended minor issues (especially minor erosion on embankment and spillway) can develop into major problems which can ultimately reduce the useful lifespan of structure.

12.11.1 Community Issues

For community owned dams, the following operation and maintenance issues should be considered. The community which will receive water from the dam has a major role to play in the operation and maintenance of the structure. During the construction (or rehabilitation) of the project the future dam operator and at least one other member of the dam committee should receive basic training in operation and maintenance aspects of the small dams. This training should concentrate on the following issues:

- Attending to minor problems which can be taken care of by the community itself, e.g. rain erosion on embankment slopes, repair of fences etc.;
- Identification of problems which require more specialized attention, e.g. erosion in spillway channel;
- Establishment of communication channels between the representatives of the community (dam committee) and the responsible organisation or administration at county level;

Once the ownership of the dam has been formally transferred to the community, it will then be the responsibility of the dam committee to carry out regular inspections and basic repairs and maintenance works. A handing-over report including specific information and instructions on existing problems for the concerned dam should be elaborated. Information regarding problems which require more specialized attention will be passed on to the relevant authorities without delay.

12.11.2 Embankment

Erosion due to rain or surface run-off on the embankment (downstream slope) must be controlled. Erosion rills on the embankment slopes should be re-filled with compacted material and grassed.

Any population of rats or other rodents should be removed, as they constitute a serious risk for the water-tightness and the stability of the dam.

Fences should be kept in good condition and repairs carried out when required. No livestock should be allowed to wander on the embankment.

Surface cracks should be noted and filled in as soon as possible. Longitudinal cracks along the crest indicate significant soil movement which could occur from settlement or as an early sign of slumping along the downstream or upstream slopes. Filling cracks with compacted material is an effective way to prevent them being filled with water which will further weaken the structure.

Erosion from wave action should be dealt with either through placement of additional riprap or by planting grass or reeds in the affected area.

Trees should be removed from the embankment before they can become established. Small brush and grass is the preferred embankment cover.

Any settlement along the crest should be filled in and the crest level should be kept at its design level.

Slumping on the embankment faces can be addressed by adding material to the base of the embankment to form a berm and reducing the length of the embankment slopes.

Seepage at the embankment toes should be noted and any standing water should be given a drainage pathway. The downstream area of the dam should not be allowed to become saturated.

12.11.3 Spillway

Spillway erosion requires lining with rip-rap or construction of gabions to stabilise the channel banks. If there is significant spillway erosion concrete sills may need to be installed to control spillway levels.

The spillway channel should be kept clear of high vegetation as this impedes the discharge capacity. Clearing of the channel should be carried out before every wet season. A short grass cover in the spillway should be encouraged as this provides an excellent erosion protection.

If possible records should be kept of spillway flows. This will help provide information for any rehabilitation work on the dam.

12.11.4 Reservoir Area

Removal of silt from the reservoir using manual labour can be periodically organised during the dry season. This will prolong the useful life period of the dam.

Water level readings should be noted as should any periods when the reservoir area is empty.

12.11.5 Water Quality

Water quality within the reservoir should be noted. For newly constructed dams there are often algal blooms after the initial filling of the reservoir due to the high level of organic nutrients. Algae growth can generally be controlled by introducing fish to the reservoir and examining carefully potential sources of nutrients to the water.

Turbid water within the reservoir can be a sign of catchment degradation and action should be taken to ensure that good catchment conditions are maintained.

12.11.6 Ancillary Structures

Outlet works should be checked and the main draw-off pipe should be flushed (in order to remove possible sediments) as needed.

Blocked outlet works can be cleared using compressed air or by back-flushing clean water up the outlet pipe. Care should be taken when doing this.

Valves should be operated during inspections to ensure that they are working properly.

Cattle troughs, tap-stands and water kiosks should be inspected and maintained. Drainage around these structures should be maintained to ensure no standing water is present.

Fencing should be repaired as needed and additional live fencing should be planted at the onset of rainy seasons.

12.11.7 Safety Issues

Observation of excessive seepage, wet patches or small slides on downstream slope, turbidity of seepage water etc. should be carried out. If seepage is observed, plans should be made to monitor the flow rate and determine if the seepage is increasing or is fairly constant and whether the discharge is correlated to the water level. Drainage channels should be installed to direct seepage away from the embankment and into the river course.

Turbid seepage indicates that the seepage is eroding its flow pathways and is a serious issue. It must be dealt with. It is best dealt with by excavating where the seepage is emerging and then backfilling with a sand/ballast/hardcore layered filter arrangement. The filter should retain any soil particles that are being carried by the seepage.

If there are concerns about the dam safety, water levels should be lowered as quickly as possible.

Fencing should be maintained to prevent uncontrolled access.

Floats or life preservers should be available and well maintained for dams in areas with human traffic.

Warning signs should be repainted as needed and placed prominently to warn people of the risks posed by the reservoir.

Contact lists of important government, riparian landowners and civil societies should be maintained and updated yearly. If there are concerns about the dam safety, the contact lists should be used to inform everyone of the situation.

If possible, inspections of the dam and reservoir should be carried out monthly, while special attention is required during the rainy seasons.

12.11.8 Inspection Schedule

Regular inspection of the structure is required to ensure problems are identified early and remedial action taken. The WRM Rule 2007 (Fourth Schedule) imposes the frequency and expertise required for inspections of dams according to the class of dam as shown in Table 12-9.

Table 12-9: Frequency and Expertise Required for Dam Inspections

Class of Dam	Frequency of inspection	Inspection by
A (Low Risk)	Once in 5 years	Panel I C1, Panel I C2, Panel II C
B (Medium Risk)	Once in 3 years	Panel I C2, Panel I C1
C (High Risk)	Once every 2 years	Panel I C2

Table 12-10 provides a simple form to support dam inspections.

Table 12-10: Sample Inspection Form

DAM NAME:		**DATE**	
MONITORED BY:		**WATER LEVEL:**	
ITEM	***CONDITION***	***ACTION REQUIRED***	***BY WHOM***
CATCHMENT AREA			
Erosion			
EMBANKMENT (cracks, vegetation, erosion, slumps, leaks, animals)			
Crest			
Downstream slope			
Upstream slope			
Lining Material			
SPILLWAY (debris, vegetation, erosion)			
Sill Area			
Culverts under Road			
Channel			
River confluence			
DRAW-OFF WORKS			
Status of Pump and Meter			
WATER QUALITY			
Colour/turbidity			
Smell			
Chemical Analysis required?			
ENVIRONMENT			
Details of Any Animals			
Details of Any Plants			
Accessibility to dam			
SIGNED BY			

12.12 Rehabilitation of Small Earth Dams

If properly designed, constructed and maintained, the type of small earth dams under consideration, should reach a useful lifetime of 20 to 25 years before the need for significant rehabilitation work occurs.

12.12.1 Selection of Dams for Rehabilitation

The aim of rehabilitation works carried out on small earth dams should not only be to restore the dam and reservoir into their original condition, but also to upgrade the structure where possible so as to bring it in line with the design guidelines set out herein.

It is important to select structures for rehabilitation where the costs involved will yield the maximum benefits, and where a reasonable chance exists that climatological, agro-ecological and community involvement factors are such that after rehabilitation, the dam will have a useful lifetime in line with set expectations.

On the technical/economical side it should always be kept in mind that physical removal of sediment deposits using ordinary earth-moving methods is rarely feasible. It is nearly always more interesting to either raise an existing embankment where the topography allows, or otherwise construct a new dam downstream of the old silted reservoir, whereby the old reservoir will further act as a silt-trap. Removal of sediments by scooping provides a ratio (water storage volume / earth fill volume) of 1, where even a dam-site rated as poor will have a ratio of 3. Physical removal of silt does make sense on sites where the sediments removed can be used to widen and raise the embankment crest and where significant ancillary structures have been built around the existing dam.

Another important factor which needs to be assessed before rehabilitation works are decided upon is the agro-ecological situation, the erosion and the expected sediment yield of the catchment area.

Finally the potential for successfully involving the beneficiaries not only in a number of construction activities but particularly in operation and maintenance of the structure and their willingness to make long term investments in catchment improvement and protection works, is another factor which needs to be taken seriously into consideration.

12.12.2 Cause of Failure

Prior to any site rehabilitation, a careful analysis should be made to establish the cause of the failure or condition that has necessitated the rehabilitation. This may require a number of steps including:

- Interrogating any locals or witnesses to the event that caused damage;
- Obtaining original design or as-built drawings and reports;
- Obtaining photographs of the as-constructed structure;
- Obtaining rainfall and streamflow records for the event that caused damage;
- Conducting topographical surveys to ascertain high water marks with respect to spillway and crest levels. This can help to ascertain whether overtopping of the embankment occurred;
- Conducting topographical surveys to establish whether excessive embankment settlement took place. Observations of longitudinal cracks on the embankment could also signify slumping and slope failure;
- Soil sampling and analysis to establish embankment material which may be different to surrounding soils and may be different across the cross section of the embankment.

The cause of failure analysis is important to avoid investing in the rehabilitation of a structure that has a fundamental weakness that is difficult or expensive to overcome (e.g. no impervious core resulting in excessive seepage, embankment built out of dispersive soils). In addition, if the failure is caused by insufficient attention to operation and maintenance tasks, then a substantive review of the O & M schedule and those responsible should be undertaken. In many instances, the cause of failure is not immediately obvious and may be attributed to a number of different factors taking place in combination.

Once there is sufficient confidence that the cause of the failure or damage has been established, then efforts to design the rehabilitation works can commence.

12.13 Rehabilitation Works

The rehabilitation works must address the cause of failure as well as returning the structure to operational status.

12.13.1 Embankment Repairs

Figure 12-12 shows how an old embankment can be raised. All trees and vegetation are removed from the old (possibly eroded) embankment, after which the embankment is trimmed down (min. 0.30 m). Any loose material should be removed or stockpiled for reuse. The trimmed old embankment is used as an upstream core for the construction of the new embankment. Suitable soil has to be used for the fill, and the usual compaction procedures should be followed. It is recommended that a core-trench should be incorporated in the new embankment. Care should be taken to ensure a good contact between the old and new sections of the embankment, and between the foundation and new material.

At the same time, various other works to improve the embankment can be carried out if required, such as:

- rip-rap protection on up-stream slope;
- incorporation of a filter blanket and toe drain etc.

Raising of an embankment will normally require the filling of the old spillway channel and the construction of a new spillway. Alternatively, the concrete spillway sill can be raised and erosion protection provided downstream of the sill.

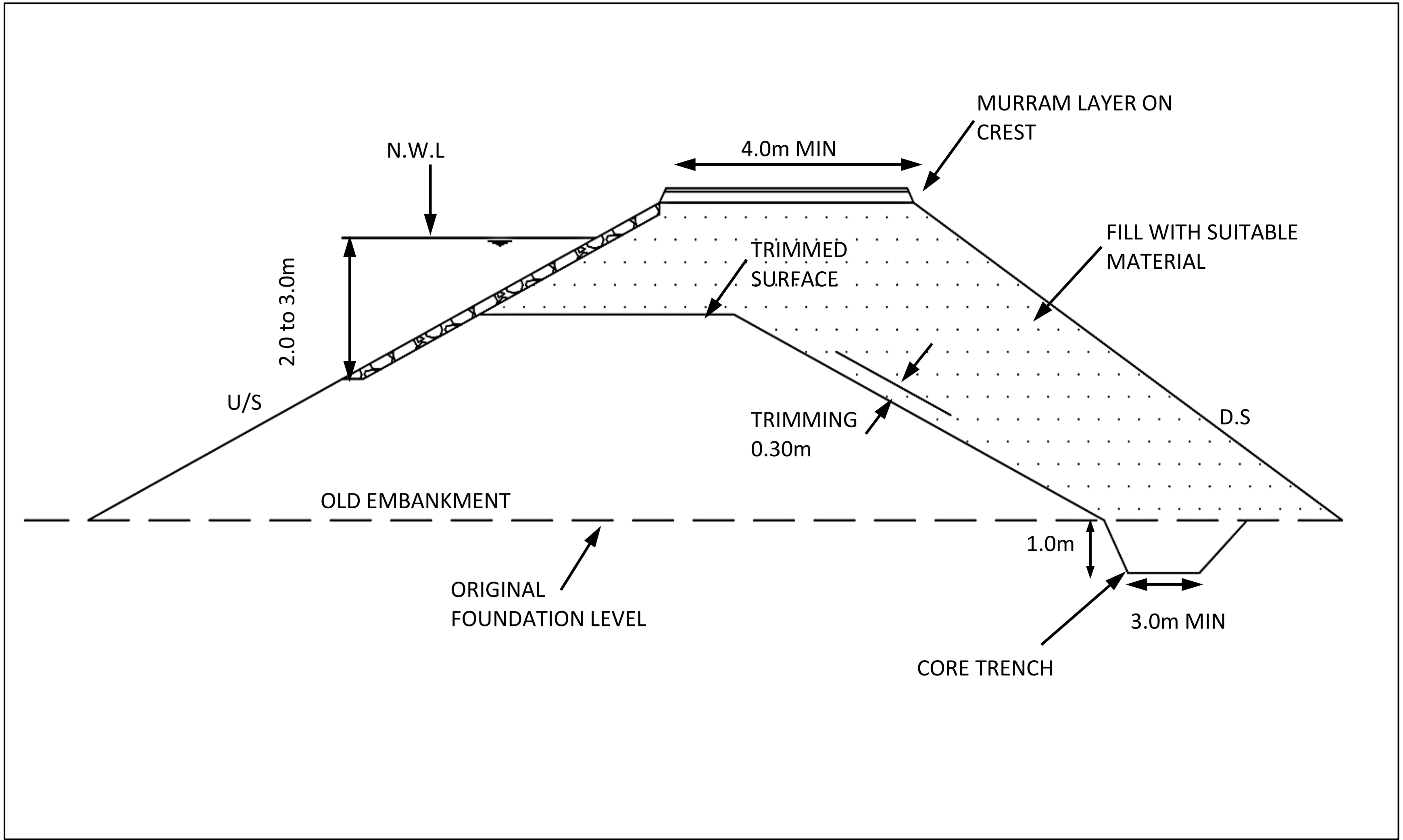

Figure 12-12: Raising of Embankment

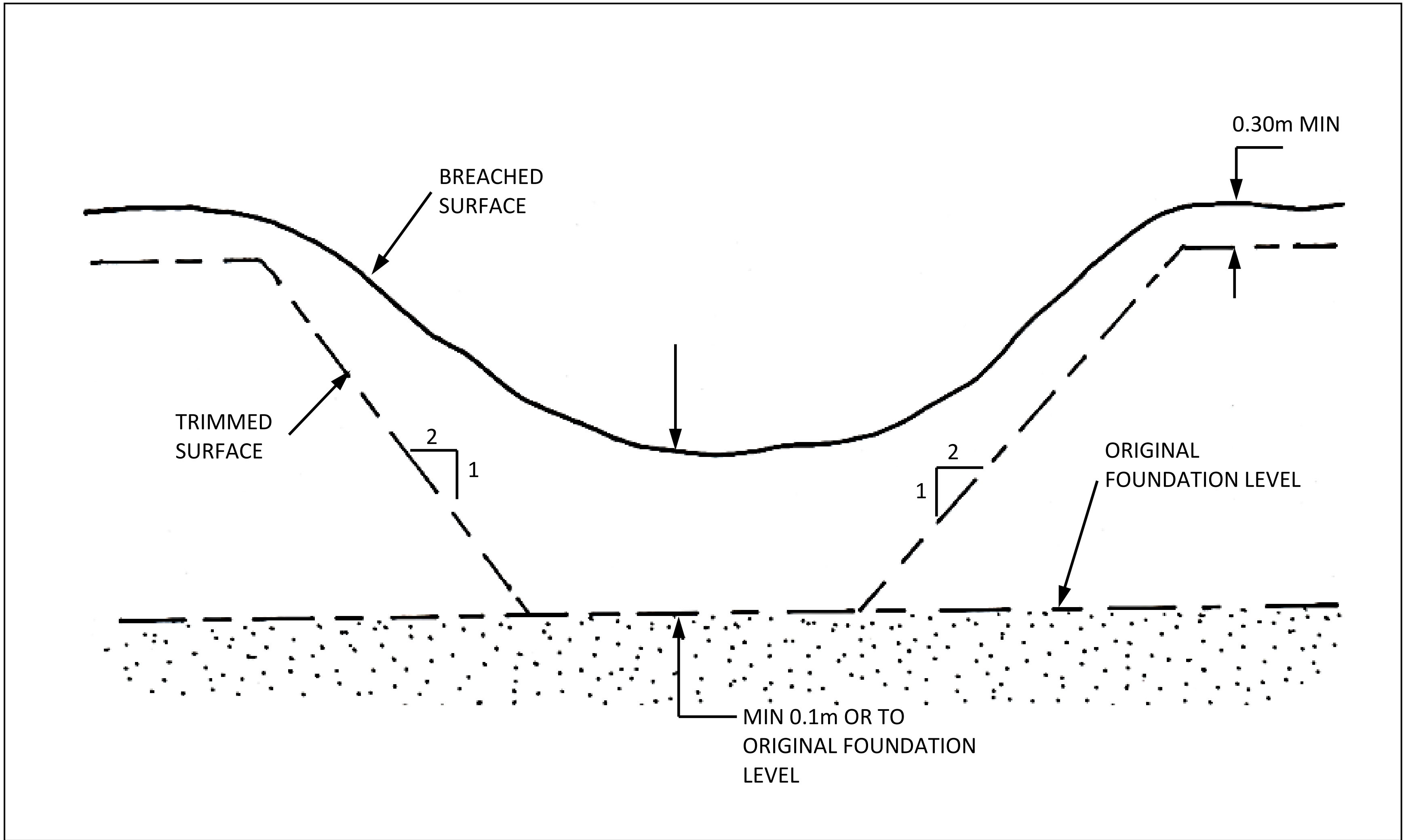

Figure 12-13: Trimming a Breached Embankment

Figure 12-13 and Figure 12-14 explain how a breached embankment is repaired. The breach is trimmed to a 2 to 1slope on the sides, the bottom is trimmed to the level of the original foundation (or at least one metre deep). All gully deposits (sand, gravel etc.) should be carefully removed from the bottom of the breach. Filling of the breach should be carried out using suitable soil and following the usual construction procedures for earthfill embankments.

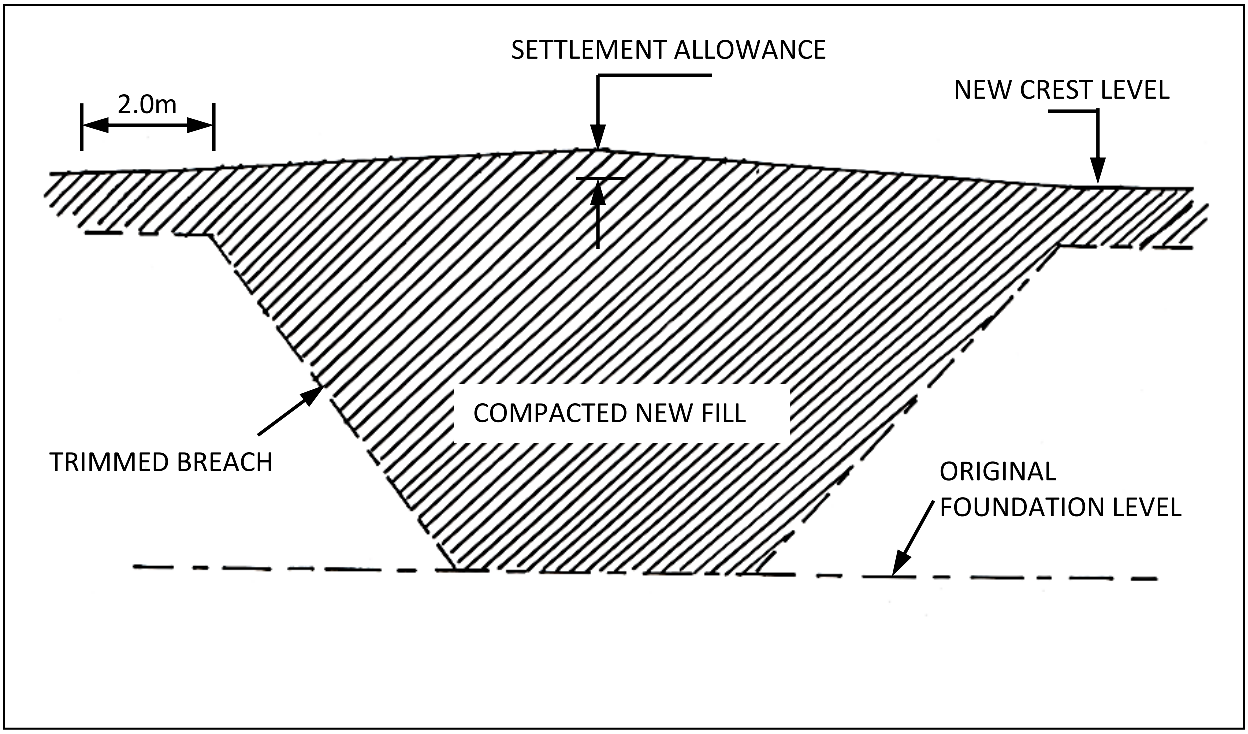

Figure 12-14: Repair of a Breached Embankment

12.13.2 De-silting

Prior to de-silting any reservoir by mechanical means, it must be emptied. This can be done by either pumping or cutting part of the spillway channel to the required depth. Breaching of the embankment is not recommended. The digging of a number of test pits in the reservoir, in order to establish the depth of the silt layer, prior to the scooping is recommended. This will also permit study of the stratification of the silt layer, which might eventually lead to determining its origin within the catchment.

If de-silting is carried out by traditional earth-moving methods, the most effective way would be to make use of a bulldozer (100-125 kW will generally be suitable) for the removing the silt, a wheel loader for loading it and tipping lorries for the transport of the silt. This basic machinery can be assisted by a number of smaller machines to perform more specific tasks.

If the removed soil is judged suitable, it can be re-used for repairing the embankment. In case of very small dams (below 5 metres of height), not involving drainage blankets or toe drains, removed soil, judged unsuitable for re-use as backfill can be used to reduce the downstream slope (see Figure 12-15).

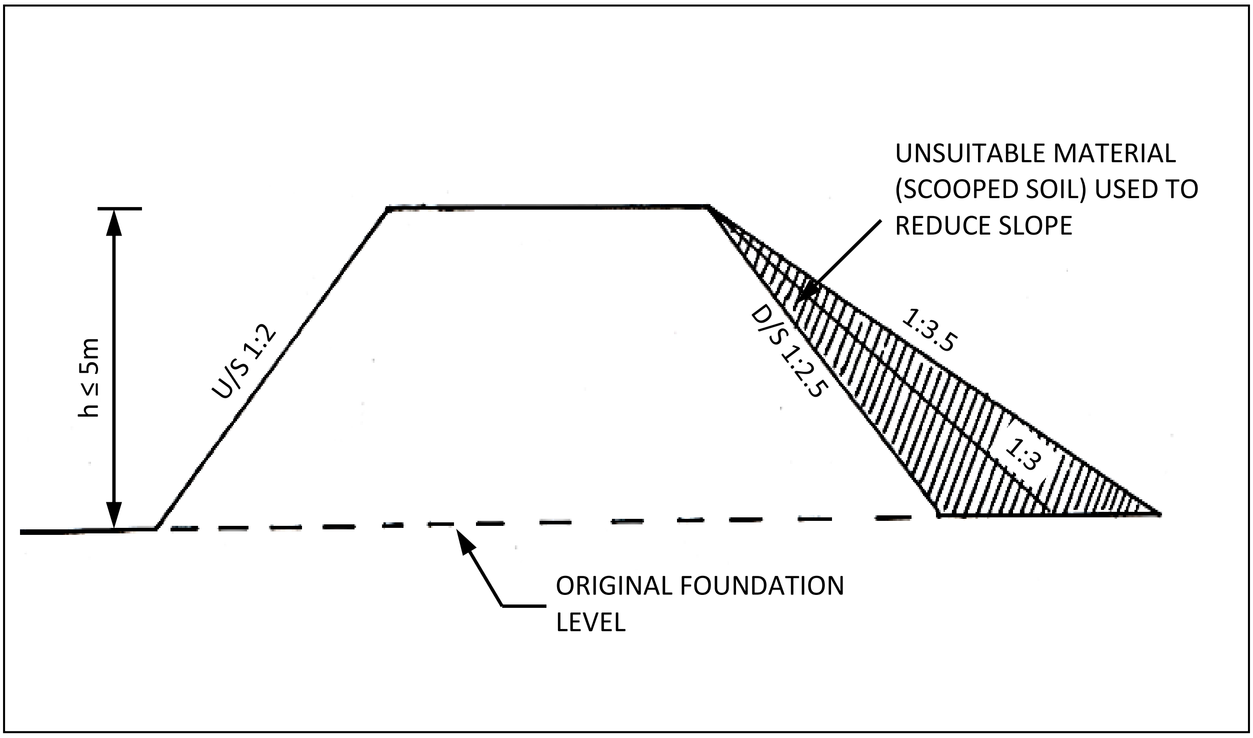

Figure 12-15: Use of Scooped Soil to Reduce Downstream Slope

12.13.3 Spillway Repairs

Repairs of old (eroded) spillways will usually involve the construction (or rehabilitation) of sills, lining eroded stretches with rip-rap (at least 0.30 m thick and compacted by a dozer), and stabilisation of banks by the use of gabions.

Whenever spillway repairs are undertaken it is suggested that the original spillway design calculations be confirmed and revised if needed due to changing catchment conditions or improved data on streamflow or rainfall.

CHAPTER 13

DESIGN OF MASS GRAVITY DAMS

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13 DESIGN OF MASS GRAVITY DAMS

Mass gravity dams are usually concrete or masonry walls constructed across water courses. They rely on the mass of the structure to provide stability against sliding and overturning. Although they can be designed and built for heights of up to 15m (or more), the taller structures become expensive in terms of materials and often alternative structures (concrete arch dams, buttressed walls, earth embankments, etc…) can offer more economical alternatives. In Kenya, mass gravity dams are most often built in the 3 to 5m height range and are most often used as controlling/offtake structures in river courses. When placed in a river channel to assist with water offtake or flow measurements mass gravity dams in Kenya are often referred to as weirs.

In keeping with low and medium risk structures, this manual only deals with mass gravity dams up to 5m tall. Mass gravity dams can be built much taller than this. For projects that exceed the 5m height limit, the following references can be consulted. *Design of Small Dams,* (United States Department of the Interior - Bureau of Reclamation, 1987) *Hydraulic Structures* (Novak, P., et al, 2006) or *The Indian Standard, Criteria for the Design of Solid Gravity Dams* IS6512-1998 (Civil Engineering Division Council, 1998).

Mass gravity dams can also be found in composite structures (part earth embankment, part mass gravity dam). Composite structures are not considered in this manual.

Curved or buttressed dams are not mass gravity dams as they use the curvature and/or buttresses to provide structural support to the dam. Curved or buttressed dams are not considered in this manual.

13.1 Typical Mass Gravity Dam Projects

Mass gravity dams are most suitable in areas where there are firm bedrock foundations and where the valley sides are also of a rocky nature. They are most often used as:

- Structures in river channels to raise water levels for water offtakes or diversions;
- Structures in river channels to assist in flow measurements;
- Small scale storage of water in arid rocky valleys;
- Storage and diversion structures in rock catchments;
- Storage structures for sand dams or other groundwater/subsurface dams.

Their main advantage over earth embankment dams is that the entire crest of the structure can serve as a spillway and that they are therefore less susceptible to erosion damage when overtopping. Other advantages include simpler and less costly offtakes as the offtake passes through a relatively thin wall compared to earth embankments.

Their main disadvantages over earth embankments are their foundation requirements (firm, stable rock foundations) and their costs. Costs for gravity dams can be quite high as transporting materials to site (sand, aggregate, cement, formwork, etc.) can be a significant expense.

13.2 General Considerations

The following factors should be considered when choosing a mass gravity dam.

13.2.1 Foundation

Foundations should be on bedrock in order to avoid seepage under the dam and for stability reasons. In case the rock formation is weathered, this profile should be completely excavated before the dam foundation is constructed. The presence of open fracture zones should also be investigated. If the presence of fracture zones is suspected, the rock surface should be cleaned and simple infiltration tests carried out by pouring water on the cleaned surface. If fractures are discovered, they need to be sealed, and expensive grouting might be required.

Mass gravity walls should not be built on softer foundations, but if there is no alternative (say for a proposed river offtake), then the wall should be keyed into the river bed (if possible a distance of ½ the height of the wall would be appropriate) and upstream and downstream aprons should be constructed. The wall will almost certainly have to be extended into the river banks as well. Typically a 5m extension into each bank should be sufficient to avoid erosion issues on the river banks.

13.2.2 Dimensions

Length of Wall: In general, walls longer than 30 to 35m should be avoided. Longer walls may require buttresses or other supports.

Height of Wall: In general wall height should not exceed 5m. Wall height will be limited to the depth of the river channel and must leave an allowance for flood flows such that the expected flood flows will still remain within the channel.

Wall Geometry: In general, crest widths range from 0.3m to 0.5m depending on construction materials and height of wall. Base widths range from 0.6 to 1.0 times the height (h) depending on construction materials and design of the wall. The front face of the wall can be vertical or with a slight slope toward the crest. The rear face generally has a slope of 1 horizontal to 2 vertical or similar.

13.2.3 Overflow Sections

The design flood and the broad crested weir formula can be used to calculate flow depths and approach heights for mass gravity dams. Depending on bank conditions, it may be necessary to leave an overflow section in the wall to keep flow away from erodible areas. This can be done by building wall sections slightly lower (0.30- 0.40 m) as shown in Figure 13-1.

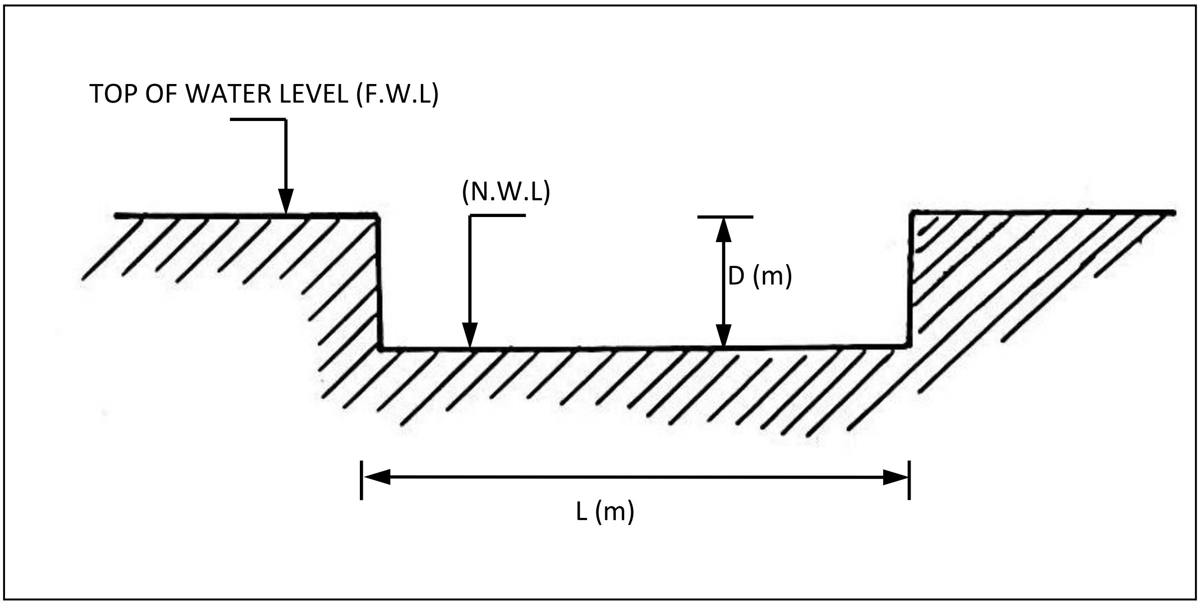

Figure 13-1: Overflow Sections for Gravity Walls

For walls situated on firm foundations and with rocky banks, the entire crest can be allowed to act as an overflow. If this is the case, the rear crest edge should be rounded and the downstream face should have a gentle slope. The transition area between the downstream face and the riverbed should be gently rounded and the river bed must be firm rock that will not erode.

13.2.4 Upstream and Downstream Aprons

Depending on foundation conditions, it may be necessary to construct both upstream and downstream aprons.

Upstream aprons are usually required where the foundation rock is weathered or where the mass gravity wall is being built on softer materials. A 300mm thick reinforced concrete apron extending 1 to 2 meters upstream and covering the entire width of the riverbed will normally be sufficient. The apron should be cast on the foundation material without any hardcore underneath it. Placing material under the apron will lead to seepage problems.

Downstream aprons are required where there are concerns about erosion at the downstream toe of the mass gravity wall. They must be positioned and sized appropriately to avoid erosion at the downstream toe of the mass gravity wall. As with the upstream aprons a 300mm thick reinforced concrete apron extending across the entire river bed is recommended. The length of the downstream apron (where length is the distance the apron extends downstream from the toe) depends on the height of the wall. As a quick rule of thumb, the apron should extend a distance equal to the height of the wall.

Downstream apron positioning and design is highly dependent on foundation conditions. In easily erodible channels, it might be necessary to construct a stilling basin rather than a simple apron. It may also be necessary to consider other forms of energy dissipation in order to reduce water velocities. This can be done with rocks or boulders set in the apron.

13.2.5 Wing Walls

Wing walls are walls constructed both upstream and downstream of the mass gravity wall. They are positioned parallel to the river/valley banks and serve to protect the banks from erosion. They are effectively retaining walls and should be designed as such. They must extend above the sill level of the mass gravity wall in order to fully protect the banks during high floods. They normally do not extend more than 5m upstream and 3m downstream from the mass gravity wall.

Wing walls can often be used for positioning offtakes. As they are out of the main river channel they are better protected from flood water damage and can be less susceptible to siltation.

13.2.6 Draw-Off System

Draw-off systems for gravity dams can be similar to those for small earth dams as shown. A simple through-the-wall draw-off arrangement is shown in Figure 13-4.

A through-the- wall system will involve a perforated or slotted GI pipe (usually laid horizontally) surrounded by a hardcore/ballast/sand filter, a draw-off pipe through the gravity wall (diameter preferably 75 mm or larger), and a small valve chamber. To minimise the risk of seepage along the pipe two or more (welded) steel collars can be provided where the pipe traverses the wall. Additionally, several scrap iron lugs should be welded to the pipe where it passes through the wall to ensure that the pipe will not spin in the wall when fittings are screwed on or off. To save money, a reducer and smaller gate valve can be used with appropriate fittings.

Alternatively, in situations where the dam is used to control flows, an overflow section that spills into a collection chamber can be placed in the wall. All pipe connections can then be made from the collection chamber. Appropriate screens and washouts should be included in the collection chamber design. This overflow and collection arrangement is called a side chamber offtake in Kenya and is very useful for ensuring an agreed upon distribution of flows. Where the dam is used to control flows for common intakes and self-regulating intakes, multiple overflow sections and collection chambers can be installed.

The final position of collection chambers is highly dependent on site topography and final pipeline routes. They are generally positioned to reduce the risks of erosion, to minimize construction costs and to reduce pipeline river crossings.

13.2.7 Scouring System/Compensation Flow

For gravity walls used as weirs in river courses, it is often desirable to install a scouring system or to ensure compensation flow.

Compensation flow can be addressed with a pipe through the wall as described above. Pipe size can be roughly calculated based on pipe cross section area and an assumed velocity of 1m/s. Compensation flow pipes can often be blocked and can become contentious issues during low flow periods. If there is concern about compensation flow during low flow periods, then it is safest to us an overflow offtake with a downstream collection chamber and install an overflow section for the compensation flow that ensures the priority of the compensation flow.

Scouring pipes through the wall must be extremely large diameters to work effectively. They are most often used as part of the diversion works during construction. Pipes with flanged fittings on the downstream side are most appropriate. The flange can be sealed with a blank plate during normal use. The plate can be loosened and then pivoted about one securing bolts during scouring operations. This removes the need for a large diameter gate valve on the scour pipe.

Even with a large diameter scour pipe, it will be necessary to manually assist in silt removal. Deep deposits must often be pushed into the area where they can be flushed through the scour pipe.

13.3 Specific Calculations

In addition to the previously discussed calculations on hydrology, storage and evaporation, water use, etc., the following standard calculations are useful in the proper design of gravity dams:

- Storage capacity;
- Check for overturning;
- Check for sliding;
- Broad crested weir flows;
- Inlet screen flows.

Larger mass gravity wall designs (over 5m tall) should also consider:

- Foundation stresses;
- Uplift pressures;
- Earthquake forces;
- Wind and wave pressures;
- Tension forces under various loads.

13.3.1 Storage Capacity of the Reservoir

The (total) storage volume of the reservoir can be estimated from the geometry of the riverbed (see Figure 13-2) using Equation 13-1.

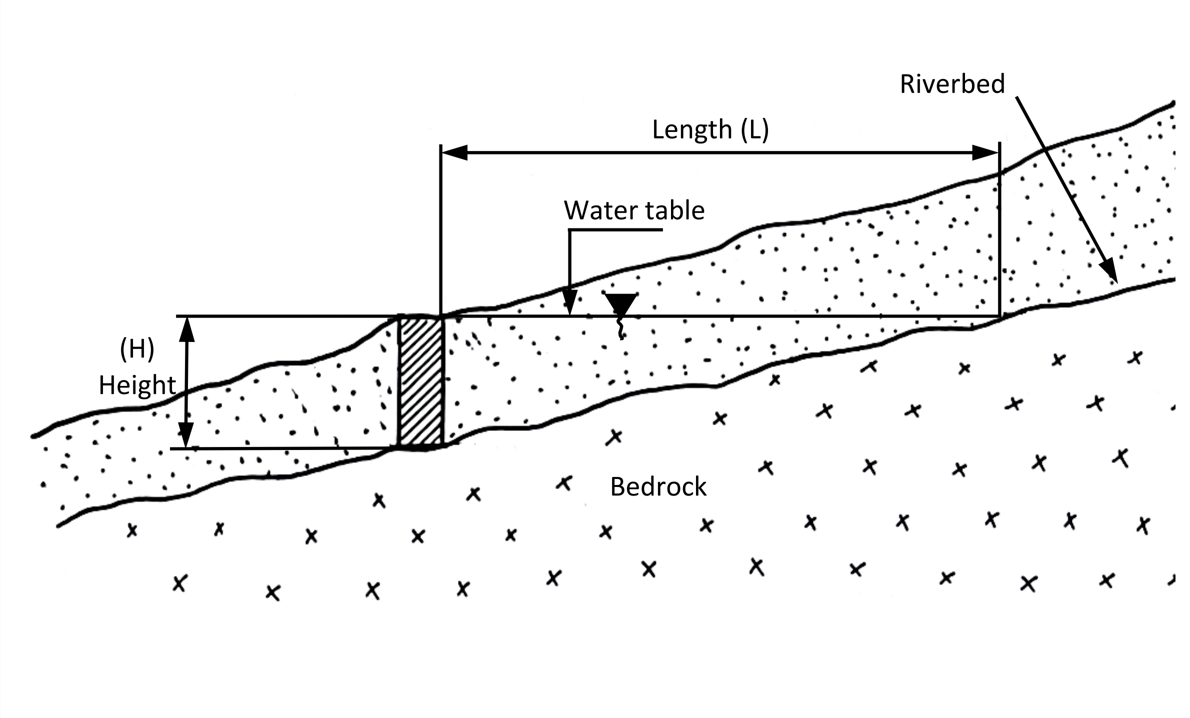

Figure 13-2: Geometry of River Bed for Storage Volume Calculations

Equation 13-1 $\mathbf{V = \frac{1}{2}\,(h \times L \times W)}$

Where: V= Storage Volume [m^3];
H = height of wall [m];
L= horizontal distance water will occupy in meters and can be calculated as
L = [h x 100] / [slope of riverbed (%)].

13.3.2 Design of the Gravity Wall

Gravity walls can be built from anything that provides sufficient mass. In Kenya they are generally built from concrete (1:2:4 mix, not reinforced) or from stone masonry using blocks obtained from natural rock (hard rock is preferable) or from rubble stone masonry with waterproof mortar.

For design purposes, the density of concrete can be taken as 2,400 kg/m^3 and the density of masonry can be taken as 2,300 kg/m^3 (IS 6512-1984). Actual measured densities for proposed materials can be used for final designs.

Mass concrete is defined by the American Concrete Institute as:

> *Any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.*

For mass gravity walls under 5m tall, it is unlikely that hydration heats will cause temperature problems in the concrete. However, if the minimum dimension of a concrete pour for a gravity dam is greater than 1m, then additional advice on mass concrete works and temperature control should be taken. BS8110 is an excellent reference for mass concrete.

The foundation of the wall should be on rock and the foundation area should be either horizontal or slightly sloping towards the reservoir. It is preferable to anchor the wall at least 0.50 meters into the foundation (see Figure 13-3). In order to ensure the water-tightness of the reservoir, special attention should be paid to the contact zone between the rock and the wall. When a masonry wall is used, it should be plastered (20--30mm) with cement mortar on the reservoir side or alternatively waterproof mortar can be used in the masonry wall (a 1 to 3 cement/sand mix with 1kg waterproofing cement to 50kg regular cement is a general ratio for waterproof mortar mixes).

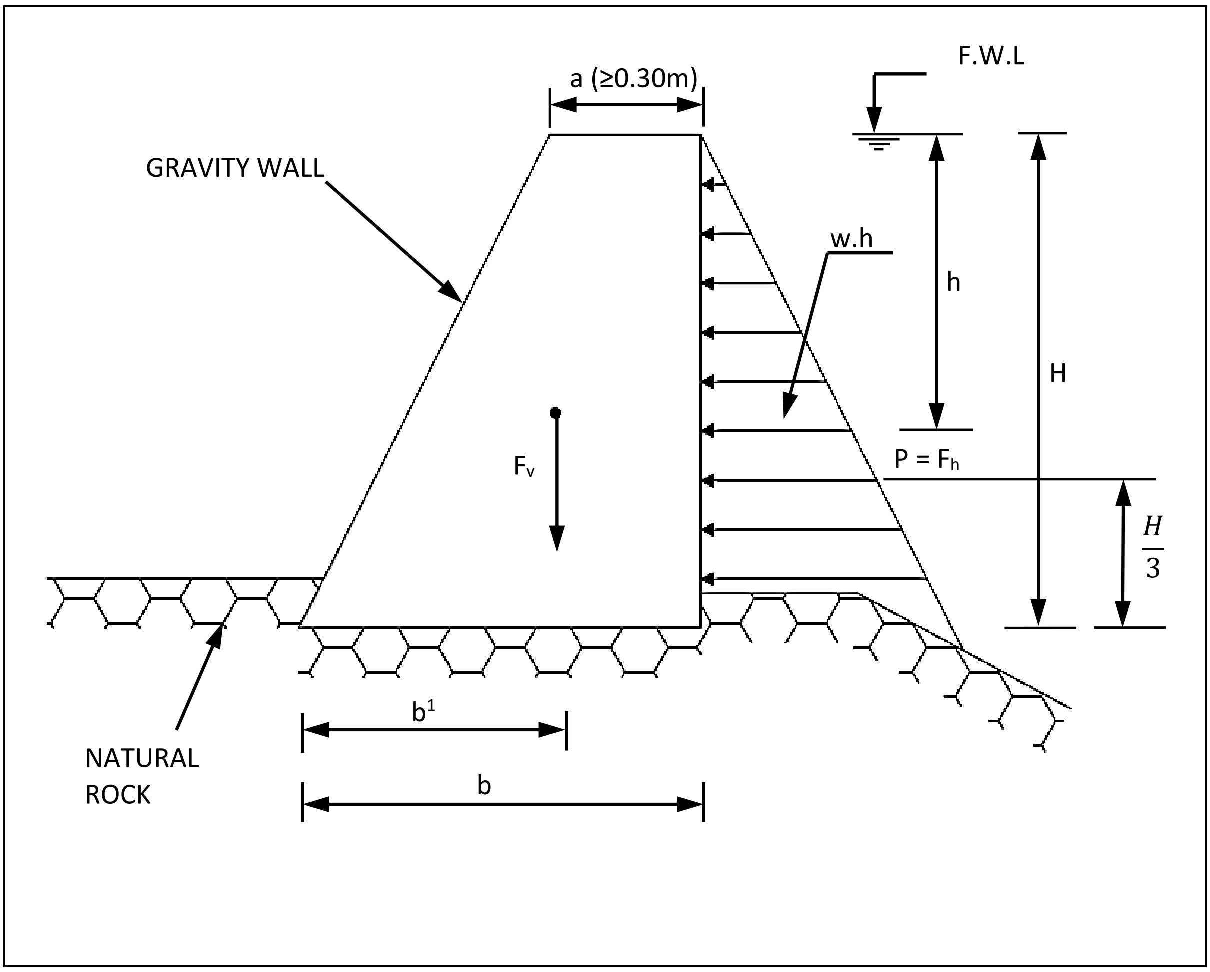

Figure 13-3: Stability Computations for Gravity Walls

The wall should be dimensioned to resist water pressure. Stability against sliding and over turning should be verified. The following formulae are applicable (see Figure 13-3):

Equation 13-2: $\boldsymbol{F_h = P = \frac{w.H^2}{2}}$

Where: F_h is the horizontal force on the wall in kN per m of width;
P is the pressure in kN per m of width;
w is the unit weight of water = 9.81 kN/m^3;
H is the height/depth of water in m.

The overturning moment can be calculated as:

Equation 13-3: $\boldsymbol{M_o = \frac{P\ x\ H}{3}}$

Where: M_o is the Overturning Moment in kN m per m of width;
P equals F_h is the horizontal force on the wall in kN m per m of width;
H is the height/depth of water in m.

Equation 13-4: $\boldsymbol{M_r = \frac{b'.F_v}{2}}$

Where: $M_r$is the Resisting Moment in kN m per m of width;
b' is the moment arm in m (calculated depending on wall geometry and should align with the centre of mass of the structure);
F_v is the weight of the wall in kN per m of width.
(Suggested density values for typical construction materials are 22-24kN/m^3 for mass concrete, 23-25kN/m^3 for reinforced concrete and 24-27kN/m^3 for stone masonry using natural rocks)

Safety factors are calculated as:

Equation 13-5: $\boldsymbol{F_s = \frac{r.F_v}{F_h}}$

Where: F_s is the Factor of Safety for Sliding;
r is the static friction coefficient (suggested 0.55 for foundation on solid rock, 0.45 for foundation on material with coarse granulometry which does not contain silt or clay and 0.35 for all other cases);
F_v is the weight of the wall in kN per m of width;
F_h is the horizontal force on the wall in kN per m of width.

Equation 13-6: $\boldsymbol{F_{o.s} = \frac{M_o}{M_r}}$

Where: $F_{o.s}$ is the Factor of Safety for Overturning;
M_o is the Overturning Moment in kN m per m of width;
M_r is the Resisting Moment in kN m per m of width.

Safety factors with respect to sliding (F_s) and overturning ($F_{o.s}$) should not be less than1.5.

In general, the top width of the wall (a) should not be less than 0.30 m. For walls higher than 2.50m, the top width should preferably not be taken less than 0.50 m.

13.3.3 Overflow Considerations

The most important overflow calculation is to use a suitable design flood to confirm the approach height that will result when the weir is passing the design flood and then to ensure that the approach height will not result in any "out of bank" flows in the stream bed around and upstream of the weir. The broad crested weir formula can be approximated as follows in Equation 13-7. This equation can be rearranged to solve for the depth of flow or the discharge can be calculated for various flow depths until a flow depth that matches the design flood flow is determined.

Equation 13-7: $\boldsymbol{Q = 1.78 \times L \times d^{3/2}}$

Where: Q = discharge [m^3/s]
L = length of the overflow section [m]
d = depth of flow over the weir [m] measured far enough up-stream

A factor of safety is prudent to ensure that the design flood Q can be passed within the overflow section. Equation 13-8 provides an estimate for the final depth of the overflow section.

Equation 13-8: $\boldsymbol{D = 1.5d + 0.1}$

Where: D = Final depth of overflow section [m]

Equation 13-7 can also be used to determine flow through various widths and depth of openings when the weir is used as a flow regulation device.

When the length of the weir is small compared to the flow depth, alternative equations may give more accurate flow measurements.

13.3.4 Uplift Pressures

In the event that the mass gravity dam is constructed on a semi-permeable surface, or where the foundation may have cracks, concerns may arise on the effect of uplift pressures on the structure. This can be resolved by incorporating a filter drain on the downstream section of the structure to dissipate such pressures. Reference should be made to detailed design documents if there is the possibility of uplift pressures.

13.3.5 Draw-off Pipe Sizing

Draw-off pipes in gravity dams are generally a fairly large diameter where they pass through the wall in order to ensure a good contact area between the pipe and the mass gravity wall. Three inch diameter, heavy gauge GI pipe is recommended for most applications. A quick flow check can be made using the proposed pipe cross section area and an assumed velocity of 1m/s. Inlet screens can be positioned as shown in Figure 13-4 .Inlet screen sizing can be carried out based on a required open area calculation. If velocities in the screen are restricted to 0.025m/s, then Equation 13-9 will give a figure for the required open area in the inlet screens. This can then be used along with perforation size and positions to determine a length of inlet screen required.

Equation 13-9: $\boldsymbol{A = Q/V}$

Where: A is the required open area in the inlet perforated pipe [m^2];
Q is the desired flow from the dam [m^3/s];
V is the velocity of the flow in the inlet screen and is set at 0.025 m/s.

13.4 ALDEV Weir Dimensions

The ALDEV weir design was developed by the Arid Land Development Programme in Kenya in the 1950s. It has proven to be a robust design suitable for mass gravity dams less than 5.0m in height and less than 50m in length. Figure 13-4 shows a typical cross section for the ALDEV Weir. The crest and base widths are calculated from the height of the wall. The crest width is 0.2xMax Height and the base width is 0.75xHeight.The upstream face slope is approximately a 7 degree slope toward the crest. A more detailed drawing is provided in Appendix B.

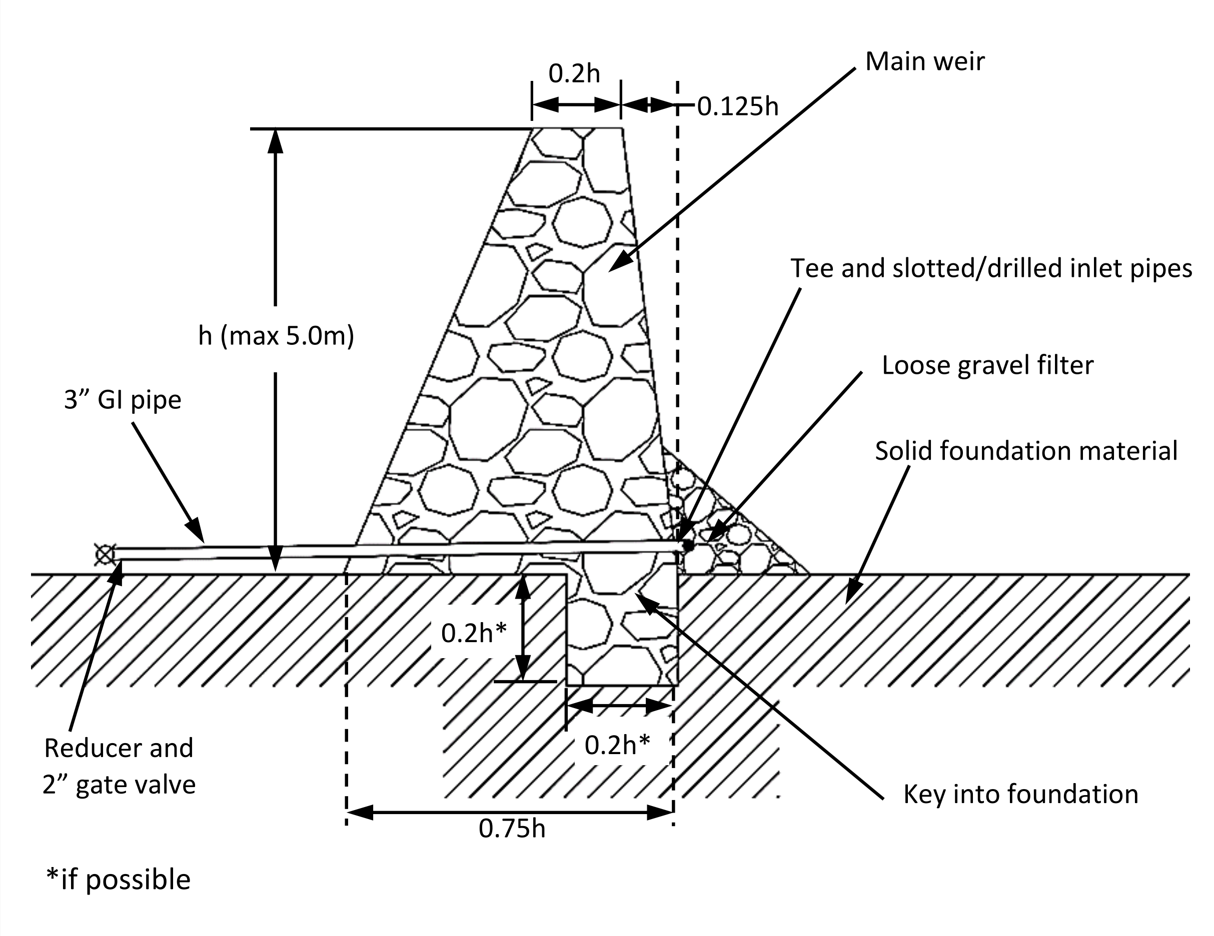

Figure 13-4: Typical Cross Section for ALDEV Weir

13.4.1 Typical ALDEV Wall Dimensions and Volumes

Table 13-1 shows typical values for varies wall heights. Material volumes can be calculated from cross sectional areas and the length of the wall. Typically a ratio of 75% stone to 25% mortar (of 1:3 cement:sand mix) is used. Mortar should incorporate waterproof cement (usually 1kg per 50kg cement).

Table 13-1: Typical Dimensions for Various Heights of Aldev Weir

Height (m)	Crest Width (m)	Key Depth (m)	Key Width (m)	Base Width (m)	Cross Section Area (m^2)
0.0	0	0	0	0.6	0.0
0.5	0.1	0.1	0.1	0.6	0.2
1.0	0.2	0.2	0.2	0.8	0.5
1.5	0.3	0.3	0.3	1.1	1.2
2.0	0.4	0.4	0.4	1.5	2.1
2.5	0.5	0.5	0.5	1.9	3.2
3.0	0.6	0.6	0.6	2.3	4.6
3.5	0.7	0.7	0.7	2.6	6.3
4.0	0.8	0.8	0.8	3.0	8.2
4.5	0.9	0.9	0.9	3.4	10.4
5.0	1	1	1	3.8	12.9

Notes:

- Achieving key depth and width can be very challenging.
- Use a fixed crest width based on maximum wall height.

13.5 Sample BoQ, Specifications and Reporting

A sample BoQ and Specifications for a rubble stone mass gravity dam are available on the website. A sample design report format is available in Chapter 19 of this document.

13.6 Construction Issues

Mass gravity dams should make as much use as possible of locally available materials. This often means constructing of local stone and mortar.

13.6.1 Foundation Preparation

Foundations must be cleaned of all loose material. In the event of weathered or cracked foundations, it may be necessary to grout or infill with a cement slurry. It may be necessary to use a wider base width to ensure a water tight seal along the foundation. Where possible a key trench should be excavated in the foundation material.

13.6.2 Materials Preparation

Sand, ballast, cement, steel (if required) and formwork should be stockpiled on site. Sufficient water must be available for the volume of concrete or mortar required. Water availability is often the largest constraint in constructing mass gravity structures in arid areas. Temporary water tanks and bowsers may be required during construction and during the curing period after construction.

13.6.3 Construction Schedule

For mass gravity dams less than 5m tall and less than 50m in length, construction can generally be carried out in 1 to 3 months with an approximate schedule given below.

Site preparation:	2 weeks;
Material delivery:	2 weeks;
Construction:	4 to 8 weeks;
Curing:	3 to 4 weeks;
Site reinstatement:	1 week.

13.7 Equipment

For the size of mass gravity walls described in this document, the following specialized equipment will be needed.

- Cement mixer;
- Wheel barrows;
- Hosepipe level (or construction level);
- Poker vibrator (if mass concrete is to be used in the foundations or aprons).

Most foundation works can be undertaken with manual labour and normal hand tools (hoes, shovels, mattocks, crowbars, sledge hammers…..). Most masonry works can be undertaken with manual labour and normal masonry tools.

13.8 Construction Supervision

While a site engineer or clerk of works is generally sufficient to ensure reasonable quality control for most mass gravity dam projects, for larger structures, it may be beneficial to put in place strict quality control measures. These may include both cone tests during concrete pouring and cube tests to ensure proper concrete quality.

13.9 Operation and Maintenance

Operation and maintenance of mass gravity walls is fairly simple. The main concerns are erosion (either at the downstream toe or around the wall in the river banks) and siltation.

Erosion should be monitored and repairs carried out as needed during low flow periods. Repairs to the downstream apron will generally involve concrete works to fill in eroded areas. Repairs to the riverbanks/wing walls may involve gabions or additional masonry wall construction.

Siltation is best dealt with during normal flow periods. Silt can be removed through the use of washouts (if installed) or manually.

Where mass gravity walls are used as offtake structures, offtakes can be blocked with leaves and floating debris and should be regularly monitored and cleaned.

13.10 Rehabilitation

Rehabilitation of failed mass gravity dams depends on the original cause of failure. Erosion failures (either below the wall or around the wall) can be repaired with either mass concrete or gabions. Structural failures (either from poor original construction or from improper design) are more difficult to address and it may be better to construct an entirely new wall.

CHAPTER 14

DESIGN OF WATER STORAGE PANS AND LAGOONS

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14 DESIGN OF WATER STORAGE PANS AND LAGOONS

Water storage pans are excavated surface water storage facilities of limited capacity (generally not exceeding 20,000m^3) which are mainly constructed in locations where the topography does not allow the construction of a small dam and instead favours excavation. Excavation of larger pans (up to 150,000 m^3) is possible and can be done, especially near populated centres, but the construction cost is generally high due to the 1 to 1 excavation to storage ratio. Often natural depressions can be enlarged to produce pans with a slightly better storage to earthworks ratio.

Recently in Kenya, lined pans (or lagoons) with storage capacities of up to 70,000 cubic meters have become popular with horticultural farmers.

Pans are excavated below the natural ground level, and with the exception of pans constructed on inclined locations, the volume of earth excavated will be equal to the storage capacity of the pan. Compared to a small dam, the water to earth ratio (water storage volume / earth excavated volume) – (see Section 10.7) is unfavourable. However, when a suitable inclined location can be identified for the construction of the pan a somewhat more favourable ratio can be obtained. An example of a pan constructed on an inclined location is shown in Appendix B Type Drawing VIII - Water Pan on Inclined Location. Storage pans tend to be relatively expensive constructions when compared to small earth dams.

Water storage pans are subject to the same limitations regarding sedimentation and evaporation as small dams (see Section 3.3 and 8.13). Due to their shallow depths (usually 2.50 m to 5.00 m) water storage pans are usually not suitable as permanent water sources for high evaporation areas, while for catchment areas subject to erosion, silt traps will have to be included in the design.

14.1 Merits and Demerits of Pans and Lagoons

Despite their increased use for water storage, pans and lagoons have a number of advantages and disadvantages as outlined in Table 14-1:

Table 14-1: Some Merits and Demerits of Unlined Pans and Lagoons

	Unlined Pans	**Lagoons**
Merits	Less complicated construction compared to dams	Can be constructed on any soil type
	Structural failure not a concern	Lined so seepage is not a constraint
Demerits	Elevation restricts conveyance by gravity	Expensive to construct
	Unfavourable water to earth ratio	Small tear in lining can lead to failure
	High evaporation losses	

A typical pan design should include the following drawings:

- Site map (google, 1:50,000 or 1:100,000 contour map);
- Site layout (showing water flows);
- Inlet channel cross section and longitudinal profile;
- Outlet channel cross section and longitudinal profile;
- Cross section (pan and silt trap);
- Control/spillway/overflow sill details;
- Inlet/outlet details/ramp details.

The following sections look at unlined pans in arid areas and lined lagoons for commercial water storage.

14.2 Guidelines for the Design of UNLINED Water Storage Pans in ASAL Areas

Unlined pans in arid areas are generally less than 5 m depth and water is mainly used for livestock. Appendix B gives the following drawings for unlined storage pans:

- Type Drawing VI: 11,600 cubic meters;
- Type Drawing VII: 20,000 cubic meters;
- Type Drawing VIII: 17,000 cubic meters on inclined site.

Recent experience (2000 to 2014) has shown that unlined pans can generally be constructed for approximately 500 ksh per cubic meter of soil excavation.

14.2.1 Location

Pans for the purpose of surface water storage can be constructed wherever a sufficient quantity of water can be intercepted to create a small reservoir. Pans are basically used in such locations where no topographically suitable site can be found for the construction of a small dam, or where no suitable construction materials for the construction of a dam can be found.

Apart from the two factors mentioned above (topography and availability of construction materials), the same basic principles for selection of appropriate locations as explained in Chapter 10 will apply. Particular attention should be paid to:

i. The water-tightness of the reservoir in sandy areas: Contrary to earth dams, lining of the reservoir with an impervious clay blanket can often present a solution for pans, since reservoir dimensions in this case are limited.
ii. The natural drainage and flow pattern of the intercepted water: Spillways are not generally provided for pans, but an overflow structure for any excess water towards the natural drainage is included in the design. The slopes of any overflow or diversion channel must be considered carefully to reduce erosion.
iii. Silt trap requirements. Silt traps are normally included in the construction of unlined pans. Often, the silt trap is combined with the overflow structure.
iv. Sedimentation, evaporation and ecological impact along the same lines as set out under Chapters 3, 6 and 8.
v. Pans are generally laid out in a rectangular arrangement and careful thought should be given to the specific alignment of the pan. Pans constructed on an inclined surface should be positioned to minimize earthworks. This usually means placing the long side of the rectangular pan parallel to the natural ground contours (See Figure 14-1 for a typical pan layout).

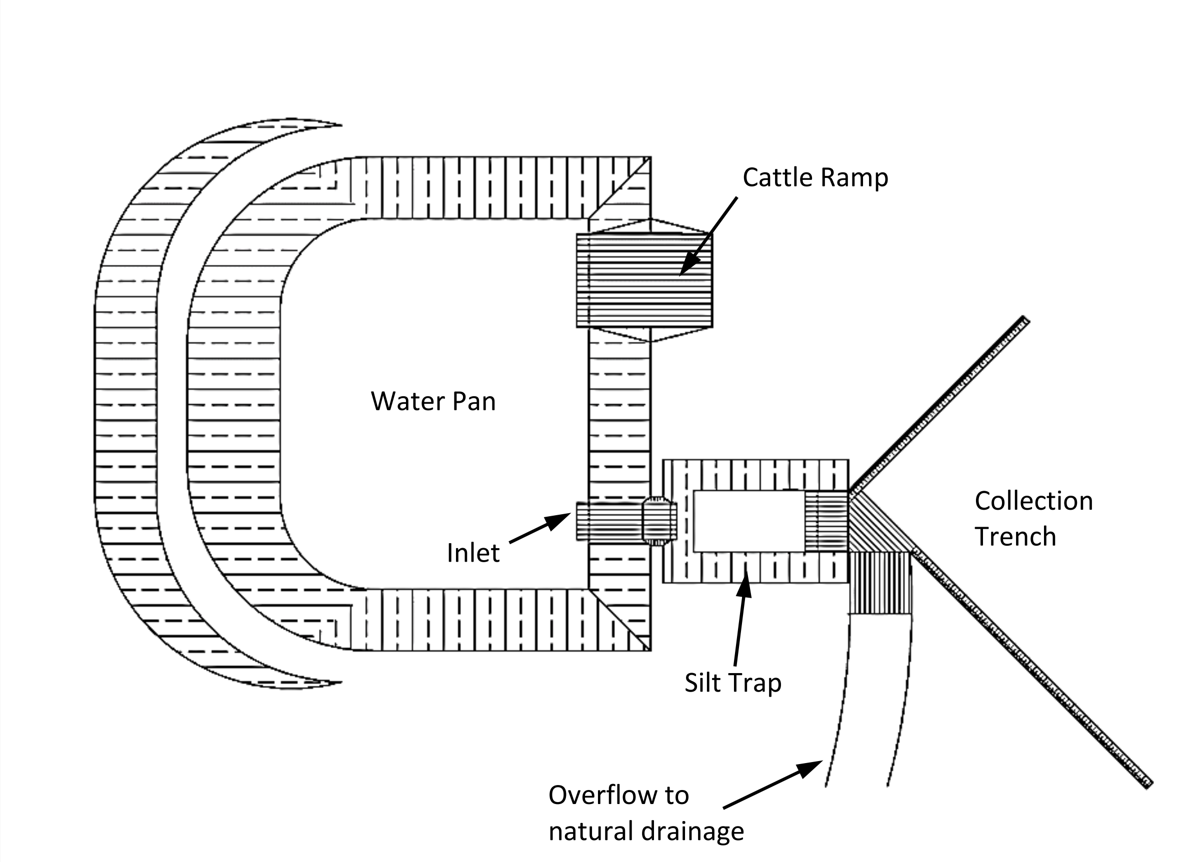

Figure 14-1: Typical Site Layout showing Silt Trap and Overflow

vi. Storage sizes should be considered carefully and need to address the expected inflows and the expected water use. Generally pans in arid areas should be sized with emphasis on availability of grazing (i.e. the pan should dry out just as the available grazing is finished). Large pans may result in overgrazing in the area around the pan.

14.2.2 General Consideration

The size of the pan will generally be a function of the required storage. The maximum capacity of a single unlined water storage pan in an arid area is generally 20,000m^3. If the required storage exceeds this capacity, more than one pan should be constructed. The exception to this rule is in built up areas where the pan will serve as a permanent water source. In such cases, larger pans can be justified. Pans of up to 150,000 m^3 of storage have been constructed in Kenya.

The depth of water storage pans will generally not exceed 5.00 metres, and will in no case be less than 2.50 metres. Which depth is adopted depends mainly on the type of soil in which the pan is excavated. Deep pans are possible in soils with a good granular distribution and impervious properties. In the case of sandy soils or heavy clay soils, the depth of excavation will be limited in order to assure the stability of the slopes of the reservoir walls and shallow pans will be preferred. From the point of view of evaporation, a deep pan is of course preferable to a shallow one.

The shape of the pan depends mainly on the available land. Generally, pans are rectangular or square. Round pans are possible but they can be quite difficult to lay out properly.

The slopes of the excavated pan (and silt trap) will generally be 1 (v) to 2.5 (h), except when relatively deep pans need to be constructed in sandy or heavy clay soils. In these cases, slopes of 1 (v) to 3(h) or shallower can be adopted.

The construction of water storage pans in relatively pervious soils is only possible if sufficient water-tightness of the reservoir can be achieved. In cases where the excavated reservoir is judged insufficiently impervious (sandy soils, fissured rock bottom etc.), lining of the reservoir with a 300 mm thick clay blanket can be a solution, if sufficient clay is available within the vicinity of the site.

14.2.3 Inlet and Overflow Structures

In order to convey water into the pan, two collection trenches (see Type Drawing VI- Water Storage Pan for details) are usually constructed. To avoid high flow velocities and erosion, the slope of those trenches should not exceed 1%. They can either be constructed with an angled dozer or grader blade. Length and exact location for each pan site has to be determined as a function of the topography of the site.

Whether the construction of a silt trap will be required or not, depends largely on the expected sediment yield from the catchment (see Section 8.13). Flat catchments with good vegetative cover produce less silt than steep or overgrazed catchments. Depending on the expected silt load, the dimensions of the silt trap will usually be either 15.00m x 25.00m or alternatively 20.00m x 30.00 m. Depth of silt traps will usually be 2.00 metres. The trap efficiency (see Figure 8-13) of the silt trap will gradually decrease as the silt trap fills up. Hence it is highly desirable to de-silt the silt trap at regular intervals.

Flow of water through the pan should be avoided in order to prevent large sediment deposits in the reservoir. As a general rule, once the maximum water level in the pan is reached, any excess water should be diverted away from the pan. Hence the inlet sill level should be determined accordingly.

Details of inlet and overflow sills are given in Appendix B Type Drawing VI-Water Storage Pan. Width of the inlet sections to silt-trap and pan should not be less than 6.00 m.

Inlet and overflow sections need to be protected against erosion by an adequate rip-rap protection of at least 300 mm thick. Impervious clay blankets need to be extended to these areas in cases where they are required (see Type Drawings VI , VII and VIII).

14.2.4 Outlet Structures

The choice of outlet structure depends mainly on the expected water users. If the pan is to be used solely for livestock, then a cattle ramp can be installed (see Type Drawing VII). This requires minimal maintenance and is the simplest “outlet structure” to construct and maintain.

If the pan is to be used for domestic water supply, the best option is generally a well dug beside the pan. The well can be equipped with a hand-pump or a bucket and windlass (see Type Drawing VI and VIII). Water flow into the well can be improved through the use of an infiltration gallery. Provision should be made for a cattle trough to be supplied with water from the well.

As small petrol pumps have become more common in Kenya, water use can be done through the use of a pump. In such cases, several level area where pumps can be positioned should be established during construction. Care must be taken to avoid polluting the water with spilled petrol or oil. Permanent pipe installations are not recommended as they are prone to damage and theft.

14.2.5 BoQs and Specifications

BoQs and Specifications for a selection of unlined pans are available on the website.

14.2.6 Construction Details

Type Drawing VI - Water Storage Pan - shows a typical rectangular pan, 4.00 metres deep, and with a total storage capacity of 11,600 m^3, equipped with a large (20.00m x 30.00m) 2.00 metres deep silt trap. Construction details of concrete sills and collection furrows are also given.

Sills should be made of Class 25 (1:2:4 mix) mass concrete. They can either be 0.70 metres (preferably) or 0.50 metres deep.

This drawing also shows the usual way of disposing of the excavated material. Excavation of pans is started by “stripping" the proposed area of 0.20 m - 0.30 m of topsoil. This topsoil is then stockpiled for later use as a cover to the excavated material. The excavated soil is disposed of as shown in Type Drawing VI, in a berm, not higher than one metre, with slopes of 1 to 10, and covered with topsoil to induce vegetative growth. The width of the berm depends on the quantity of excavated soil available.

Properly compacted rip-rap protection (compaction by dozer) of not less than 300 mm thick should be used where indicated. Angular rip-rap (100 - 200 mm) from solid rock is preferable. For cattle ramps the rip-rap layer should be covered with gravel.

Type Drawing VII - Standard Water Pan 20,000 m^3 - shows the construction details for a more elaborate water storage pan, where the excavated soil has been utilised to construct a 3.00 metre high embankment around the whole pan. This type of embankment needs to be properly compacted. In this case, the storage pan can remain unfenced. This Type Drawing also illustrates the use of an impervious clay blanket lining.

Type Drawing VIII - Water Pan on Inclined Location/Dug Well and Filter - shows details of a 11,700 m^3 capacity pan on an inclined location, where a berm made from compacted excavated material has been used on the lower side, as well as all relevant details of the well and filter for abstracting water for human consumption.

14.2.7 Equipment

Although small pans can be constructed with manual labour (usually up to about 10,000 cubic meters of storage and usually done under food for work or cash for work programmes) it is most common to use mechanized equipment.

A small pan can be constructed with the use of a bulldozer where the bulldozer simply excavates and pushes material. Push distances should be kept less than 60m.

Larger pans will also require a bulldozer as well as some sort of loader (wheeled or tracked) and several tippers. When push distances exceed 60m it is generally less expensive to load tippers and then haul and dump at the appropriate spoil area.

While larger excavators can be used to both excavate and load tippers, they do not produce uniform excavated slopes unless they have a skilled operator.

14.2.8 Construction Supervision

Construction supervision should be most intensive during the initial site layout. Once laid out weekly visits to ensure that the design is being followed should be sufficient.

Competent small contractors can usually shift 300 to 700 cubic meters of soil per day. Larger contractors may shift up to 1,000 cubic meters of soil per day. Using those figures, a 20,000 cubic meter pan could take 20 to 66 days to construct.

14.2.9 Operation and Maintenance of Unlined Pans

The main problems encountered with unlined pans are:

- Not filling;
- Excessive seepage;
- Excessive sedimentation;
- Controlling livestock access;
- Poor water quality;
- Difficulties in extraction of water.

In the event of a constructed pan not filling, efforts should be made to increase the catchment area for the pan. This can be done by constructing contour bunds to collect water from a larger area. Alternatively inlet structures and canals to capture flood flows from nearby rivers or laggas can be constructed.

For a pan that fills but then quickly loses water to seepage, lining should be considered. Typically a 200 to 300 mm thick clay lining will be required. Artificial linings will not withstand long empty periods and are easily damaged by livestock and wildlife. Bentonite or other commercially available clays can be added to the existing soil to reduce permeability. These can be purchased from local mining companies.

Excessive sedimentation can be addressed through improvements to silt traps, improved control of inflows or catchment improvement campaigns.

Livestock access can be controlled by fencing. If possible live fencing is recommended. Community awareness raising can also be undertaken to reduce uncontrolled livestock access.

Poor water quality can be address with an improved offtake. An offtake that incorporates a sand/ballast filter is recommended.

Difficulties in extracting water can be addressed through improved offtakes and or pumped offtakes. Most offtake improvements will significantly increase the operating costs of the reservoir.

14.2.10 Rehabilitation of Unlined Pans

Rehabilitation works will mainly concentrate on removal of silt (effectively re-excavating the pan) or improvements/repairs to the offtake works.

During rehabilitation it might also be necessary to extend or improve any inflow channels.

14.3 Guidelines for the Design of Lined Lagoons

Recent changes in the availability of HDPE and LDPE lining material have resulted in lagoons being adopted by commercial farmers. Commercial lagoons can be up to 70,000 cubic meters in storage and may have water depths of up to 7m.They are generally positioned to collect runoff from greenhouses or other farm structures. Such lagoons will have a lined spillway and it should be sized to meet the expected maximum runoff flow.

In the context of this manual, lagoons differ from pans in that they are always lined with an artificial lining material and they are normally constructed on sloping land. Construction on sloping land means that part of the lagoon will be an earth embankment dam and part of the lagoon will be an excavated pan.

A typical lagoon drawing with details on topography and associated structures is provided in Appendix B Type Drawing XII – Lined Lagoon on Inclined Location. It should be noted that the final design/layout of a proposed lagoon will depend on the topography of the land and the availability of space.

Lagoons can generally be constructed for KSH 250 to 350 per cubic meter of storage (in Kenya in 2015). This is slightly better than for pans because lagoons are generally constructed on sloping ground and have a better than one to one earthworks to storage ratio. Lining material costs must be added on to this figure. Lining can cost 2 to 3 USD per square meter (installed).

14.3.1 Location

Lagoons are generally positioned in the lower elevations of commercial farms so that they can collect runoff. In general, they are best positioned on land with 1 to 3 percent slope. They should be designed so that the crest level is equal to the ground level at the highest spot in the original ground as this will allow runoff drainage to fill the lagoon.

As the lining is very susceptible to damage by wildlife or livestock, lined lagoons should not be considered unless they are in an area with controlled wildlife and livestock access.

14.3.2 General Consideration for Lined Lagoons

The size of the lagoon will generally be a function of both the available area and the expected runoff. In general, lagoons are in the range of 30,000 to 70,000 cubic meters of storage and have water depths of 5 to 7 metres.

The shape of the lagoon depends mainly on' the available land. Generally, lagoons are rectangular or square. Other shapes are possible but they can be quite difficult to lay out properly.

Lagoon slopes will generally be 1 (v) to 2.5 (h). Although steeper slopes are possible, care should be taken with upstream slopes as any slumping beneath the lining will result in catastrophic lining failure.

Crest widths are generally kept at 4 to 5m wide to allow machinery access during construction.

Soils in the embankment are usually compacted but not to the same degree as for unlined dams. In general, compaction that allows tippers to run along the crest/working surface is considered sufficient.

14.3.3 Lining Materials

Lining materials can vary widely in terms of price and availability. 1mm to 1.5mm thick LDPE lining is a popular lining material. HDPE lining is possible but is often very rigid and does not conform to the finished earthworks as well as LDPE lining material.

Lining generally comes in rolls (7.5m to 8m wide, 180 to 200m long) and is cut and welded on site with specialist equipment. Welding lining material requires an adequate electricity supply at the site (generator or mains).

Most lining leakages occur through poorly welded seams and most poorly welded seams can be traced back to low voltages or inadequate electricity supply at the site. This is especially the case where welding equipment has been run using long (over 200m) extension cable runs. It is always preferable to use a generator at the site for welding lining material.

14.3.4 Drainage Blankets

Many lagoons are designed with a drainage blanket to collect any seepage that occurs between the lining and the final ground levels. This seepage must be allowed to drain through the embankment to prevent "blisters" and possible failure of the lining. For most sites, a 1m x 1m trench, passing under the embankment and filled with hardcore and ballast is a sufficient drain.

For sites with significant groundwater emergence, it may be necessary to design larger drains or consider a sump and pump arrangement underneath the lining material.

14.3.5 Inlet and Overflow Structures

Inlet structures generally consist of farm drains. Care must be taken to prevent water from seeping between the lagoon liner and the final ground surface. This can be done by extending lining 10 to 15m along the drain or by installing concrete sills to fix the lining in place at the end of the drain.

Silt traps are generally not necessary as most runoff is from greenhouses and is very clean.

Spillway designs should be carried out and spillways should be placed as dictated by site topography. Spillways should be lined with the same lining material as the lagoon. Concrete sills are not necessary and can actually cause damage to the lining material. Spillway lining should extend the entire length of the spillway plus 10 to 15m. Spillways should terminate in a concrete sill which fixes the lining in place.

14.3.6 Outlet Structures

Outlet structures can be pipes that pass through the lining and lagoon wall. These should be sized according to expected water use. Flanged connections should be used where pipes pass through the lining.

Alternatively, floating offtakes can be used. Either pontoons with a pump can be floated on the lagoon or a crest mounted pump can be outfitted with a flexible suction line that extends into the lagoon.

Solar pumps with floating offtakes are becoming a popular option for moving water from one lagoon to another in large farms or from a lagoon to an elevated tank for water projects.

14.3.7 Safety Concerns

Lining materials are slippery when wet and it is almost impossible to climb unaided from a full, lined lagoon. Safety ropes and floats should be left in the water to assist any person who accidently falls into the lagoon. All lagoons should be fenced and sign posted to control access.

14.3.8 BoQ and Specifications

Typical BoQs and Specifications for a lined lagoon are given on the web site.

14.3.9 Construction Details

Figure 14-2 below shows an example of a typical lagoon. If a silt trap is required, it can be placed at any convenient point along the inlet channel.

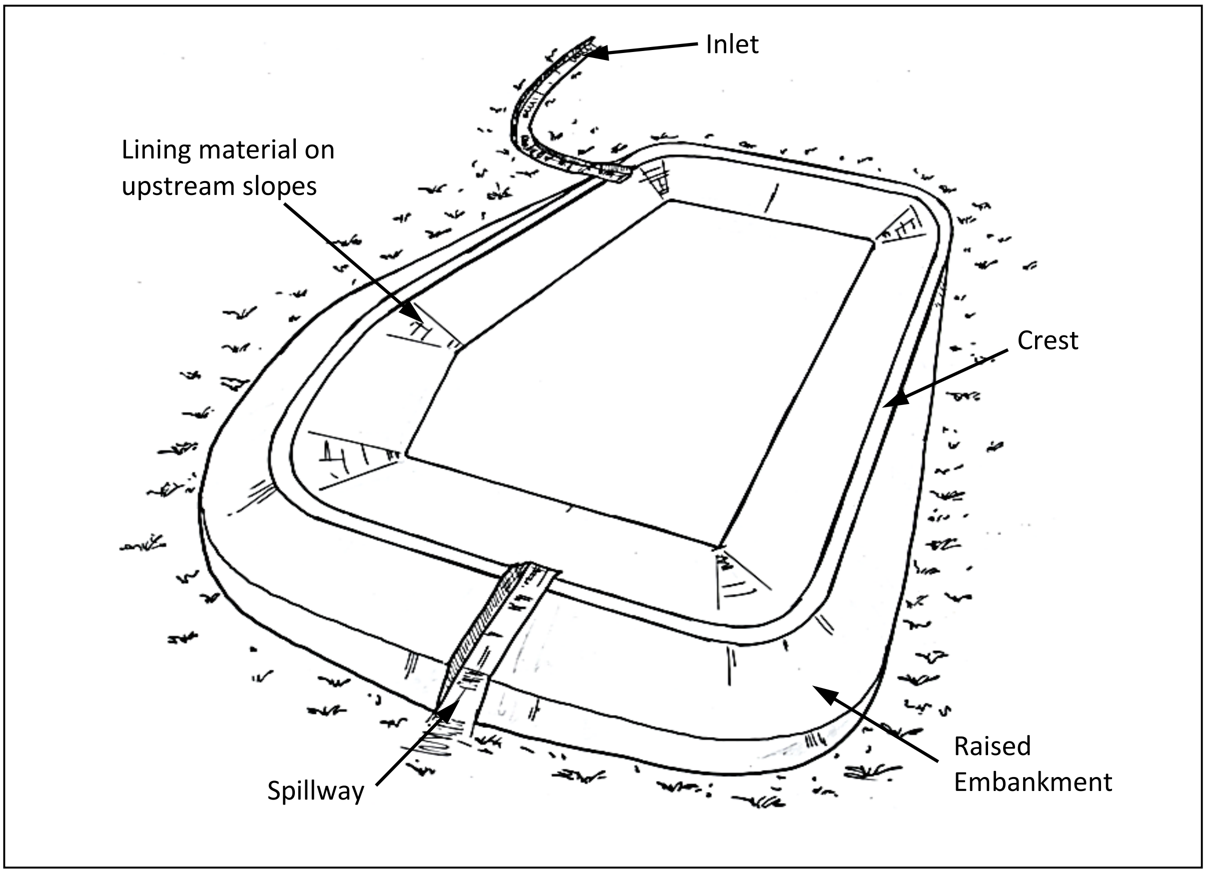

Figure 14-2: Typical Lagoon

Construction is similar to an earth embankment. Excavation and embankment slopes are normally 1 (v) to 2.5 (h). The embankment is constructed in 300mm layers. Compaction can normally be carried out with bulldozers, tippers or other heavy equipment. In general, compaction that allows vehicle access along the working surface is sufficient for the lined structure.

Care must be taken to allow any seepage between the lining and the final ground levels to pass through the embankment. This can generally be done with a 1m x 1m hardcore/ballast filled drainage trench.

The drainage trench can also be used to pass any offtake pipes through the wall.

Considerable care must be taken where offtake pipes pass through the lining. Concrete anchors and bolted flanged connections are preferred.

Considerable care must be taken when securing the lining around the crest of the lagoon. This is normally done with a 50cm x 50cm trench. The lining is placed in the trench and then the trench is backfilled. During the initial filling, the lining should have sufficient slack to allow for any settlement in the embankment. This can be done by securing the lining in the trench with a sandbag every 6m.Once the lagoon has been filled for several weeks, the trench can be backfilled properly and the lining will be securely fixed.

14.3.10 Equipment

Lagoons are generally constructed with mechanized equipment. The following equipment is required:

- Bulldozer (For excavation, trimming slopes and spreading embankment material);
- Loader (for loading tippers);

- Tippers (transport of excavated material to embankment or to spoil, at least two usually four or five);
- Water bowser (for watering material as it is spread and placed in embankment, can be done with hosepipe and existing water supply on many farms);
- Grader (for trimming slopes and spreading embankment material);
- Roller (for compacting material in embankment, not necessary but preferred).

14.3.11 Construction Supervision

Construction supervision should be most intensive during the initial site layout. Once laid out weekly visits to ensure that the design is being followed should be sufficient.

Competent small contractors can usually shift 300 to 700 cubic meters of soil per day. Larger contractors may shift up to 1,000 cubic meters of soil per day.

Supervision of lining placement is important to ensure that all seams are properly welded and that all offtake pipes are properly positioned.

14.3.12 Operation and Maintenance of Lined Lagoons

The main problems encountered with lined lagoons are:

- Torn or damaged lining;
- Algae blooms;
- Blistering of lining material.

Torn or damaged lining material requires that the lagoon be emptied, the tears and damage be identified and then repairs carried out. Welding LDPE or HDPE lining requires that all surfaces are dry. It should be noted that large tears or leaks may endanger the lagoon embankment and that if a significant tear or leak is suspected, the lagoon should be drained as quickly as possible. Any wildlife or livestock that enters a lagoon will damage the lining as it struggles to get out. Access to the lagoon must be rigidly controlled with fences and locked gates.

Algae blooms have been noted in many shallow horticulture farm lagoons. This is partly due to the temperature of the stored water (the black lining helps warm the water) and to fertilizers in some of the runoff water. They can be controlled by introducing fish, adjusting pH or with the use of other chemicals.

Blistering of lining material is usually a sign of excessive seepage between the ground surface and the lagoon lining material. It can be addressed through interception drains uphill from the lagoon or by improved drainage under the lining. Improved drainage under the lining will require that the lagoon be emptied and the lining removed while the drains are installed.

14.3.13 Rehabilitation of Lined Lagoons

Rehabilitation works will mainly concentrate on replacement of lining and filling in any eroded embankments. During rehabilitation works, improved drainage can be installed under the lining.

CHAPTER 15

DESIGN OF SAND DAMS

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15 DESIGN OF SAND DAMS

Sand dams are structures (usually mass gravity walls) that are constructed in a watercourse and are designed to retain both sand and water. When properly designed and constructed, the reservoir area will be filled with coarse sand and water will be stored in the voids between sand grains.

Several manuals address sand dam design and construction in detail and are relevant to sand dam projects in Kenya. "A Practical Guide to Sand Dam Implementation" by the Rainwater Harvesting Network is available online. "Building Sand Dams: A Practical Guide (Madrell & Neal, 2013) is also available online.

Sand dams are most appropriate in areas with high evaporation rates. They are always located in watercourses. An ideal sand dam site is a seasonal river with solid rock bars running perpendicular to the water course. River banks should be rocky material and the banks should be quite deep (at least 2 meters and possibly up to 3 or 4 meters). The *lagga* must have a heavy sand load when it is in flood. The slope of the *lagga* bed upstream of the site should be shallow (so that the sand will fill a large distance upstream of the sand dam).

The construction of a successful sand dam is heavily dependent on proper site selection. Before undertaking a sand dam project, it is highly recommended that a successful sand dam be visited. Many NGOs in Kenya have implemented intensive sand dam campaigns in Kajiado, Kibwezi and other suitable areas and will often assist with technical or community visits to discuss and demonstrate what makes a good sand dam site.

Sand dams offer a viable water storage alternative in arid and semi-arid areas where surface water reservoirs or the construction of small earth dams might not always offer the most appropriate solution. This may be due to a variety of reasons including topographic conditions, availability of suitable construction material within an affordable distance from the dam site and high evaporation rates.

15.1 Introduction

A sand-storage dam impounds water in sediments caused to accumulate by the dam itself.

The general principle of a sand-storage dam is illustrated in Figure 15-1. By the construction of a weir of suitable height across a riverbed, sand carried by heavy flows during the rainy season is deposited upstream of the weir, and the reservoir fills up with sand hence creating an artificial aquifer. This aquifer is replenished during the rains, and water for use during the dry season is stored. The weir must be built in stages of approximately 0.3 - 0.5 metres each and allowed to fill with coarse sand before the next stage is built. Failure to do this will result in the reservoir area filling with fine silts rather than coarse sand. Fine silts will not store appreciable amounts of water and will result in a failed project.

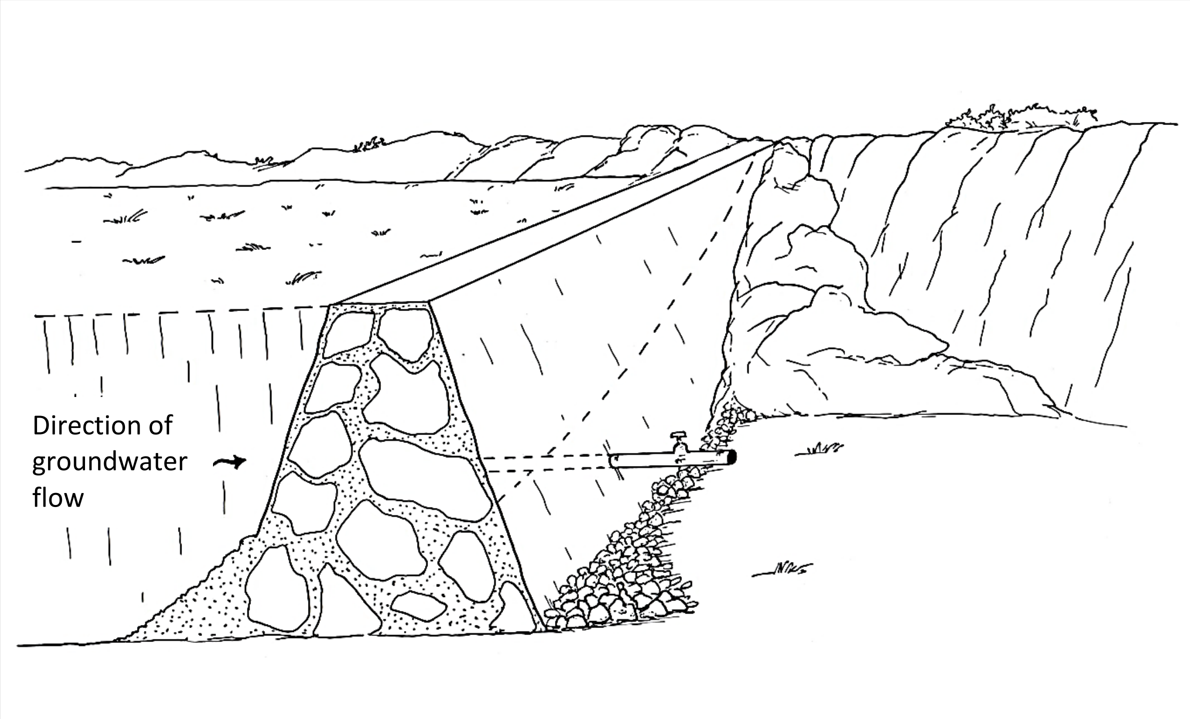

Figure 15-1: Typical Sand Dam

The main advantage of using sand dams instead of conventional dams is a considerable reduction of water losses through evaporation, which is particularly interesting in arid and semi-arid areas. According to Nilsson[1]evaporation from a fine sand storage structure is 50% of that from an open water surface. When keeping the water level at 0.6 m depth in medium sand, evaporation can be reduced to 10% of the open surface value. The granulometry of the material has also an influence on evaporation losses. Evaporation as a function of particle size obtained by Hellwig (Journal of Hydrology 18-1973) is shown in Figure 15-2.

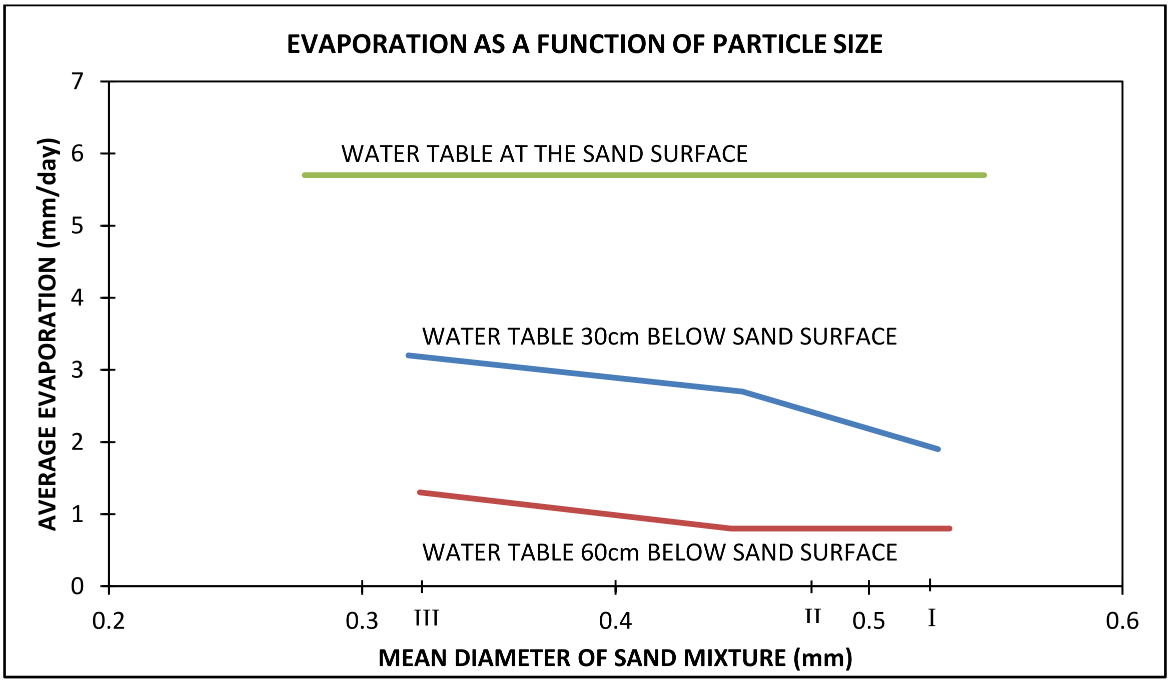

Figure 15-2: Evaporation in Sand Dams

[1]Ake Nilsson (1988): "Groundwater Dams for Small-Scale Water Supply"-IT Publications, London

Water stored in sand storage dams is also less susceptible to pollution and less likely to introduce health hazards associated with mosquito breeding.

15.2 Site Selection

Site selection should be considered the most important part of the sand dam project.

15.2.1 Topography/Geology

Topographical and geological conditions determine to a large extent the technical possibilities of constructing sand dams. Storage volumes should be as high as possible, and at the same time dam heights and wall volumes should be kept minimal. The water tightness of the basin in which the water will be stored should be guaranteed. The presence of impermeable bedrock across the floor and both river banks to key the weir into is a pre-requisite for a trouble free sand dam.

The particle size of sediments accumulated in riverbeds is generally proportional to the riverbed slope, whereas depth and lateral extent of the deposits is generally inversely proportional to the slope. The most favourable sites for the construction of sand dams occur when an optimum relation between both factors is achieved. According to Nilsson such sites are most often found on the gentle slopes in the transition zone between hills and plains.

15.2.2 Sediments

The deposition character of sediment influences construction of sand dams in river beds. Sediments originate from parent rocks in the catchment area through weathering and erosion. Coarse sand and gravel particles are desirable in the reservoir. The most favourable rocks are granite, quartzite and sand stone, but also dams constructed in gneiss and mica-schist areas have been successful. Areas underlain by basalt and rhyolite tend to be less favourable however.

The total extent of erosion is largely dependent on rainfall intensity, slope and land use. Catchments with steep slopes and little vegetative cover are usually favourable for the construction of sand dams.
When surveying an area to find a suitable site for a sand storage dam, the absence of sand deposits along the river is not necessarily an indication that no such deposits will occur when the dam is built. It could be that such heavy peak flows occur that sand deposition is not possible under natural conditions.

Total rates of erosion in catchments and rates of sediment deposition in (large) dams are generally reasonably well known, but are not directly applicable here, since sand storage dams use only a fraction of the total sediment load. In order to estimate the useful portion of sediment deposit, a particle size analysis of sediment samples can be helpful. Sediment samples for analysis should as much as possible be obtained during the floods.

15.3 Site Investigations

The construction of sand dams is rather inexpensive and straight-forward. However, many sand dams have been unsuccessful due to unforeseen seepage losses through underlying fracture zones, damage due to improper foundation, erosion around the walls etc. It is therefore important to conduct a minimum number of site investigations before the construction of a sand dam is decided.

15.3.1 Desk Study

Existing geological and hydrological reports and papers, together with topographical and geological maps, satellite images (e.g Google Earth) and aerial photographs can yield valuable information in order to specify target locations for field reconnaissance investigations.

15.3.2 Field Reconnaissance

Field reconnaissance is required in order to identify specific potential sites. The field reconnaissance should focus on:

- Establishing the nature of sediments in the water course;
- The size and condition of catchment;
- The condition of the river banks;
- The depth to and nature of bed rock, both on the floor and sides of the water course to ensure a secure and water tight foundation for the wall;
- The longitudinal profile upstream and downstream of the site;
- The cross section of the potential site.

In order to establish the depth to bed rock, test pits can be excavated, making sure to secure the side walls from collapse. Alternatively, a metal rod (e.g. 10 mm diameter round bar with pointed end) can be used to poke into the sand to test for bedrock.

If there is any doubt about the impermeability of the bedrock foundations for the sand dam, the foundation area should be exposed and evaluated for cracks and weathering. An impermeable foundation is required.

Sand deposits should be evaluated. Test pits should evaluate whether sand deposits are homogeneous or layered. Homogeneous coarse sand is ideal. A repeating thick layer of coarse sand with a thin layer of finer sand and silt on top of it also indicates a good sand dam site. Deep deposits of fine sand and silt indicate marginal sites. Samples from the sand deposits should be taken and should be submitted to a laboratory for the following tests:

- Determination of porosity (the ratio of the volume of voids to the total volume of the aquifer);
- Determination of specific yield (the quantity of water that can be extracted from a saturated sand volume, expressed as a percentage of the total volume);
- Particle size analysis.

The tests above can also be carried out in the field with a minimum amount of specialized equipment.

A survey of any other materials which can be used during the construction of the dam should also be carried out during the field reconnaissance (rocks for rubble stone, nearest source of cement, nearest source of water, ballast, formwork, secure storage area, etc.).

15.3.3 Laboratory Soil Tests

Figure 15-3 shows the test cylinder used for measuring porosity and specific yield. Porosity is determined as follows: A test cylinder (capacity > 2000 ml) is filled with 1000 ml of water and sand is added until the sand level reaches the 2000 ml mark. The water level will be higher than the 2000 ml mark. Water is drained from the test cylinder to adjust the water level to the 2000 m1 mark. The volume of the drained water is recorded.

The porosity (%) is calculated as follows:

Equation 15-1: $\mathbf{P = (V/T)*100}$

Where: P = Porosity [%] (typically 20 – 35%)
V = Volume of water remaining in the test cylinder [ml] = $(1000ml - V_{drained})$
T = 2000 [ml]

To avoid air voids in the test cylinder, it is essential to put the sand sample into the water and not the other way round.

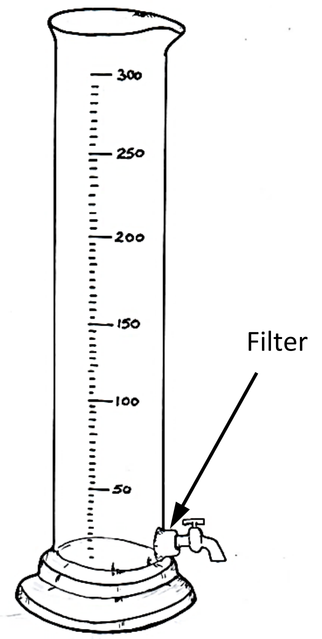

Figure 15-3: Test Cylinder for Measuring Porosity

To determine the specific yield, the test cylinder (capacity >2000 ml) is filled with 1000 ml of water and sand is added until the sand level reaches the 2000 ml mark. The water level will be higher than the 2000ml mark. Water is drained from the test cylinder to adjust the water level to the 2000 m1 mark. This water is discarded. The test cylinder is now drained for 30 minutes and the volume of water released during the 30 minutes is recorded. From this the specific yield is determined as follows:

Equation 15-2: $$S_y = (V/T) \times 100$$

Where: Sy = Specific Yield [%]
V = Volume of water drained over 30 minutes [ml]
T = 2000 [ml]

A particle size analysis can be carried out as described in Section 9.4.4

15.4 Hydrology

In general, a valley that carries sufficient sand load to justify constructing a sand dam will almost always carry sufficient water to fill the sand dam. Hydrological calculations can be carried out to determine the annual volume of water available for storage and the expected maximum flood at the site. Hydrological calculations are described in detail in Chapter 8 and simple calculations are briefly described below.

A quick estimation of the water available for storage in a sand dam can be made using a 5% runoff factor, the catchment area and annual rainfall as shown in Equation 15-3.

Equation 15-3: $$V = 0.05 * A * R$$

Where: V = yearly runoff volume available for storage [m^3]
A = catchment area [m^2]
R = annual rainfall [m]

The expected maximum flood can be estimated from Table 15-1 by multiplying the catchment area in km^2 times the appropriate Q_{100} figure given in the table.

Table 15-1: 100 Year Return Period Discharge

Catchment Area (km^2)	Q_{100} ($m^3/s/km^2$)
< 1	15
1 - 5	12 - 10
5 - 25	3 - 6
25 - 100	3 - 2
100 - 1000	1 – 0.4
> 1000	< 0.3

The broad crested weir formula can then be used to determine expected flood depths (approach heights) upstream of the sand dam/weir. The final sand dam crest level must be positioned so that when the maximum flood passes over the sand dam, the flood is still contained within the river banks. In this case, the crest must be at least "D" meters below the level of the river bank (see Equation 15-5). If this is true, then the expected maximum flood will not result in any "out of bank" flows in the stream bed around and upstream of the weir.

For sand storage dams, the broad crested weir formula can be approximated as shown in Equation 15-4. This equation can be rearranged to solve for the depth of flow and the discharge can be calculated for various flow depths until a flow depth that matches the design flood flow is determined.

Equation 15-4: $\boldsymbol{Q = 1.78 \times L \times d^{3/2}}$

Where: Q = discharge [m^3/s]
L = length of the overflow section [m]
d = depth of flow over the weir [m] measured far enough up-stream

A factor of safety is prudent to ensure that the design flood Q can be passed within the overflow section. Equation 15-5 provides an estimate for the final depth of the overflow section.

Equation 15-5: $\boldsymbol{D = 1.5d + 0.1}$

Where: D = Final depth of overflow section [m]

15.5 General Design Considerations

The following items should be considered.

15.5.1 Foundation

Sand dam foundations should be on bedrock, in order to avoid seepage under the dam and for stability reasons. In case the rock formation is weathered, this profile should be completely excavated before the dam foundation is constructed. The presence of open fracture zones should also be investigated. If the presence of fracture zones is suspected, the rock surface should be cleaned and simple infiltration tests carried out by pouring water on the cleaned surface. If fractures are discovered, they need to be sealed, and expensive grouting might be required.

15.5.2 Downstream Aprons

Downstream aprons should be considered if there is any possibility of erosion at the downstream toe of the sand dam. Flow velocities at the downstream toe of the dam can be very high and it might be necessary to construct a concrete apron to prevent erosion. In general a 300mm thick mass concrete apron extending across the river bed and extending downstream a distance equal to the height of the sand dam will be sufficient to prevent erosion in the immediate area of the downstream toe.

15.5.3 Wing Walls

As sand dams are most suited to deeply incised water courses, it is often necessary to ensure that the river banks are protected. This can be done by masonry walls along the banks. For most sites wing walls should be constructed for 5m upstream of the dam and 3m downstream of the dam.

Wing walls should extend a sufficient height above the sand dam sill so that large floods are still contained in the river course and do not cause erosion outside of the river channel.

15.5.4 Storage Capacity of the Reservoir

The (total) storage volume of the dam can be estimated from the geometry of the river bed (see Figure 15-4) using the following simplified formulae:

Equation 15-6: $$\boldsymbol{SV = \frac{1}{2} H \times L \times W}$$

Where:

SV = Storage Volume [m^3] **(Note: This is total volume, not water volume)**
H = wall height [m]
L = length of throwback or fetch [m].
L can be measured or calculated as: = [H x 100] / [slope of riverbed (%)]
W = average width of the reservoir [m]

The expected water yield from the dam can be estimated as:

Equation 15-7: $$\boldsymbol{Y = \frac{Sy\ (\%)}{100} \times SV}$$

Where:

Y = yield [m^3]
Sy = Specific Yield (%)
SV = Storage Volume [m^3]

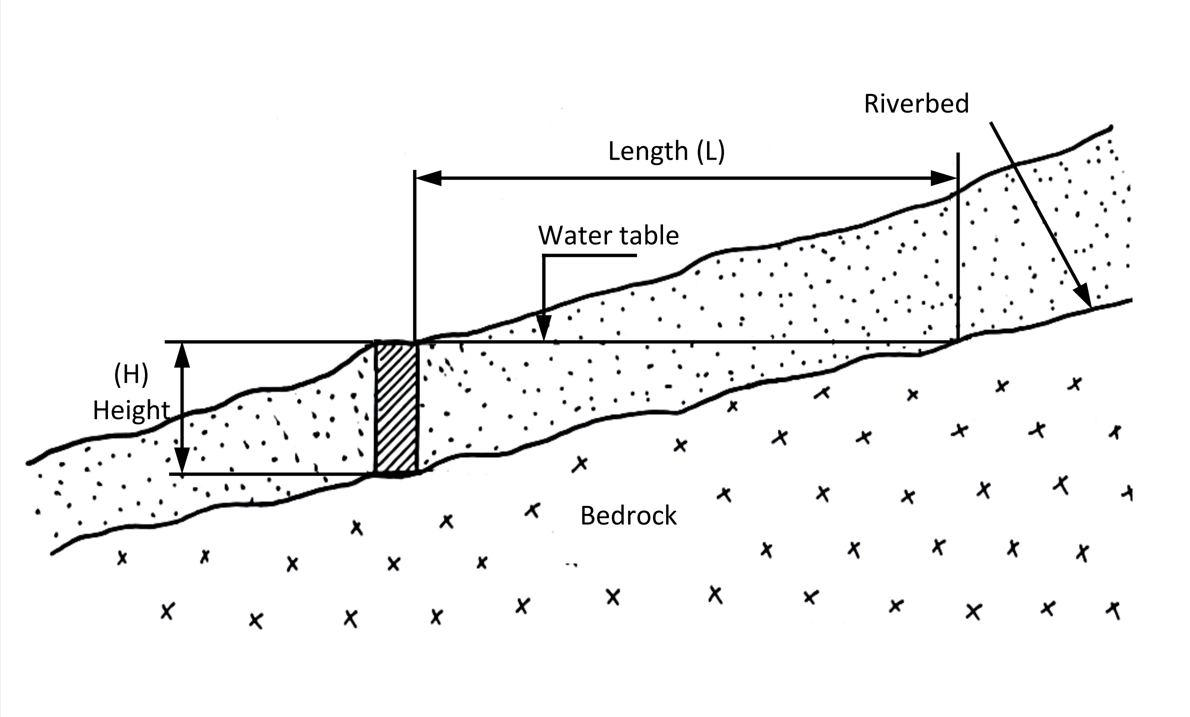

Figure 15-4: Typical Geometry of River Bed for Sand Dam

15.5.5 Types of Walls

The design of the wall for a sand-dam should make use of locally available materials and should meet the following design criteria:

- be sufficiently massive to resist the horizontal pressures exerted on the wall by sand and water;
- provide a water tight barrier against seepage either under or through the wall;
- be sufficiently robust to withstand flood flows over the wall;
- should penetrate sufficiently into the banks with side/wing walls to ensure that flows do not erode the bank and bypass the wall.

The following types of walls are commonly used for sand dams:

- Mass concrete using lean concrete mix;
- Rubble stone masonry wall built with waterproof mortar (1Kg waterproof cement per 50Kg cement).

Gabion walls with artificial lining materials have been used in the past but are no longer recommended.

15.5.6 Dimensioning the Dam

Sand storage dams should be designed for stability against sliding and overturning[2]. Safety factors with respect to sliding (F_S) and over- turning ($F_{O.S}$) should not be less than 1.5.In general, sand dam heights should not exceed 3 to 3.5m without careful design. Sand dam lengths should not exceed 25 to 30m without careful design.

[2]This approach is only valid when the foundation of the dam is on solid rock

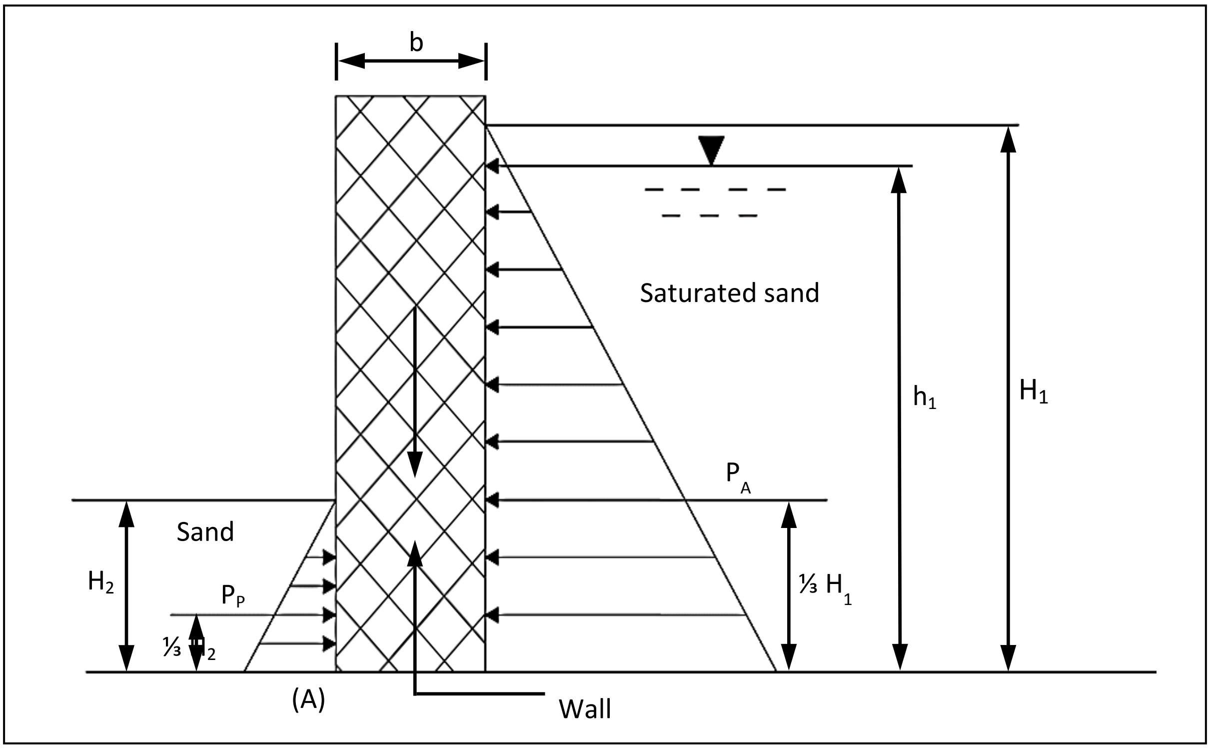

Figure 15-5: Stability Calculations

Figure 15-5 shows an example of a stability computation for a vertical wall. Although the water should not be allowed to rise to the surface for reasons of avoiding pollution, it is assumed that the whole sand layer is saturated (i.e. $H_1 = h_1$). Computation of safety factors requires Equation 15-8 through Equation 15-12.

Equation 15-8:	$P_A = \frac{K_A.w_3.H_1^2}{2}$	**Active earth pressures**
Equation 15-9:	$P_P = \frac{K_P.w_2.H_2^2}{2}$	**Passive earth pressures**
Equation 15-10:	$M_O = \frac{P_A.H_1}{3}$	**Overturning moments**
Equation 15-11:	$M_r = \frac{P_P.H_2}{3} + \frac{b.F_v}{2}$	**Resisting moments**
Equation 15-12:	$F_s = \frac{r.F_v}{F_h}$; $F_{o.s} = \frac{M_o}{M_r}$	**Safety factors**

Where:

K_a ;K_p = coefficients of active and passive earth pressure
$K_A = \frac{1-\sin\partial}{1+\sin\partial}$; $K_P = \frac{1+\sin\partial}{1-\sin\partial}$
δ = angle of friction of the sand [°] (typically 30 - 40°)
w_2 ; w_3= unit weights of unsaturated and saturated sand [kN/m^3][3]
F_v = Weight of wall [kN][4]
F_h= Sum of horizontal forces [kN] = (P_P - P_A)
R = Friction coefficient [-][5]
H_1 ; H_2 ; b = Dimensions [m as indicated on Figure 15-5]

15.5.7 Outlet Works

Outlet works are designed so that water can be drawn from the sand dam in a controlled manner. The main point of having outlet works is to prevent the need for excavation of shallow wells in the sand reservoir area which can lead to human and animal traffic on the sand surface which can lead to pollution of the water in the sand. In addition, shallow wells excavated in the sand reservoir will be destroyed during each flood event. Two alternative arrangements can be considered:

1. Outlet pipe through the wall. This arrangement can consist of a 2 - 3” GI pipe through the wall with a gate valve below the wall. A perforated length placed horizontally at the upstream toe of the wall covered in a ballast surround provides suitable infiltration into the pipe. This option has a number of advantages, including:

    a. Water is passed through the dam wall, reducing traffic on the sand reservoir;
    b. Water is drawn by gravity and can be conveyed to a water point, cattle trough or field for use;
    c. The reservoir can be emptied should the need arise.

    The option also has some disadvantages which include:

    a. The reservoir can be emptied if the flow control is not maintained in good working order;
    b. A pipe through the wall can introduce a line of weakness for seepage;
    c. The pipe can get blocked if the inlet arrangements do not provide sufficient protection from debris entering the pipe.

---

[3] Typical density of dry, unsaturated and saturated sand 15, 18, 21 kN/m^3
[4]Tentative values of the density of construction materials: Concrete(not reinforced)22-24 kN/m^3,(reinforced)23-25 kN/m^3, Stonemasonry(using natural rocks) 24~27 kN/m^3
[5]The following values are suggested for r: r= 0.55 in case of a foundation on solid rock; r = 0.45 in case of a foundation on material with a coarse granulometry, which does not contain silt or clay: r = 0.35 in all other cases.

2. Shallow well with handpump constructed upstream of the wall and adjacent to the river bank. This arrangement may require a graduated sand/gravel infiltration drain (1 x 1 m) placed at the river bed level to connect the sand reservoir to the shallow well. A 100 mm dia heavy gauge perforated PVC pipe can be incorporated into the sand/gravel infiltration drain to enhance the capacity of the drain. This option has the important advantage that the water from the sand dam cannot be accidentally released as is the case with an outlet pipe. This option also has some disadvantages which include:

    a. The handpump requires routine maintenance;
    b. The handpump and shallow well may be exposed to damage from high floods.

15.6 Typical Drawings

Typical drawings for a sand dam include:

- Layout showing site, offtake position and any other relevant features;
- Cross section of the proposed wall showing offtake if relevant;
- Longitudinal profile of the site, showing wall and crest positions and showing any relevant offtake structures.

Appendix B Type Drawing IX - Standard Cross Sections - Concrete Sand Storage Dams, shows the basic construction details for sand-storage dams built in several stages, with lean concrete (1:3:6 mix) as construction material. An upstream cut-off trench (depth depending on the height of the dam) is incorporated in the dam. As shown in the drawing, PVC water-stop profiles have been incorporated in the joints between the various construction stages.

Examples are also given on how the type cross-sections can be used with vertical shuttering, or with shuttering built from stone masonry. The final overflow section should be dimensioned for the expected maximum flood as detailed in Section 15.4.

15.7 BoQs, Specifications and Reporting Requirements

Sample BoQs and specifications for a sand dam are given on the web site.

15.8 Construction of Sand Storage Dams

Sand-storage dams are commonly constructed in stages (see Figure 15-6). The basic idea is to limit the height of each stage in order to keep a sufficiently high water velocity, so that fine particles are washed out while the coarse particles settle. Each stage is typically 0.30 to 0.50 m in height. Each stage should be filled with course sediments before the next layer is added. This is the main reason why the sand dams must be constructed in stages. It is possible to achieve the construction of multiple layers in one rainy season with careful monitoring of the sediment levels.

It is important to ensure that the stages properly adhere to each other once a new level is constructed. This is done by preparing the existing surface (chipping away loose, weathered material) and using a slightly wetter mortar mix in the joint area. The existing surface can be treated with a cement slurry (1:3) in order to facilitate bonding if required.

The method of constructing the dam by periodically adding a new stage means that construction costs will be higher compared to if the dam was constructed to the full height at once. A method which has been successfully used to avoid having to construct the dam in stages is leaving notches in the dam/wall, so that accumulation of sediments is allowed only up to a certain height. The openings are then filled in in stages as the main reservoir fills with sand. This allows the bulk of the construction costs to be incurred at the beginning of the project. If the method of leaving notches is adopted, the notches should be sufficiently wide to prevent the accumulation of fine sediments (silts, clays) behind the section of wall that rises above the notch level.

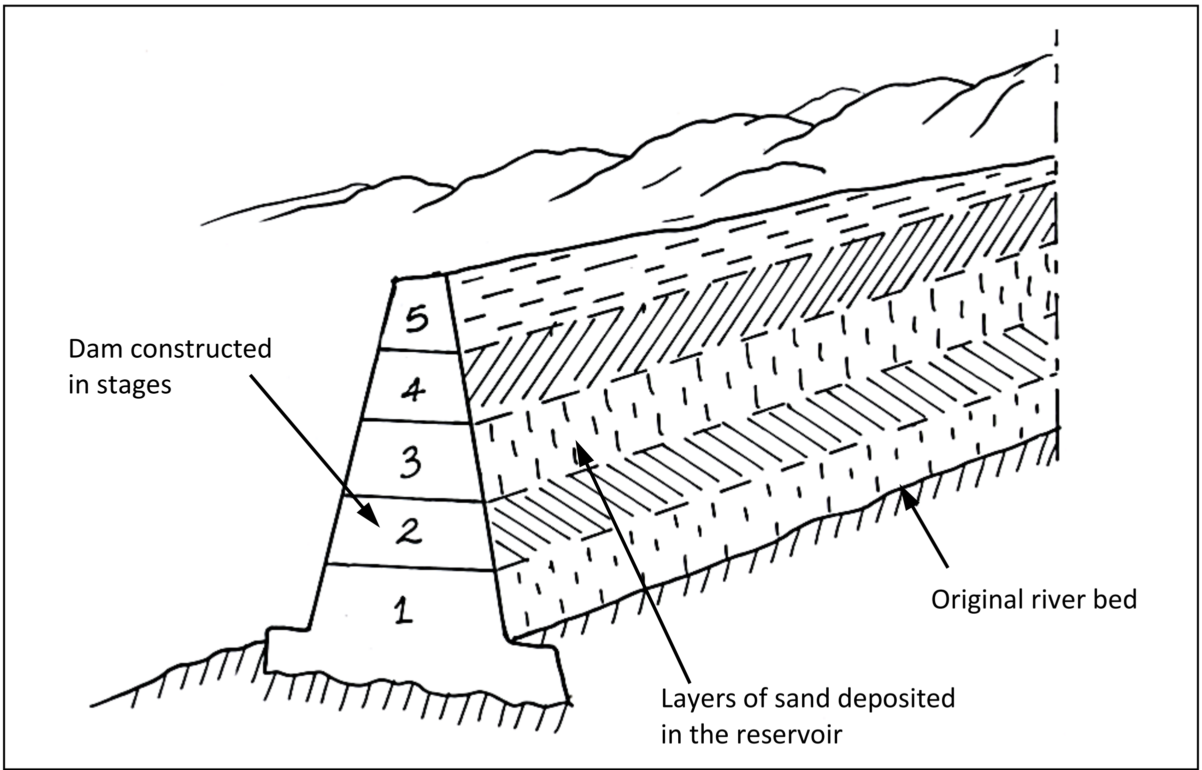

Figure 15-6: Sand Storage Dam Constructed in Stages

A good bond is required between the wall and the foundation rock. In order to achieve this the following steps should be taken:

- All loose rock, dirt, and organic matter should be removed from the rock base;
- The rock surface should be chipped and roughened to remove any loose stone, expose any fissures and to create a key for the wall;
- The rock surface should be swept and cleaned;
- A cement slurry (1:3) should be placed on the rock prior to building the rock surface.

In the event that the rock foundation has steep slopes, it may be advisable to drill and place starter bars into the foundation. This can include the abutments as well. Generally a pattern of 12mm bars placed with 0.5 to 1.0 metre spacing and set 150 to 300 mm into the foundations will be appropriate.

The final wall must protect the river banks from erosion. This can be done by extending the wall into the abutments on either side of the river (5m to 10m on each side may be required) or by constructing wing walls upstream and downstream of the main sand dam. Extending the main wall into the abutments is the preferred erosion protection method as it is generally the least expensive option.

15.8.1 Construction Water Requirements

Water is required for the construction of a sand dam. In the initial stages, it is unlikely that any water will be stored in the reservoir itself so provision must be made for water for mortar/concrete mixing and water for curing concrete and mortar. A small plastic tank can be placed on site to provide temporary water storage during construction. Tanks sizes of 3,000 to 5,000 litres are easily transported on a pickup and can then be filled by a bowser or from existing water sources.

15.9 Equipment

Sand dams can be constructed with manual labour. For larger walls it may be preferable to ensure that a concrete mixer and poker vibrator are available on site. A small pump for dewatering or for pumping water for construction and curing may be beneficial.

15.10 Construction Supervision

Construction supervision can best be carried out by a competent foreman. Engineer visits are usually only necessary during initial site layouts and then periodically to check on progress.

As construction of the complete sand dam wall will usually extend over several rainy seasons, it is preferable to involve local artisans in the construction and to make sure that they can raise the wall level in stages as rains and coarse sand filling allows.

15.11 Operation and Maintenance

Operation and maintenance of sand dams will generally be concerned with ensuring that the top layers of the sand remain coarse and allow water to infiltrate, dealing with erosion or seepage in the area around the dam and with maintaining the offtake structures.

If water infiltration is a concern due to silt/sediment build up on the top sand surface, then the silt/sediment must be removed by raking or digging out. During the next rainy season, coarse sand will backfill where any silt/sediment was removed. Vegetation should not be allowed to grow on the reservoir area.

Erosion or damage to the structure should be monitored and repaired as quickly as possible. Sand bags can be used for temporary fixes, but permanent repairs should be made before the next rainy season. Erosion around the sand dam, reservoir and abutments should also be monitored and repaired as the need arises. This may involve filling in gullies and erosion points with material borrowed upstream or downstream of the structure.

Maintenance of offtake structures may involve digging out the screened inlet areas and removing built up silt. For sand dams equipped with handpumps, spares (U seals, foot valves, handle bushes, etc.) should be kept in stock and used as needed.

15.11.1 Trouble Shooting

Troubleshooting sand dams is fairly straightforward and should cover:

- Outlet problems: For screened outlets, digging out and cleaning screens is the simplest solution. Compressed air can be used to back-flush outlets with some success. For well and handpump outlets, hand pump maintenance and repair should be carried out as needed.
- Sediment problems: When fine sediment build up is problematic, the fine sediments must be manually removed before they have a chance to mix with the coarse sand. If the sand dam is still under construction, consider reducing the stage height for the remaining stages. This will ensure that only coarse sand is captured in each new stage.
- Water quality problems: Poor water quality can be caused by many factors. Pollution by uncontrolled livestock access is a common problem. Chlorination of the sand dam water by digging application pits and applying granular chlorine has proven successful in dealing with biologically polluted water. Chlorine applications must be calculated carefully.
- Height concerns: Sand dams constructed with local artisans should not exceed 3 to 4 meters in height. A series of low dams is often a better alternative than one tall sand dam.

- Construction in layers: Construction must take place in such a way that coarse sand layers are built up. Fine sand and sediments will not store water as efficiently as coarse sand. Determining the best step height or stage height to build for each layer is a matter of judgement, experience and monitoring how sand is deposited. There is considerable trial and error involved.
- Erosion around abutments: Any erosion around the wall can threaten the functional performance of the sand dam and must be remedied properly and urgently. Options include extend the main wall into the abutments or building wing walls along the abutments.

15.12 Rehabilitation

Rehabilitation of sand dams will be required if there are structural problems in the wall or excessive erosion around the structure. It is generally dealt with on a case by case basis.

Rehabilitation of offtake structures (especially wells and hand pumps) may be required at a regular interval.

15.13 NGOs and Capacity Building in Sand Dam Construction

Sand dam construction has been undertaken extensively in the NGO sector as it offers an alternative water storage solutions, and at the same time maintains reasonable water quality while minimising losses through evaporation. A variety of local and international NGOs are actively constructing sand dams in Kenya. Check with county water officers and WRMA officers to see if there are any active programmes in the proposed project area.

Much of the NGO work on sand dams involves building capacity of local artisans. As such, local, trained artisans may be available to assist in proper project implementation.

CHAPTER 16

DESIGN OF SUB-SURFACE DAMS

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16 DESIGN OF SUB-SURFACE DAMS

The sub-surface dam (sometimes referred to as a "groundwater dam") is very similar in nature to the sand dam and so the reader is advised to make reference to Chapter 15 on the Design of Sand Dams. The manual "Sub Surface Dams: a simple, safe and affordable technology for pastoralists" by VSF (VSF & TLDP, 2006) details experiences with sub-surface dams (SSDs) in Turkana. In addition, ASAL Consultants have produced the manual "Sub-surface and Sand-storage Dams". (Nissen-Petersen & Lee, 1990)

The sub-surface dam is distinguished from the sand dam by a number of significant features including the nature of site conditions, outlet options and construction technique. These are elaborated in this chapter.

16.1 Introduction

A sub-surface dam is constructed below ground level and arrests the flow in a natural aquifer, whereas a sand-storage dam impounds water in sediments caused to accumulate by the dam itself.

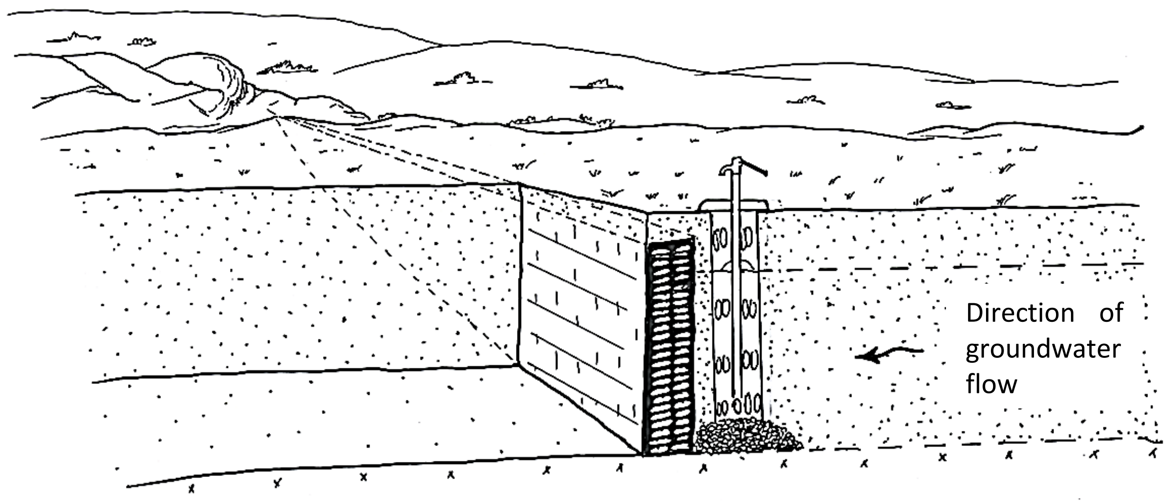

Figure 16-1: Sub Surface Dam

The general principle of a sub-surface dam is shown in Figure 16-1: a trench, reaching down to bedrock, has been dug across a valley, which contains an aquifer consisting of permeable alluvial sediments. An impervious wall has been constructed in this trench which arrests the flow in the aquifer. The impervious wall can be constructed of clay, masonry, gabions (with artificial lining), concrete or a combination of these materials. Excess groundwater will flow above and over the dam crest. In this way, the sub-surface dam does not interfere with flood flows and is not at risk of siltation.

The main advantage of using groundwater dams instead of conventional dams is a considerable reduction of water losses through evaporation, which is particularly interesting in arid and semi-arid areas.

Water stored in sub-surface dams is also less susceptible to pollution and is less likely to generate public health hazards such as breeding grounds for mosquitos.

16.2 Site Selection

Sub-surface dams are only practical in a very narrow range of sites.

16.2.1 Topography/Geology

The topographical/geological conditions for a sub-surface dam are similar to those of a sand dam. The notable exception is that a sand dam is usually placed where the bed rock is exposed or near to the surface and will normally be located in a deeply incised water course.

The sub-surface dam aims to exploit a rock bar hidden beneath a depth of sand or permeable material in a fairly wide water course with poorly defined shallow banks. In general sub-surface dams are most common where the water course has a shallow gradient which results in sandy sediments accumulated along the entire length of the water course.

The location of a suitable rock bar may be identified through analysis of geological formations visible from aerial photography or satellite imagery. Alternatively, the rock bar would need to be identified by probing during field surveys.

Resistivity testing equipment (as used for groundwater exploration) can be used to aid in locating rock bars.

16.2.2 Sediments

The nature of sediments suitable for sub-surface dams is similar to those described in Section 15.2.2 for sand dams. In general river courses containing at least 1m of coarse sand are considered appropriate. Coarse sand has large voids for storage of water from where up to an average of 30 % specific yield can be achieved. Table 16-1 shows specific yield values for a variety of materials that might be found in a sub-surface dam reservoir area. Specific yield is the percent of the reservoir volume that can be used for storing useable water.

Table 16-1: Specific Yields for Various Materials Found in Sub-Surface Reservoirs

	Specific yield %		
Material	**Maximum**	**Minimum**	**Average**
Coarse Gravel	26	12	22
Medium Gravel	26	13	23
Fine Gravel	35	21	25
Gravelly Sand	35	20	25
Coarse Sand	35	20	27
Medium Sand	32	15	26
Fine Sand	28	10	21
Silt	19	3	18
Sand Clay	12	3	7
Clay	5	0	2

(Source: Johnson (1967) as quoted by CW Fetter)

The deposition character of sediment influences construction of sub-surface dams along water courses. Sediments originate from parent rocks in the catchment area through weathering and erosion. Coarse sand and gravel particles are desirable in the reservoir. The most favourable rocks are granite, quartzite and sand stone, but also dams constructed in gneiss and mica-schist areas have been successful. Areas underlain by basalt and rhyolite tend to be less favourable however. Riverbeds with salty rocks or saline water should not be considered for sub-surface dams.

Catchments with shallow slopes and little vegetative cover are usually favourable for the construction of sub-surface dams.

16.2.3 Traditional Water Holes in Riverbeds and Riverine Vegetation

Sections of river beds with seasonal or perennial waterholes are a good pointer to the presence of natural dykes in the immediate downstream area. Such holes are reliable water sources during the dry season and settlements can often be seen in such areas. In addition, sections of river beds suitable for sub-surface dams can also be identified by observing water-indicating vegetation (see Table 16-2).

Table 16-2: Typical Tree Root Depths

Botanical name	Depth to water-level
Cyperus rotundas	3 m to 7 m
Delonix elata	5 m to 10 m
Grewia	7 m to 10 m
Markhamia hildebranditi	8 m to 15 m
Hyphaene thebacia	9 m to 15 m
Borassus flabellierfer	9 m to 15 m
Ficus walkefieldii	9 m to 15 m
Ficus natalensis	9 m to 15 m
Ficus malatocapra	9 m to 15 m
Gelia aethiopica	9 m to 20 m
Piptadenia hildebranditi	9 m to 20 m
Acacia seyal	9 m to 20 m

16.3 Site Investigations

Detailed site investigations are essential to the identification, assessment and selection of suitable sites for sub-surface dams. Key issues to be addressed during site investigations include:

- Presence of firm water tight bedrock formation on which to position the wall;
- Presence of suitable alluvial sediments with high porosity and yield characteristics;
- Water tightness of the water course;
- Water level within the alluvial sediments;
- Access to site for construction.

16.3.1 Desk Study

Aerial photos and satellite imagery, combined with geological maps and reports can help to identify geological formations that may yield suitable sites for sub-surface dams.

16.3.2 Field Reconnaissance

An essential part of the field reconnaissance for sub-surface dams is to establish the cross section and longitudinal profile of impermeable layers along the targeted length of the water course. These topographical features cannot be readily seen and so must be established through topographical survey and probing techniques.

The presence of dykes can be confirmed either by digging trial pits or probing with iron rods hammered into the sand or by use of VES equipment. Alternatively, having a hybrid by combining either two of the methods can improve reliability of the results.

Probing centreline of Lagga: Probing helps to determine the sub-surface profile of the *lagga*. In an area showing potential for water storage in the sand river, probing is done to try and identify possible natural dykes. Probing is usually done downstream of existing traditional scooped wells in the riverbed as they indicate a sub-surface obstruction that causes water to be retained in the riverbed. Such a sub-surface dyke would form the basis for a sub-surface dam, thus raising the level of water in the sub-surface sand storage reservoir. Probing is also done upstream to find the deepest points in the sub-surface sand reservoir and to determine the extent of storage. Water from the sub-surface reservoir is ideally extracted from the deepest points. Thus, the investigation of a site starts with probing along the centreline over a length of roughly 400 m (200m downstream; 200m upstream of the point identified). The topographical survey of the ground levels of the area being probed is also done to show the surface topography of the area as well as the levels of the riverbanks and surrounding areas.

Probing sections across river for potential dykes and galleries: Where analysis indicates potential sites for a sub-surface dam, probing is done on the section across the riverbed. This is to follow the possible dyke and to determine the dimensions of the sub-surface dam to be designed. Probing is also done across the deep points identified where galleries may be built to extract the water from the riverbed. With potentially large volumes of water stored in the sandy riverbeds, there is a possibility for identifying sections for more than one gallery and/or shallow well for each sub-surface dam.

Confirmation probing: Confirmation probing is done in close proximity to the sections identified for construction of sub-surface dams and galleries. This is done 5m to 10m upstream; and 5m to 10m below the initial section probed. Analysis of the results obtained from the confirmation probing enables the selection of the optimum site for the sub-surface dam; it also enables the selection of the most appropriate site for the gallery and shallow well.

After determining the presence of sub-surface dykes or impermeable layers, test pits should be excavated along the targeted length of the water course to establish the homogeneity of the soil profile across and along the water course.

If there is any doubt about the impermeability of the bedrock foundations for the dam, the foundation area should be exposed and evaluated for cracks and weathering. An impermeable foundation is required.

Aquifer material should be evaluated. Test pits should evaluate whether the aquifer is homogeneous or layered. A homogeneous coarse aquifer is ideal. Deep deposits of fine sand and silt indicate marginal sites. Samples from the aquifer should be taken and submitted to a laboratory for the following tests:

- Determination of porosity (the ratio of the volume of voids to the total volume of the aquifer);
- Determination of specific yield (the quantity of water that can be extracted from a saturated sand volume, expressed as a percentage of the total volume);
- Particle size analysis.

The tests above can also be carried out in the field with a minimum amount of specialized equipment.

The availability of suitable local materials should be established during the field investigations. This will include the availability of:

- Clay or clayey soil for the wall;

- Rubble stones;
- Water.

A survey of any other materials which can be used during the construction of the dam should also be carried out during the field reconnaissance (nearest source of cement, ballast, formwork, secure storage area, etc.).

16.3.3 Laboratory Soil Tests

Figure 16-2 shows the test cylinder used for measuring porosity and specific yield. Porosity is determined as follows: A test cylinder (capacity > 2000 ml) is filled with 1000 ml of water, and sand is added until the sand level reaches the 2000 ml mark. The water level will be higher than the 2000 ml mark. Water is drained from the test cylinder to adjust the water level to the 2000 m1 mark. The volume of the drained water is recorded.

The porosity (%) is calculated as follows:

Equation 16-1: **P = (V/T)*100**

Where: P = Porosity [%] (typically 20 – 35%)
V=Volume of water remaining in the test cylinder [ml] = (1000ml-$V_{drained}$)
T= 2000 [ml]

To avoid air voids in the test cylinder, it is essential to put the sand sample into the water and not the other way round.

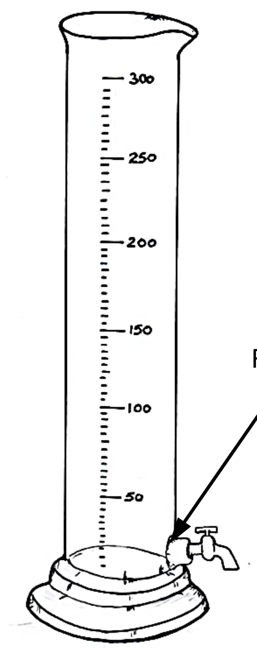

Figure 16-2: Test Cylinder for Measuring Porosity

To determine the specific yield, the test cylinder (capacity >2000 ml) is filled with 1000 ml of water and sand is added until the sand level reaches the 2000 ml mark. The water level will be higher than the 2000ml mark. Water is drained from the test cylinder to adjust the water level to the 2000 m1 mark. This water is discarded. The test cylinder is now drained for 30 minutes and the volume of water released during the 30 minutes is recorded. From this the specific yield is determined as follows:

Equation 16-2: $$S_y = (V/T) \times 100$$

Where:
Sy = Specific Yield [%]
V = Volume of water drained over 30 minutes [ml]
T = 2000 [ml]

A particle size analysis can be carried out as described in Section 9.4.4.

16.4 Hydrogeology

The most favourable type of aquifer for sub-surface dams is surfacial river beds made up of sand and gravel. In situ weathered layers, and deeper alluvial aquifers have also been dammed with success, although such aquifers generally have smaller storage capacities.

Sub-surface dams are very sensitive to permeability values. Difficulties which could be encountered when a groundwater dam is constructed in an aquifer with fine-grained material are small available water storage volumes and difficult water extraction possibilities, due to low permeability of the material.

Improving the permeability of the aquifer by water jetting the ground upstream of the dam is not economically feasible for the type of water supply under consideration. The same can be said concerning the construction of groundwater dams in fractured hard-rock aquifers using grout curtains.

Groundwater dams are, mainly for reasons concerning the excavation techniques used, best suited for shallow aquifers.

16.5 Hydrology

Although sub-surface dams are filled by and rely on groundwater flows, for dams located in shallow aquifers, an estimate of available water can be made from surface hydrology assumptions. Hydrological calculations can be carried out to determine the annual volume of water available for storage and the expected maximum flood at the site. Hydrological calculations are described in detail in Chapter 8 and simple calculations are briefly described below.

A quick estimation of the water available for storage in a sub-surface dam can be made using a 5% infiltration factor (experience shows 2 to 22 percent as an acceptable range for shallow aquifers), the catchment area and annual rainfall as shown in Equation 16-3. This assumes that the catchment area for the shallow aquifer is the same as the catchment area for the surface catchment.

Equation 16-3: $$V = 0.05 * A * R$$

Where
V = yearly volume available for storage [m^3]
A = catchment area [m^2]
R = annual rainfall [m]

16.6 General Design Considerations

Flood events, even on water courses with shallow gradients, can be turbulent, and can result in the movement of a significant volume of sediments. The sub-surface dam wall is contained primarily within the body of the sediments which implies that the wall design should be based on hydraulic loading only. However, if there is concern that the wall is likely to be exposed to turbulent flows and shifting sediments then the design should consider stability against sliding and overturning (see Section 15.5.6).

Sub-surface dam foundations should be solid bedrock impervious clay or a consolidated murram formation and investigated as outlined in Section 15.5.1.

The (total) storage volume of the dam can be estimated from the geometry of the river bed as discussed in Section 15.5.4 from which the volume of useable water can be estimated using specific yield figures.

In general sub-surface dams are less than 5m deep (below surface level) and less than 50m long.

16.6.1 Types of Walls

The different types of walls which can used for sub-surface dams include:

- Compacted impervious (clayey) soil, with a minimum crest width of 3 m and side slopes of 0.75 (h) : 1 (v). Clay walls have a risk of erosion from groundwater flows. This is especially true if the clay is not fully compacted. In addition, clay walls have a risk of cracking and so should only be used for shallow sub-surface dams;
- Rubble stone masonry wall built with waterproof mortar;
- Mass or reinforced concrete.

The key advantage of using masonry or concrete is that they allow the option to raise the wall above the current river level and turn the reservoir into a hybrid sub surface/sand dam.

Other types of walls that could be considered but are not in common use in Kenya are:

- Ferrocement dams;
- Brick wall;
- Plastic sheet;
- Steel sheet;
- Injection screen.

The use of plastic (HDPE or LDPE) lining material in the construction of sub-surface dams offers the possibility of significant cost savings over traditional methods. At present lining material still requires specialized installation tools and lining materials are prone to damage during installation.

16.6.2 Draw-off Works

A sub-surface dam does not usually lend itself to a gravity piped offtake through the wall. In general, a shallow well, placed upstream of the wall and adjacent to the river bed is the most suitable draw off arrangement.

A graduated sand/gravel drain or infiltration gallery connecting the aquifer upstream of the wall to the shallow well may be required. This will depend on the permeability of the river banks.

The paragraphs below describe the construction of an infiltration gallery and shallow well for use with a sub-surface dam.

Infiltration Gallery: Extraction of water from the sub-surface dam can be via an infiltration gallery laid from the deepest section of the sand reservoir saturated with water, towards the nearest river bank with a shallow well. The excavation for the gallery includes removal of all sand, silts, clay, rocks and other materials as well as the control of the high water table, which may rise in the excavated trench.

The infiltration gallery is constructed of 4 inch heavy gauge PVC piping system with holes punched around the pipe surrounded by gravel with size not exceeding 1.5". After laying the piping system, gravel with size not exceeding 1.5" is placed around the pipe and up to a level of 10cm above the pipe. A plastic liner with thickness of at least 0.3 mm is then laid around the gravel pack to prevent sand filling the spaces between the gravel particles. The trench will be backfilled with coarse sand placed in layers above the infiltration gallery.

Shallow well: The shallow well with an external diameter of 1.55m; internal diameter of 1.25m, will be made of culverts or concrete blocks. Manual excavation will be done at the inside of the ring thus lowering it as a caisson to the required depth. Permeable concrete building blocks or culverts are used up to the elevation of the maximum water level in the sub-surface reservoir. Above this elevation, regular building concrete blocks, or precast culvert rings are to be used.

The cast in-situ or caisson lining shall be continued above ground level to form a head wall. The height of this headwall shall be a minimum of 60cm above the original ground level.

A hole will be excavated in the lower concrete blocks or culverts to enable connecting the permeable PVC pipes to the shallow well. These will be connected at the bottom of the shallow well excavation.

16.7 Typical Drawings

Typical drawings for a sub-surface dam include:

- Layout showing site, offtake position and any other relevant features. As the dam will be covered, it is important that the drawings show accurately where it is positioned and any other relevant details;
- Cross section of the proposed wall detailing final dimensions and materials;
- Longitudinal profile of the site, showing wall and crest positions and showing any relevant offtake structures.

16.8 BoQs, Specifications and Reporting Requirements

Sample BoQs and specifications for a sub-surface dam are given on the web site.

16.9 Construction of Sub-Surface Dams

Ideally, the construction of sub-surface dams should be carried out at the end of the dry season, when there is little water in the aquifer. Flow occurring in the river will have to be pumped out.

The preparation of the foundation should follow the description in Section 15.8 in order to ensure that a secure water tight bond is created between the wall and the foundation bedrock.

Where a clay soil material is being used to build the wall, the soil should be compacted at optimum moisture content (OMC). The excavation and re-filling of the trench can be carried out by labour intensive methods since the depths involved are usually limited (< 5m).

Figure 16-3 shows recommended minimum dimensions for sub-surface dams. Sufficient attention should be paid to compaction especially when manual labour is used, and to the contact zone between the dam and bedrock. The use of a key in the rock is recommended.

While sub surface dams have been successfully made with thinner crests and steeper slopes, the dimensions in Figure 16-3 will result in a more stable and robust structure.

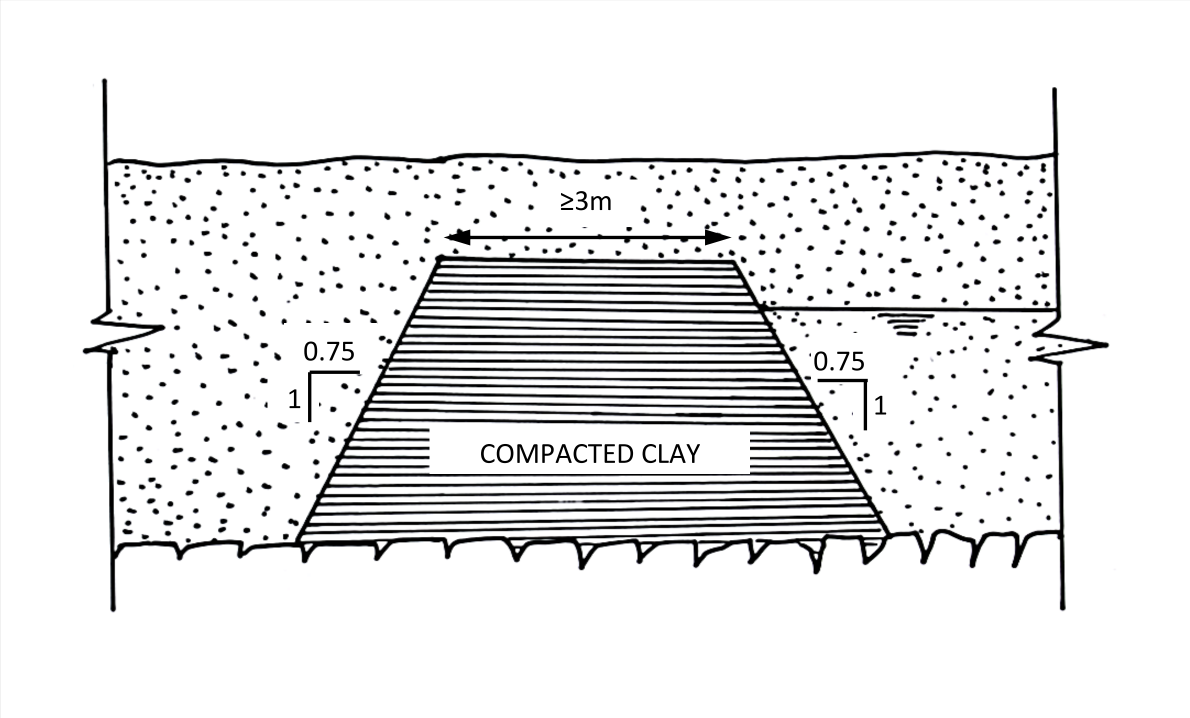

Figure 16-3: Minimum Dimensions for Sub Surface Dams

After completing the excavation, the backfilling should be carried out in stages of around one metre. After filling and compacting the clay dyke, the remaining gap between dam and excavation is backfilled with lightly compacted sand at every stage (see Figure 16-4).

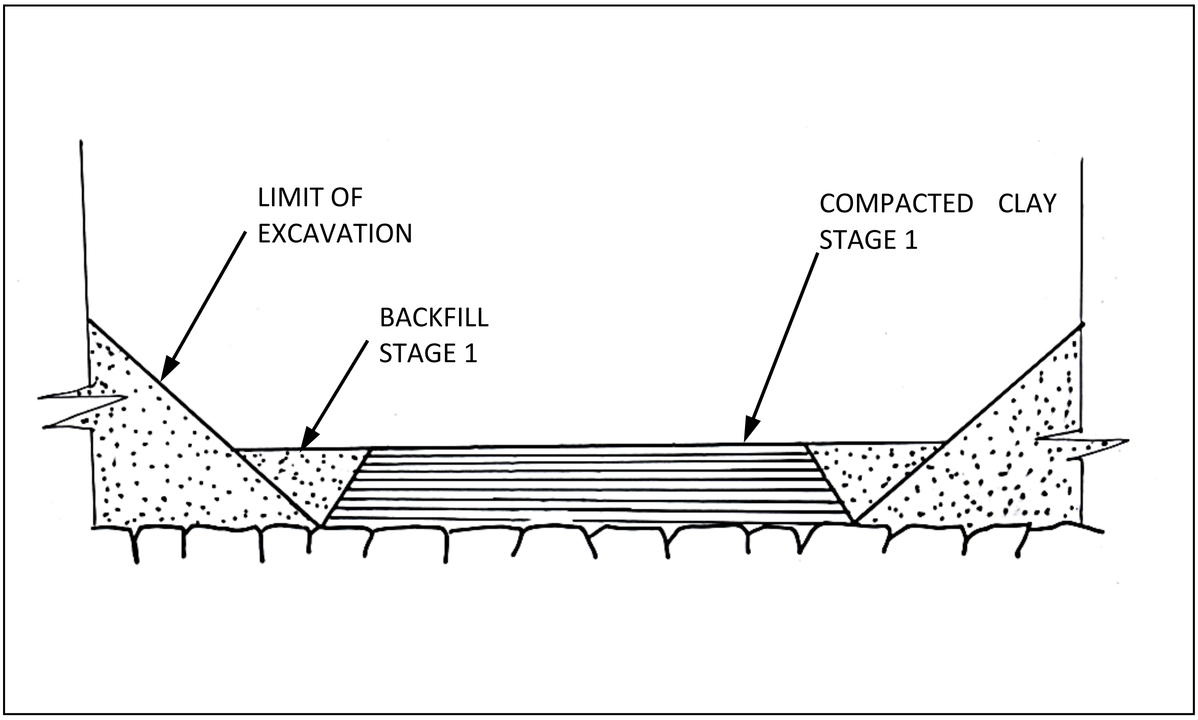

Figure 16-4: Construction Sequence for Sub-Surface Dams

Where a masonry or mass concrete wall is to be used, the centre line of the wall should be excavated to a width of 2 meters. The excavation should extend 0.3 to 0.6m into the foundation material and at least 5 to 10 meters into the banks. The wall can be constructed in masonry, rubble stone or concrete. The upstream face should be plastered with 50mm of 1:3 mortar (with water proof cement added at 1kg per 50kg back of cement). The sill of the wall should be brought to the riverbed level. When completed, the upstream and downstream spaces should be backfilled with well watered and compacted material.

16.10 Equipment

Manual labour is usually sufficient for the construction of sub-surface dams. For masonry or mass concrete walls a concrete mixer, poker vibrator and good wheelbarrows will be required.

16.11 Construction Supervision

Construction supervision is essential. The final wall design, alignment and height will depend on what is revealed during the excavations. Having a trained artisan or experienced engineer on site throughout construction will allow the final wall to be tailored to any unforeseen issues that arise.

16.12 Operation and Maintenance

Operation and maintenance of sub-surface dams will generally be concerned with dealing with erosion or seepage in the area around the dam and with maintaining the offtake structures.

Erosion or damage to the structure should be monitored and repaired as quickly as possible. Sand bags can be used for temporary fixes, but permanent repairs should be made before the next rainy season.

Maintenance of offtake structures may involve digging out the infiltration gallery and removing built up silt. For sub-surface dams equipped with hand pumps, hand pump spares (U seals, foot valves, handle bushes, etc) should be kept in stock and used as needed.

16.12.1 Trouble Shooting

Troubleshooting sub-surface dams is fairly straightforward and should cover:

- Outlet problems: For well and handpump outlets, hand pump maintenance and repair should be carried out as needed.
- Water quality problems: Poor water quality can be caused by many factors. Pollution by uncontrolled livestock access is a common problem. Chlorination of the sub-surface dam water by digging application pits and applying granular chlorine has proven successful in dealing with biologically polluted water. Chlorine applications must be calculated carefully.

16.13 Rehabilitation

Rehabilitation of sub-surface dams will concentrate mainly on the offtake structures. Cracks or leakage in the wall will require significant excavations for repairs to be carried out.

CHAPTER 17

DESIGN OF ROCK CATCHMENTS

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17 DESIGN OF ROCK CATCHMENTS

Rock catchments collect water from rock faces and either store the water on the rock face or pipe it to storage tanks located elsewhere. The manual "Manual Number 3 Rock Catchment Dam with Self Closing Water Tap" (Nissen-Petersen & Lee, 1990) provides further reading on design and construction of rock catchments.

17.1 Introduction

Rock catchment dams consist of a concrete or masonry gravity wall constructed around a depression in a rock-surface, so as to retain the water running off this surface. The gravity walls can be any shape or height, and reservoirs of considerable capacity have been created this way. The height of the walls can be either determined by the topography of the dam site or by the expected run-off from the catchment. Gutters or training walls to extend the natural catchment area and to direct runoff along the rock catchment face are often essential to the operation of the rock catchment.

Rock-catchment dams are subject to the same limitations regarding evaporation as earth dams and pans. In order to reduce the evaporation from the reservoir, in certain cases, parts of rock catchment reservoirs have been shaded using a variety of materials including synthetic shade netting. This will not completely eliminate the evaporation but might reduce it by 30 to 40 %.

In order to reduce evaporation and preserve water quality, many rock catchments are now designed to drain into covered tanks.

Due to the nature and size of the catchment areas involved, sedimentation will usually not be problematic in terms of storage capacity reduction. Further reduction of the quantity of silt entering the reservoir can be obtained by fencing off the catchment area (e.g. planting sisal), and by removing soil and vegetation from the catchment before every rainy season. Silt should also be removed from within the reservoir before every rainy season.

17.2 Site Selection

Rock outcrops, presenting a depression or dip, which can be transformed into a reservoir by building either a single (straight) gravity wall, several straight sections of gravity wall or a V or U-shaped gravity wall are suitable sites for rock-catchment dams. Figure 17-1 shows an example where two sections of gravity wall are used to constitute the reservoir. (Sometimes, these dips are filled with soil and need to be emptied first!). No training walls have been used to extend the catchment area. All water is stored on the rock face.

In most cases, even a rock face without any significant natural depressions can be developed with a U shaped gravity wall to allow water collection and storage. Figure 17-2 shows a rock catchment dam on a rock face with a small depression. The proposed wall has been sized to create just enough storage for the expected runoff from a 110mm storm with 80% runoff. Training walls have been used to maximise the catchment area. A pipeline has been provided to allow water to drain (through a sand filter) into a closed tank located some distance away from the catchment. Ideally no water is stored on the rock face.

Choice of a suitable rock face can be confusing. Rock outcrops with vast vertical faces are not suitable for rock catchments as their actual catchment area can be very small. Vertical faces do not collect runoff. Horizontal rock outcrops or outcrops with uniform sloped faces are preferable as they offer the best catchment areas and produce the most runoff.

Rock faces can, in rare instances, be permeable and if possible a quick check can be carried out by pouring water on the rock face and evaluating the runoff. Permeable rock faces cannot be used for rock catchments.

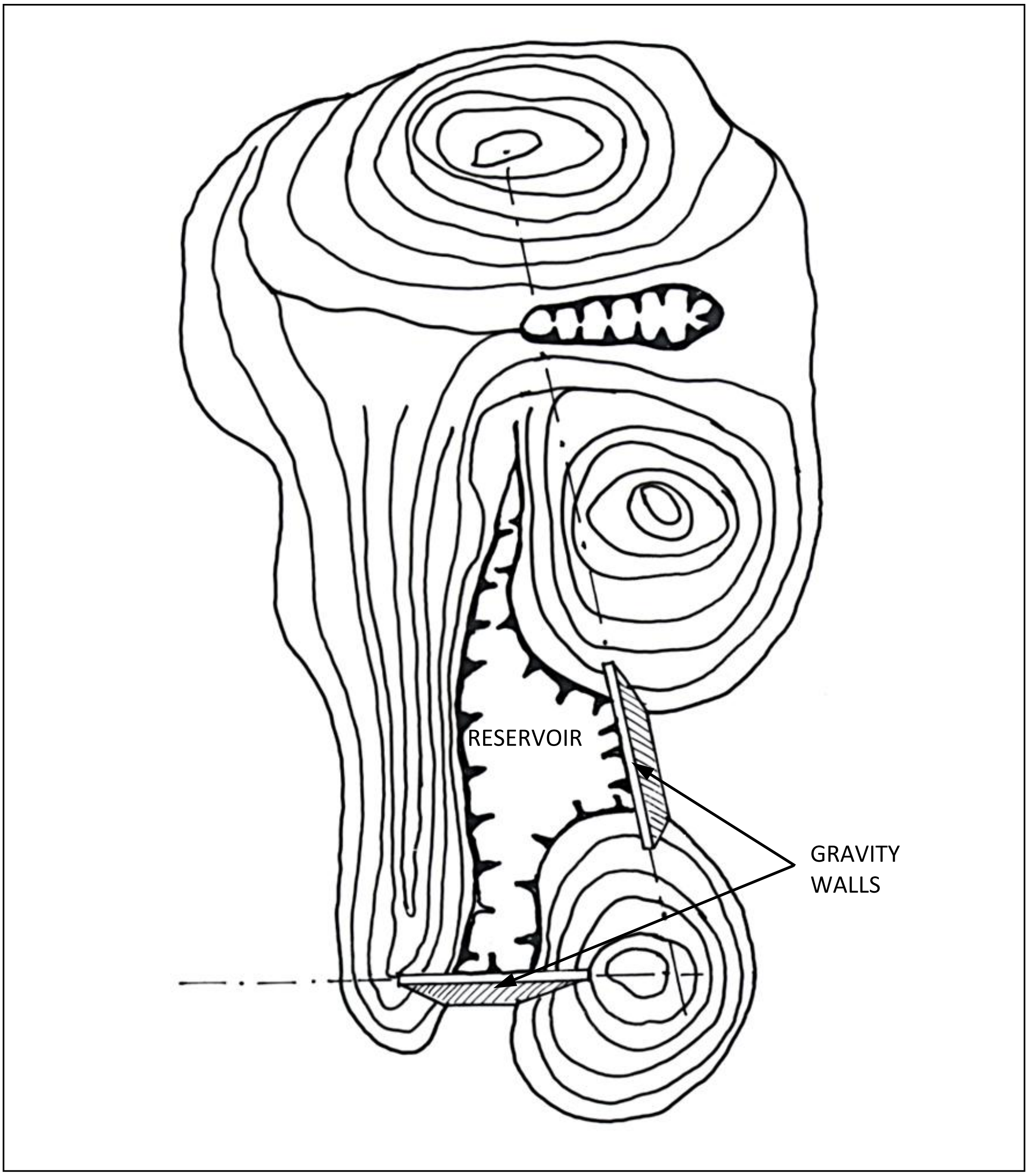

Figure 17-1: Rock Catchment Reservoir on Rock with Depressions

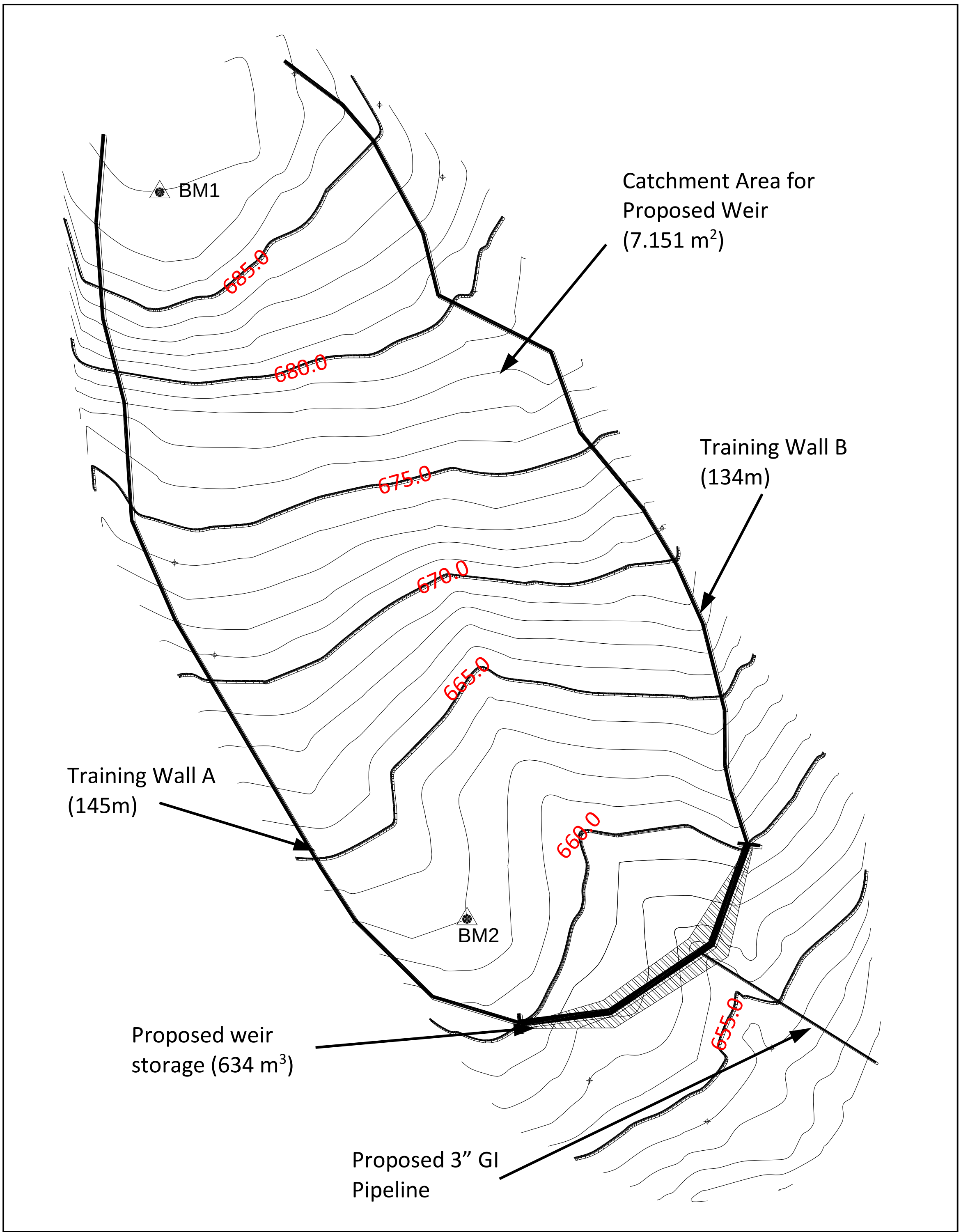

Figure 17-2: Rock Catchment Reservoir on Uniformly Sloped Rock

17.3 Site Investigations

Site investigations should evaluate the extent of the rock face and examine suitable areas for wall construction. If possible judgements should be made as to how hard the rock surface is to work. If possible a topographic survey of the rock face should be carried out as the resultant contour map will allow accurate calculations with regards to gutter placement and length, gravity wall placement, alignment and length and outlet works placement and pipe requirements (See Figure 17-2).

Site investigations should also examine site access, paying special attention to how sand, cement and reinforcement steel can be brought to the site.

17.4 Hydrology

The hydrology for a rock catchment is very straightforward but it depends slightly on the type of rock catchment to be constructed. For a rock catchment where water is to be stored on the rock face, the hydrological calculations should estimate the annual runoff to be collected. For a rock catchment where the water is to be piped to separate storage, the hydrological calculations should estimate an expected maximum runoff volume from a single storm.

In either case, a quick estimation of the runoff can be made using an 80% runoff factor, the catchment area and the expected rainfall as shown in Equation 17-1.

Equation 17-1: $$\boldsymbol{V = 0.8 * A * R}$$

Where V = volume available for storage [m^3]
A = catchment area [m^2]
R = expected rainfall [m]

For an annual estimate, the expected rainfall can be the average annual rainfall in meters. For a single storm estimate, the expected rainfall can be the expected storm rainfall in metres. Within Kenya, rainfall intensity maps for a variety of duration and recurrence intervals are available (See Appendix A). If information is not available, then a single storm estimate can be assumed. In most cases an estimated maximum storm in the 100mm to 150mm range can be used but it should be noted that it is an assumed storm value.

17.5 General Design Considerations

The following items should be considered in the design.

17.5.1 Approach

Essentially there are two basic approaches to the design of rock catchment dams:

(a) **Gravity wall provides storage reservoir**. The advantage of this arrangement is that there are no further costs related to building storage. One disadvantage of this system is that any seepage through the wall affects the reliability of the water source. Another disadvantage is that the water can become contaminated as it is accessible to both human and wildlife pollution and sunlight which enables algae growth;

(b) **Gravity wall provides temporary flood storage reservoir**. In this arrangement the long term water storage is provided by storage tanks situated below and separate to the gravity wall. The advantage of this option is that the gravity wall is only designed to provide storage sufficient for a single design runoff event and the tanks, which need to be water tight and controlled, can be placed at sites which are more suitable for construction and access. The water from this arrangement tends to retain better quality especially if the storage tanks are covered (this also reduces evaporation) and a basic graduated sand/gravel filter is incorporated within the rock catchment offtake works. If the long term runoff volume exceeds the storage tank volumes then additional storage tanks can be added as budget allows.

The selection of design approach is a function of:

- Site conditions: Sites with no pronounced depressions are most suitable to approach (b).
- Nature of water demand: Water for domestic use is most suited to approach (b), water for livestock or wildlife is suited to approach (a).

- Likelihood of contamination of storage reservoir: If contamination appears likely, approach (b) provides better quality water.
- Budget: Approach (b) has higher costs as separate storage tanks are constructed as needed. It does allow the construction costs to be spread over a longer period as all storage tanks do not need to be constructed at one time.

17.5.2 Storage Capacity

In cases where the gravity wall is intended to provide the storage reservoir, the rock-catchment will normally be developed to its maximum capacity. Otherwise the size of the reservoir is to be determined as a function of the water demand and required storage reliability as explained in Sections 3.3 and 8.12.

In order to estimate whether the reservoir can be filled by the expected run-off it is necessary to compute the run-off volume to be expected from the catchment, For the purpose of computing the expected run-off a run-off factor (Kr) of 0.8 is usually appropriate for small rock catchments. The catchment area can in many cases be slightly enlarged by the construction of stone masonry gutters[1]. The possibilities offered by the construction of such gutters depend largely on the topography of the dam site (see Figure 17-3). The gutters are built of flat stones set in mortar as shown on Figure 17-4. The gutter slope should preferably not exceed 2 to 3 % in order to keep flow velocities within acceptable ranges.

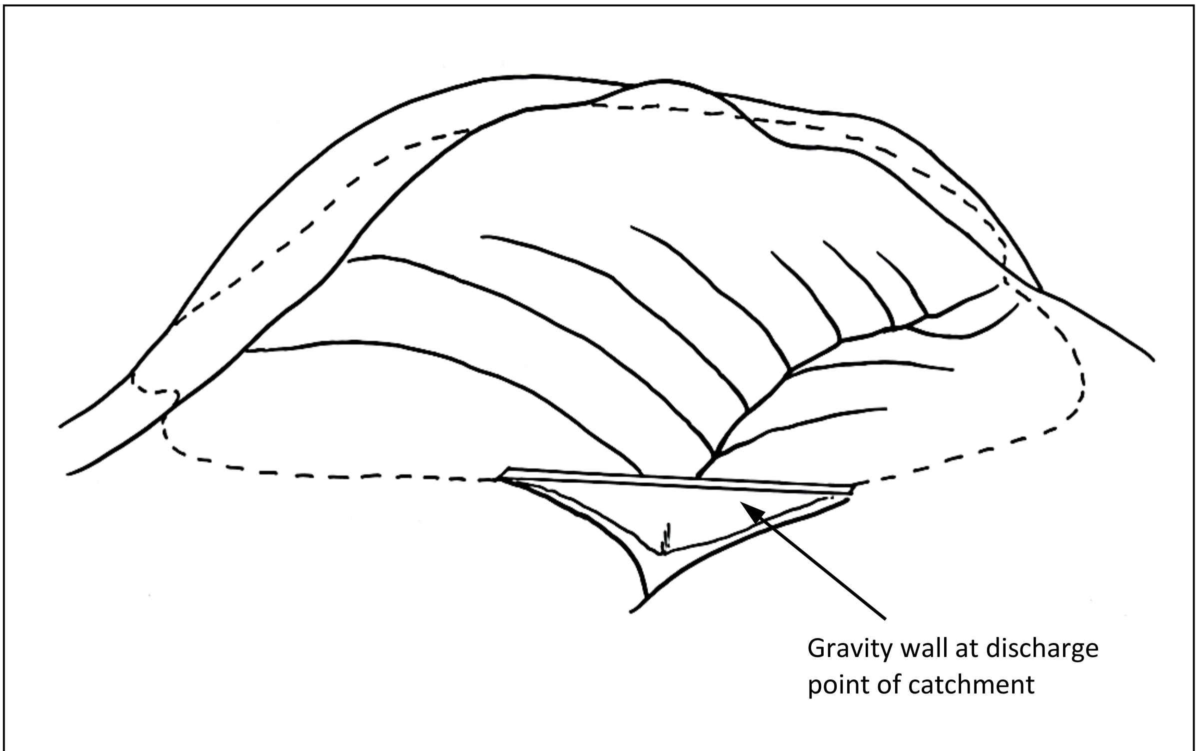

Figure 17-3: Layout of Gutters to Enlarge a Catchment Area.

[1]See Nissen-Peterson E. and Dr. Michael Lee (1990): "Harvesting Rainwater in semi-arid Africa - Rock Catchment Dam with Self Closing Water tap"-ASAL Rainwater Harvesting, Nairobi.

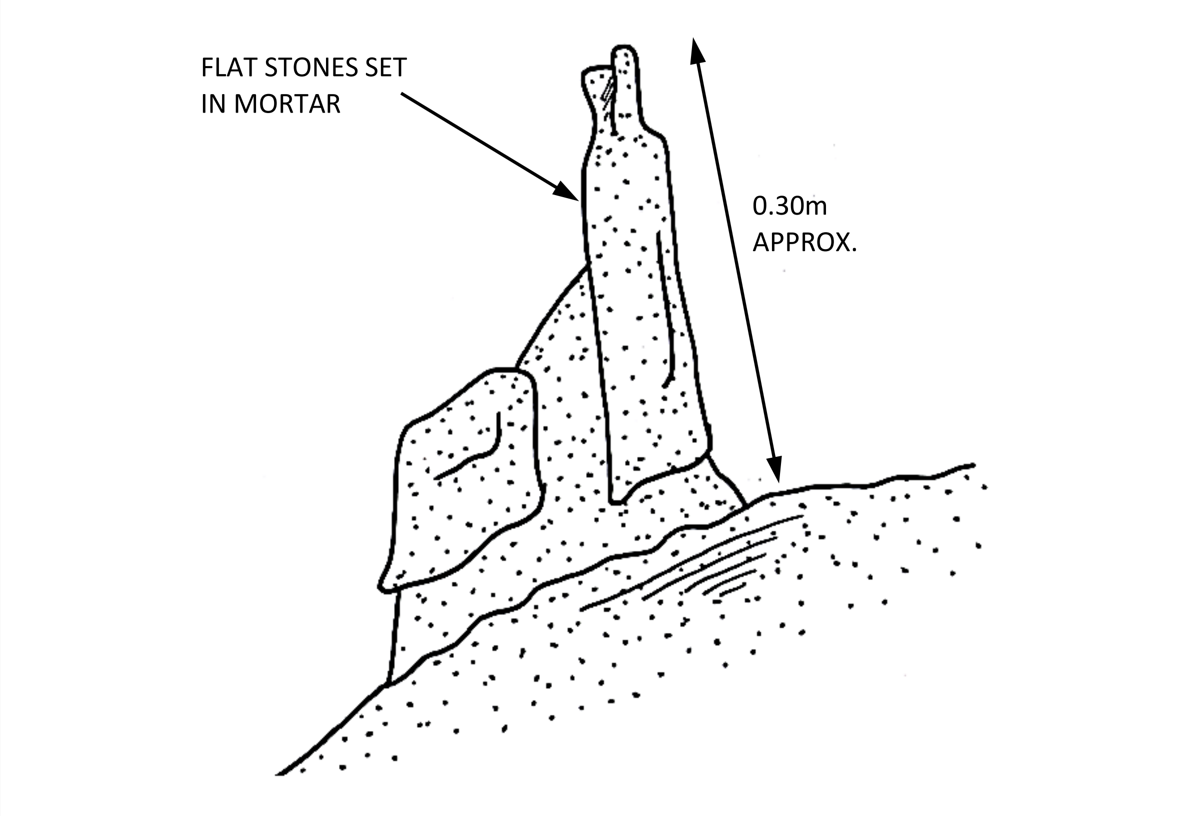

Figure 17-4: Cross Section of Typical Gutters

17.5.3 Design of the Gravity Wall

Gravity walls will be built from mass concrete (1:2:4 mix, not reinforced) or from stone masonry using blocks obtained from natural rock or from rubble stone masonry (hard rock is preferable). The foundation of the wall should be on rock, and the foundation area should be either horizontal or slightly sloping towards the reservoir. It is preferable to anchor the wall at least 0.50 m into the foundation (see Figure 17-5). In order to ensure the water-tightness of the reservoir, special attention should be paid to the contact zone between the rock and the wall. When a masonry wall is used, it should be plastered (20--30mm) with waterproof cement mortar on the reservoir side.

Wall dimensions should follow typical ALDEV dimensions (see Section 13.4). Wall height should be limited to 3.5m and wall length without buttresses or corners should be limited to 30m.

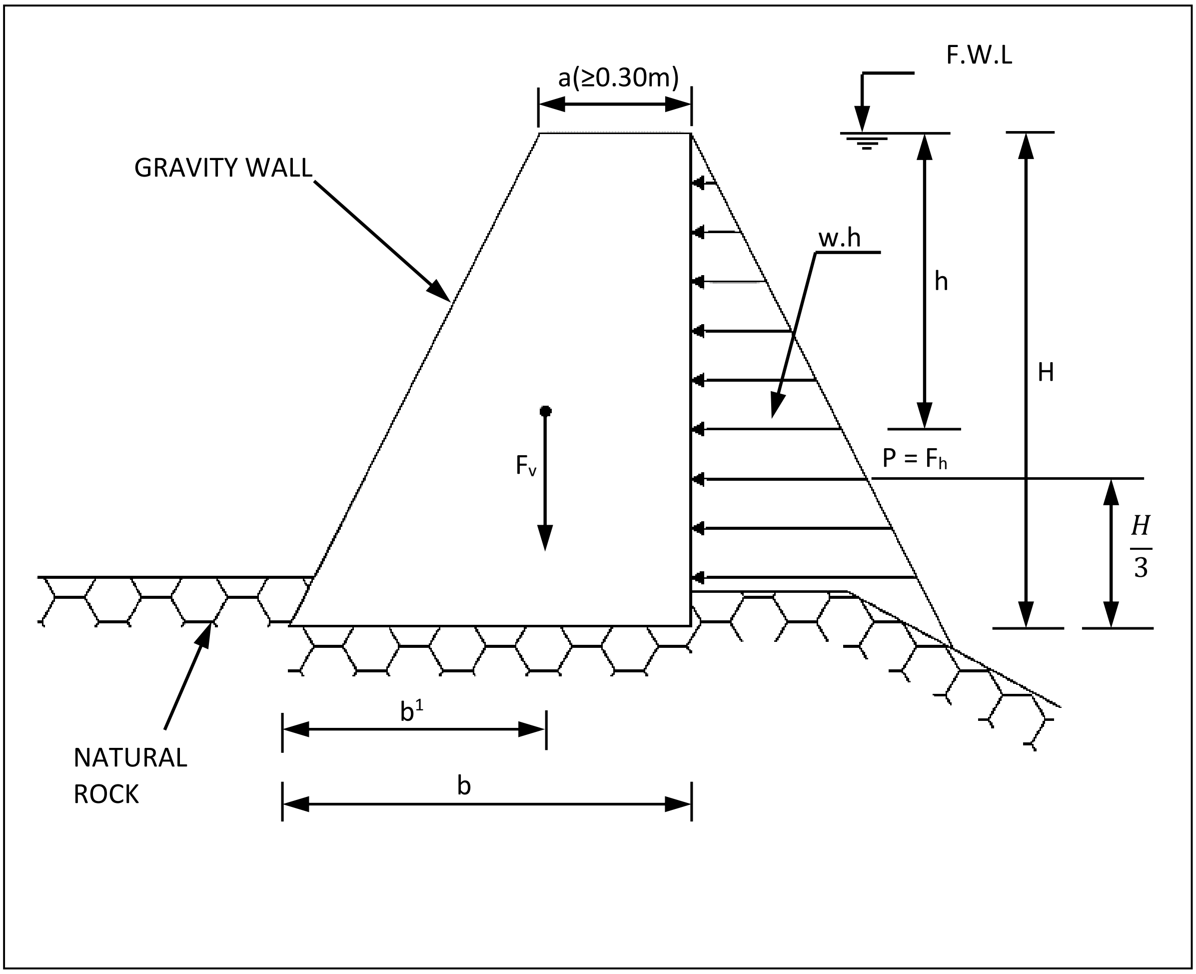

Figure 17-5: Typical Gravity Wall

The wall should be dimensioned to resist water pressure. Stability against sliding and overturning should be verified. The same approach as for sand storage dams or mass gravity walls can be used. The following formulae are now applicable (see Figure 17-5):

Equation 17-2 $\boldsymbol{F_h = P = \frac{w.H^2}{2}}$(kN) **Hydrostatic Pressure**

Equation 17-3 $M_o = \frac{P.H}{3}$; $M_r = \frac{b'.F_v}{2}$ **Overturning and Resisting Moments**

Where: w = Unit weight of Water = 9.81 kN/m^3

Safety factors with respect to sliding (F_S) and overturning ($F_{O.S}$) should not be less than 1.5.

The top width of the wall (a) should not be less than 0.30 m. For walls higher than 2.50m, the top width should preferably not be less than 0.50 m.

17.5.4 Overflow Sections

If properly constructed, the entire crest of the gravity wall can be used as an overflow section and there is no need for a detailed design.

There is also no need for a detailed inflow flood design. The rock catchment is simply designed to fill up and then overtop.

In the event that it is desirable to direct overflow water away from downstream infrastructure or into alternative storage sites, then spillway and inflow design flood calculations must be carried out and appropriately sized diversion channels can be constructed.

17.5.5 Draw-Off System

Draw-off systems for rock-catchment dams generally consist of a perforated GI pipe, laid horizontally at the upstream toe of the gravity wall, surrounded by sand/gravel filter material. The draw-off pipe passes through the gravity wall (diameter preferably 75 mm) to a gate valve located in a valve chamber below the wall. To minimise the risk of seepage along the pipe two or more (welded) steel collars can be provided where the pipe traverses the wall.

Leaking draw-off systems can result in considerable wasted water. Failed valves will result in the rapid emptying of a rock catchment. Whenever possible, redundant valves should be placed in the draw-off system. Other alternative offtake arrangements, such as the self-closing water tap, can be considered.

17.5.6 Storage Tanks and Ancillary Structures

There are various ancillary structures that are commonly associated with rock catchments. These include storage tanks, kiosks and cattle troughs. Standard Ministry designs, bills of quantities and drawings for masonry storage tanks of various sizes (10, 25, 50, 100, 135, 225 m^3), water kiosks, standpipes, and cattle troughs are available on the web site.

17.6 Typical Drawings

A rock catchment project should have the following drawings:

- Site layout showing rock face and final water use location;
- Cross section of any gravity walls;
- Longitudinal section of any storage areas.

Examples are given in Appendix B.

17.7 BoQs Specifications and Reporting Requirements

Sample BoQs and specifications are available on the web site.

17.8 Construction of Rock Catchments

Construction of rock catchments requires planning. As they are normally built in arid environments, care must be taken to ensure that sufficient water is available for both the construction and the curing of any concrete and mortar.

It is essential to ensure that all materials are delivered to the construction site in a timely fashion. Ideally everything should be on site (and transported to the rock face) before construction begins.

On sloping foundations, the use of starter bars is recommended. They can be drilled into the rock face as needed. 12mm bars set 150 to 200mm into the rock face and placed at a 50cm spacing throughout the foundation area have proven to be effective.

Foundations must be cleaned of all loose rock and any fractures should be filled and plastered before construction begins.

Wall and gutter alignments can be marked out with paint or chalk on the rock face.

17.9 Equipment

Site access is normally severely limited and all construction will be carried out by labour intensive methods. Water storage may be required if there is no alternative water source available. If the site allows vehicle access a cement mixer and wheelbarrows will be required.

17.10 Construction Supervision

Once marked out, construction can be carried out by local artisans with an appropriate level of supervision which can focus on specific aspects of construction – setting out, foundations, construction process, and finishes.

17.11 Operation and Maintenance

Operation and maintenance of rock catchments concentrates on repairing cracks in the gravity walls and ensuring that the offtake arrangements are working properly.

For open catchments, it may be necessary to desilt or remove any animal/wildlife waste during the dry season.

17.11.1 Trouble Shooting

If the rock catchment does not fill, even during average rainy seasons, then the catchment area must be extended with guttering.

If the rock catchment fills and then empties, all fractures and seepage zones in the rock and wall must be filled and plastered.

If poor water quality is a common problem, then piping the water to covered storage tanks should be considered.

17.12 Rehabilitation

Rehabilitation of rock catchments will consist of repairing large leaks and upgrading offtake systems.

CHAPTER 18

ANCILLIARY STRUCTURES

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18 ANCILLARY STRUCTURES

The ancillary structures needed for a particular water storage project will be site specific. This chapter presents guidelines for the design, operation and maintenance for a variety of ancillary structures that commonly occur with storage structures. Material from this chapter is drawn from standard ministry (MWIS) designs and WSTF[1] (2011). Relevant and available drawings are presented in Appendix B and on the website to this manual. For more detail, additional references that address each type of ancillary structure should be consulted.

18.1 Water Tanks

18.1.1 Design Considerations

The design considerations for a water tank include the following aspects.

a) Tank Capacity

The appropriate size of tank will be a function of inflow and demand (See discussion in Chapter 3 and 8). In a case where the storage tank serves to balance the daily peak demand then the tank capacity should be approximately 50% of the daily water demand. It should be noted that a tank of this capacity cannot be expected to extend reliability of the service beyond one day in the event of a disruption to tank inflow.

Table 18-1 provides tank sizes for which the design and drawings have been prepared by MWIS and WSTF (2011).

Table 18-1: Standard Tank Sizes

Source	MWI	WSTF (2011)	WSTF (2011)	WSTF (2011)	WSTF (2011)	WSTF (2011)	
Type	Circular ground masonry	Circular ground masonry	Rectangular Ground Masonry	10m High Elevated Steel	6m High Elevated reinforced Concrete	Under-ground *(Berkad)*	Plastic (uPVC) tanks
Tank Capacity (m_3)	10	20	5	64	20	215	Sizes range from 100l to 24m^3
	25	25	10	100	25		
	50	50		150	50		
	100	60			60		
	135						
	225						

b) Elevation

The required elevation of the tank will be a function of whether the water is pumped into the tank or flows by gravity and similarly for the outflow. This will determine whether the tank is placed on or near the ground surface or has to be elevated.

c) Materials

The choice of material for the tank is a function of:

- Size and structural requirements;

[1] WSTF 2011. Projects Unit Costing Report

- Availability of construction materials;
- Cost;
- Foundation conditions;
- Durability against weather, livestock, wildlife and misuse/vandalism;

Plastic tanks have gained popularity in Kenya recently due to their ease of installation, diversity in capacity and relatively lower costs when compared to other conventional storage options. They are also non-reactive and therefore compatible with different water quality types. The tanks can be surface or subsurface, and their capacities range from 100 litres to 24m^3. The only main consideration for the plastic tanks would be the foundation/base. This can be constructed from treated wood, concrete or compacted earth. Regardless of the material used, one should be careful to ensure that the surface of the base is as even and level as possible. This will prevent failure and damage to the tank due to localised pressures on or near the base.

d) Roofing/Cover

A roof or cover to a tank is recommended, especially where the water will be used for domestic purposes because sunlight on raw water can induce algae growth and open tanks can be contaminated. Any roof or cover should provide adequate size opening to facilitate access into the tank for cleaning and maintenance. A roof should include mesh covered ventilation pipes to facilitate "breathing" of tank during filling and emptying cycles.

e) Inflow, Outflow and Washout Arrangements

The inflow, outflow and washout arrangements should be carefully considered as these components are generally the most likely to require maintenance and possibly replacement. All valves at or near ground level should be protected in a valve chamber with a lockable lid. The valve chambers should provide adequate space to facilitate the use and replacement of the valves. Float valves should be placed within easy access of the roof hatch to facilitate monitoring, maintenance and replacement as needed. The level of the washout should be placed below the outlet pipe. Finally consideration should be given to the safe disposal of flow from the wash out.

18.1.2 Operation and Maintenance

Operation and maintenance tasks for tanks include:

- Inspect all valves and test function by opening/closing as necessary;
- Service and/or replace any faulty valves;
- Inspection of tank wall, roof and base for cracks and leaks;
- Determine and implement appropriate repair of cracks;
- Observe and record amount of silt accumulation;
- Clean tank of accumulated silt;
- Check washout and observe any signs of erosion related to washout water.

These tasks should in general be conducted at least once per month.

18.2 Stand Pipe/Community Water Point

18.2.1 Design Considerations

Key issues on the design of the standpipe include siting, ensuring the up-stand pipe is firmly secured against regular use, ensure a firm platform at the appropriate height for the water-drawer to place his/her container, stop-cock/gate valve near the standpipe to facilitate repairs of the tap, and the overflow water drains away from the collection point to avoid muddy pools of water impeding access to the stand pipe and creating a potential breeding ground for insects. Any meter should be placed in a secure meter chamber.

The most common problem with stand pipes include unsecure mounting which results in movement of the up-stand which ultimately causes damage to the joints and/or delivery pipe. Other common problems include damage/wear to the tap and inadequate drainage of spilled water.

18.2.2 Operation and Maintenance

Regular inspection of the gate valve, tap and drainage is required. An easily solvable problem like a leaky or non-functioning tap can result in the loss of supply from the whole project. If the stand pipe is within a fenced compound, then inspection and repair of the fence should be undertaken to ensure that uncontrolled livestock cannot cause damage to the stand pipe. The meter, if applicable, should be checked that it is operating properly and read on a regular basis.

18.3 Water Kiosk

18.3.1 Design Considerations

Design considerations for community water kiosks include:

1) Storage and the placement of storage;
2) Water-drawing arrangements for pedestrians and, if appropriate donkeys and hand carts;
3) Drainage;
4) Operator room/office space;
5) Security for cash and records;
6) Tools and spare parts for kiosk and water supply.

Reference should be made to standard designs and drawings. Improved designs have incorporated an elevated tank (usually plastic) placed on the concrete roof slab of the water kiosk. Such designs are ideal for locations with space restrictions. Typical drawings and reinforcement schedules for this arrangement are provided on the website.

18.3.2 Operation and Maintenance

The key operation and maintenance tasks associated with a water kiosk include:

- Check the fence and repair;
- Check structure of the water kiosk and repair as required;
- Check tap(s) and service as required;
- Check gate valves and meters and service as required. The meter may have a sieve that should be removed, cleaned and reinstalled;
- Tidy kiosk and surrounding area of garbage, litter, and any animal dung.

18.4 Cattle Trough

18.4.1 Design Considerations

When specifying cattle trough dimensions it is important to know approximate herd sizes that will be using the trough and if the trough will be predominantly used by goats, cattle, camels or a mix of livestock.

Standard designs are available for animal troughs which are specific to cattle/donkeys, sheep/goats and camels. Gate valves should be placed securely in close proximity to the livestock trough to facilitate servicing of the ballcock.

The livestock trough should be water tight so careful attention should be made to the foundation conditions (to avoid differential settlement that can induce tension cracks in the walling) and the quality of the plasterwork.

18.4.2 Operation and Maintenance

Operation and maintenance tasks are similar to a water kiosk but particular attention should be given to the structure and any plaster cracks repaired without delay. The ballcock needs servicing (replace pin and washers) and the area around the trough should be inspected for poor drainage. The combination of livestock dung and stagnant water can result in an unpleasant environment.

18.5 Cattle Ramp

18.5.1 Design Considerations

Cattle ramps provide a low cost way to ensure livestock can access the water in a dam or pan. Use of a cattle ramp reduces erosion around the reservoir and contamination of the water reservoir through uncontrolled contact with livestock. However, use of a cattle ramp will diminish the quality of the water making it unfit for human consumption without treatment. If a dam or pan is to be used only for livestock watering, it is not necessary to install any offtake structures.

A 15m wide (or larger) gently inclined cattle ramp can be constructed by placing 300mm of rolled hardcore on a 300mm sand bed for sections above the water line. Below the water level, the surface of the ramp should be protected by a 300 mm rip-rap layer, on an impervious clay blanket where required, which should be covered by gravel (See Type Drawing VI, VII and VIII for details).

18.5.2 Operation and Maintenance

The key maintenance task for a cattle ramp is to ensure that the surface is not covered in sediments thereby providing no advantage over other points of access to the water. Any places where the ramp material is broken up or buried should be covered with new hardcore/rip rap that is firmly secured in place.

18.6 Water Level Gauges

18.6.1 Design Considerations

Water level gauges are useful for estimating the volume of water present in a dam or pan. They can be constructed of metal pipe (usually 2 inch) set in concrete. 3.0m sections of pipe set at least 0.5m into a concrete footing are recommended. Scrap lugs should be welded to the bottom 0.5m of the pipe to ensure that it is securely held by the concrete. Pipes should be painted in 0.5m intervals. Finer intervals can be used for more accurate volume estimations (see Figure 18-1). Longer pipe sections are not recommended as they tend to be more easily damaged or destroyed.

It is advisable for owners and/or users of dams and pans to maintain proper records of regular readings of the water level in the dam or pan.

Careful attention should be given to the datum and zero level for the staff gauges to get the maximum benefit out of the staff gauges. The zero level can be set either at the lowest point in the reservoir or at the mouth of the outlet, in which case the staff gauge will just read the depth of live storage.

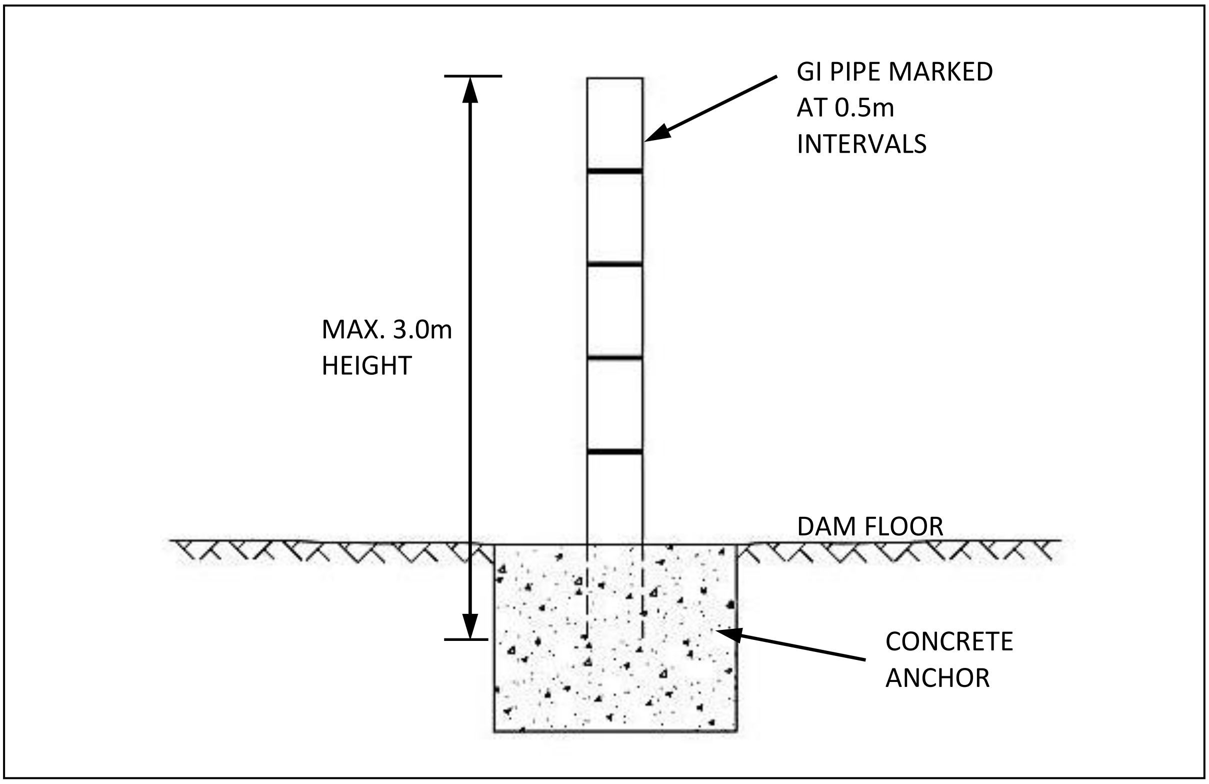

Figure 18-1: Typical Water Level Gauge

18.6.2 Operation and Maintenance

Operational tasks related to the staff gauges include making regular observations of the water level and keeping proper records. The primary maintenance task is to ensure that the markers are legible. The gauge should be repainted whenever required.

18.7 Fencing

Allowing either people or livestock near or in the reservoir increases the risk of polluting the reservoir water. Fencing a reservoir area helps to reduce the risk of uncontrolled access and drowning. Consequently, reservoirs areas should be fenced, and a draw-off system included in their design.

Fencing of dams and pans is, in theory, a very good idea. In practice, with the exception of commercially managed storage, fences very rarely last as long as planned.

18.7.1 Design Considerations

For the type of dams under consideration, barbed wire fencing on wooden posts will usually be adequate. Timber posts to be used for fencing will preferably be well seasoned straight wooden posts of 100 mm to 150 mm of diameter and 2.00 m in length.

The posts shall be firmly embedded 0.6 m deep in the ground. The portion of the posts buried underground should either be encased in concrete or treated against attack by insects.

Strutted straining posts (150mm diameter) should be provided at ends, corners and acute changes of direction or level, and at intervals not exceeding 100m in straight line fence. Struts (100mm diameter 2.40m long) must be secured to the straining posts at an angle of 45°, preferably with a spiked through bird’s mouth rebated joint. Intermediate posts shall be provided at intervals not exceeding 3 meters.

Fences consisting of three or four strands of barbed wire, spaced at 0.30m to 0.40m from each other, will usually be adequate to restrict cattle. All wires shall be strained tight by an appropriate strainer (e.g. ratchet winder) before being attached to the intermediate posts.

In order to minimize interference of the fenced and impounded area with wildlife habitat, it can be desirable to vary fence design so that livestock (and humans) are excluded but wildlife is allowed to pass. Figure 18-2 shows examples of fences which exclude cattle while permitting small antelope to pass under the fence (Note the smooth bottom strand!).

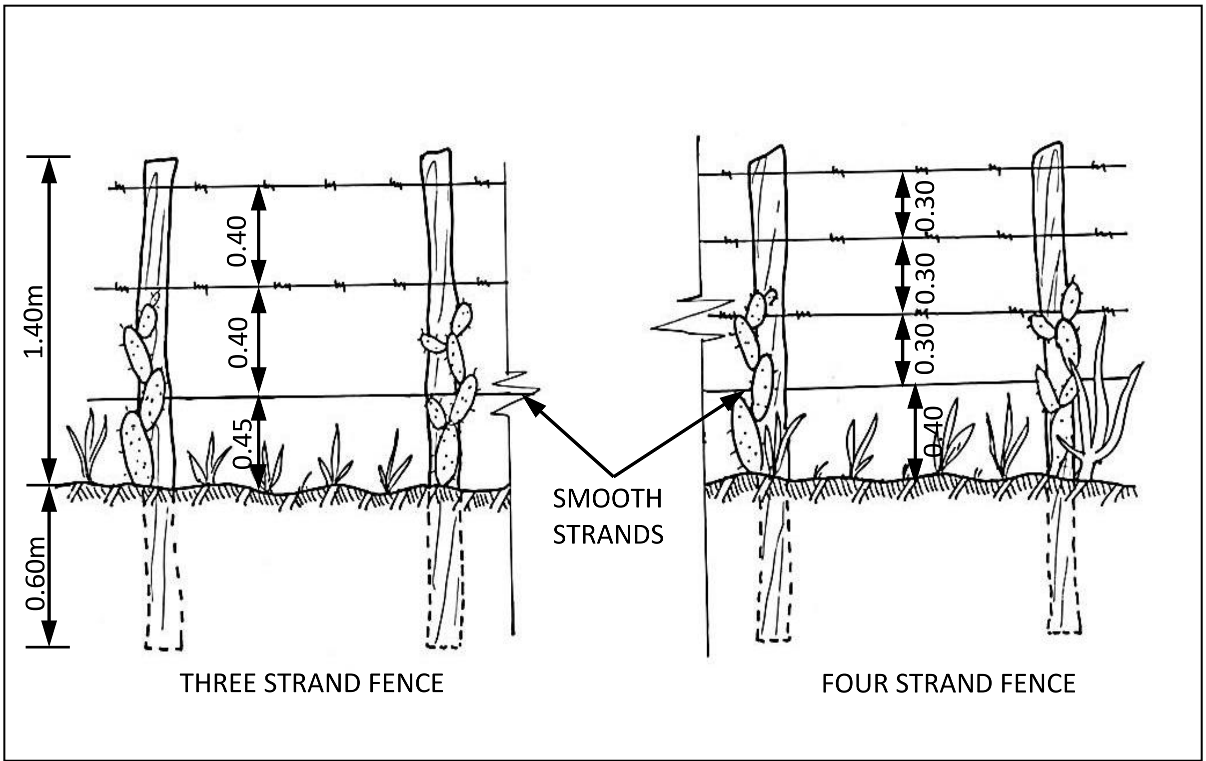

Figure 18-2: Fences for Dams and Pans

The use of live fences composed of thorny bushes should be combined with barbed wire fences. Recommended plants for live fences include Cactus and Sisal, which will deter livestock from entering the dam and reservoir area. Live fences should be planted as soon as the wire fence has been constructed. The selection of plant for the live fence should consider the viability of the plants given the local climate and rainfall pattern. Some live fencing material can be very invasive and while it may serve well as a fence, it may present other problems around the reservoir area.

18.7.2 Operation and Maintenance

Fences will deteriorate and must be regularly inspected and repaired immediately if they are to serve the intended purpose. Live fences should also be maintained which may involve trimming, infilling gaps, and replanting sections as needed.

18.8 Offtakes

There are a number of offtake options for most water conservation structures. The analysis of options is specific to the type and site conditions of the water conservation structure. Detailed design documents should be referred to once a choice of structure has been made. This section provides an overview on various design considerations.

18.8.1 Design Considerations

The design of the offtake structure should consider the following factors:

- Water quality and type of use to which water will be put. This may necessitate the use of specially designed filter and filter media to remove as many impurities as possible;
- Maintaining the water tightness of the water storage structure;
- Pumping or gravity options;
- Risk of emptying the reservoir;
- Accessibility to conduct inspection and maintenance activities. Essentially any offtake structure that is covered or buried should be designed to be robust as accessing the component for repair may not be possible;
- The need for a meter to measure abstraction.

Due to fairly low water quality of stored runoff water, abstraction of water for human consumption should be, wherever possible, through a specially constructed infiltration gallery with a sand or sand-gravel filter as shown on Type Drawing III. The thickness of the filter should not be less than 1.00 m, while its granulometric composition should satisfy the recommended USBR filter criteria (United States Department of the Interior - Bureau of Reclamation, 1987). Filters that incorporate a geotextile layer are possible but are not commonly used in Kenya.

In areas where it is not desirable to pass an offtake pipe through the wall of a storage structure (dam, sand dam), an offtake well can be constructed. The well can be linked to the reservoir via a pipe or via a filter drain as shown in Type Drawing VIII. The well should be lined and water can then percolate into the well through a filter ring made of no-fines concrete as indicated in Type Drawing VIII. The well should be located in a convenient place, preferably away from any inlet structure and cattle ramp. The well can be equipped with a hand-pump. Detailed references for the design of shallow wells and selection of hand-pump type should be consulted.

This structure has the advantage of avoiding any possibility of draining the reservoir through careless operation of valves.

In lined lagoons and pans where water is removed by pumping, it is often common to use submersible pumps (usually a borehole pump positioned inside a large diameter “sleeve” to avoid damaging the lining). It is also common to use floating pumps (usually a small raft with a walkway access) connected to supply pipelines via a section of flexible pipe.

Preliminary offtake pipe sizing can be done based on required discharge rates.

Table 18-3 on the following page gives low, average and high discharge rates for a variety of uPVC and GI pipe sizes. The discharge rates are based on flow velocities where low flow rates are determined using a velocity of 0.5 m/s velocity, average uses 1.0 m/s and high flow rate uses 3.0m/s.

Table 18-2: Typical Offtake Pipe Flow Capacities

Pipe Size		Pipe Diameter and Class	Low Flow	Avg Flow	High Flow
Inch Size	(mm)		m^3/hr	m^3/hr	m^3/hr
12"	315	315mm D	113	225	676
		315mm E	107	213	640
10"	280	280mm D	89	178	533
		280mm E	84	168	504
8"	225	225mm D	57	115	344
		225mm E	54	109	326
6"	160	160mm C	30	60	180
		160mm D	28	56	169
		160mm E	26	53	159
4"	110	110mm C	14	28	84
		110mm D	13	27	80
		110mm E	12	25	75
3"	90	90mm C	9	19	56
		90mm D	9	18	53
		90mm E	8	17	50
2.5"	75	75mm C	7	13	39
		75mm D	6	12	37
		75mm E	6	12	35
3"	GI	3inches	8	16	48
4"	GI	4inches	14	28	85
6"	GI	6inches	32	64	191
8"	GI	8inches	57	113	339

For offtakes that use a perforated or slotted offtake pipe, it is useful to be able to quickly estimate the perforated or slotted area required to achieve design discharge rates. If velocities in the screen are restricted to 0.025m/s, then Equation 18-1 will give a figure for the required open area in the offtake screens. This can then be used along with perforation size and positions to determine a length of perforated or slotted pipe.

Equation 18-1: $$\boldsymbol{A = Q/V}$$

Where: A is the required open area in the inlet perforated pipe [m^2];
Q is the desired flow from the dam in [m^3/s];
V is the velocity of the flow in the inlet screen and is set at 0.025 [m/s].

18.8.2 Siphon Offtakes

In certain circumstances, siphon offtakes can be used to pass water over the top of an embankment. In many cases they are positioned through the top of embankment walls (normally at the normal water line or just below it). In general the maximum height that a siphon can draw water over is 5.0m. This maximum height is determined by atmospheric pressure and varies depending on the elevation of the reservoir.

Siphons require heavy class pipes (thinner classes are susceptible to crushing) and are also extremely sensitive to friction losses from pipe fittings (especially entrance and exit losses).

A siphon arrangement will require some form of priming mechanism in order to start the siphon process.

Siphon pipe flow velocities are limited as per Table 18-2 and should generally be kept toward the higher velocity ranges.

18.8.3 Operation and Maintenance

The operation and maintenance tasks are specific to the type of offtake structure under consideration. The components of an offtake structure are frequently buried or covered and so are inaccessible. An important consideration is to monitor the quantity and quality of flow through the offtake structure as this can indicate whether the structure is blocked, or flow is impeded in some way and the condition of the material surrounding the offtake structure. For example, very turbid water would indicate that any filter medium is not working, that sediments are reaching the offtake structure, and that siltation in the offtake structure may be a problem.

The typical maintenance tasks are associated with inspection, servicing and replacement of valves, meters, pumps and piping.

18.9 Culverts

Culverts are often required for inflow channels or for outlet channels.

18.9.1 Design Considerations

Where inlet channels and spillways cross roads, the road crossing should be done with properly constructed culverts. Concrete culverts are generally available in 600, 900 and 1200mm diameters. On spillway channels it will usually be necessary to install multiple culverts to handle the expected flows.

Culvert flows can be calculated using the Manning's equation for flow through a conduit, and can be used to determine the required size of culvert to carry a specified discharge. The required discharges can be determined from runoff estimates of the area under consideration or from design flood estimates in the case of spillway channels.

From the simplified continuity equation:

Equation 18-2: $\boldsymbol{Q = vA}$

And from Manning's Equation:

Equation 18-3 $\boldsymbol{v = \frac{1}{n} R^{2/3} s^{1/2}}$

Where $R = \frac{A}{P}$

Combining the two formulas:

Equation 18-4: $\boldsymbol{Q = vA = \frac{1}{n} R^{2/3} s^{1/2} \pi r^2}$

Where
Q = discharge [m^3/s]
R = wetted perimeter [m]
S = slope [m/m]
N = Manning's roughness coefficient

Table 18-3 gives suggested values for Manning's coefficient for a wide variety of surface materials.

Table 18-3: Values for Manning's Coefficient

Surface Material	Manning's Roughness Coefficient - *n* -
Concrete - steel forms	0.011
Concrete (Cement) - finished	0.012
Concrete - wooden forms	0.015
Concrete - centrifugally spun	0.013
Corrugated metal	0.022
Earth, smooth	0.018
Earth channel - clean	0.022
Earth channel - gravelly	0.025
Earth channel - weedy	0.03
Earth channel - stony, cobbles	0.035
Galvanized iron	0.016
Gravel, firm	0.023
Masonry	0.025
Metal - corrugated	0.022
Natural streams - clean and straight	0.03
Natural streams - major rivers	0.035
Natural streams - sluggish with deep pools	0.04
Natural channels, very poor condition	0.06
Plastic	0.009
Polyethylene PE - Corrugated with smooth inner walls	0.009 - 0.015
Polyethylene PE - Corrugated with corrugated inner walls	0.018 - 0.025
Polyvinyl Chloride PVC - with smooth inner walls	0.009 - 0.011
Rubble Masonry	0.017

18.9.2 Operation and Maintenance

Culverts should be inspected for blockages, any settlement, sediment accumulation, and damage by wildlife and livestock. Excessive sediment would be an indication of insufficient slope.

18.10 Inlet Channels

18.10.1 Design Considerations

For an offline storage structure, it will often be necessary to construct inlet channels to bring flood water to the reservoir. Design considerations include:

- Geometry of the channel. A trapezoidal channel is frequently used as the side slopes are stable, and there is easy access for maintenance and cleaning of the channel;
- Size and longitudinal slope. This is a function of the desired conveyance capacity, material, and topographical conditions;
- Water-tightness and need for lining;
- Control of excess flows. A freeboard and section spillways should be provided to release excess flow at desired locations, where the overflow can be conveyed safely to the water course;
- Handling surface runoff that may introduce unwanted flow and/or debris into the channel. This may necessitate earth embankments on the upper side of a channel to prevent unwanted ingress into the channel.

Lined channels are recommended to minimise conveyance losses. Final lining choices will depend heavily on availability of local materials. A trapezoidal channel such as shown in Figure 18-3 is recommended.

Calculations for flow in trapezoidal channels can be done using Manning's equation as described in Section 18.9 Culverts.

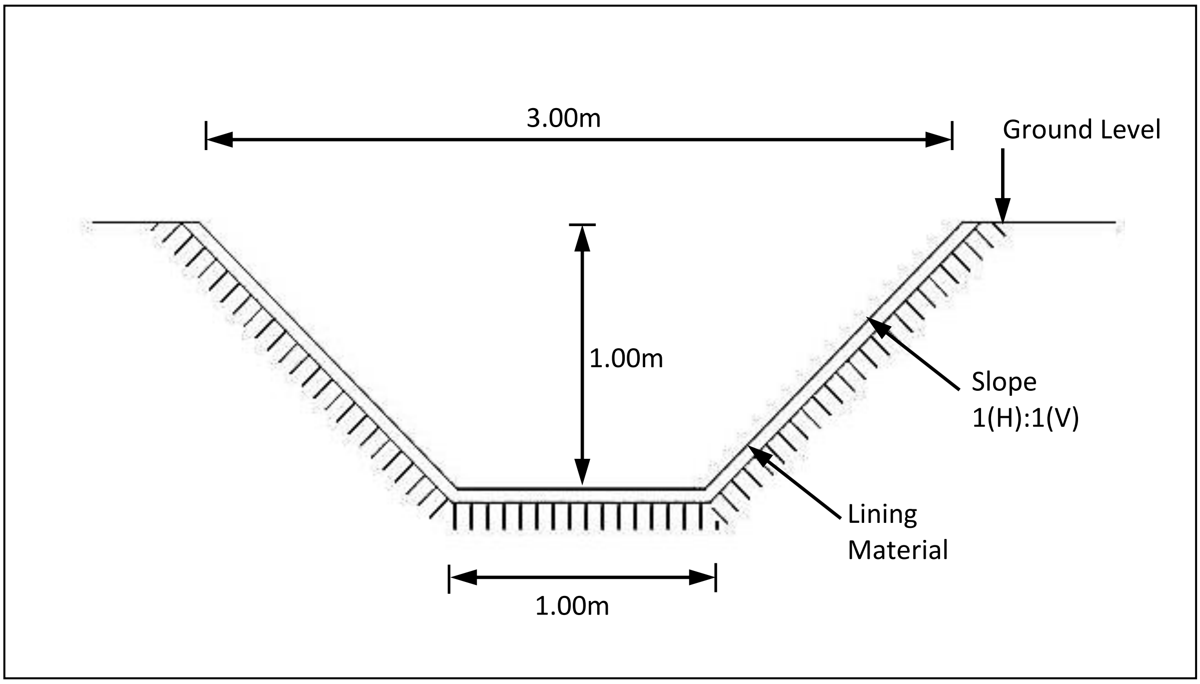

Figure 18-3: Typical Trapezoidal Channel (Section)

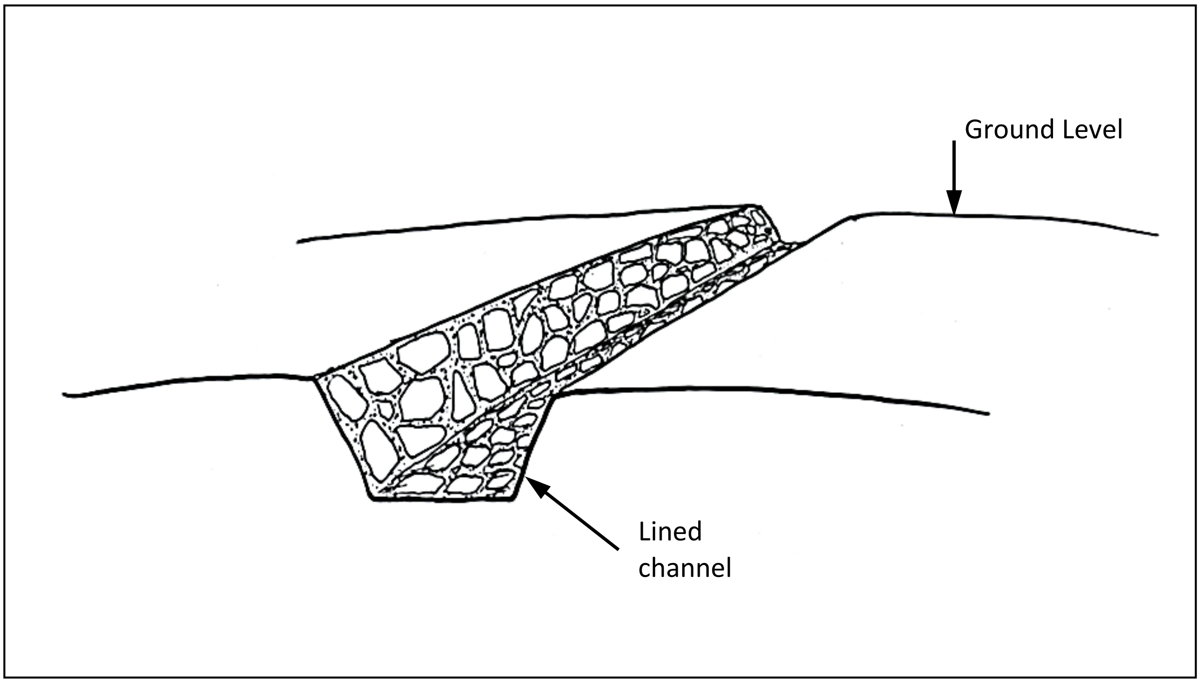

Figure 18-4: Typical Trapezoidal Channel

Channels can be lined with HDPE and LDPE lining material but these must be protected from trampling by livestock. The HDPE and LDPE materials results in a reasonably water tight channel with very favourable flow characteristics. The material does frequently need replacement and maintenance.

18.10.2 Operation and Maintenance

Channels should be inspected for blockages, any settlement, sediment accumulation, uncontrolled overflows, and damage by wildlife, livestock, and traffic. Sites showing signs of settlement, sediment accumulation or uncontrolled overflows should be addressed as these are indicators of other issues that need to be addressed, namely a weak foundation, insufficient slope, insufficient freeboard, and excess inflow, all of which can lead to more serious problems.

18.11 Self Regulating Weirs

For reservoirs that rely on river offtakes for inflow, self-regulating weirs can be used to ensure that river water is abstracted strictly in compliance with the water permit and WRM Rules where water is allocated not only by quantity but also by timing in that flood water abstraction can only take place when river flows are in a flood state, i.e. above Q80 (flows that are exceeded 80% of the time).

18.11.1 Design Considerations

A self-regulating weir typically consists of a mass gravity wall across the river. Two openings are left at the top of the weir; one that passes flow downstream and one that flows into a collection box that leads to the conveyance pipe or channel. The river opening is sized and positioned to ensure that at low flows equivalent to the environmental reserve (Q95), all river flow passes through the river opening. The abstraction opening is sized and positioned so that as river flows increase, water begins to pass through the abstraction opening.

The idea is to ensure that the reserve flow is maintained during periods of low river flows and that higher flows are divided appropriately between the river and the abstraction up until the maximum abstraction rate is reached after which all additional river flow passes over the main weir crest and remains in the river channel.

A typical cross section of a self regulating weir is shown in Figure 18-5

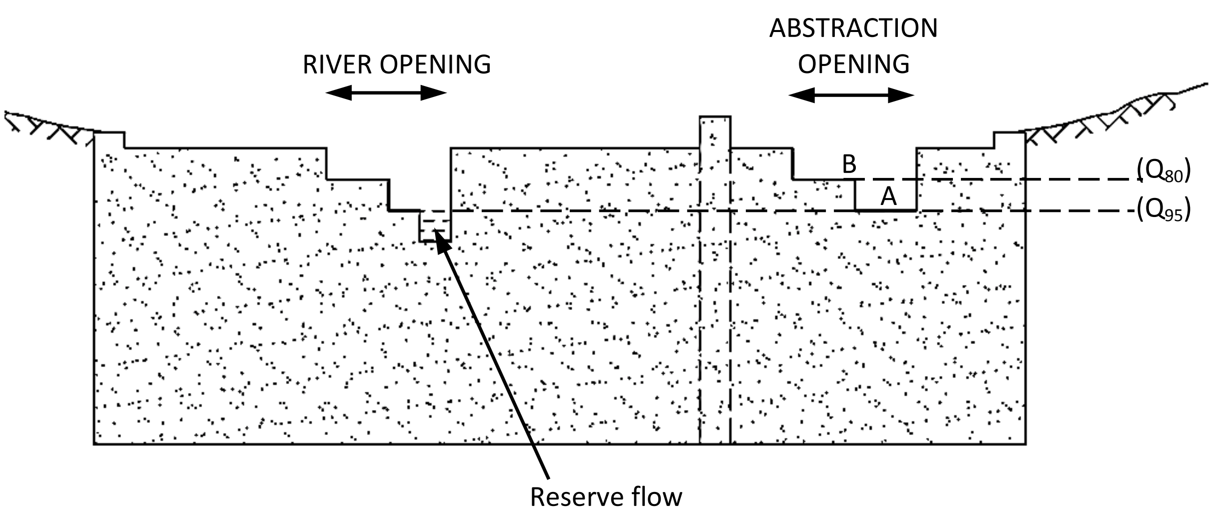

Figure 18-5: Typical Self Regulating Weir (Front view)

In general, abstraction should not begin until the river flow reaches the Q_{95}value in order to safe guard the reserve flow. In the section where the flow is above Q_{80}, the width of the opening for the river and abstraction will determine the proportion of the river flow that is going into the abstraction chamber. A guideline value is 50% so that flows are shared equally between the abstraction and river. Once flows exceed the maximum abstraction capacity all the flow will go downstream.

In some instances, mostly where there are other storage structures or offtakes downstream of the self regulating weir, it may be necessary to direct a greater percentage of the flood flow into the river and a smaller percentage into the abstraction opening. This can be done by appropriately sizing the opening widths.

The broad crested weir formulas can be used for sizing openings and determining flows as given in Equation 18-5:

Equation 18-5: $$Q = b \times \sqrt{g \times \left(\frac{2}{3} \times H_a\right)^3}$$

Where: Q_{max} = Discharge over the weir [m^3/s]
b = width of the weir [m]
H_a = approach height of the water [m] (measured upstream of the weir)
g = acceleration of gravity [9.81 m/s^2]

Note that H_a is equal to $\frac{3}{2} \times H_c$ where H_c is the critical flow depth over the weir.

Various width and approach height options for each opening can be tried until a combination that allows flows as described above is achieved.

Reference can be made to Thomas (2001)[2] for further details.

[2] 2001 Self Regulating River and Spring Intakes. *Kenya Engineer*. July/August Issue. Vol. 24, No. 4

18.11.2 Operation and Maintenance

One of the advantages of the self-regulating weir is that the arrangement of opening sizes and levels means that abstraction is regulated according to the river flows, not by means of opening and closing gate values. However, it is still important to inspect the weir for any cracks and the control valves/gates for wear and/or damage.

18.12 V-Notch Weir

The v notch weir is an example of a sharp crested weir, and is especially useful for measuring low flow rates, because the flow area decreases rapidly as the head over the v notch gets smaller, as would be the case for seepage from an embankment.

Flow over a v-notch weir is calculated using Equation 18-6.

Equation 18-6 $$Q = C_d \frac{8}{15}\sqrt{2g}\tan\frac{\theta}{2}H^{5/2}$$

Where:
Q = flow over the weir [m^3/s]
C_d = dimensionless coefficient of discharge
θ = angle of v notch
H = height of water above apex of v notch [m]

For a v notch weir to work effectively, the upstream chamber should be of sufficient length and depth to ensure tranquil flow over the weir.

18.12.1 Design of the V-Notch Weir

Some guidelines when designing a v-notch weir include:

1. The centre line of the weir notch should be parallel to the direction of flow;
2. The face of the weir should be vertical, not leaning upstream or downstream;
3. Water surface downstream of the weir should be at least 6cm below the bottom of the V to allow a free flowing waterfall;
4. For accurate measurements, the depth over the crest should not be more than one third the length of the crest.

18.12.2 Operation and Maintenance

Operation and maintenance requirements for the v notch weir would include regular inspection of the blade to ensure that it is upright, regular inspection of training walls and rehabilitation as need arises.

18.13 Break Pressure Tanks

Break pressure tanks (BPTs) are a useful component of a water reticulation system for sections of the system whose pressures exceed the classes of the pipeline installed. As the name suggests, the main function of a BPT is to keep flow pressures within the design limits. The tank may be elevated to generate downstream flow.

18.13.1 Design Considerations

The volume of the BPT should be large enough to give a retention period of up to two minutes. The inlet should end close enough to the floor of the tank to prevent trapping of air from the falling jet of water.

Typical BPT drawings are provided on the website.

18.13.2 Operation and Maintenance

Operation and maintenance requirements would include regular desilting, inspection and repair of pipe connections and valves as the need arises.

18.14 Pump Stations

For pumped distribution systems, a typical pump station should be included. Its function will be to house the controls, switches, pump and motor (in case of a surface pump) and standby spares and equipment. The pump house should be designed to allow for access to serviceable parts, as well as allow for sufficient ventilation and provide adequate security.

Drawings of a typical pump house are provided on the website.

18.15 Water Treatment

This section presents simple water treatment methods that can be adopted at the household level. Treatment options for bulk water can be obtained from the Water Supply Manual (Ministry of Water and Irrigation, 2005).

18.15.1 Boiling

This is the most commonly used household water treatment method. It involves raising the water temperature to 100°C to kill off microbial pathogens. However, the cost of fuel would be an impediment to this choice of treatment, as well ask the risk of burning (UNICEF, January 2008).

18.15.2 Chlorination

This is another common low cost water treatment solution. It involves the addition of Sodium Hypochlorite to the water for disinfection. This method offers a quicker option when compared to boiling, and has fewer cost implications (UNICEF, January 2008).

18.15.3 Filtration

Filtration kits have been scaled down to units that can be used at the household level, and are typically ceramic or slow sand filters. The benefits of using these include improving the aesthetic value of the water and providing treatment without affecting the taste of the water. Filters treated with bacteriostatic silver have been documented to remove at least 99% of protozoa and bacteria.

18.15.4 Solar Disinfection

Also known as Sodis, this alternative option for water treatment involves bottling the water in 2 litre clear plastic bottles and exposing these to UV radiation by placing the bottles on rooftops for 6 to 48 hours, depending on the intensity of insolation. This method has been shown to be effective in reducing microbial pathogens in drinking water (UNICEF, January 2008). However, this method is only effective for low turbidity water.

18.15.5 Combined Flocculation and Disinfection

This method can be adopted for high turbidity water and will involve addition of flocculants such as alum to settle the suspended matter, as well as a disinfectant to kill off pathogens. Products such as PUR have been developed to provide low cost, household level solutions for the treatment of water with turbidity. These can be easily obtained from retail outlets or local pharmacies.

CHAPTER 19

TECHNICAL REPORTS

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19 TECHNICAL REPORTS

This chapter addresses the content and structure of the technical reports that may be required as part of the development of a water conservation project. Technical reports should provide a concise and legible record of the findings, considerations and results of the site investigations, feasibility analysis, design, construction and operation and maintenance aspects of a project. This section is provided as a guide to ensure that critical elements regarding the documentation of a project are not overlooked. In addition, as many of the documents are required for regulatory authorities, consistency in regard to the structure of the document facilitates the assessment of the document by the regulatory authorities.

19.1 Site Reconnaissance Report

19.1.1 Introduction

The Site Reconnaissance Report provides documentation of the findings from the site visit and presents the conclusions and recommendations of the design engineer to enable the project owner to make the decisions required as to whether to proceed with the project to the next stage of investigations. The report is not a requirement for a regulatory authority.

19.1.2 Reconnaissance Report Outline

1. Background and purpose of the proposed structure
   - Owner of the project;
   - Purpose of the project;
   - Location of the site;
   - Details and date of site visit.
2. Findings from site visit and investigations
   - Land ownership and site access;
   - Project stakeholders;
   - Legal compliance aspects;
   - Topography;
   - Hydrology and sediment load;
   - Soils and geology;
   - Construction materials;
   - Environmental and social considerations.
3. Analysis of alternative options
4. Design issues
   - Limitations to scale.
5. Construction issues
   - Approach;
   - Construction period;
   - Timing.
6. Operation and maintenance/sustainability issues
7. Conclusions and recommendations
   - Does the professional conclude that the site(s) is suitable for the intended purpose?
   - What measures or steps should the project owner take to advance or alter the project to achieve the intended objectives?

19.2 Feasibility Report

19.2.1 Introduction

Once the feasibility assessment has been conducted and all relevant information collected, the design engineer should prepare a feasibility report. The information presented in this report will guide decision makers on whether or not to proceed with the proposed project to the design phase.

19.2.2 Feasibility Report Outline

1. Executive summary
    - Summary of project details and feasibility analysis. This is a concise summary of the key features of the project and can be used to share project details with project stakeholders like WRMA, WRUA, County government, community and potential development partners.
    - Summary of conclusions and recommendations regarding the economic, social, legal, technical, and environmental feasibility of the project.
2. Background and purpose of the proposed structure
    - Owner of the project;
    - Purpose of the project;
    - Location of the site;
    - Details of site visits and investigations.
3. Analysis of alternative options to meet project objectives
4. Analysis of water requirements to meet project objectives
5. Site investigations
    - Topography;
    - Soils and geotechnical.
6. Environmental and social considerations
    - Scoping for EIA study;
    - Stakeholders;
    - Legal implications.
7. Hydrological analysis
    - Estimated inflow and design flood;
    - Risk of sedimentation.
8. Identification of design issues
    - Criteria;
    - Estimated spillway sizes;
    - Offtake structures;
    - Ancillary structures.
9. Project construction plan
    - Approach – labour based, mechanised, etc.
    - Project construction plan should identify the key steps and expected duration of the project going forward. It should cover the project from design through construction to certification.
10. Cost estimate
    - A budget for each option should be produced based on the BoQ and estimated rates.
11. Economic analysis
    - A cost/benefit analysis should be conducted even if the scale of the project and the detail at this stage does not warrant a detailed analysis. However the total project cost, covering design, construction, supervision, and all environmental and social mitigation measures can be estimated. This may not reflect the true cost of all the impacts but assumptions regarding the estimation process can be described.
    - The benefits of the project should be described and estimated and compared to the costs. The intention is to avoid making investments in projects that cannot be justified due to the cost.

12. Project financing
    - Identification of financing sources and issues.
13. Analysis of risks and proposed mitigation measures
14. Conclusions
    - Legal, social and environmental feasibility;
    - Technical feasibility;
    - Financial details/feasibility;
    - Economic feasibility.
15. Recommendations
    - Measures required to enable the project to meet its objectives;
    - Way forward.

19.3 Hydrological Assessment Report

19.3.1 Introduction

As per Section 27 of the WRM Rules (2007) WRMA may require a Hydrological Assessment Report prepared by a Qualified Water Resource Professional to accompany a permit application. The Second Schedule of the WRM rules (2007) sets out the outline of the Hydrological Assessment Report.

19.3.2 Hydrological Assessment Report Outline

1. Name and details of applicant;
2. Location and description of proposed activity;
3. Details of climate;
4. Details of river or water body (name, nearest river gauging station, sub-catchment);
5. Details of catchment (area, slopes, soils);
6. Details of vegetation and land use;
7. Details of registered and non registered abstraction on the resource;
8. Details of all other permits related to this application;
9. Hydrological characteristics and analysis (annual, monthly, extreme events, flow duration or probability of events occurring);
10. Hydrochemistry;
11. Analysis of the reserve;
12. Assessment of availability of flow;
13. Impact of proposed activity on flow regime, water quality, other abstractors;
14. Recommendations on proposed activity.

19.4 Environmental Impact Assessment

19.4.1 Introduction

A described in Chapters 4 and 6, the Environmental Impact Assessment (EIA) Report is part of the process to obtain an EIA license from NEMA and is applicable to the type of structures within the scope of this document.

19.4.2 EIA Report Outline

1) Project description
    a. Project objectives/purpose;
    b. Location (GPS coordinates; grid reference);
    c. Nature and scope of project components;
    d. Expected benefits.

2) Process of public disclosure and consultation
    a. Stakeholder analysis;
    b. Public meetings;
    c. Key informant interviews;
    d. Findings from public disclosure and consultation process.
3) Project alternatives
    a. Alternative location;
    b. Alternative design;
    c. Alternative options to meet same objectives;
    d. No project alternative.
4) Legislative framework
    a. Environmental laws and regulations;
    b. Other relevant laws and regulations - water, agriculture, livestock, forestry, fisheries, land planning, physical planning, land control, etc.
5) Baseline situation
    a. Physical environment;
    b. Ecological environment;
    c. Social environment;
    d. Economic environment.
6) Anticipated environmental and social impacts
    a. Pre-construction phase;
    b. Construction phase;
    c. Operational phase.
7) Mitigation measures
    a. Pre-construction phase;
    b. Construction phase;
    c. Operational phase.
8) Environmental monitoring and management plan
    a. Pre-construction phase;
    b. Construction phase;
    c. Operational phase.
9) Recommendations

19.5 Dam Design Report

19.5.1 Introduction

The Dam Design Report is one of the requirements to accompany the water permit application. The outline provided below complies with the Water Resource Management Rules (2007) Section 64.

The dam design report should be complemented by the Environmental and Social Impact Assessment Report and the Hydrological Assessment Report as annexes to the design report. However, the main findings and results of the ESIA and HAR should be included in the Dam Design Report.

The dam design report presented below is customised to the case of a small earth embankment dam. The same report outline should be followed for other structures (mass gravity dams, pans, rock catchments, sand dams, sub-surface dams) but should be customised appropriately for the specific type of structure.

19.5.2 Dam Design Report Outline

A. Executive Summary

- Table of salient features.

B. Introduction

- Dam name;
- Physical location (grid reference);
- Administrative location (county, sub-county,….sub-location);
- Land reference number;
- Drainage area;
- Name of river or water body;
- Class of dam and hazard assessment;
- Owner of the land and structure;
- Designer / engineer;
- Dates: design completed, planned construction.

C. Purpose

Brief description of the purpose of the project and the expected benefits.

D. Reference Drawings

List of reference drawings, listing their subjects (titles) and reference numbers.

E. Hydrological Assessment

This information should be consistent with and drawn from the Hydrological Assessment Report.

- Climatic conditions
    - Mean and monthly annual rainfall (mm);
    - Mean and monthly annual evaporation (mm).
- Catchment features
    - Catchment area (km^2);
    - Longest path (length of river) in the catchment area (km);
    - Maximum altitude (m);
    - Altitude at the dam site (m);
    - Catchment conditions (land use, vegetation cover, slopes, soils).
- Hydrological analysis
    - Description and quality of data used for analysis;
    - Existing upstream and downstream abstractions;
    - Hydrological characteristics (annual, monthly, extreme events, flow duration or probability of events occurring);
    - Reserve or environment flow requirements;
    - Inflow design flows for specified return periods.

F. Assessment of Environmental and Social Impacts

Brief description of the principal findings of the environmental and social impact assessment, covering both the short term (construction period) and long term (operational phase).

G. Details of the Reservoir

- Impounded area (ha);
- Storage capacity (m^3), dead storage volume (m^3), net storage volume (m^3);
- Expected annual evaporation losses (m^3);
- Fetch (m);
- Maximum depth (m);
- Height-volume-area (HVA) curves.

H. Details of Embankment

- Height above lowest foundation (m);
- Gross freeboard (m), minimum freeboard (m);
- Dam crest: width (m), length (m), allowance for settlement;
- Embankment slopes (h:v): upstream slope, downstream slope;
- Core slopes;
- Total fill volume (m^3);
- Details of embankment fill: volumes and type of material of core fill (m^3), shell fill (m^3) filter material (sand, gravel) (m^3), rock-fill (m^3), rip-rap (m^3);
- Foundation excavation volume (m^3);
- Upstream, downstream and crest protection measures.

I. Details of Spillway

- Inflow design flood (m^3/s) and return period;
- Number of spillways;
- Width(s) (m) and lengths (m);
- Number of sills, material and location;
- Slope(s);
- Excavation Volumes: total excavation (m^3), rock excavation (m^3), soil excavation (m^3);
- Spillway protection measures.

J. Details of Draw-off System

- Brief description of the draw-off system and its principal components;
- Diameter of draw-off pipe (mm);
- Length of draw-off pipe (m) and anti-seepage arrangements;
- Ground level and details of intake structure;
- Outlet level at valve.

K. Details of Scour or Compensation Flow Arrangement

- Brief description of the scour or compensation flow arrangements and its principal components;
- Diameter of pipe (mm);
- Length of pipe (m) and anti-seepage arrangements;

- Ground level and details of intake structure;
- Outlet level at valve.

L. Details of Ancillary Structures

- Water level gauges;
- Fencing;
- Pump houses;
- Valve chambers;
- Consumer points (cattle troughs, water kiosks, etc).

M. Catchment Protection Works

- Brief description of the principal catchment protection works.

N. Cost Estimate

- Estimated construction cost (KES).

O. Construction Schedule

- Details of diversion works, if any;
- Construction schedule.

Figure 19-1 shows an elaborate example of a bar chart construction schedule for the construction of a new dam.

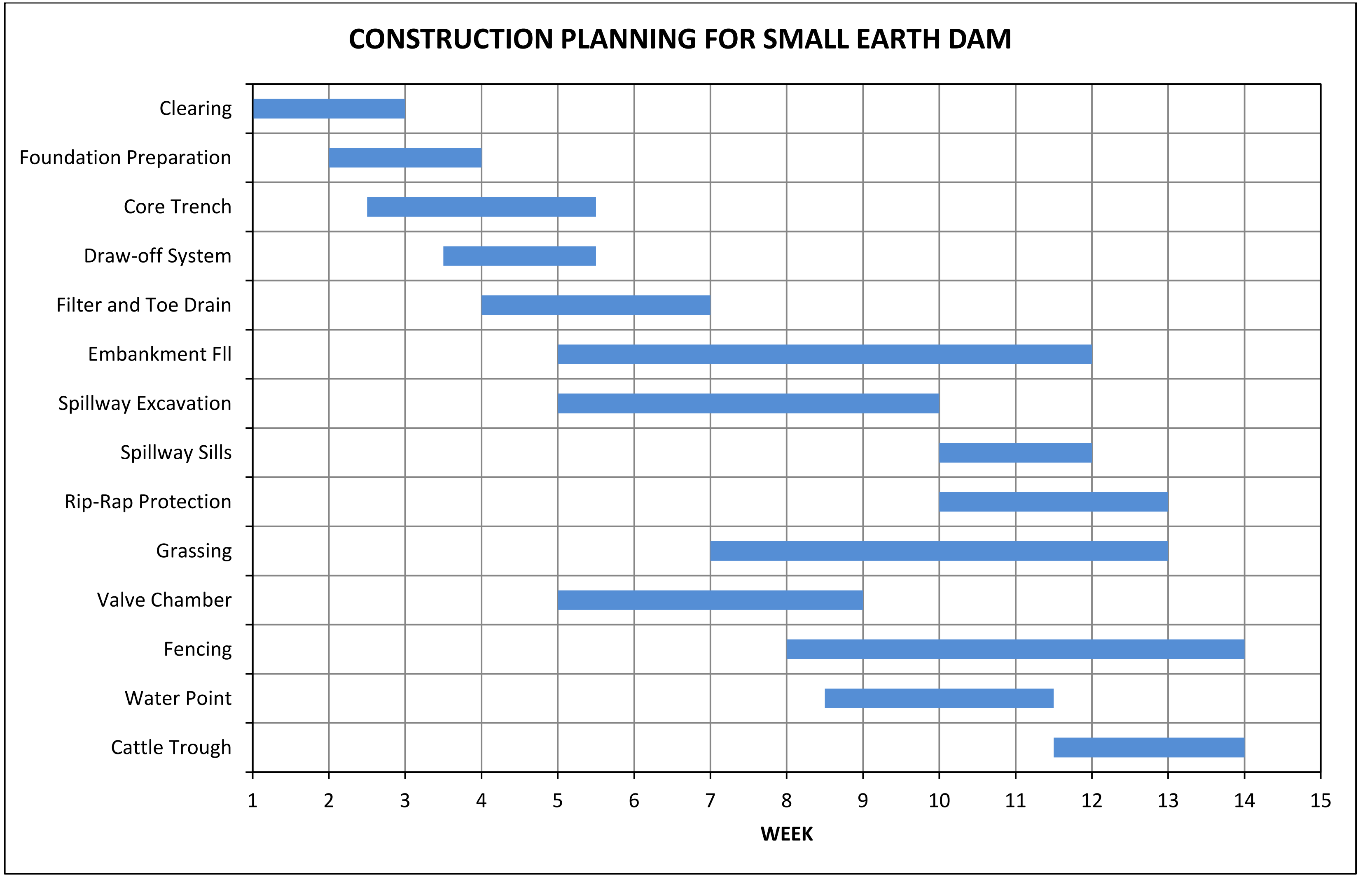

Figure 19-1: Construction Schedule for Small Earth Dam

P. Operational Rules

- Description of operating rules;
- Regime of environmental flow releases.

Q. Procedures to notify and protect downstream inhabitants, infrastructure and environments

This material should be consistent with and drawn from the Dam Safety Plan (EAP).

- Hazard risk analysis;
- Notification procedures and details.

R. Schedule of Inspection and Maintenance

- Inspection schedule and details;
- Maintenance schedule and details.

19.6 Dam Completion Report

19.6.1 Introduction

Section 66 of the WRM Rules (2007) requires the submission of a Dam Completion Report to WRMA. The report should be provided to the dam owner and users in addition to WRMA. The Report should be prepared by the engineer or technical staff involved in the project.

The outline of the Dam Completion Report, presented below, also meets the requirements of the Dam Operation Report as anticipated in Section 66 of the WRM Rules (2007). It is possible to submit one report that comprehensively addresses the as-constructed conditions and the operational aspects.

19.6.2 Dam Completion Report Outline

A. General Information

- Details of owner and dam users;
- Location.

B. As-Constructed Details

- Technical design details, revised for as-constructed conditions.
- Changes between the original design and as-constructed conditions with an explanation for the changes.
- The final as-constructed details should be clearly specified. Particular attention should be paid to aspects of the project that are buried and would be difficult to ascertain without proper documentation. Final HVA curves should be presented.

C. As-Constructed Drawings

- A complete set of as-constructed drawings should be included in the Dam Completion Report with particular attention to site bench marks, level monitoring pegs on the embankment (if any), water level gauges, and spillway crest levels, all of which are relevant to future monitoring of settlement of the embankment or changes to the as-constructed conditions over time.

D. Operation and Maintenance Issues

- Release rules, including releases for environmental flows;
- Schedule of operation and maintenance tasks and responsibilities around dam, reservoir and catchment area;
- Stocks needed to perform the maintenance tasks;
- Schedule of monitoring tasks, including relevant forms and data management systems;
- Schedule of inspection, including relevant forms, and remedial measures if needed;
- Catchment protection.

E. Emergency Procedures

- Notification Flow chart drawn from the Emergency Action Plan.

F. Recommendations

- General recommendations on catchment conservation which may cover issues of overgrazing and soil conservation activities;
- Means to control pollution of the dam;
- Measures to minimise and manage water use conflicts, should they arise.

19.7 Emergency Action Plan

19.7.1 Background

The Emergency Action Plan (EAP) is an operational document that provides the agreed procedure and contact details for those responsible in the event of an emergency. It should be structured to enable quick and easy reference to key information.

19.7.2 Emergency Action Plan Outline

Title Page/Cover Sheet
Table of Contents

1. Notification flowchart;
2. Statement of purpose;
3. Project description;
4. Emergency detection, evaluation and classification;
5. General responsibilities under the EAP;
6. Preparedness;
7. Inundation maps;
8. Appendices;
9. Investigation and analyses of dambreak floods;
10. Plans for training, exercising, updating and posting;
11. Site specific concerns;
12. Approval of EAP.

19.8 Dam Failure Report

19.8.1 Introduction

Section 68 of WRM Rules (2007) requires that a report be submitted to WRMA in the event of a dam failure, regardless of whether harm or damage was caused downstream. This report provides documentation on the events leading up to and causing the failure. It is important that the document is prepared immediately after the dam failure as information regarding the cause of failure may be lost with time.

19.8.2 Dam failure Report Outline

1. Details of dam location;
2. Date and time of dam failure or damage;
3. Preceding climate;
4. Preceding hydrology;
5. Cause of dam failure or damage;
6. Steps taken to notify downstream inhabitants;
7. Nature and extent of damage caused to the dam or caused by the dam failure.

CHAPTER 20

BIBLIOGRAPHY

20 BIBLIOGRAPHY

Agrawal, Y. C. (n.d.). *Soil properties and their influence on design of dams.* Jaipur, India: Minor Irrigation schemes Poject.

Arkansas Soil and Water Conservation Commission. (2002). *Inspection and maintenance manual for Arkansas dam owners.* Little Rock, Arkansas: Arkansas soil and water conservation commission.

Blackie, J.R, Edwards, K.A and Clarke, R.T. (1979). Hydrological Research in East Africa. *East African Agricultural and Forestry Journal, Vol. 43*(Special Issue).

Burke, C. (2003). *Indiana Dam Safety Manual.* Indianapolis, Indiana: Department of natural resources, Division of water.

Charman J., Lubtcho K., Minnetti L., Stoutesdijk J. and Tricoli D. (2001). *Small Dams and weirs in earth and gabion materials.* Rome: Food and Agriculture Organisation of the United Nations.

Civil Engineering Division Council. (1998). *Indian Standard; Criteria for Design of Solid Gravity Dams.* New Delhi: Bureau of Indian Standards.

DHI. (2013). *Building Adaptive Capacity to Climate Change in Kenya: Regional Climate Modelling - Key Results.* Horsholm, Denmark: DHI.

Egan, G. (April 1962). *Notes on the Design and Construction of Small Earth Dams in Kenya.* Nairobi, Kenya: Ministry of Works, Hydraulic Branch.

Eriksson, A., & Kidanu, A. (August 2010). *Guidelines for Prevention and Control of Soil Erosion in Road Works.* Nairobi: International Labour Office.

Fiddes D., Forsgate J .A and Grigg A. O. (1974). *The prediction of storm Rainfall in East Africa.* Crowthorne, Berkshire: Transport and road research laboratory, Department of the environment.

Herzog, M. (1999). *Practical Dam analysis.* London: Thomas Telford Publishing.

ICOLD, A. H. (2011). *Small Dams: Design, Surveillance and Rehabilitation.* International Committee on Large Dams.

ICOLD, A. H. (2011). *Small Dams: Design, Surveillance and Rehabilitation.* International Committee on Small Dams.

Institution of Civil Engineers. (1991). *Standard Method of Measurement (CESMM3)* (3rd Edition ed.). London: Thomas Telford.

Interagency committee on dam safety. (2005). *Technical manual for dam owners: Impacts of plants on earthen dams.* U.S.A: Federal Emergency Management Agency.

Interagency committee on dam safety. (April 2004). *Federal guidelines for dam safety: Emergency action planning for dam owners.* USA: Federal Management Agency.

International Finance Corporation. (2002). *Handbook for Preparing a Resettlement Action Plan.* Washington DC: IFC.

JICA. (January 1992). *National Water Master Plan Sectoral Report (B) Hydrology.*

JICA; Nippon Koei Co.Ltd. (2013). *The Project on the Development of the National Water Master Plan 2030. Final Report Voulme II Main Report (1/2).* Nairobi.

Knappett, J., & Craig, R. (2012). *Craig's Soil Mechancs.* New York: Spon Press.

Lindqvist A.K. (2005). *Water from ponds, pans and dams: A manual on planning, design, construction and Maintenance. Technical handbook No. 32.* Nairobi, Kenya: Regional Land Management Unit (RELMA)/World Agroforestry Centre Eastern and Central Africa Regional Programme.

Linsley, Ray K.; Joseph B. Franzini. (1979). *Water Resources Engineering.* Ohio: Mc-Graw Hill.

Mekdaschi S. R and Liniger H. (2013). *Water Harvesting: Guidelines to good practice.* Rome: Centre for Development and Environment, Rain Water Harvesting Implementation Network and International Fund for Agricultural Development.

Ministry of Water and Development. (1978). *Rainall Frequency Atlas of Kenya.* Nairobi.

Ministry of Water and Irrigation. (1992). *The National Water Master Plan.*

Ministry of Water and Irrigation. (2005). *Practice Manual for Water Supply Services in Kenya.* Nairobi.

Ministry of Water and Irrigation. (May, 2010). *National Water Harvesting and Storage Management Policy.* Nairobi: MoWI.

Ministry of Water Development. (1986). *Design manual for water supply in Kenya.* Nairobi, Kenya.

Natural Resources Conservation Service. (October 1990). *Earth Dams and Reservoirs.* Washington D.C: United States Department of Agriculture.

Nelson, K. D. (1985). *Design and construction of small earth dams.* Melbourne, Australia: Inkata Press.

Nelson, R. D. (n.d.). *Investigation, Design and Construction of Small Earth Dams.* British Army.

New England College. (August 1974). Safety of small dams. *Engineering Foundation Conference.* New York: American Society of Civil Engineers.

Nilsson, A. (1988). *Groundwater Dams for Small Scale Water Supply.* London: IT Publications.

Nissen-Petersen, E., & Lee, D. M. (1990). *Harvesting Rainwater in Semi-Arid Africa; Sub-Surface and Sand-Storage Dams.* Nairobi: ASAL Rainwater Harvesting.

Nissen-Petersen, E., & Lee, M. (. (1990). *Harvesting Rainwater in Semi-Arid Africa - Rock Catchment Dam with Self Closing Watertap.* Nairobi: ASAL Rainwater Harvesting.

Novak, P., et al. (2006). *Hydraulic Structures.* London: Taylor and Francis.

Ondieki, C. (1993, June). *The Hydrological Characteristics of the Arid and Semi-Arid Parts of Kenya. Ph.D.* University of Nairobi: Kenya.

Sharma R. K and Sharma T. K. (1992). *Dam Engineering Including water power engineering.* New Delhi: Oxford and IBH Publishing Co. PVT Ltd.

Soil Conservation Service. (July 2005). *Earth Dams and Reservoirs.* Washington D.C: U.S Department of Agriculture.

Special Public works programme. (1988). *Training element and technical guide for Special labour intensive public works booklet no 5, small earth dams.* Geneva: International Labour Organisation.

Stephens, T. (1991). *Handbook of small earth dams and weirs: A guide to siting, design and construction.* Bedford, U.K: Cranfield Press.

Stephens, T. (2010). *Manual on small earth dams: A guide to siting, design and construction.* Rome: Food and Agriculture Organisation.

Technical engineering and design guide as adapted from the U.S army corps of engineers No 27. (1999). *Construction control for earth and rockfill dams.* Reston, Virginia: American Society of Civil Engineers.

The Community Land Bill. (2013). Kenya Gazette Supplement No. 152.

The Constitution of Kenya. (2010).

The Environmental Management and Coordination (Environmental Impact Assessment and Audit) Regulations. (2007). L.N No. 101 of 2003.

The Environmental Management and Coordination (Waste Management) Regulations. (2006). L.N No. 121 of 2006.

The Environmental Management and Coordination (Water Quality) Regulations. (2006). L.N No. 120 of 2006.

The Environmental Management and Coordination Act. (1999).

The Land Act. (2012).

The National Construction Authority Regulations. (2014). Kenya Gazette Supplement No. 84.

The Public Procurement and Disposal Act. (2006).

The Registration of Land Act. (2012).

The Water Act. (2002).

The World Bank. (2011, December). *The World Bank.* Retrieved November 2014, from OP 4.12, Annex A - Involuntary Resettlement Instruments: http://web.worldbank.org

Thomas D.B. (ed). (1997). *Soil and Water Conservation Manual for Kenya.* Nairobi: MALDM.

Thomas, M. K. (2001, July/August Vol. 24). Self Regulating River and Spring Intakes. *Kenya Engineer*.

U.S Army corps of Engineers. (2004). *Earth and Rock dams: General design and construction considerations.* University Press of the Pacific: Honolulu, Hawaii.

U.S Army corps of engineers. (2004). *General design and construction considerations for earth and rock fill dams.* Washington D.C: Department of the Army.

U.S Army Corps of Engineers. (May 1994). *Engineering and design: Arch dam design.* Washington D.C: Department of the Army.

UNICEF, FAO and Oxfam GB. (2012). *A Trainers Manual for Community Based Water Supply Management in Kenya.* Nairobi: UNICEF-Kenya Country Office, FAO & Oxfam GB.

United States Department of the Interior - Bureau of Reclamation. (1987). *Design of Small Dams.* Washington DC: U.S Government Printing Office.

VSF, & TLDP. (2006). *Sub-Surface Dams: A simple, safe and affordable technology for pastoralists.* VSF.

Water Resouces Management Rules. (2006). L.N No. 76 of 2006.

Water Resources Management Rules. (2007). L.N No 171 of 2007.

Water Services Trust Fund. (2011). *Projects Unit Costing Report.* Nairobi: WSTF.

Waweru D.W and Ngigi S. N. (December 1999). *Sand dams and sub-surface dams: Proceedings of a workshop held at the garden hotel Machakos.* Machakos, Kenya: Kenya Rain Water Association.

Weaver, K. and Bruce, D. (2007). *Dam Foundation grouting.* USA: American Society of Civil Engineers.

Wenner, C. (1981). *Soil Conservation in Kenya, Especially in Small Scale Farming in High Potential Areas using Labour Intensive Methods.*

Wolf P. and Ghilani C. (2006). *Elementary Surveying, An introduction to Geomatics eleventh edition.* New Jersey: Pearson Prentice Hall.

Woodhead, T. (1968). *Studies of Potential Evapotranspiration in Kenya.* Nairobi: East African Agriculture and Forestry Research Organization.

World Meteorological Organisation. (1994). Guide to Hydrological Practices; Data Acquisition and Processing, Analysis, Forecasting and Other Applications.

GLOSSARY OF TERMS AND DEFINITIONS

Back-filling: The on-site filling of a trench or similar excavation with suitable material. In dam construction this is usually done on a layer by layer basis accompanied by wetting and compaction.

Berm: A step on the shoulder of an embankment constructed for access or to meet design slope requirements.

Borrow Area: An excavated area or pit (preferably in the basin area) providing suitable material for the embankment construction.

Breaching: Complete removal of a section of the dam embankment and can be due to erosion, foundation movement or excessive flood.

Casing: The shoulder or shell of an embankment.

Catchment Area: The land area upstream of the dam that takes in all streams and rivers that supply the dams.

Catchment Yield: Average annual run-off for the catchment area of the dam.

Cement Wash: A cream-like slurry of cement and water used to seal and assist anchorage of concrete/masonry to rock or to other layers of concrete/masonry.

Clay Blanket: A 'blanket-like' layer of clay laid on the dam or in the basin to seal an area against excessive seepage.

Compaction: An essential process to increase the density of soil by mechanical means to improve its water tightness and structural characteristics.

Core: The central section of the embankment constructed with highly impermeable material to prevent water from moving from the basin through the dam wall. The ground below the section of the core is often referred to as the 'cut-off'.

Crest: The top of the embankment.

Cut-off: Impermeable material placed below ground level so as to impede seepage.

Dam: Barrier developed on a water course for the purpose of retaining water.

Dam-Scoop: A tractor drawn scooper up to 2.0m^3 capacity.

Drop Inlet Overflow: A trickle flow outlet through the embankment just below spillway level and designed to carry small discharges so as to protect the spillway from erosive minor flows.

Erosion: The removal of soil and rock by natural agencies such as rainfall, river and flood flows, undermining or gravity.

Expansion Joint: A sealer filled joint in a concrete weir to minimize cracking and to allow for sealing under pressure.

Flood Flows: Abnormal river flows following excessive rainfall and/or run-off.

Footing: A firm secure foundation for any embankment or weir.

Freeboard: The difference in height between the dam crest and the spillway channel. Estimated at the design stage and with the correct width of spillway must be sufficient to allow for the safe carrying away of flood flows when the basin is full.

Grout curtain: A cement or bentonite slurry pumped under pressure into fractured rock so as to minimise seepage.

French Drain: Below ground rock-gravel drain beneath the downstream shoulder to improve drainage and reduce the seepage levels in the embankment.

Full Supply Level: The maximum water level the dam is designed for. The same as the spillway level. Normally shown on drawings as NWL (Normal Water Level).

Internal drainage system: Arrangement of pervious material placed in such a manner to draw in the phreatic line.

Head Wall: A concrete retaining wall keyed into an embankment or valley side to limit erosion and safely link an earth section to concrete or masonry.

High Flood Level: The water level in the reservoir when the spillway is flowing at its design capacity. Often shown on drawings as FWL (Flood Water Level).

Key Trench: (i) Excavated area to allow 'mating' sections of an embankment to its older counterpart- usually in repair work or when raising the height of the dam. (ii) A trench cut in rock to secure a concrete weir in its foundation.

Key Wall: Masonry or similar wall constructed on firm foundation to anchor the embankment.

Lagoon: Alternative term for a pan.

Masonry: Construction work carried out with stone and concrete. Often required on spillway sides to protect the sensitive areas against erosion.

Normal Water Level: Also referred to as the Full Supply Level this is the maximum water level in the reservoir prior to flood excess.

Outfall: This is the area at the end of the spillway where it discharges to a stream or similar. Erosion often starts here if the outfall has not been properly prepared or maintained.

Overtopping: This is when excessive floods pass over the embankment due to insufficient spillway capacity. Erosion always follows and of severe can lead to major damage.

Pan: Structure created by means of excavation to retain water

Perennial Flow: A stream that flows all year around is said to be 'perennial'. The alternative, where a stream dries up periodically is said to be 'seasonal'.

Piping: Erosion within the embankment due usually to uncontrolled seepage.

Ramming: When erosion occurs on an embankment or spillway, it is remedied by 'ramming' soil and grass into holes or gullies as they develop. The ramming action compacts the soil and assists the bonding of new material to old.

Relief well: Arrangement of pervious material placed in such a way to alleviate pore water pressures

Reservoir: Water impounded by means of a wall or excavation.

Rock Toe: Apex of the downstream shoulder constructed of rock and/or gravel to improve drainage, reduce instability and provide anchorage for the embankment.

Sand Bag: A sack filled with sand, earth and/or cement mixture used in temporary construction work or in protecting parts of the dam from rainfall, run-off or floods.

Seepage: Water moving through or under the embankment is said to be 'seepage'. All dams will 'seep' to some extent and this is not normally serious.

Settlement: Any earth structure, however well built, will settle to some extent. Provision for this should be made at the time of construction (the 'settlement allowance') and maintenance should be allowed for to treat any cracks as they develop.

Slumping: The movement of earth (i.e. through erosion, water-logging or too steep slopes) away from either face of the embankment.

Spillway: The overflow section of the dam catchment constructed to dimensions suitable to safely carry away expected peak floods when the dam is full.

Throwback: The distance between the embankment and the upstream apex of water stored in the reservoir at a required level (usually at Full Supply Level).

Training Bank: Secondary embankment constructed to channel spillway flows and protect the downstream toe area of the dam.

Turbulence: Rapid, irregular flow that highly is erosive. To be avoided in grass or earth spillways by flat slopes and wide, shallow channels.

Water-line: The level of water in the basin ore reservoir is referred to as the water-line or water-level. The maximum water-level possible is the Full Supply Level.

Waterlogging: A soil completely saturated with water is 'water-logged'. The downstream section of the dam can become unstable if allowed to become water-logged – free drainage is therefore important in this area of embankment.

Wing wall: A head wall constructed at the end of the masonry spillway.

APPENDIX A

MAPS

- Annual Rainfall
- Annual Evaporation
- 50 Year Rainfall Intensity 12 Hour Storm
- 100 Year Rainfall Intensity 12 Hour Storm

Note: Additional rainfall intensity maps are available on the web site.

Return Period	Storm Duration							
5 years	10 min	30 min	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour
10 years	10 min	30 min	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour
25 years	10 min	30 min	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour
50 years	10 min	30 min	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour
100 years	10 min	30 min	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour

APPENDIX B

REFERENCE DRAWINGS

- 1/17 Type Drawing I: Small Earth Dam;
- 2/17 Type Drawing II: Spillway for Small Earth Dam (1/2);
- 3//17 Type Drawing II: Spillway for Small Earth Dam (2/2);
- 4/17 Type Drawing III: Intake, Drawoff and Valve Chamber for Small Earth Dam;
- 5/17 Type Drawing IV: Cattle Trough (1/2);
- 6/17 Type Drawing IV: Cattle Trough (2/2);
- 7/17 Type Drawing V: Communal Water Point;
- 8/17 Type Drawing VI: Water Storage Pan, Capacity 11,600 Cubic Metres (1/2);
- 9/17 Type Drawing VI: Water Storage Pan, Capacity 11,600 Cubic Metres (2/2);
- 10/17 Type Drawing VII: Water Storage Pan, Capacity 20,000 Cubic Metres (1/2);
- 11/17 Type Drawing VII: Water Storage Pan, Capacity 20,000 Cubic Metres (2/2);
- 12/17 Type Drawing VIII: Water Storage Pan on Inclined Location, Capacity 17,000 Cubic Metres, Dug Well and Filter (1/2);
- 13/17 Type Drawing IX: Standard Cross Sections Concrete Sand Storage Dams;
- 14/17 Type Drawing X: Gabion Sand Storage Dam (1/2);
- 15/17 Type Drawing X: Gabion Sand Storage Dam (2/2);
- 16/17 Type Drawing XI: ALDEV Weir Cross Section;
- 17/17 Type Drawing XII: Lagoon on an Inclined Site, 38,000 Cubic Meters.

Note: Additional drawings (Autocad and PDF) available on the web site.