# THE UNITED REPUBLIC OF TANZANIA

MINISTRY OF WATER | GOVERNMENT CITY
MAJI STREET | P. O. BOX 456
DODOMA

MKUKI NA NYOTA
WWW.MKUKINANYOTA.COM

ISBN 978-9987-08-432-6
9 789987 084326

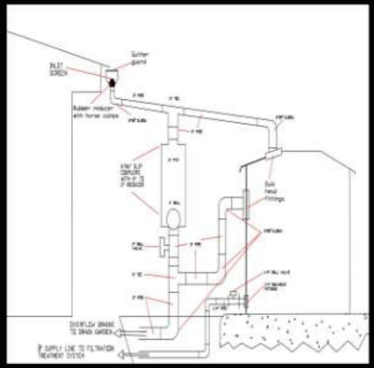
DCOM Manual Volume I

THE UNITED REPUBLIC OF TANZANIA
MINISTRY OF WATER

# | DESIGN |
# | CONSTRUCTION SUPERVISION |
# | OPERATION & MAINTENANCE |
# (DCOM) MANUAL

## VOLUME I

## DESIGN OF WATER SUPPLY PROJECTS



MAY 2020

4TH EDITION

# THE UNITED REPUBLIC OF TANZANIA

# MINISTRY OF WATER

## DESIGN, CONSTRUCTION SUPERVISION, OPERATION AND MAINTENANCE (DCOM) MANUAL FOURTH EDITION

## VOLUME I

## DESIGN OF WATER SUPPLY PROJECTS

**Edited by Ninatubu Lema,
Mengiseny Kaseva and William Sabaya**

**PROJECT PREPARATION, COORDINATION AND DELIVERY UNIT (PCDU)**

Published for
Ministry of Water
Project Preparation, Coordination and Delivery Unit (PCDU)

Government City
Maji Street
P. O. Box 456
Dodoma

Tel +255 26 2322602/+255 26 2322602

Website: www.maji.go.tz
Email: ps@maji.go.tz

Published by:
Mkuki na Nyota Publishers Ltd

Publication Title:
Design, Construction Supervision, Operation and Maintenance (DCOM) Manual Fourth Edition
Volume I: Design of Water Supply Projects



ISBN 978-9987-084-32-6

# PREFACE

The Government of the United Republic of Tanzania, through the Ministry of Water, oversees the implementation of the Water Supply and Sanitation projects in the country. The Ministry of Water has published several editions of the relevant Design Manuals. The First edition was the Water Supply and Waste Wastewater Disposal Manual of 1985/86. The Second edition was titled “Design Manual for Water Supply and Wastewater Disposal of 1997”. The Third edition was titled “Design Manual for Water Supply and Wastewater Disposal of 2009”. These manuals guided the Ministry and the general public in the planning and design of water supply and sanitation projects in the country.

As it is now well over ten years since the Third Edition of the Design manual was adopted, and since many scientific and technological changes have taken place, including the conclusion of MDGs and adoption of the SDGs in 2015 as well as useful lessons learnt out of implementation of the WSDP I and WSDP II (which is still on-going), it has become necessary to revise the 2009 design manual. Notably, the 3$^{rd}$ Edition Design Manual has, among other things, limited coverage on the impact of climate change, application software and sanitation management issues.

The Ministry is now at various stages of instituting policy and legal reforms that are deemed necessary for futuristic improvement in the design, construction supervision, operation and maintenance of water supply and sanitation projects in Tanzania. Therefore, the 4$^{th}$ Edition of the Design, Construction Supervision, Operation and Maintenance (DCOM) Manual will make invaluable contribution in this regard. It is important to recall that the Government has established the Rural Water Supply and Sanitation Agency (RUWASA), which is responsible for the supervision, execution and management of rural water supply and sanitation projects. RUWASA is expected to improve the existing responsibility and accountability in the management of water and sanitation services in rural areas. The 4$^{th}$ Edition DCOM Manual will support the sector development and implementation institutions (including RUWASA, Water Supply and Sanitation Authorities, development partners, and civil society organisations), and will provide valuable information relating to implementation of water supply and sanitation projects in their various stages, from pre-feasibility and feasibility studies, to planning, designing, construction supervision and operation and maintenance.

It is expected that the 4$^{th}$ Edition of the DCOM Manual will position the Ministry well to systematically and comprehensively implement the design, construction supervision, operation and maintenance of water supply and sanitation projects in order to ensure the sustainability of water supply and sanitation projects in the country. This is also expected to contribute in realising the water sector's contribution towards achieving the Tanzania Development Vision 2025, as well as the various national and international commitments and milestones in the water sector as also specified in the Agenda 2063 in the "Africa that we want" and the Sustainable Development Goals (SDGs) on water and sanitation (SDG No. 6).

The preparation of this Water Supply and Sanitation Projects DCOM Manual required contributions in form of both human and financial resources. The Ministry of Water, therefore, takes this opportunity to thank the members of the Special Committee for Reviewing and Updating the $3^{rd}$ Edition of the Design Manual for Water Supply and Wastewater Disposal of 2009, specifically for their efforts in preparation of this comprehensive $4^{th}$ Edition of the DCOM Manual. Thanks are also due to the World Bank for financing the major part of the activities, and to all others who contributed in the preparation of this new DCOM Manual.

In the future, the Ministry plans to periodically review and update the DCOM Manual in order to keep in pace and address emerging changes in policy and societal needs, emerging technologies, and sustainability concerns in the implementation of water supply and sanitation projects in the country.

Prof. Makame Mbarawa (MP)
**Minister**
**Ministry of Water**

**May, 2020**

## ACKNOWLEDGEMENTS

Changes of policy and technology have necessitated the preparation of this new edition of the DCOM Manual for the design, construction supervision, operation and maintenance of water supply and sanitation projects in Tanzania. The 4th Edition of the DCOM Manual is expected to guide engineers and technicians in their design work, construction supervision as well as in operation and maintenance of relevant projects. The manual is to be adopted for all water supply and sanitation projects in the country.

The 4th Edition of the DCOM Manual has been developed using the following approaches:

- Review of the 3rd Edition, including benchmarking with design manuals from other countries,
- Website reviews and review of other manuals prepared by consultants who have had working experience in Tanzania,
- Review of Literature data collection and design methods review,
- Data collection from stakeholders, namely: Primary stakeholders-MoW technical and management staff; Private companies that deal with implementation of water supply and sanitation projects; Beneficiaries of water supply and sanitation projects,
- Collection and digitization of existing standard drawings after conversion into metric units as felt necessary,
- Review of the 4th Edition drafts by various stakeholders including MoW staff and other stakeholders outside the MoW,
- Revision of the 4th Edition by incorporating comments and views from all the stakeholders,
- Preparation and submission of the 4th Edition of the DCOM Manual.

The review and updating of the 3rd Edition of the DCOM Manual is considered to be a continuous process involving regular updating to incorporate changes in policy and societal needs, emerging issues, technologies or methods. The MoW welcomes comments on this new edition of the DCOM Manual from users in order to facilitate further improvement of future editions.

The new features in the 4th Edition of the DCOM Manual include mainstreaming of climate change impacts and use of various types of software in the design of water supply and sanitation projects. These features have facilitated the faster and more accurate analysis of pertinent data. The DCOM manual has also

encouraged the use of Supervisory Control and Data Acquisition Systems (SCADA) for large urban and generally national projects where local capacity building can be guaranteed by the providers. It should be borne in mind that relevant software allows a wide variety of scenarios to be considered. However, it should also be noted that, despite the critical role of software/models in guiding decision-making, its limits should be realised so as to avoid its becoming a substitute for critical practical evaluation.

I wish to thank the different stakeholders for their active participation and support in contributing towards the various inputs during the course of preparation of this DCOM Manual. They include those from within and outside the Ministry of Water as well as Development Partners, NGOs, Consultants, Suppliers and Contractors as well as other Ministries. The review team of engineers and technicians from MoW, RUWASA, WSSA who worked with the Special Committee for three days in March 2020 are hereby gratefully acknowledged.

Finally, I take this opportunity to thank the members of the Special Committee on Reviewing and Updating the 3rd Design Manual of 2009 under the Chairmanship of Eng. Prof. Tolly S. A. Mbwette for diligently undertaking this assignment.

Prof. Kitila Mkumbo
**Permanent Secretary**
**Ministry of Water**

**May, 2020**

# MEMBERS OF THE SPECIAL COMMITTEE ON THE REVIEW AND UPDATING OF THE 3RD EDITION OF THE DESIGN MANUAL FOR WATER SUPPLY AND WASTEWATER DISPOSAL OF 2009

| NAME OF MEMBERS | | POSITION | INSTITUTION | SIGNATURE |
|---|---|---|---|---|
| 1. Eng. Prof. Tolly S. A. Mbwette | - | Chairperson | UDSM | |
| 2. Eng. Alex J. Kaaya | - | Deputy Chairperson | Consultant | |
| 3. Eng. Dr. Deogratias Mulungu | - | Secretary | UDSM | |
| 4. Eng. Christer Mchomba | - | Member | MoW | |
| 5. Eng. Prof. Karoli N. Njau | - | Member | NM-AIST | |
| 6. Eng. Amos Mtweve | - | Member | RUWASA | |
| 7. Eng. Dr. Richard Kimwaga | - | Member | UDSM | |
| 8. Dr. Juma Hemed Lungo | - | Member | UDSM | |
| 9. Mr. Issa Mlimbila | - | Member | MoW | |
| 10. Mr. Fadhili Saulo Ollo | - | Member | RUWASA | |
| 11. Mr. Alex N. George | - | Member | MoW | |
| 12. Eng. Masoud Almasi | - | Member | MoW | |

**Dodoma May, 2020**

# TABLE OF CONTENTS

## CHAPTER TWO
## PROJECT PLANNING . . . . . 17

**CHAPTER EIGHT**

## LIST OF FIGURES

# LIST OF TABLES

## LIST OF ABBREVIATIONS

| | |
|---|---|
| ATAWAS | Association of Tanzania Water Suppliers |
| CBWSO | Community Based Water Supply Organisations |
| COWSOs | Community Owned Water Supply Organizations |
| CMIP5 | Coupled Model Inter-comparison Project Phase 5 |
| DAWASA | Dar es Salaam Water Supply and Sanitation Authority |
| DEWAT | Decentralized Wastewater Treatment Systems |
| DCOM | Design, Construction Supervision, Operation and Maintenance |
| EIA | Environmental Impact Assessment |
| EIS | Environmental Impact Statement |
| EMA | Environmental Management Act |
| ENSO | El Nino-Southern Oscillation |
| ERB | Engineers Registration Board |
| EWURA | Energy and Water Utilities Regulating Authority |
| GCMs | Global Climate Models |
| GHGs | Greenhouse gases |
| IHP | International Hydrological Programme |
| ICOLD | International Commission on Large Dams |
| ITCZ | Inter-tropical Convergence Zone |
| IOD | Indian Ocean Dipole |
| IPCC | Intergovernmental Panel on Climate Change |
| IWA | International Water Association |
| NAWAPO | National Water Policy |
| NEMC | National Environmental Management Council |
| NMAIST | Nelson Mandela African Institution of Science and Technology |
| NWSDS | National Water Sector Development Strategy |
| RCPs | Representative Concentration Pathways |
| RUWASA | Rural Water Supply and Sanitation Agency |
| SADC | Southern Africa Development Community |
| SDGs | Sustainable Development Goals |
| SST | Sea Surface Temperature |

## LIST OF ABBREVIATIONS

| | |
|---|---|
| UNESCO | United Nations Educational, Scientific and Cultural Organisation |
| UNFCC | United Nations Framework Convention on Climate Change |
| UWSSA | Urban Water Supply and Sanitation Authority |
| UDSM | University of Dar es Salaam |
| URT | United Republic of Tanzania |
| TAWASANET | Tanzania Water Supply and Sanitation Network |
| WMO | World Meteorological Organization |
| WSDP | Water Sector Development Programme |

Chapter 1

# INTRODUCTION

The preparation of this DCOM manual was preceded by an overview of five important global considerations of Water Supply and Sanitation prior to reviewing the water and sanitation sector in Tanzania. This was followed by an explanation of the rationale for the preparation of the 4th edition. The introductory chapter is concluded by presenting the organization of the manual as well as the purpose and content of this volume of the DCOM manual.

## 1.1 GLOBAL CONSIDERATIONS ON WATER SUPPLY AND SANITATION

### 1.1.1 Sustainable Development Goals (SDGs)

In 2015, world leaders convened at the United Nations Headquarters in New York and adopted the 2030 Agenda for Sustainable Development. Governments responded to the common development challenges then faced and to the changing world around them by uniting behind a truly forward-looking, yet urgent plan to end poverty and create shared prosperity on a healthy and peaceful planet. The central principle of Agenda 2030 is leaving no one behind in achieving the 17 SDGs through 169 targets.

The 2030 Agenda for Sustainable Development adopted at the UN Summit includes SDG 6 on *Water and Sanitation* and in December 2016, the United Nations General Assembly unanimously adopted the resolution "International Decade for Action-Water for Sustainable Development" (2018–2028) in support of the achievement of SDG 6 on water and sanitation and the related targets (United Nations, 2015). It should also be noted that, water and sanitation are at the heart of the Paris Agreement on climate change 2015 (UNFCC (2015).

Ensuring availability and sustainable management of water and sanitation for all has therefore been, for a long while, an important topic at the United Nations and is now turning this vision into a reality, through national leadership and global partnerships. Water and sanitation are at the core of sustainable development and the range of services they provide, underpin poverty reduction, economic growth and environmental sustainability. The world needs to transform the way it manages water resources and the way it delivers water and sanitation services for billions of people.

The designers and engineers, therefore, have the responsibility to support the Government of Tanzania in achieving the SDG 6, where population growth and rapid urbanisation have intensified demand for water and sanitation services beyond all past thresholds.

### 1.1.2 Climate Change and Resilience to Climate Change

Climate change is now recognized as one of the defining challenges for the 21st century. More frequent, intense and extreme weather events continue to result in higher incidences of floods and droughts around the planet. The ensuing adverse impacts of climate change on water and sanitation services constitute a serious threat to human health and overall development of nations. Ensuring optimal resilience of water and sanitation services in a globally changing climate context will continue to be crucial for maintaining the momentum of making progress in health and general socio-economic development. Climate variability is already a threat to the sustainability of water supplies and sanitation infrastructure.

Flood occurrences continue to cause shocks for affected populations and to challenge water and sanitation managers. In many places floods are likely to become more frequent with intensification of climate change, thus;

- Floods can have catastrophic consequences for basic water and sanitation infrastructure. Such damages can take years to repair.
- On a smaller scale, drinking-water infrastructure can be flooded and put out of commission for days, weeks or months.
- Where flooding of sanitation facilities occurs, there may not only be a break in services, but the resultant flooding may distribute human excreta and its attendant health risks across entire neighbourhoods and communities.

Droughts occur unpredictably worldwide. In many places they are likely to become more frequent and more widespread with climate change. For example: Falling groundwater tables and reduced surface water flows can lead to wells drying up, extending distances that must be travelled to collect water, and increasing water source pollution. In response, drilling rigs, which would otherwise be used to increase access, may be redeployed to renew or replace out-of-service wells, slowing the actual progress in extending access.

Since climate change is likely to affect water sources and infrastructure in Tanzania, it must therefore be taken into consideration (i.e. ensure enhanced adaptation capacity) in design, operation and maintenance of water and sanitation infrastructure or projects. Globally, climate change studies are coordinated by the United Nations Framework Convention on Climate Change (UNFCCC) and the Inter-Governmental Panel on Climate Change (IPCC). Accordingly, designers should use the latest information, data and model predictions available and include statements on what measures, if any, have been allowed for in order to cope up with (or adapt to) climate change within the time frame of pertinent project design (i.e. design period).

### 1.1.3 Public Private Partnership in Water Supply and Sanitation Projects in Developing Countries

One of the key challenges faced by water authorities in Developing Countries (DC) is how best to manage service delivery obligations to rural communities. Even in decentralized sectors, water authorities may find it hard to provide services to remote rural communities. It is recognized that water user associations and/or local private operators may be best placed to provide services as they are close to the users. The majority of the agreements are currently in place in the short term (1 to 3 years) management or operation and maintenance contracts for existing systems that involve minimal investment from the private sector. One key issue that arises repeatedly though is how to effectively regulate and monitor performance of activities under these contracts.

Globally, activities undertaken in 2005 suggest that private participation in the water sector is entering a new phase. New private firm involvement is continuously focusing on smaller projects and bulk facilities. Contractual arrangements involving utilities are combining private operations with public financing and new players are entering the market.

In an infrastructure-intensive sector, improving access and service quality to meet the SDGs cannot be done without massive investment. Around the developing world, the water sector is chronically under-funded and inefficient in addition to giving low priority to sanitation. In this context, Public-Private Partnerships (PPPs) can be a mechanism (among others) to help Governments in funding the much needed investment and deploying technologies and efficiency that can improve the performance and financial sustainability of the water and sanitation sector.

Governments are currently using private firms in the water and sanitation sector increasingly to finance and operate bulk water supply and wastewater treatment. New technologies and innovations such as desalination and wastewater re-use are currently being increasingly introduced, where traditional water sources have become scarce. Utilities are drawing on specific expertise, such as Non-Revenue Water (NRW) reduction and pressure management, to promote efficiency and improvement of services. Private investors and providers are increasingly becoming local and regional, and so raising competition and pushing down charges.

Most utilities are increasingly turning to the private sector for turnkey solutions to the designing, building and operating water and wastewater treatment plants, and in some cases they also provide financing. With new technologies such as membrane filtration and in wastewater treatment; utilities have faced challenges in finding the capacity to operate and maintain these facilities and in selecting the most appropriate technology.

Where a utility has the funds or is seeking financing to develop water or wastewater treatment plants but wishes to draw on the private sector to Design, Build and Operate (DBO) a facility, then the DBO approach is used.

The International Financial Institutions (IFIs) are being asked to finance such approaches. In response, the WB has recently developed a suite of documents for DBO deployment in water and sanitation projects, including an initial selection document; a Request for Proposal (RFP) with DBO document based on The International Federation of Consulting Engineers (FIDIC), an acronym for its French name *Fédération Internationale Des Ingénieurs-Conseils*) Gold Book and a guidance note with guidance on when the DBO approach is appropriate and how to approach such projects; draft framework for Employer Requirements and draft Terms of Reference for Consultancy support to carry out the requisite studies and develop the documents (World Bank, 2010).

### 1.1.4 International Water Law

The URT is riparian to the following trans-boundary International River Basins: Congo River Basin, Kagera River Basin, Nile River Basin and Zambezi River Basin. These water sources are managed using international law on trans-boundary resources.

International law is a culture of communication that "constitutes a method of communicating claims, counter-claims, expectations and anticipations, as well as providing a framework for assisting and prioritizing such demands" (Shaw, 2008). International water law is the law of non-navigational uses of international watercourses.

In international water law, there are two substantive principles that ought to be taken into consideration when sharing international waters:

- The principle of *equitable utilization* which is a more subtle version of the doctrine of absolute sovereign territory. It argues that a (nation) state has absolute rights to all water flowing through its territory.
- The principle of *no significant harm* is the delicate version of the doctrine of both absolute riparian integrity (every riparian state is entitled to the natural flow of a river system crossing its borders) and historic rights (where every riparian state is entitled to water that is tied to a prior or existing use) (Wolf, 1999).

There are two relevant international water conventions for trans-boundary water cooperation. The 1997 Convention on the Law of the Non-navigational Uses of International Watercourses (i.e. UN Watercourses Convention, 1997), and the 1992 UNECE Convention on the Protection and Use of Trans-boundary Watercourses and International Lakes (i.e. UNECE Water Convention, 1992) which recently broadened its membership beyond the EU to a global audience. In March 2016, Water Convention became a global multilateral legal and Inter-Governmental framework for trans-boundary water cooperation that is open to accession by all UN member states. The soft law of the SDGs provides further impetus to the management of trans-boundary water resources directly through Goal 6.5: "*Implement integrated water resources management at all levels, and* *through trans-boundary cooperation as appropriate*", and indirectly through Goal 16: "*Promote peaceful and inclusive societies for sustainable development*". In this case, the contribution of designers and engineers is in the provision of tools and information or data to support the needed decision making.

The management of water resources that entails extraction of shared international water resources in the form of rivers, lakes, seas and oceans as sources are guided by International Conventions and/or Protocols that have to be subsequently ratified by respective national Parliaments before they become enforceable. Because Tanzania is a member of the EAC, SADC and the African Union, it has ratified a number of the conventions and/or protocols that are associated with water resources management and water supply and sanitation services. At an African level, Tanzania fully subscribes to the *Agenda 2063* that ensures African development is guided by African experts to attain the aspirations of "The Africa that we want" with respect to water supply and sanitation services. Furthermore, as a member of the United Nations, Tanzania's water supply and sanitation services are guided by the UN SDGs of 2015 as well as the UNFCCC (2015) as mentioned earlier on.

## 1.2 DEVELOPMENT AGENDA AND WATER AND SANITATION SECTOR IN TANZANIA

The Tanzania Development Agenda includes the Tanzania Development Vision (TDV) 2025). The realization of TDV is carried out through Five Year Development Plans. Currently, the GoT is implementing the Second Five Year Development Plan (FYDP II), 2016/17 – 2020/21.

The Government adopted the TDV in the mid-1986s for socio-economic reforms and the same continues to be implemented to date. Better and improved water and sanitation services contribute to one of the attributes of Vision 2025, which is on high quality livelihood. Thus, the review and update of this manual better shapes the future in which water and sanitation services will be delivered to enhance the health and improved livelihoods of normal citizens who are a critical national labour force.

The FYDP II has integrated development frameworks of the first Five Year Development Plan (FYDP I, 2011/2012-2015/2016) and the National Strategy for Growth and Reduction of Poverty (NSGRP/MKUKUTA II, 2010/2011-2014/2015) further extended to 2015/2016 - 2019/2020. The FYDP II is built on three pillars of transformation: industrialization, human development, and implementation effectiveness, and is aligned to the relevant SDGs. Importantly, industrialization placeshigh demand on utility supplies e.g. energy and water, so subscribing on addressing the SDG Goals 6: on water and sanitation.

Chapter 4 of the FYDP II, sub-chapter 4.3.4 on Water Supply and Sanitation Services sets key targets by 2020: Access to safe water in rural areas, 85%; regional centres and Dar es Salaam, 95%. Proportion of rural households with improved sanitation facilities, 75%; regional centres, 50% and Dar es Salaam, 40%. Non-revenue water (NRW) for regional centres, 25%; for Dar es Salaam, 30%. The Key targets by 2025 are: Access to safe water in rural areas, 90%; regional centres and Dar es Salaam, 100%. Proportion of rural households with improved sanitation facilities, 85%; regional centres, 70% and Dar es Salaam, 60%. Non-revenue water (NRW) for regional centres, 20%; for Dar es Salaam, 25%. One of the tools towards achieving the key targets of water supply and sanitation is the effective application of the DCOM manual.

The Government has a comprehensive framework for sustainable development and management of water resources where there is an effective policy, legal and institutional framework. The water sector policy and strategy contains operational targets to be achieved in terms of coverage and timescale for improving water resources management, water supply and sanitation. The targets are reflected in the National Water Sector Development Strategy (NWSDS) of 2006. Based on the targets of the ruling party manifesto on water coverage for rural areas and urban areas are 85% and 95% by 2025, respectively which are also articulated in the WSDP.

In the context of water supply and sanitation services in Tanzania Mainland, the Water Supply and Sanitation Authorities (WSSAs), in collaboration with Rural Water Supply and Sanitation Agency (RUWASA), are responsible for management of water supply and sanitation services mostly in the urban, towns and rural areas as well as in areas that used to be managed by National Water Utilities. The water sector status report of 2017/18 has set water coverage targets of 95% for Dar es Salaam, 90% for other WSSAs and rural areas, 85%.

The Community Based Water Supply Organisations (CBWSOs) are the basic units responsible for management of water supply and sanitation services in rural areas under the overall coordination of RUWASA. The WSSAs are regulated by the Energy and Water Utilities Regulating Authority (EWURA), while CBWSOs are regulated by the RUWASA under the Ministry of Water that is in turn responsible for rural water supply and sanitation services in Tanzania. As part of the on-going reforms in the MoW, a number of small WSSAs have been clustered with urban WSSAs leading to reduction of WSSAs from 130 to 71. RUWASA has been charged with the task of supervising the operations of 50 small town WSSAs in addition to the CBWSO managed projects.

The regulatory role of WSSAs is provided by the Energy and Water Utilities Regulatory Authority (EWURA) and to some extent by RUWASA. With regard to sanitation, the water sector status report 2017/18 has estimated an average coverage of sewerage systems to be 30% (2018) in urban areas. On sanitation achievements, the same report indicates that by 2018, safely managed sanitation was available to only 21.2% of the population compared to the target of 25%. When this is compared to the SDG target of 100% by 2030, it can be seen that Tanzania is lagging behind by far.

### 1.2.1 National Water Policy

The National Water Policy (NAWAPO) of 2002 guides the management of the water sector in Tanzania with major emphasis being on the active participation of communities, the private sector and the local governments in protecting and conserving water sources, supplying water and management of water and sanitation infrastructure. Currently, the review of the NAWAPO is at fairly advanced stages.

The main objective of the National Water Policy of 2002 was to develop a comprehensive framework for sustainable development and management of the Nation's water resources, in which an effective legal and institutional framework for its implementation was put in place. The policy aimed at ensuring that water beneficiaries participate fully in planning, construction, operation, maintenance and management of community based domestic water supply schemes. This policy sought to address cross-sectoral interests in water, watershed management and integrated and participatory approaches for water resources planning, development and management. Also, the policy laid a foundation for sustainable development and management of water resources in the changing roles of the Government from service provider to that of coordination, policy and guidelines formulation, and regulation. Other objectives of the water policy included: increasing the productivity and health of the population through the assurance of improved water supply and sanitation services to the water users and to identify and preserve water sources.

### 1.2.2 Legal and Institutional Framework for Water Supply and Sanitation Services

Basically, the water and sanitation sector is governed by two main broad legal frameworks namely:

I. Water Resource Management Act No.11 of 2009
II. Water Supply and Sanitation Act No. 5 of 2019.

In the institutional framework, there are several organs under the Ministry of Water, which coordinate water supply and sanitation delivery service: the Directorate of Program Preparation, Coordination and Delivery Unit (PCDU), Directorate of Water Resources Management, Basin Water Boards (BWBs), Directorate of Water Supply and Sanitation, Directorate of Water Quality Services, Rural Water Supply and Sanitation Agency (RUWASA) and Water Supply and Sanitation Authorities (WSSAs). Special attention is hereby paid to RUWASA as, in collaboration with respective regional or district authorities is responsible for planning and managing, and supervising the rural water supply and sanitation projects, including financial and procurement management, as well as monitoring and evaluation for contracting consultants and local service providers to assist with planning and implementation of the projects at the district level and in the communities.

Through implementation of WSDP I and II (up to 2019) projects, the role or participation of the beneficiaries in planning, construction, operation, maintenance and management of community based domestic water supply schemes was guaranteed in most of the implemented projects through establishments of COWSOs in every completed project that was given all the mandate of making sure the project is sustainable. Among the lessons learnt from the implementation of WSDP I & II projects was the need for engineers and consultants to use the MoW Design manuals in order to reduce or eliminate the many design flaws already observed.

However, according to the Water Supply and Sanitation Act No. 5 of 2019, the COWSOs were replaced by CBWSOs and these are expected to have the frontline responsibility for sustaining rural water supply and sanitation services on behalf of the beneficiaries (communities). The members of CBWSOs are drawn from the users but their qualifications and experiences have been better specified under the Act No.5. The minimum qualifications of the technical staff employed by CBWSOs has also been explicitly specified to ensure they have the requisite capability and experience. Their roles as well as the assumed responsibility of CBWSOs are also explicitly highlighted in the Act No.5 as well as the roles of RUWASA at different levels.

### 1.2.3 Coverage and Access to Water Supply Services

While the responsibility for provision of sanitation services in rural areas is principally under the Ministry of Health, Community Development, Gender, Elderly and Children (MoHCDGEC); following enactment of the Water and Sanitation Act No. 5 of 2019, RUWASA has also been given some responsibility to coordinate delivery of sanitation services in areas that are under its jurisdiction. In areas served by former National Project Water Utilities (WSSA), it is expected that the MoHCDGEC will liaise closely with both the latter and RUWASA to deliver sanitation services. It is estimated that by 2019, on average 21.2% of Tanzanians had access to safely managed sanitation (MoW AGM, 2019) against a National target of 25%.

### 1.2.4 Policy Environment for Water and Sanitation Services in Tanzania

The management of water resources in Tanzania is guided by the National water policy of 2002 (URT, 2002) that has been in use over the last 18 years and was further articulated by the National Water Sector Development Strategy of 2006 - 2015 (URT, 2008) and the WSDP of 2006-2025. There are currently efforts to update the national water policy by the Ministry responsible for Water. The most important national legislation guiding water resources management include the Water Resources Management Act No.11 (URT, 2009) and all subsequent amendments as well as the various regulations prepared by the Ministry responsible for Water.

The Water Supply and Sanitation Act No.5 (URT, 2019) and the associated regulations prepared by the Ministry responsible for Water guide the development of water supply and sanitation services in Tanzania. The users of this manual are referred to the URT website for further information. As regards sanitation, the Public Health Act of 2009 and The Health Policy of 2007 provide the relevant legal guidance. Other relevant guiding documents include The National Guidelines for Water, Sanitation and Hygiene for Tanzania Schools (MoEST, 2016), National Guidelines for Water, Sanitation and Hygiene in Health Care Facilities (MoHCDGEC, Oct. 2017), Guidelines for the Preparation of Water Safety Plans (MoW, Oct. 2015), National Guidelines on Drinking Water Quality Monitoring & Reporting (MoW, Jan. 2018) and Guidelines for the Application of Small-Scale, Decentralized Wastewater Treatment Systems; A Code of Practice for Decision Makers (Mow, Dec. 2018). Another Swahili document is titled "*Mwongozo wa Ujenzi wa Vyoo Bora na Usafi wa Mazingira*" (Guidelines for Construction of Toilets and Sanitation), (MoHCDGEC, 2014).

### 1.2.5 Major Stakeholders in Water Supply and Sanitation Projects

Effective and efficient implementation of water supply and sanitation projects will be achieved through the contribution of a number of stakeholders. The stakeholders of significant importance are described below.

(a) **Regulatory Authorities**

In order to ensure the smooth implementations of water supply and sanitation projects, various regulatory authorities have been established from time to time. The latter, monitor professional conduct of the different parties involved in water and sanitation projects. These include:

(i) Public Procurement Regulatory Authority (PPRA),
(ii) Tanzania Bureau of Standards (TBS),
(iii) Engineers Registration Board (ERB),
(iv) Contractors' Registration Board (CRB),
(v) Energy and Water Utilities Regulating Authority (EWURA),
(vi) The National Environmental Management Council (NEMC).

(b) **Contractors and Consultants**

*Contractors* are the firms that perform the actual construction of the water projects according to the agreed terms in the contracts. *Consultants/Project Managers* are firms that design water supply and sanitation projects and supervise the construction works depending on the terms and conditions specified in their respective contracts. Moreover, the consultant, on behalf of the client, approves completed structures with regards to the specifications given and the standards required as elaborated in chapter twelve of Volume I of the DCOM manual

(c) **National Water Supply and Sanitation NGOs and networks**

The following is a sample list of Non-Governmental Organizations (NGOs) that deal with water supply and sanitation services in Tanzania and hence have a contributing role to the Ministry of Water (MoW):

(i) Association of Tanzania Water Suppliers (ATAWAS),
(ii) Tanzania Water Supply and Sanitation Network (TAWASANET),
(iii) Tanzania Global Water Partnership (GWPTZ).

### 1.2.6 Water Supply and Sanitation Public-Private Partnership in Tanzania

The national water policy (NAWAPO) of 2002 (URT) envisaged devolution elements to be introduced as well as public and civil service reforms. It had assumed that the Central Government would provide technical and financial support, coordination and regulation of water supply development while the private sector was expected to support the communities in planning, design, construction and supply of materials, equipment, spare parts and to support operations in some cases. The Development Partners (DPs), NGOs and CBOs were expected to provide funding and technical assistance to supplement the Government’s efforts through basket funding.

In support of the Government the Public-Private Partnership (PPP) policy of 2009 as also supported by EWURA which prepared the PPP guidelines for water supply and sanitation (EWURA, 2017) and the relevant legislation that was stipulated in NAWAPO 2002, the MoW has created the necessary environment for supporting the private sector such that, a sizeable proportion of the works, services and goods are procured from private sector Service Providers (SPs) hence assisting the Government in fulfilling its roles.

Essentially, one of the successes of NAWAPO 2002 has been the inclusion of the private sector in water supply and sanitation projects implementation. Notwithstanding the good experiences, the MoW (2018) indicated that even though the Water Sector Development Programme (WSDP) Project Implementation Manual gave a lot of opportunities to the private sector that procured most of the works, field experience has shown that the capacity of the private sector in Tanzania is limited in terms of having only a few staff and thereby failing to supervise the works properly.

On the other hand, the Ministry of Water organized a forum on enhancing public private partnership in the water sector. This was held in Dar es Salaam from 19 to 20 July 2018. In this forum, discussions were held with the private sector stakeholders where experiences, challenges and recommendations were obtained with regard to implementation of rural water supply projects in Tanzania. The forum was a follow up of the Five-Year Development Plan (FYDP) 2016/17-2020/21. The fourth priority area of the FYDP is strengthening project implementation effectiveness, which earmarked water supply and sanitation one of the key interventions for achievement. In the forum, the following key issues were captured:

(a) Contract management issues such as delays in decision making by the client,
(b) Payment problems,
(c) Procurement problems,
(d) Policy issues on Tax exemption for imports,
(e) Political interference in the execution of works,
(f) Knowledge gap on current technology available for groundwater exploration based on quality and quantity of water,
(g) Shortage of contractors with capacity to execute water supply projects,
(h) Database issues especially on water resources information, which often end up with over- or under- designing water supply facilities.
(i) Design specifications based on the use of obsolete technologies was identified as a critical problem.

Privatization of some or all functions of Operation and Maintenance can be considered to achieve: (i) efficiency (ii) economy (iii) professionalism and (iv) financial viability of the system. In order to achieve the above stated objectives, the private entrepreneur needs to possess: (i) adequately trained, qualified staff for operation and supervision of the services (ii) equipment, material, testing and repairing facilities (iii) experience in operating similar systems (iv) financial soundness (v) capacity to meet the emergency situations.

In order to assist service providers/operators in ensuring financial viability of their projects through Public-Private Partnerships, the following were recommended:

(a) The MoW, through the established in-house Design Unit should provide an option for on demand engagement of the private sector at the project level, in cases where in-house capacity or technology is limited;
(b) Enhancement of awareness on other operational modes in PPP as per water policy;
(c) Where applicable, private operators should be engaged in operation and maintenance of water supply and sanitation services after due diligence; The same applies to contracting personnel with specialized skills for the repair and maintenance of specialized equipment or instrumentation as specialized services for maintenance of such equipment instead of employing additional staff. Such a practice may ensure proper functioning of the equipment with least cost;

Private operators should be supervised closely to avoid challenges in operation and maintenance of water supply and sanitation projects (i.e. water supply connections, facilities and finances).

## 1.3 RATIONALE FOR THE PREPARATION OF THE FOURTH EDITION DCOM MANUAL

The need to review and update the 2009 Design Manual was emphasised during the Private-Public Partnership (PPP) stakeholders' meeting hosted by the MoW in 2018. During that meeting, the issue of providing designs/specifications that use old technologies in procurement was indicated as a concern as well as stressing the need to adopt the latest and appropriate technology. Among the Recommendations of the Special Committee on Audit of WSDP I & II projects in rural areas in Tanzania (URT, Nov. 2018), the need to review and update the design manual and to ensure that all consultants use it was emphasized. The four volumes of the DCOM manual have been prepared in order to facilitate effective complimentary planning, design, construction supervision as well as operation and maintenance of water supply and sanitation projects for urban, peri-urban and rural areas of Tanzania.

The manuals will also assist the staff of the Ministry responsible for water and sanitation projects to effectively undertake their supervisory and coordination roles and the consultants to undertake designs using the guidelines recommended in the MoW manual. For Urban and National WSSA or RUWASA staff who may be involved in design, construction supervision of projects using the *Force Account* mode of implementation, the four manuals will prove to be useful in facilitating step by step supervision.

On the other hand, for staff who will be implementing water supply and sanitation projects, the manuals will provide guidance on how they should involve all the principal stakeholders including the Community Based Water Supply Organisations (CBWSO) as foreseen in both the NAWAPO (URT,2002) as well as the NWSDS (URT, 2008). The manuals have been formatted in order to be more user friendly by allowing navigation within and across the manuals as well as having the capability to navigate into or from website links with ease using subject indices that enable a user to search for the needed information almost instantly. It is hoped that, the manuals will contribute towards improvement of the contract management capacity of the staff involved in project management and will eliminate the recurring problem of consultants designing water supply and sanitation management projects that are below minimum quality standards.

## 1.4 ABOUT THE FOURTH EDITION OF THE DCOM MANUAL

The 4$^{th}$ edition of the DCOM Manual was prepared in 2020, following the review and updating of the Third Edition of the Water Supply and Wastewater Disposal Design Manual of 2009. The former manual was prepared in three separate volumes. These volumes included eight chapters on water supply, three chapters on wastewater disposal and one chapter on water pipelines standards and specifications. It should be remembered that the 2$^{nd}$ Edition of the Design Manual that was titled *Design Manual for Water Supply and Waste and Wastewater Disposal* was prepared in July 1997 in two volumes with eight chapters and three chapters, respectively. The 1st Edition of the Design Manual was prepared in 1985/86, a few years after the conclusion of the International Water and Sanitation Decade that ended in 1981. Thus, the current edition of the DCOM is adequately informed by previous edition reviews which incorporate topical and existing challenges and issues.

A Special Committee of twelve members from The Ministry of Water, RUWASA, University of Dar es Salaam (UDSM), The Nelson Mandela African Institution of Science and Technology (NMAIST) and Private Sector undertook the preparation of the four volumes of this manual. The process of preparing the design manuals entailed a number of participatory consultations with key stakeholders from the water and sanitation sector as well as from Ministries of Education, Science & Technology, Ministry of Health, Community Development, Gender, Elderly and Children (MoHCDGEC), President's Office Regional Administration and Local Government (PORALG) as well as Consultants, Contractors, Materials suppliers and Development Partners. It also involved undertaking an extensive search of literature from libraries, conference proceedings, journal publications, websites of various entities and design manuals from various global, East African and SADC countries.

## 1.5 ORGANISATION OF THE 4TH EDITION OF THE DCOM MANUAL

The 4th Edition of the DCOM Manual has been prepared in four separate volumes that are divided as follows:

- **Volume I** which presents *Design of Water Supply Projects* organized into thirteen chapters;
- **Volume II** that dwells on *Design of Sanitation Projects* and is divided into six chapters;
- **Volume III** titled *Construction Supervision for Water Supply and Sanitation Projects* has been structured into five chapters; and
- **Volume IV** titled *Operation and Maintenance for Water Supply and Sanitation Projects* is organized into nineteen chapters. This Volume IV is organized into five parts as indicated below, and can be used as separate packages for training of different groups of users from the water sector:

  Part A: Essentials of Operation & Maintenance,

  Part B: O&M of the Water Supply Sources and Network,

  Part C: O&M of Water Treatment, Water & Wastewater Quality Compliance,

  Part D: O&M of Sanitation Projects,

  Part E: Water Audit, Revenue and Community Participation Management.

## 1.6 PURPOSE OF THIS VOLUME

This volume has been prepared with the main aim of providing engineers and designers with step by step design approaches for water supply projects. Observations gathered during the audit of water projects by special audit committee revealed that one of the reasons for poor performance of water projects is an outdated design manual. This very first volume of DCOM has been organized in such a manner that it starts with planning for water projects. The water projects planning chapter which comes after the introductory chapter underlines and underscores the significant and major part of the water projects. Also, the volume covers detailed account of assessment of safe yield of water sources. Water intakes, treatment and pipelines hydraulic analysis have been well covered in this volume. Chapter four provides description designs and specification. The role of stakeholders on water projects has been stipulated in the last chapter.

The preparation of this volume aimed to provide an opportunity to guide well engineers who have been given the responsibility for the design of either a complete water supply scheme or any component of the same as currently presented under 16 different topics. Volume I of the DCOM Manual has also provided the opportunity to link or hyperlink to many other websites and also to use the index provided at the end of the volume for ease of instant search.

## REFERENCES


MALCOLM N. SHAW 2008, International Law, Sixth edition, Cambridge University Press, Cambridge Uk.

Rocha Loures F & Rieu-Clarke A (eds) (2013). *The UN Watercourese Convention in Force: Strengthening international law for trans-boundary water management.* Earthscan, Routledge.

SIWI (2015). International water law. Retrieved from: https://www.siwi.org/icwc-course-international-water-law

UNFCC (2015). Paris Agreement on climate change 2015. Retrieved from: https://unfccc.int/process-and-meetings/the-paris-agreement/the-paris-agreement

UNFCCC (2015). Paris Agreement. United Nations Framework Convention on Climate Change (UNFCCC).

United Nations (2015). Helping governments and stakeholders make the Sustainable Development Goals (SDGs) a reality. Retrieved from: https://sustainabledevelopment.un.org/

URT (2000). The Tanzania Development Vision 2025. Ministry of Finance and Planning. https://www.mof.go.tz/mofdocs/overarch/vision2025.htm.

URT (2002). The National Water Policy (NAWAPO). United Republic of Tanzania (URT).

URT (2008). The National Water Sector Development Strategy (NWSDS). United Republic of Tanzania.

URT (2014). *Mwongozo wa Ujenzi wa Vyoo Bora na Usafi wa Mazingira*. Ministry of Health, Community Development, Gender, Elderly and Children (MoHCDGEC).

URT (2016). Five Year Development Plan (FYDP II), 2016/17 – 2020/21. Ministry of Finance and Planning. Retrieved from: https://mof.go.tz/mofdocs/msemaji/Five%20 2016_17_2020_21.pdf

URT (2016). The National Guidelines for Water, Sanitation and Hygiene for Tanzania Schools. Ministry of Education, Science and Technology (MoEST).

URT (2016). The Second Five Year Development Plan (FYDP II), 2016/17 – 2020/21. Ministry of Finance and Planning.

https://mof.go.tz/mofdocs/msemaji/Five%202016_17_2020_21.pdf.

URT (2017). National Guidelines for Water, Sanitation and Hygiene in Health Care Facilities. Ministry of Health, Community Development, Gender, Elderly and Children (MoHCDGEC).

NSGRP II & III

Wolf 1999, cited in https://www.siwi.org/icwc-course-international-water-law/ visited on 08, March, 2020.

WorldBank (2010). Water and Sanitation Public-Private Partnership Legal Resource Centre. https://ppp.worldbank.org/public-private-partnership/sector/water-sanitation

Chapter 2

# PROJECT PLANNING

Arguably, planning of water supply projects is considered to be one of the most important stages in the design. Thus, it is strongly advised and emphasized that much time should be spent to undertake proper project planning. A proper project planning will ensure effective, efficient and successful completion of the project. In this chapter, project planning is presented in seven parts that are detailed below. These include:

- Planning considerations for water supply projects
- Project Planning steps
- Consulting the Integrated Water Resources Management and Development Plan
- Consult guideline for preparation of Water Safety Plan – resilient to climate change.
- Environmental and Social Impact Assessment (ESIA) and Strategic Environmental and Social Impact Assessment (SESIA) Compliance
- Potential Impacts of Climate Change on Water Supply Projects
- Participation of CBWSOs in Project Planning Stages

## 2.1 PLANNING CONSIDERATIONS FOR WATER SUPPLY PROJECTS

Before the commencement of any development of a water project, it is essential to conduct project planning. Planning is a process that should entail the following:

- Undertake ESIA and SESIA studies,
- Engagement and involvement of the local community to instil the ownership, provide and take advantage of local knowledge, project buy-in and accommodate community needs and requirements,
- Assess safe and reliable yield or discharge and quality of water source,
- Determination of the system layout,
- Conduct design of the water supply project,
- Implement the project in terms of construction, operation and maintenance,
- Work out and obtain a sound and robust project financing.

It should be emphasized and stressed that the collection of good quality, reliable, credible and enough data should be given high priority at all stages of project planning. Population projection methods and their relevance for rural and urban settings or areas as recommended by the National Bureau of Statistics (NBS) have to be evaluated. Demographic features such as social and economic conditions have to be studied before design projections can be established. Also, water source reliability should be carried out before any further stage of project planning and implementation.

## 2.2 PROJECT PLANNING STEPS

Project planning involves a series of steps that determine how project goals will be achieved. The goals may be solicited from the existing community or a strategic plan. In an event that there is not any plan, project plans can be developed through community meetings and gatherings, councils or board meetings, special focused group discussions or other planning processes.

The main steps of project planning include:

Step 1: Initiation

Step 2: Pre-feasibility study

Step 3: Feasibility study

Step 4: Preliminary and Detailed Design

Step 5: Project phasing

Step 6: Procurement

Step 7: Implementation/Construction

Step 8: Operation and Maintenance (Management)

Step 9: Performance Monitoring

The planning of water supply projects can be represented diagrammatically by a step wise planning cycle as shown in Figure 2.1.

## 2.3 DESCRIPTION OF THE PLANNING STEPS

The following sections provide a brief description of each of the twelve project planning steps.

### 2.3.1 Initiation

Initiation or sometimes referred to as triggering stage, is a step where initial ideas of the project are presented. Community mobilization through awareness raising is conducted at this stage. The whole idea is to inform the community on the start of the project, solicit community input and knowledge about the project area. Project common understanding is also expected to be realized at this step.

The outcome of this stage of planning is to acquire an understanding of the community conditions and identify problems that prevent the community from achieving its long-range goals. Community conditions which must be collected include aspects of the community such as:

- Its geographic location,
- Demographics,
- Ecosystem,
- Cultural norms,
- History, e.t.c

The data collection for the above information should employ community assessment methods.

**WATER PROJECTS PLANNING PROCESS**

**1 Initiation Phase**

**Community Needs Assessment**
- Identify specific community problems and produce community problem statement
- Develop goal and objectives
- Create work plan to address problems & attain goal
- Describe beneficial impacts
- Determine level of resources

**2 Planning Phase**

**Pre-Feasibility Study**
- Initial data collection
- Establish alternative solutions
- Establish human resources requirement
- Estimate time requirement
- Develop initial budget estimate

**Feasibility Study**
- Collection of sufficient Design Data
- Appraise and evalute alternative solution
- Ranking of alternative solutions
- Share alternative solutions with community
- Conduct Environmental impact assessment
- Conduct preliminary engineering design & cost estimate for all alternative solutions
- Select most feasible project

**Preliminary & Detail Design**
- Statistical analysis of data
- Structural and stability computations
- Calculations for pumps, motors, power generator
- Detailed Engineering Drawings and Contract Documents
- Detailed design report
- Project Phasing

**3 Implementation Phase**

**Procurement Process**
- Preparation of Tender Documents
- Issue of tender documents
- Submission and receipt of tenders
- Opening of tenders
- Evalution of tenders
- Post qualification & Disqualification of service providers
- Award of Tenders
- Signing of Contract Agreement
- Use of Force account

**Contract Management**
- Contract Management Plan
- Contract delivery follow-up
- Stakeholder management plan
- Issue and claims management
- Contract termination
- Acceptance of works
- Contract close out

**Construction supervision**
- Develop and adherence to work program
- Develop and adherence to Quality assurance plan
- Measurement of work and management of payments
- Supervision of works

**4 Operation & Maintenance Phase**
- Preparation of O&M plan
- Planning for Capacity building of O&M personnel
- Planning for provision of tools spares and tools
- Plan for water audit and leakage control
- Plan for Financial Management
- Develop O&M schedules
- Reports and record keeping
- Water Audit, Revenue and plan for information education communication
- Develop and use Water safety plans

**Figure 2.1: Projects Planning Process**
(Source: Modified after Design Manual 3rd edition, 2009)

## 2.3.1.1 Community Assessment Steps

- Identify specific community problems that stand in the way of meeting community goals. Produce a community problem statement,
- Create a work plan to address the problems and to attain the goals,
- Describe measurable beneficial impacts to the community that result from the project's implementation,

- Determine the level of resources or funding necessary to implement the project.
- Solicit community social economic assessment report from Local Government Authorities for use in the choice of technology to achieve project sustainability

## 2.3.1.2 Methods of Conducting Community Assessment

Two methods can be employed for conducting community assessment. They are comprehensive and strategic planning.

### *2.3.1.2.1 Comprehensive Community Assessment*

This process should involve:

- Completing a community-wide needs assessment to engage the community in identifying and prioritizing all long-range goals and the community problems preventing the achievement of those goals,
- Next, the community is involved in the process of developing a method to accomplish the long-range goals,
- Discussing initial ways to overcome the problems
- Develop measures to monitor progress towards achieving those goals.

Comprehensive plans require at least a year to complete and should cover a five- to ten-year time span.

### *2.3.1.2.2 Strategic Community Assessment*

This is a process used when a community or an organization already has a comprehensive plan and wants to move forward to achieve its long-range goals. Strategic planning involves:

- Participation of the community in identifying problems that stand between the community and its goals and to move the community towards realizing its long-range vision.

The product of strategic planning, simply called the "strategic plan," builds on pre-established long-range goals by designing projects related to one or more of these goals. A strategic plan generally takes at least six months to complete.

## 2.3.2 Pre-feasibility Study

The pre-feasibility study stage involves initial fieldwork and studies of alternative water resource development plans. The report issued is an outline of possibilities and a list of all the fieldwork activities that need to be accomplished at feasibility study or even preliminary engineering design stage.

The objective of this initial study is to determine whether it is worthwhile to proceed with more detailed investigations. In other words at this stage, various projects or alternatives are screened and this should normally reduce the number of options considered feasible to no more than three or so. The report should contain recommendations on the proposed project and how to proceed with the detailed investigations.

These should include indications on the following:

- Data to be collected,
- Remaining alternatives to be considered and investigated,
- Professional human resources required,
- Estimate of time that will be taken or needed,
- Budgetary financial requirements.

The above are considered taking into account:

- Long term needs,
- Deficiencies in the existing system (if any),
- Phases of project implementation.

Briefly, the pre-feasibility reports should give an outline of the future development, which seems most appropriate to provide the project area with water in the long term. The other major aim may be to select a short-term project that may be implemented to overcome any immediate needs (crash programme) while the long-term project is being prepared.

### 2.3.3 Feasibility Study

The feasibility study stage develops the pre -feasibility work further and ends with a Report which normally concentrates on the project alternatives that were recommended for more detailed consideration at the pre-feasibility stage.

The study has to be carried out by a team of competent and experienced personnel from the Ministry, RUWASA and WWSA or with the help of a private sector. At this stage the following should be achieved:

- Collection of sufficient design data,
- Appraisal of alternatives,
- Alternative plans (projects) adequately studied and evaluated,
- Socio-economic analysis adequately conducted and completed
- Solicitation of views and preferences of the community in an open meetings
- Discussion of merits and demerits of the project with community representatives
- Rank alternatives on the basis of appropriate costing method and perceived ability of community to afford the costs of operation and maintenance and reach agreement in principle with concerned water officials.
- Inform the community about the reasons for selection of alternative(s) and seek their agreement and approval.

- Conduct Environmental Impact Assessment (EIA). For larger projects a statement on Life Cycle Assessment should be included indicating the extent of quantitative and other relevant information currently available,
- Preliminary engineering design done, including a review of alternative materials,
- Preliminary cost estimates done,
- Economic internal rate of return and financial internal rate of return,
- Carry out design to a level sufficient to enable construction to proceed either using local (District) based contractors or a Force Account approach using local sub-contractors as considered feasible and appropriate,
- Most feasible project (least cost) selected,
- Feasibility report prepared and presented to the authorities for approval

The report may also include interim progress reports, appendices of data collected during the detailed study. The feasibility report should be presented as a supporting document to apply for financing from the financing agencies.

## 2.3.3.1 Water Supply Projects Ranking and Technology Selection Criteria

### *2.3.3.1.1 Ranking of Projects*

Projects to be implemented should be ranked based on the following criteria:

- Type of technology,
- Quantity of water,
- ESIA Report comments,
- Negative environment impact,
- Quality of water available,
- Cost/benefit analysis,
- Walking distance scheme complexity.

### *2.3.3.1.2 Choice of Technology*

As far as ranking of project selection is concerned, technology choice should be based on progressive consideration of:

- Hand pump(s) from proven permanent deep hand dug well(s) or shallow borehole(s),
- Gravity scheme from protected spring,
- Medium or deep well with appropriate hand pump (rotary type),
- Pumped / Piped Scheme Electrical Driven,
- Pumped/Piped Scheme Solar Driven.

For point water sources or simple distribution systems, a prime location for a domestic point should be the village primary school followed by a village health facility (if any). Provision of improved sanitation and hand washing facilities at both primary school and health facility should also receive priority consideration in any village scheme. Use the relevant WASH guidelines for design of the washing facilities.

### 2.3.4 Preliminary and Detailed Design

After the feasibility report is presented and approved, the preliminary and the preferred alternative should be selected and the finances sought. The following should be considered while conducting the design:

- The Engineer should prepare the preliminary engineering design and then the detailed or final project report,
- These reports should provide the basis for implementation,
- The initial report has to provide the design basics which are then developed further in the detailed design of the project including working drawings and tender documents,
- They should include a review of all relevant aspects of this DCOM Manual and either accept or otherwise indicate, complete with detailed reasoning, why different criteria are proposed.

In addition the report should address the following:

- The issue of costing being adopted and requirement for extent of whole life cycle analysis and adaptation of costing,
- Consideration of the environmental impacts of the project and its envisaged elements,
- Issue of climate change and its possible effects on the project being designed.

It should be noted that the conceptual designs provided at the feasibility study or preliminary engineering stages are generally inadequate for the construction of the project. Foremost, the Engineer arranges for any outstanding detailed field investigations, surveys and data collection. Based on the detailed field data collected; detailed designs, plans and estimates are prepared.

Detailed designs should include:

- Statistical analysis of data collected for the population and demand projections; hydrological, hydrogeological and meteorological data,
- Least cost lay-outs for different components of the project, i.e. treatment plants, hydraulic and structural works,
- Structural and stability computations of different structures,
- Calculations for pumps, motors, power generators and other machinery and equipment,
- Engineering analysis for deciding the most economic size of delivery mains.
- Hydraulic computations for the distribution system,
- Bills of quantities.

Detailed design should include the following items:

### 2.3.4.1 Detailed Engineering Drawings

These should include:

- Index plan showing overall layout of the project,
- Schematic diagram showing levels of salient components of the project (may not necessarily be to scale),
- Detailed plans and sections in scale for the headwords, treatment plants, clear water storage tank, pumping station, in a scale 1:20 to 1:100 depending on the details and size of the works,
- Detailed structural plans for structures, intake, treatment plant, clear water reservoir etc., in a scale of 1:20,
- Index plan of the distribution system normally in an appropriate scale,
- Longitudinal sections of the delivery main and details of appurtenances in scales: Horizontal scale 1:500 to 1:5000 depending on distance and details Vertical scale 1:20 to 1:100 depending on the terrain surface undulations.

### 2.3.4.2 Detailed Estimates of Capital Costs

Project cost estimates should be based on unit costs derived from recent projects of a similar magnitude, complexity and remoteness from or proximity to ports or major urban areas.

### 2.3.4.3 Detailed Estimates of Recurrent Costs

As far as possible this should be based on unit costs provided by the operating authority or from schemes of a similar size and nature.

### 2.3.4.4 Anticipated Revenue

These should be based on the recommendations made regarding tariff structures or provided by the operating authority or regulator.

### 2.3.4.5 Detailed Design Report

A report should accompany the detailed designs, plans and estimates elaborating on the:

- Engineering aspects,
- Financial aspects,
- Administrative aspects,
- Tender documents
- Specifications.

### 2.3.4.6 Project Write-up to be Submitted to Potential Financiers

Each Development Partner may have a different pattern of project presentation for financial request. The project document should therefore follow guidelines indicated by the financiers or the local funding sources where applicable.

### 2.3.5 Project Phasing

Sometimes the implementation of a project is carried out in phases due to among other things, the following reasons:

- Financial resources available,
- Opportunity cost of money,
- Economies of scale,
- Growth rate in the area,
- Rate of development in the area,
- The design (working) life of various installations,
- Development in levels of service,
- New technologies or methods that need piloting before rolling them out.

Once the basic design period is decided (usually between 10 and 20 years) and water demand is computed for different years, the different elements can be phased. Exceptions do occur where financial assistance capital is being used and there is fear or a probability that further funding will not be available just a few years later.

Generally, phasing should be undertaken as follows:

- Dams, river and spring intakes, should be implemented in a single phase to cover all of the ultimate design demand or the hydrologically calculated water availability. This is particularly significant for dams as flood spillways form an expensive integral part and the need to raise a spillway inlet and deal with the additional energy at its exit and this is usually very costly.
- Boreholes to be constructed in Multiple Phases according to the growth in demand.
- Treatment plants and storage tanks to be constructed stepwise or in phases, according to the projected growth in demand.
- Mechanical installations to be implemented in Multiple Phases according to the design life of the equipment.
- Pump houses constructed in a Single Phase with space for additional mechanical plant.
- Rising mains and main conduits between units to be constructed to cover the ultimate demand in a single Phase.
- Long transmission mains to be constructed as two parallel lines in a single Phase where funds allow or in Two Phases where not. It can be advantageous to dedicate one of two parallel transmission mains to supplying water to the terminal reservoir whilst using the second for a mix of local distribution (daytime) and conveyance to the terminal reservoir (night time).
- Distribution systems to be constructed according to the growth in development in Multiple Phases.

## 2.3.6 Procurement

### 2.3.6.1 Preparation of Tender Documents

The Procurement Management Unit (PMU) using the approved templates as guided by PPRA documentation undertakes the preparation of tender documents. In preparing the tender documents, a job undertaken by PMU, unit rate contract is normally adopted for project components such as intake, delivery mains, distribution system, storage tanks and other appurtenances. For specialized areas like treatment plants and pumping stations it may be necessary to prepare separate tenders for the supply and installation of such facilities. The superstructure may still be included in the main contract bill of quantities. As much as possible one contract is preferred. The suppliers of such specialized equipment would then be included as sub-contractors of the main contractor. Important documents in the contract include:

- Letter of Invitation to Tender
- Instruction to Tenderers
- General Conditions of Contract
- Special Conditions of Contract
- Drawings
- Specifications
- Bills of Quantities
- Tender Forms
- Security Forms
- Anti-bribery Pledge
- Schedule of Additional Information
- Information Data

### 2.3.6.2 Tendering Process

This process involves the use of Public Procurement Act to select service providers as detailed in following steps:

- Issue of tender documents
- Submission and receipt of tenders
- Opening of tenders
- Evaluation of tenders
- Award of tender
- Signing of contract agreement

## 2.3.7 Implementation/Construction Stage

Construction stage includes contract management, Contract supervision and administration.

### 2.3.7.1 Contract Management

Contract management entails the following;

- Contract Management Plan (CMP),
- Contract Delivery Follow-up,
- Work progress monitoring & control,
- All projects executed must have a completion report (as constructed, built reports and drawings). It is essential that Engineers or Foremen keep an up to date record of all project activities including all changes to the original design with reasons for this clearly indicated as well as the approving authority,
- Initial and Final Acceptance of the Works,
- Contract Close Out.

### 2.3.7.2 Contract Supervision and Administration

During the construction stage, it is necessary to consider the following;

- Each phase of the project implementation should be planned in detail using techniques such as the Critical Path Method (CPM) or Programme Evaluation or/and Review Technique (PERT ) to ensure time control,
- Work together with the Contractor to prepare a Quality Assurance Plan which shall narrate the scope of the works and the expected quality requirements for the project and the role of the participants in ensuring quality requirements are met,
- Obtain a cash flow forecast from the contractor, and make the Client aware of his payment obligations based on the forecast,
- Keep a close track of all contractors' approved claims and adjust the contract price to reflect increases or decreases in the contract price.

Detailed information in procurement, contract management, contract supervision and administration is detailed well in chapter three and four of **Volume III Construction Supervision for Water Supply and Sanitation Projects.**

## 2.3.8 Operation and Maintenance Stage

This process takes over after the project completion and it involves;

- Preparation of O&M Plan,
- Development of Individual Unit Plans for O&M,
- Plan for capacity Building of O&M Personnel,
- Plan for Providing Spares and Tools,
- Plan for Water Audit and Leakage Control,
- Plan for Efficient Use of Power,
- Plan for sound financial management system,
- Plan for Information Education Communication for Water and Sanitation Services,

- Reports and Record Keeping,
- Develop appropriate maintenance schedule and check lists,
- Utilize Standard Operating Procedures,
- Utilize Water Safety Plans.

Detailed information on planning for operation and maintenance is found in chapter three and four of **Volume IV Operation and Maintenance of Water Supply and Sanitation Projects**

### 2.3.9 Performance Monitoring

The aim of the project is to provide the services uninterrupted. To ensure this, a proper monitoring mechanism of the performance of the project should be prepared. Such a mechanism should include proper procedures for procurement and distribution of spare parts, fuel, replacement, a maintenance programme for the project including personnel at the village, District and if necessary at Regional and National levels. Likewise a water quality surveillance procedure should be instituted in the framework of the existing mechanism.

## 2.4 CONSULTING THE INTEGRATED WATER RESOURCES MANAGEMENT AND DEVELOPMENT (IWRMD) PLANS

It is imperative that during planning of a water supply and sanitation project, the designer should consult the Integrated Water Resources Management and Development (IWRMD) plan for a basin where the project is planned to be executed. The development of an IWRMD Plan is a key objective of the water resources component of the Water Sector Development Programme 2006-2025. It is a legal requirement provided for in the Water Resources Management Act, No. 11 of 2009. The plan provides a blueprint for sustainable development and management of the basin's water resources.

Thus, a water supply and sanitation project designer is advised and encouraged to consult the IWRMD plans as they provide:

- The status of water resource availability (both quantity and quality) in the basin,
- Water data and information necessary for the design of the projects,
- Framework for water allocations among its competing demands,
- Water demand for water related sectors,
- Stakeholders' consultation plan.

### 2.4.1 Status of Development and Implementation of IWRMD Plans

By the time of development of this DCOM manual, IWRMD plans had been developed for six (6) out of the nine basins. The six basins are:

- Rufiji River Basin,

- Ruvuma and Southern Coast Basin,
- Lake Tanganyika Basin,
- Lake Nyasa Basin,
- Internal Drainage and
- Lake Rukwa Basin

It was reported that the development of IWRMD plans for Lake Victoria Basin and Wami/Ruvu basins were on-going.

IWRMD plans implementation challenges have been observed in some basins. These include:

- Inadequate funding to implement plans recommendations,
- Some plans are not implementable because of inclusion of unrealistic recommendations,
- Some plans are considered to have been more of studies than plans,
- Inadequate human resources capacity to implement the projects,
- As required by EMA, ESIAs have not been conducted, contrary to statutory requirement

### 2.4.2 Components of IWRMD Plans

The developed IWRMD plans are expected to have the following main components:

- Component 1: Inventory and review of water availability, use and demand,
- Component 2: Institutional, Policy and legal framework,
- Component 3: Sector/Thematic Water Plans,
- Component 4: Integrated Water Resources Management and Development Plan,
- Component 5: IWRMD Plan Implementation Strategy and Action Plan.

The production capacity of a source is very important in planning a water supply system. An estimate of the water that can be reliably produced by a water source like a well or spring which gives the planner a basis to decide for or against its development. For the source(s) to be considered adequate, they must at least satisfy the maximum daily demand of the area to be served.

## 2.5 CONSULT GUIDELINES FOR THE PREPARATION OF WATER SAFETY PLANS - RESILIENT TO CLIMATE CHANGE

Water Safety Plan (WSP) is the most effective means of consistently ensuring the safety of a drinking-water supply through the use of a comprehensive risk assessment and risk management approach that encompasses all steps in water supply from the catchment to the consumer (WHO, 2017). The approach enables the operators and managers of water utilities to know the system thoroughly, to identify where and how problems could arise, to put multiple barriers and management systems in place to stop the problems before they happen and making all parts of the system work properly so as to ensure the safety and acceptability of a drinking water supply intended for human consumption and other domestic uses as summarized in the WHO safe water chain frameworks.

Thus, during the planning phase, a designer should consult the guidelines for the preparation of Water Safety Plans - Resilient to Climate Change, which has been prepared and published by The Ministry of Water (MoW, 2015)

## 2.6 ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT COMPLIANCE

Section 81 of the Environmental Management Act (Cap 191)requires all developers of projects identified in the 3rd Schedule of the Act and detailed in the 1st Schedule of the Environmental Management (Environmental Impact Assessment And Audit) (Amendment) Regulations, 2018, to undertake Environmental Impact Assessment (EIA). Section 82 of EMA (Cap 181) requires that the EIA be carried out prior to the commencement or financing of the project. The procedures for carrying out the EIA, identified under the EIA and Audit (Amendment) Regulations of 2018 identify eight steps to be followed. According to EIA and Audit (Amendment) Regulations of 2018, projects are classified into the following categories, namely:

(a) "A" category for Mandatory projects;
(B) "B1" category for Borderline Project;
(c) "B2" category for Non-Mandatory; and
(d) "Special Category

So it is imperative that a proponent and developer of any water supply and sanitation project categorizes their project prior to actual project implementation for the same.

### 2.6.1 Procedures for Conducting ESIA in Tanzania

The procedures for carrying out the ESIA, identified under the EIA and Audit (Amendment) Regulations of 2018 identify eight key steps to be followed in the EIA process in Tanzania. These are:

- Step 1: Registration,
- Step 2: Screening,
- Step 3: Scoping,
- Step 4: Environmental Assessment,
- Step 5: Review,
- Step 6: Recommendations of the Technical Advisory Committee (TAC),
- Step 7: Submission to The Minister responsible for Environment,
- Step 8: Approval of the EIS.

It is recommended to consult NEMC guidelines and Environmental Management (Environmental Impact Assessment And Audit) (Amendment) Regulations, 2018 for more details. Also, the following Ministry of Water guidelines, have to be consulted:

a) Guidelines of Good Environmental and Social Practices (GGESP) of July 2019,
b) Environmental and Social Management Framework (ESMF) of July 2019.

### 2.6.2 Strategic Environmental and Social Assessment (SESA) Compliance

The SESA addresses broad strategic issues (policies) such as those affecting more than just one water project, affecting other sectors and those that must be resolved at higher administrative levels prior to the planning of the project. The SESA serves as a broad level analysis that guides eventual EIA. It helps to focus the EIA to key areas of interest.

Section 105 part (2) of the Environmental Act requires that wherever there is a major water project planned for construction, the Ministry responsible for water should conduct a Strategic Environmental and Social Assessment. The strategic environmental assessment shall assess the area marked for development and include:

- Baseline environmental conditions and status of natural resources,
- Identification of ecological sensitive and protected areas,
- Identification and description of communities around the area,
- Existing socioeconomic conditions,
- Existing economic activities and infrastructure,
- Proposed developments, including longer-term scenarios and the cumulative development of a number of different mine or oil and gas site or hydro-electric power stations,
- Infrastructure and resources required to service these development,
- Potential environmental and social impacts of mining or petroleum development or hydro-electric power or any major water projects; and
- Recommendations for land reclamation and limitations on development indifferent areas.

The strategic environmental and social assessment shall be submitted to the Minister responsible for Environment for approval before the start of the planning process.

## 2.7 POTENTIAL IMPACTS OF CLIMATE CHANGE ON WATER SUPPLY PROJECTS

It should be emphasized that immediately the project is conceived, hydrological, rainfall and other meteorological data collection must be initiated. In addition and given the long design life of such structures, consideration must be given to the possible impacts of climate change. Detailed account of predictions and impacts of climate change on water supply projects is provided in Appendix A.

The URT (2019) has recommended strategies and plans to adapt risks from climate change. The design related strategies of infrastructure, which a designer should consider while planning for water supply projects include:

- Where possible, have at least two sources of supply at different locations. Build superstructures above high flood-line level.
- Adopt energy-efficiency programmes and, where possible, select facilities which require less power consumption.
- Monitor wells near coastlines to prevent salinization. If climate change causes sea levels to rise dramatically, even aquifers that have been sustainably utilized can suffer salinization.
- Utilize renewable energy sources.

Guidelines for resiliency to climate change for urban water supply utilitieshave been published by the Ministry of Water.

## 2.8 PARTICIPATION OF COMMUNITY BASED WATER SUPPLY ORGANIZATIONS (CBWSO) IN VARIOUS PLANNING STAGES

As explained in detail in Section 2.1 of this volume, the CBWSOs must be involved in the complete life cycle of the project including ensuring their sustainability during operation and maintenance of the projects under the overall coordination of WSSAs and RUWASA.

## REFERENCES

Asadieh, B. and Krakauer, N.Y. (2016). Impacts of changes in precipitation amount and distribution on water resources studied using a model rainwater harvesting system. J. Am. Water Resour. Assoc. 52: 1450–1471.https://doi.org/10.1111/ 1752-1688.12472.

Gebrechorkos, S. H., Hülsmann, S., & Bernhofer, C. (2019). Regional climate projections for impact assessment studies in East Africa. Environmental Research Letters, 14(4), 044031. https://doi.org/10.1088/1748-9326/ab055a

Giannini, A., M. Biasutti, I. Held, and A. Sobel (2008). A global perspective on African climate. Clim. Change, 90: 359–383.

Hansingo, K., and C. Reason (2008). Modelling the atmospheric response to SST dipole patterns in the South Indian Ocean with a regional climate model. Meteorol. Atmos. Phys., 100: 37–52.

Hansingo, K., and C. Reason (2009). Modelling the atmospheric response over southern Africa to SST forcing in the southeast tropical Atlantic and southwest subtropical Indian Oceans. Int. J. Climatol., 29: 1001–1012.

Hermes, J., and C. Reason (2009). Variability in sea-surface temperature and winds in the tropical south-east Atlantic Ocean and regional rainfall relationships. Int. J. Climatol., 29: 11–21.

IPCC (2007). Summary for policymakers Climate Change 2007: Impacts, Adaptation and Vulnerability. Contribution of Working Group II to the Fourth Assessment Report of the

Intergovernmental Panel on Climate Change ed M L Parry, O F Canziani, J P Palutikof, P J van der Linden and C E Hanson (Cambridge: Cambridge University Press) pp 7-22.

IPCC (2014). Climate Change 2014: Summary for Policymakers, Synthesis Report. Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Core Writing Team, R.K. Pachauri and L.A. Meyer (eds.)]. IPCC, Geneva, Switzerland, 151 pp.

Li, J., Chen, Y.D., Gan, T.Y., Lau, N. (2018). Elevated increases in human-perceived temperature under climate warming. Nat. Clim. Chang. 8: 43–47. https://doi.org/10.1038/s41558-017-0036-2.

Marchant, R., C. Mumbi, S. Behera, and T. Yamagata (2007). The Indian Ocean dipole—the unsung driver of climatic variability in East Africa. Afr. J. Ecol., 45: 4–16.

Moss, R. H. et al. (2010). The next generation of scenarios for climate change research and assessment. Nature, Vol 463, 11 February 2010, doi: 10.1038/nature08823.

Pohl, B., N. Fauchereau, C. Reason, and M. Rouault (2010). Relationships between the Antarctic Oscillation, the Madden - Julian Oscillation, and ENSO, and Consequences for Rainfall Analysis. J. Clim., 23: 238–254.

Rouault, M., P. Florenchie, N. Fauchereau, and C. Reason (2003). South East tropical Atlantic warm events and southern African rainfall. Geophys. Res. Lett., 30, doi: 10.1029/2002GL014840.

UNFCCC (2010). The Cancun Agreements. United Nations Framework Convention on Climate Change http://unfccc.int/meetings/cancunnov2010/meeting/6266.php, 2010.

URT (2019). Water sector development programme. Environmental and social management framework (ESMF). Revised version. Ministry of Water.

Vautard, R., Gobiet, A., Sobolowski, S., Kjellström, E., Stegehuis, A., Watkiss, P., Mendlik. T., Landgren, O., Nikulin, G., Teichmann, C. and Jacob, D. (2014). The European climate under a 2°C global warming. Environ. Res. Letters. Environ. Res. Lett. 9, 034006, doi:10.1088/1748-9326/9/3/034006.

Vigaud, N., Y. Richard, M. Rouault, and N. Fauchereau (2009). Moisture transport between the South Atlantic Ocean and southern Africa: Relationships with summer rainfall and associated dynamics. Clim. Dyn., 32: 113–123.

WHO (2017). Climate-resilient water safety plans: Managing health risks associated with climate variability and change. World Health Organization. ISBN: 978-92-4-151279-4. Retrieved from: https://www.who.int/water_sanitation_health/publications/climate-resilient-water-safety-plans/en/

# Chapter 3 ANALYSIS OF WATER SOURCES

Water source is the single most important element and is key to the proper functioning and thus sustainability of any water supply project. Evidence shows that whenever proper water source analysis has not been adequately conducted, most water supply projects fall into dysfunction. This chapter presents the analysis of water sources. It includes analysis of both surface and ground water. Further, the chapter gets into the 'nitty gritty' of each water source.

## 3.1 AVAILABILITY OF WATER RESOURCES IN TANZANIA MAINLAND

Tanzania mainland is endowed with a wide range of water resources that include the main drainage systems, river basins and natural wetlands. All such sources are identified and discussed in the next paragraphs. The drainage systems of water resources in Tanzania mainland, is divided into five drainage systems that include:

- The Indian Ocean drainage system,
- The Internal drainage system to Lake Eyasi, Natron and Bubu depression,
- The Internal drainage systems to Lake Rukwa,
- The Atlantic Ocean drainage system through Lake Tanganyika,
- The Mediterranean Sea drainage system through Lake Victoria.

The drainage systems in turn comprise nine river basins with some bearing names resembling the drainage systems. These nine basins are indicated in Figure 3.1.

From the geographical point of view, Tanzania is party to at least eleven trans-boundary water resources in the form of lakes and rivers (NWSDS, 2008). These include the following:

- Lake Victoria,
- Lake Tanganyika,
- Lake Nyasa,
- Lake Chala,
- Lake Jipe,
- Kagera River,
- Mara River,

- Pangani River,
- Umba River,
- Ruvuma River and
- Songwe River.

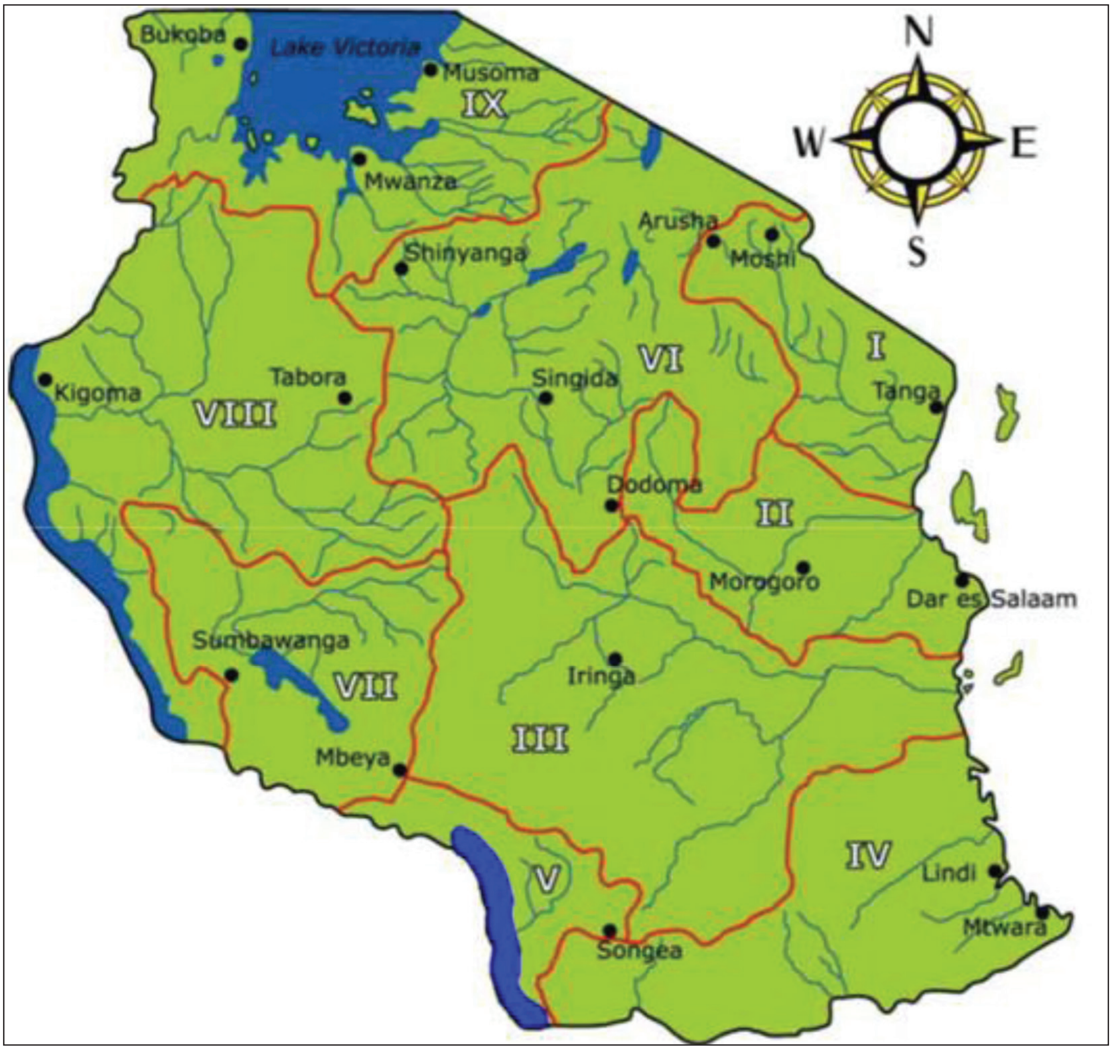



**Figure 3.1: The River Basins of Tanzania**
(Source: NWSDS, 2008)

*Legend for Figure 3.1*

(I) Pangani River Basin
(II) Wami/Ruvu River Basin
(III) Rufiji River Basin
(IV) Ruvuma and South Coastal River Basin
(V) Lakes Nyasa Basin
(VI) Internal Drainage Basin
(VII) Lake Rukwa Basin
(VIII) Lake Tanganyika Basin
(IX) Lake Victoria Basin

With its numerous water bodies, Tanzania is perceived to have abundant surface and groundwater resources for meeting its present consumptive and non-consumptive needs. However, the reality is that severe and widespread water shortages exist in many areas of Tanzania because of climate variability, poor distribution of the resource in terms of time and space, and inadequate management of the water resources (NWSDS, 2008). As a result, Tanzania experiences frequent and intense water shortages and some water use conflicts.

Furthermore, Tanzania is relatively dry with more than half of the country receiving, on average, less than 800 mm of rainfall per year depending upon air circulation patterns and the movement of the convergence zones in the region. The semi-arid Central and Northern parts of the country, including areas immediately South of Lake Victoria receive less than 700 mm of rainfall per annum and are dry for an average of seven consecutive months a year. River flows in these areas are intermittent. In the Southern, Western and Northern highlands, which receive more than 1,000 mm/year of rainfall, rivers are perennial, and some of these experience frequent floods.

As an example, in 1999 the availability of renewable freshwater resources, both surface and groundwater were estimated[1] to be about 2,700 $m^3$/capita/year. By 2018, this estimate was reduced to 2,330 $m^3$/capita/year due to increased population alone. The average figure is significantly above the level of 1,700 $m^3$/capita/year set by the United Nations as denoting water stress, or 1,000 $m^3$/capita/year denoting water scarcity. Furthermore, due to the projected population growth alone, Tanzania's annual freshwater renewal rate is projected to drop to 1,500 $m^3$/capita/year by 2025, thus categorising the country as being water stressed by then.

On the whole, Tanzania has sufficient surface and ground water resource potential to meet most of her present needs. However, differences in topography, rainfall patterns and climate account for the existing variation in the availability of water in different parts of the country. In the densely populated Pangani and Rufiji Basins, these variations have already resulted into water stress. It is estimated that the annual surface runoff from Tanzania to the world's oceans is about $74 \times 10^9$ $m^3$. The Rufiji, which drains a 177,000 $km^2$ area, contributes over 50% of the runoff. Typical annual runoffs are shown in the Table 3.1 for some of the major rivers of Tanzania.

The most abundant surface water resources exist in Lakes Victoria, Tanganyika, Nyasa, Chala and Jipe, as well as the Kagera, Mara and Songwe rivers, which are trans-boundary waters. The use of these abundant surface water resources for water supply, irrigation and other purposes is still very limited even today.

Tanzania is also rich in wetland systems that are areas which, for part of the year, have enough water to enable the development of different types of plants

[1] SADC, IUCN, SARDC, World Bank, Sida, - Environmental Sustainability in Water Resources Management in Southern Africa, 2002 .

and animals adapted to these conditions. These include the lakes of the Western and Eastern Rift Valley system, Lake Victoria, numerous small lakes, river in flood plains and permanent swamps, coastal mangrove and deltaic systems, and a number of artificial impoundments and reservoirs and fish ponds. There are numerous permanent and seasonal freshwater swamps and flood plains distributed in almost all of the country's major drainage basins, which account for some 2.7 million hectares. The largest in this category are found in the Rufiji/Ruaha river system and in the Malagarasi/Moyowosi system, while other river systems are the Kagera River, along with Ugalla River, Suiwe River, Mara River, Pangani, Wami and Ruvu Rivers. The principal wetlands of Tanzania constitute one of the country's richest and most durable resources.

**Table 3.1: Mean Annual River Discharges for Some of the Principal Rivers of Tanzania**

| River | Mean Annual Discharge (Million $m^3$/yr) |
|---|---|
| Rufiji (at Steiglers Gorge) | 22,250 |
| Kilombero (at Swero) | 14,470 |
| Malagarasi (at Taragi Ferry) | 5,060 |
| Ruvu (at Moro Bridge) | 1,370 |
| Wami (at Mandera) | 3,280 |
| Ruhuhu (at Kikonge) | 5,600 |
| Kiwira (at Kyela) | 1,900 |
| Kagera (at Kyaka) | 7,064 |
| Mara (at Mara Mines) | 1,971 |
| Pangani (at Hale) | 627 |

(Source: NWSDS, 2008)

## 3.2 WATER SOURCES AVAILABLE IN TANZANIA MAINLAND

In Tanzania, there are three main categories of water sources available, namely: rainwater, surface and groundwater.

### 3.2.1 Rainwater and Fog Harvesting

One of sources of water include rainwater and fog that can generate limited amounts of very clean water if they are properly collected and stored. In an area where other water sources are not available, consideration should be given to harvesting rainwater and fog.

### 3.2.2 Surface Water

For design purposes, the surface water sources that can be considered include;

- Rivers or streams,
- Impoundments (Reservoirs and ponds),
- Springs,
- Lakes,
- Dams (charco, sand, earth etc).

A brief description of each water source is provided below.

#### 3.2.2.1 Rivers or Streams

Rivers and streams are water sources that originate from springs located in highlands which flow down to the end of the respective drainage basin which can be lakes, seas or oceans as depicted on the map of Tanzania in Figure 3.1.

#### 3.2.2.2 Impoundments

Impoundments includes all types of reservoirs that emanate from road borrow pits, mining, human or natural activities that are utilised as sources of water for a formal water supply project.

#### 3.2.2.3 Springs

Springs include artesian or freely flowing spring water that has been tapped by an intake structure to facilitate supply of water to a designated community. A spring is a point where groundwater flows out of the ground, and is thus where the aquifer surface meets the ground surface. The spring may be ephemeral (intermittent) or perennial (continuous). Springs can be developed by enlarging the water outlet and constructing an intake structure for water catchment and storage.

#### 3.2.2.4 Lakes

Lakes found in Tanzania are either located at the end of drainage basins or are highland lakes and some of them are volcanic lakes. Tanzania is endowed with many small inland lakes in addition to the third biggest lake in the world (Lake Victoria) as well as Lake Tanganyika which is the Africa's deepest lake and the world's longest lake. Both lakes supply water to various localities around their respective catchments.

#### 3.2.2.5 Dams

Dams are classified based on the availability of construction materials. Various types of dams can be built ranging from earth fill dams, concrete dams, sand dams and charco dams. These are purposely built structures that allow impoundment of river and/or rain water for various end uses.

### 3.2.3 Groundwater

Groundwater is that portion of rainwater which has percolated beneath the ground surface to form an underground reservoir referred to as aquifer water. The upper surface of groundwater is the water table. Groundwater is often clear, free from organic matter and bacteria due to the filtering effect of the soil on water percolating through it. However, groundwater almost always contains dissolved minerals from the soil. Groundwater is often better in terms of quality than surface waters. It is less expensive to develop for use, and usually provides more adequate supply in many areas in the country. In semi-arid and the drier parts of the country, groundwater has played and will continue to play a major role as the sole water source for various uses especially in the central and northern parts of the country and the drier regions of Dodoma, Singida, Shinyanga, Tabora, Mwanza, Mara, Arusha, Coast and Southern Kilimanjaro.

Groundwater can be considered as either spring water or well (or borehole) water. Springs, offer excellent water supply opportunities, but are generally found in hilly or mountainous areas only. They may require long pipelines to bring the water to the demand area. This is a feasible source for larger and concentrated settlements but rarely for dispersed populations. For rural water supply systems, groundwater is generally preferred as a water source.

The main sub-types of groundwater and extraction methods are as follows:

#### 3.2.3.1 Infiltration Galleries/Wells

Infiltration galleries are horizontal wells, constructed by digging a trench into the water-bearing sand and installing perforated pipes in it. Water collected in these pipes converges into a "well" from which it is pumped out.

#### 3.2.3.2 Well

This is a hole constructed by any method such as digging, driving, boring, or drilling for the purpose of extracting water from underground aquifers. Wells can vary greatly in depth, water volume and water quality. Well water typically contains more minerals in solution than surface water and may require treatment to soften the water by removing minerals such as arsenic, iron and manganese. Well water may be drawn by pumping from a source below the surface of the earth. Alternatively, it could be drawn up using containers, such as buckets that are raised mechanically or by hand.

Wells are various types of artificially constructed water production wells that are designated as *shallow wells* (up to 20 metres deep) or *deep wells* (more than 20 metres deep) as designated by the Ministry responsible for water from time to time. Water is pumped out of the well into the end user containers or a storage tank using various types of pumps that can be driven manually or using various energies. Typical cross sections through such wells are given in Section 3.6.5.

### 3.2.3.3 Classification of Wells Based on the Aquifer Tapped

As mentioned, an aquifer contains a considerable amount of groundwater underground beneath layers of permeable soil material like sand or gravel. Aside from their water storage capacity, aquifers allow the underground flow of groundwater. Aquifers are recharged with rainwater that seeps down to the soil and through the permeable layers.

#### *3.2.3.3.1 Shallow Wells*

Generally, a well is considered shallow if it is less than 20 metres deep. Shallow wells tap the upper water-bearing layer underground. This permeable layer, however, usually has limited safe yield due to its great dependence on seasonal rainfalls. Therefore, the supply capacity of shallow wells could be unreliable and are sometimes intermittent. Also, the water extracted from the upper strata is usually more affected by contamination since the aquifer being tapped is near the ground surface where possible sources of contamination are abundant. Protection against contamination is therefore one of the main considerations in constructing a shallow well.

#### *3.2.3.3.2 Deep wells*

Deep wells, which are over 20 metres deep, tap the deeper unconfined aquifer. This aquifer is not confined by an overlying impermeable layer and is characterized by the presence of a water table. A deep well is less susceptible to surface contamination because of the deeper aquifer. Also, its yield tends to be more reliable since it is less affected by seasonal precipitation.

#### *3.2.3.3.3 Artesian Wells*

Artesian wells are much like the deep wells except that the water extracted is from a confined aquifer. The confining impermeable layers are above and below the aquifer. Groundwater recharge enters the aquifer through permeable layers at high elevations causing the confined groundwater at the lower elevations to be under pressure. In some cases, the hydraulic pressure of the aquifer is sufficient for a well to flow freely at the well head.

## 3.3 QUALITY SUITABILITY OF WATER SOURCES FOR WATER SUPPLY PROJECTS

When considering the different water sources for water supply projects, it is necessary to ensure that the quality of the water source expected to be utilised is monitored well preferably for a period of not less than three years consecutively prior to commencing design to ensure the variability of the quality is captured during the wet and dry seasons. When one looks at the list of the potential sources presented in the foregoing section, such a monitoring programme may not be necessary for rainwater and fog. Only short-term monitoring of the quality of these two sources should be undertaken.

## 3.4 PILOT TESTING OF WATER SOURCES FOR ESTABLISHMENT OF APPROPRIATE TREATMENT

A decision on whether the water source needs to be subjected to water treatment or otherwise will emanate from the results of the short term tests on the quality of the water which will in turn guide the decision of pilot testing of the recommended flow sheets particularly for river/streams, lakes, impoundments and dams. Groundwater will usually need only a few unit operations for removal of the identified elevated impurities that may include Iron, Manganese or Fluoride and the need to disinfect water from shallow wells in addition to maintaining residual disinfectants for the prevention of re-contamination. Rainwater and condensates from fog may not need to be pilot tested.

## 3.5 GENERAL CONSIDERATIONS FOR SELECTION OF WATER SOURCES

In the selection of a source or sources of water supply, adequacy and reliability of the available supply can be considered as the overriding criteria. Without these, the water supply system cannot be considered viable.

Sources which require little or no treatment of raw water such as springs, wells and boreholes should be given the highest selection priority provided their yields are sufficient to meet the water demands of the water supply scheme. For large supplies, surface water will continue to be the most economical alternative water source. In selecting surface water sources, rivers with upland and mostly forested catchments should be given preference. Sub-surface water drawn from a riverbed or river bank can sometimes be a viable alternative in dry areas with only seasonal flows in the river, or in rivers with a high silt load.

Sources from which water can be supplied by a gravity system are particularly more favourable than those which require pumping with significant energy costs. For household and small community water supplies, rainwater harvesting will be the most appropriate in most medium and high potential areas in Tanzania that receive sufficient rains.

These, together with the other interdependent factors should be considered as follows:

### 3.5.1 Adequacy and Reliability

Adequacy of water supply requires that the quantity of water flow from a water source be large enough to meet present and future water demand. On the other hand, source reliability can be expressed by how frequently a water system expects normal demand to go unmet, such as a one-in-25 year or even a one-in-50 year drought. Safe yield is a 1-day low flow rate that is exceeded for 96 percent of the period of record and that can be related to the determined average daily water demand in order to establish the reliability of a water source. For a river/stream, safe yield represents the minimum flow rate that will guarantee no risk to the river hydrology and its surroundings. Safe yield is estimated to check whether the planned withdrawal for water supply purposes will be met. To determine the safe yield of a river or stream, a flow-frequency/probability analysis presented in section 3.6.2should be performed. From the analysis, the determined 96% low flow index should be taken as the safe yield of the river or stream and thus considered as the water source reliability.

## 3.5.2 Quality of Water Sources

### 3.5.2.1 Surface Water Quality

The assessment of water quality of a water source is important to establish the suitability of water source for human consumption. The quality of surface water is determined by the amount of pollutants and contaminants picked up by the water in the course of its travel. While flowing over the ground, surface water collects silt, decaying organic matter, bacteria and other micro-organisms from the soil. Sources which require little or no treatment of the water should be chosen in the first instance, provided the required quantity of water can be obtained. Hence springs and ground water resources should always be exploited in the first hand. Surface water from rivers, streams and lakes will almost always require some treatment to render it safe for human consumption. However, for large supplies, surface water will often still be the most economical alternative. Rivers which have the bulk of their catchments in forest areas should be preferred.

Thus, all surface water sources should be presumed to be unsafe for human consumption without some form of treatment. The option to treat surface water to make it safe for human consumption in compliance with the latest edition of Tanzania potable water standards (TBS, TZS 789) has to be evaluated to decide on the feasibility of the water supply project.

### 3.5.2.2 Groundwater Quality

Generally, all groundwater contains salts in its solution that are derived from the location and past movement of the water. Groundwater quality parameters exhibit considerable spatial variability. The eight common groundwater parameters that are measured include:

- Dissolved oxygen DO
- Electrical conductivity EC
- Total dissolved solids TDS
- Salinity
- pH
- Turbidity
- Total suspended solids TSS

- Chloride

Industrial discharges, urban activities, agricultural, groundwater pump age, and disposal of wastewater all can affect groundwater quality. The quality required of groundwater supply depends on its purpose.

### *Natural groundwater quality*

The groundwater in natural systems generally contains less than 100mg/l dissolved solids, unless it has:

- Encountered a highly soluble mineral, such as gypsum,
- Been concentrated by evapotranspiration,
- Been geothermally heated.

### *Sources of salinity in groundwater*

All groundwater contains salts in solution ranging from less than 25 mg/l in quartzite spring to more than 300,000 mg/l in brines. The types and concentration of salts depend on the environment, movement, time in residence with a particular geological formation, and source of the groundwater. Generally, salinity increases with depth.

### *Graphic representation of groundwater quality*

In the field of hydrogeology and groundwater analysis, piper plots (also known as tri-linear diagrams) are very powerful tools for visualizing the relative abundance of common ions in water samples. Although there are other plot types that can show abundance of ions in groundwater, this plot type is especially useful because it allows one to plot multiple samples on the same plot, thus allowing for grouping water samples by groundwater facies and other criteria.

In this day and age, when groundwater is so closely monitored, it is especially important to have a plot type like the piper plot (Figure 3.2) that makes it easy to determine. Piper plot is comprised of three components: a ternary diagram in the lower left representing cations (magnesium, calcium, and sodium plus potassium), a ternary diagram in the lower right representing anions (chloride, sulfate, and carbonate plus bicarbonate), and a diamond plot in the middle which is a matrix transformation of the two ternary diagrams. Each sample is normalized (sum of cations = 100 and sum of anions = 100), so the relative concentrations are on a percentage basis.

Bottom left is a ternary plot of the cations, bottom right is a ternary plot of the anions, and top is a diamond plot of a projection from the other two plots.The diamond plot can then be analysed to tell you what kind of groundwater you're looking at. Samples in the top quadrant are calcium sulfate waters, samples in the left quadrant are calcium bicarbonate waters, samples in the right quadrant are sodium chloride waters, and samples in the bottom quadrant are sodium bicarbonate waters. See example on Figure 3.2.

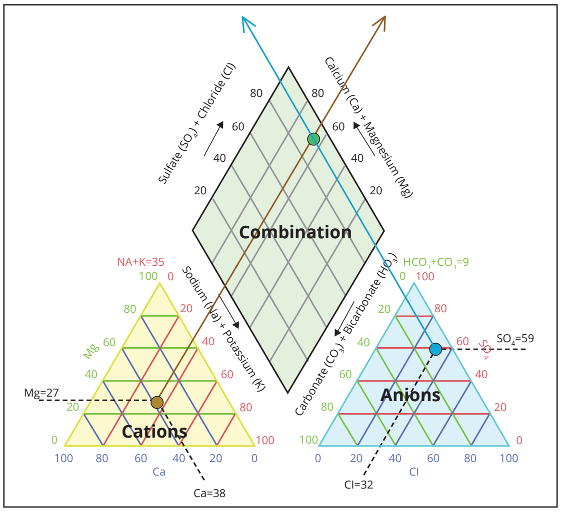



**Figure 3.2: The Three Components of the Piper Plot**
(Source: http://inside.mines.edu, Golden Software Support)

### *Interpretation of the diamond plot*

Samples in the top quadrant are calcium sulphate waters, which are typical of gypsum ground water and mine drainage. Samples in the left quadrant are calcium bicarbonate waters, which are typical of shallow fresh ground water. Samples in the right quadrant are sodium chloride waters, which are typical of marine and deep ancient ground water. Samples in the bottom quadrant are sodium bicarbonate waters, which are typical of deep groundwater influenced by ion exchange. See Figure 3.3.

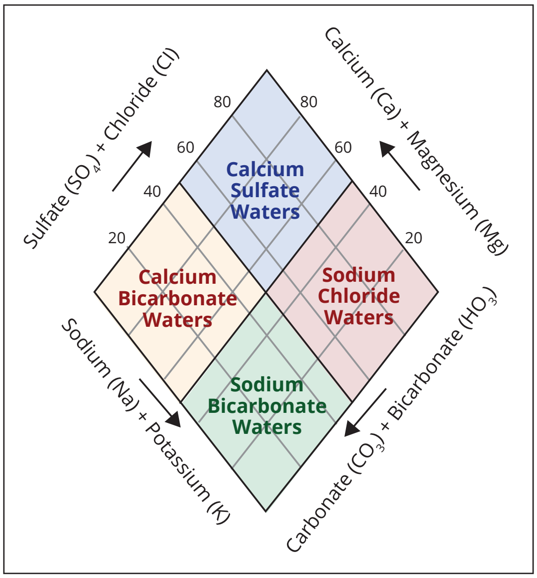



**Figure 3.3: Interpretation of the Diamond Plot**
(Source: http://inside.mines.edu, Golden Software Support)

### 3.5.3 Technical Requirements

The development of the source should be technically feasible, the operation and maintenance requirements for the source abstraction and supply system should be appropriate to the resources available.

### 3.5.4 Cost Implications to Develop a Water Source

The assessment of investment costs to develop a given water source including operation and maintenance costs has a bearing in the selection of the water source for development. Affordability of investment costs is an important factor to be considered in the selection of a water source.

### 3.5.5 Protection of Water Sources

The location of a water source is a key factor in securing the highest quality water source. In analysing a water source location, the design engineer should consider the measures necessary to protect the water source from human excreta, from industrial discharges and from agricultural run-off. In addition, measures to establish and maintain watershed control, physical protection and barriers to contamination must be considered to ensure sustainable quantity and quality of the raw water.

### 3.5.6 Legal and Management Requirements

The ownership of the land and the legal requirements of obtaining permission to abstract are also factors to consider when selecting a source. Sources on private land may cause access problems.

### 3.5.7 Distance of Water Supply Source

The source of the water supply must be situated as near to the demand area as possible. Hence, less length of pipes needs to be installed and thus economical transfer and supply of water. The source(s) nearest to the demand area is usually selected.

### 3.5.8 Topography of the Project Area and its Surroundings

The area or land between the source and the area to be served by water supply system should not be highly uneven, i.e., it should not have steep slopes because the cost of construction or laying of pipes is very high in such areas.

### 3.5.9 Elevation of a Source of Water Supply

The source of water should preferably be at a higher elevation than the demand area so as to provide sufficient residual pressure in the water for daily requirements. When the water is available at lower levels, then pumps are used to pressurize water. This requires an excess developmental and operational tasks and costs.

# 3.6 DETERMINATION OF WATER SOURCE YIELD

## 3.6.1 Rainwater and Fog Harvesting

### 3.6.1.1 Rainwater Harvesting

Rainwater harvesting is a technique of collection and storage of rainwater into natural reservoirs or tanks, or the infiltration of surface water into subsurface aquifers (before it is lost as surface runoff). The types of rainwater harvesting systems are described in the sections below.

#### *3.6.1.1.1 Types of Rainwater Harvesting*

Two types of rainwater harvesting should be considered:

- Land catchment
- Roof catchment

Important data for the design of rainwater harvesting systems are:

- Rainfall data
- Catchment/Surface Area
- Run-off Coefficient

To accurately estimate the potential rainwater supply from a catchment, reliable rainfall data for a 10-year period is required[2]. The Hydrology Section, Tanzania Meteorology Agency, and Agriculture Departments should be contacted for rainfall data wherever rainwater-harvesting technology is proposed.

The amount of rainfall collected depends on the surfaces where rain falls and the runoff coefficient K of the surface. The runoff coefficient varies with topography, land use, vegetation cover, soil type and moisture content of the soil. In selecting run off coefficients the future characteristics of the water shed are considered. If land use varies within a watershed consider the segments individually and use a weighted coefficient value to determine the total runoff for the watershed. Since most of the rainfall occurs during the rainy seasons between October and May annually, design should be mostly based on rains received during this period.

**(i) Run-off Coefficients**

Table 3:2 shows the runoff coefficients for various surfaces. They should be used for calculating the fraction of the rainfall which can be harvested.

[2] Dr.Sharafaddin Abdullah Saleh, Prof. Dr. Taha Taher and Prof.Dr. Abdulla Noaman (2017. Manual for Rooftop Rainwater Harvesting Systems. Water and Environment Center (WEC) – Sana'a University, Yemen

**Table 3.2: Run-Off Coefficients for Different Surfaces**

| Surface | Run-Off Coefficient (K) |
|---|---|
| 1. Roof catchments | |
| • Roof tiles | 0.8 to 0.9 |
| • Corrugated sheets | 0.7 to 0.9 |
| 2. Ground surface covering | |
| • Concreted | 0.6 to 0.8 |
| • Bitumen, plastic sheeting, butyl rubber | 0.8 to 0.9 |
| • Pavement of stone, bricks with open joints | 0.5 to 0.6 |
| • Pavement of stone, bricks with tightly cemented joints | 0.75-0.85 |
| 3. Compacted and smoothened soil | 0.3 to 0.5 |
| 4. Lawns, sandy soil | |
| • 2% slope | 0.05-0.10 |
| • 2.7% slope | 0.10-0.15 |
| • >7% | 0.15-0.20 |
| 5. Lawns, heavy soil | |
| • 2% Slope | 0.13-0.17 |
| • 2.7% slope | 0.18-0.22 |
| • >7% | 0.25-0.35 |
| 6. • Uncovered surface, flat terrain | 0.3 |
| • Uncovered surface, slope less than 10% | 0.0 to 0.4 |
| • Rocky natural catchments | 0.2 to 0.5 |

### *3.6.1.1.2 Components of a Rainwater Harvesting System*

(a) Catchments Area**:** The catchment of a water harvesting system is the surface which directly receives the rainfall and provides water to the system.

(b) Coarse mesh at the roof to prevent the passage of debris.

(c) Gutters to collect and transport rainwater to the storage tank. Gutters can be semi-circular or rectangular and could be made using:

- Locally available materials such as plain galvanised iron sheets (20 to 22 gauge), folded to required shapes
- Semi-circular gutters of PVC material can be readily prepared by cutting those pipes into two equal semi-circular channels. Recently, there are commercially available gutters in standard sizes for simple home or community schemes. For larger schemes, the same may be ordered from pipe manufacturing companies in the right sizes,
- Bamboo trunks cut vertically in half.

The size of the gutter should be according to the flow during the highest intensity rain. It is advisable to make them 10 to 15 per cent oversize.

(d) Conduits/pipeline that carry rainwater from the catchment or rooftop area to the harvesting system. Conduits can be of any material like polyvinyl chloride (PVC) or galvanized iron (GI), materials that are commonly available.

(e) First Flush pipe to separate first rainwater contaminants namely debris, dirt, and dust.

### *3.6.1.1.3 Estimation of the Yield*

A first estimate of the average yield of a catchments area can be found using the following expression.

S = K × I × A....................................................................................................(3.1)

Where:

S = Yield in $m^3$ / annum
A = Area of catchment/surface, $m^2$
I = Average annual rainfall m/annum
K = runoff coefficient

Determination of average runoff coefficient for the entire catchment area composed of different surfaces can be calculated as follows:

$$K_{average} = \Sigma_{i=0}^{n} K_i A_i = K_1 A_1 + K_2 A_2 + \ldots \qquad (3.2)$$

The required capacity of the collection facility should be calculated using available meteorological data showing the rainfall pattern of the area. However, for rough calculations the storage tank, capacity may be calculated as follows:

C = D × T × $10^{-3}$ .........................................................................................(3.3)

Where:

C = Capacity of tank in $m^3$
D = Total water demand in litres / day
T = Longest dry spell in days

## 3.6.1.2 Fog Harvesting

Fog harvesting technology though unconventional is an innovative technology and a very simple method of harvesting water. In this process, massive vertical shade nets (flat & rectangular) of nylon or polypropylene mesh are erected in high-lying areas located near communities that have a low supply of freshwater.[3] When fog blows through these structures tiny droplets of water are captured, coalesce and become larger, eventually flowing down along a plastic conduit to receptacles or gutters at the bottom of the structure. Collected water in the receptacles is then channelled into reservoirs, from where it is supplied to individual homes for multiple end uses.

3 www.thewatertreatmentplants.com › fog-harvesting-in-Chile

**Figure 3.4: Fog Harvesting Installation**[4]

The principle of collecting fog water requires a region subject to a lot of fog, it must be in an anticyclonic zone close to an ocean with cold water and to have a relief, or a natural obstacle, such as a high mountain above sea level. In Tanzania fog harvesting is being tested at Qameyu[5], Manyara Region to upgrade the fog-harvesting infrastructure at the Qameyu secondary school. The fog collector will supply drinking water for 17 teachers and 300 pupils.

### 3.6.2 Hydrological Analysis of Surface Waters

The Design Manual for Water Supply projects is expected to reflect the best concepts on what constitutes the basis for designing a safe, reliable and sustainable water system. Hydrological principles must be taken into consideration during the feasibility and preliminary design stages of the water supply system to ensure that from the outset, design and construction of the system is done right. The design engineer must apply hydrological principles during design of the water system to ensure that the system being designed does not result in exhausted water supply sources and empty reservoirs after construction of the project is completed. *If it is recognised from the beginning that there is water deficiency from the source, then the water source should not be considered for development.*

The following steps should be followed when undertaking hydrological analysis for the water supply projects:

Step 1: Measurement of the quantity of surface water sources,

Step 2: Low flow assessment of surface water sources,

Step 3: Flood flow estimation,

[4] https://www.appropedia.org/Run-off_rainwater

[5] https://townsoftheworld.com/Tanzania/Manyara/Qameyu/Videos

Step 4: Rainfall analysis,

Step 5: Water permit application,

Step 6: Data to support hydrological analysis.

**Step 1: Measurement of River Discharge**

In order to assess the amount of water available from an identified surface water source, a discharge measurement must be carried out during both the dry and wet seasons. The measurement of discharge will highlight the production capacity of a water source, the information which is important in the planning of a water system. An estimate of the quantity of water that can be reliably produced by a water source gives the planner a basis to decide for or against its development. For the sources to be considered adequate, they must at least satisfy the average day water demand of the area to be served by a water system. The average daily water demand is calculated from estimated average water requirements for domestic, commercial, industrial, public institutions and livestock as elaborated in Chapter Four. The following methods can be used to measure discharge.

## (a) Volumetric Method

This method is appropriate for measuring small quantities of flow from small streams and springs. Flow can be measured by measuring the volume. The equipment required are a stopwatch and a bucket or drum of known volume. The method consists of determining the time required to fill the bucket or drum. For more accurate results, the measurement is repeated several times, and the average time of these trials is taken.

## (b) V-Notch Weir Method

A weir is an overflow structure built across an open channel for the purpose of measuring the rate of flow. Weirs may be rectangular, trapezoidal or triangular in shape. The triangular or V-Notch Weir is a flow measuring device particularly suited for small flows. The V-Notch Weir often used in flow measurements is the 90° V-Notch that is placed in the middle of the channel and water is allowed to flow over it. The water level in the channel is then measured using a gauging rod. The zero point in the rod should be level with the sill or crest of weir/notch. For a known height of water above the zero in the rod, the flow in cumecs for the 90° V-Notch can be obtained by using the formula:

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan\frac{\theta}{2} H^{2.5} \qquad (3.4)$$

$$Q = 1.38 H^{2.5} \qquad (3.5)$$

Where,

Q = Discharge in $m^3$/sec

H = Height of Water level above the crest of the weir in meters

In this case, the discharge coefficient ($C_d$) of the weir is approximated to be equal to 0.58.

## (c) Current meter Measurement

The current meter is an instrument that is used to measure relatively larger quantities of flow from streams and rivers. The instrument consists of a propeller rotating freely on a well-lubricated shaft. The device is lowered into the water and the rate of revolution of the impeller is directly proportional to the velocity of the water flow. A small magnet is usually built into the shaft of the instrument and a coil detects the passage of the magnet and allows the number of revolutions of the shaft in the given time to be counted. Once the rate of revolution of the impeller is known the water velocity can be calculated using the calibration equation for the instrument, which is expressed as follows:

$$V = a + bn \quad \text{(3.6)}$$

Where,

*V* is the water velocity in meters per second,

*n* is the number of revolutions of the impeller per second,

*a,* b are instrument's specific constants.

The discharge measurement Q is determined by multiplying the velocity of flow and water flow cross-section area.

### Step 2: Low flow assessment

The assessment of low flow magnitudes of streams/rivers or springs in hydrology is important in the planning of a water supply system in view of the fact that it reflects on the water source adequacy and reliability to meet the consumer demand. In low flow hydrology, two questions are asked about a particular river identified to be a water source for a given water supply system:

- Does the river supply a particular water demand at all times?
- If not, how much water must be stored to meet any deficiency which may arise?

Flow duration curves, annual minimum flow analyses and annual drought volumes are applied to address the two questions.

## (a) Flow duration curve

The flow-duration curve (FDC) is defined as a cumulative frequency curve that shows the percentage of time specified discharges were equalled or exceeded during a given period. It combines in one curve the flow characteristics of a stream throughout the range of discharge, without regard to the sequence of occurrence. To prepare a flow-duration curve, the daily, weekly, or monthly flows during a given period are arranged according to magnitude, and then percentage of time during which specified flow values are equalled or exceeded are computed.

A flow duration curve once it is prepared, is used to determine the indices of low flow magnitudes; for example, the 96-percentile flow ($Q_{96}$), is the flow that is exceeded for 96 percent of the period of record. This discharge value is a useful index of low flow that is related to the quantity of water that can be available for water supply in the dry season.

The following steps are followed in constructing the FDC:

(i) Rank the observed stream flows in descending order (from the maximum to the minimum value).

(ii) Calculate exceedence probability (P) of each flow as follows:

$$P = 100\left(\frac{m}{(n+1)}\right) \quad \text{(3.7)}$$

Where,

P is the probability that a given flow will be equaled or exceeded (% of time),

m is the ranked position of a given flow value on the list,

n is the length of the sample.

(iii) An FDC is obtained by plotting each ordered observed streamflow value *versus the* corresponding calculated exceedence probability.

(iv) Read the indices of low flow magnitudes from the FDC corresponding to 90%, 95% and 99% probabilities of exceedence.

## (b) Low flow frequency analysis

The frequency analysis of low river flows is performed by analyzing 1-day or 7-day or 10-day annual minimum flow series obtained by selecting the lowest flow values occurring in each year of record. The set of observed annual minimum flow values recorded at any gauging station is assumed to be a random statistical sample from the population of all possible annual minima at the given site.

The selected set of observed annual minimum flow values is fitted to the Gumbel statistical distribution and then the annual minimum flow magnitudes ($Q_T$) corresponding to the design probability of failure (1/T) is then estimated from the Gumbel prediction equation:

$$Q_T = \mu + \alpha KT \quad \text{(3.8)}$$

Where,

$Q_T$ = Low flow magnitude

T = Return period of one failure in T years

$\mu$ and $\alpha$ = Gumbel Parameters

$K_T$ = Frequency factor, obtained as:

$$K_T = -\ln\left(-\ln\left(\frac{1}{T}\right)\right) \quad \text{(3.9)}$$

Results from low flow frequency analysis:

(i) If the value of $Q_T$ is large in comparison to $Q_D$, the average day water demand, then the river can be considered to be able to supply the demand satisfactorily.

(ii) On the other hand if $Q_T$ is less than or of the same order of magnitude as, $Q_D$, then the river alone without some form of flow regulation could not be considered satisfactory for supplying the demand.

## (c) Annual drought volumes Analysis

On the basis of the results obtained from low flow frequency analysis, in case the annual minimum flow magnitudes ($Q_T$) corresponding to the design probability of failure (1/T) is found to be less than $Q_D$, demand flow, then storage will be required to meet the established water demand. The required storage is determined by carrying out deficiency/drought Volumes Analysis.

The storage required on a river to meet a specific demand depends on the following factors:

- Variability of the river flow
- Magnitude of the demand
- Degree of reliability of meeting the demand

The capacity of the reservoir required to augment the river flow in any year can be determined from the analysis of the series of annual maximum deficiencies (drought volumes) as follows:

Drought volumes $V_1$, $V_2$, $V_3$,..., Vn are computed from a hydrometric record of the river flow ($Q_i$), with reference to the demand flow ($Q_D$), i.e., ($V_i = Q_i – Q_D$).

The set of observed annual maximum deficiencies at any gauging station is assumed to be a random statistical sample. The annual maximum deficiency ($V_T$) corresponding to the probability of failure (1/T) is estimated from the series of annual maximum deficiencies using a statistical distribution, e.g., Gumbel distribution as illustrated in previous section. The design storage of the reservoir can be made equal to the volume $V_T$ corresponding to a risk of one failure in T years.

### Step 3: Dependable Rainfall Analysis

Rainfall analysis is carried out when the need arises to determine dependable rainfall in a given area for the purpose of designing a rainwater harvesting system for domestic use. Frequency analysis of recorded annual rainfall data from a given area, enables the determination of the 90% dependable annual rainfall.

This is the value of rainfall magnitude that will be exceeded 90% of the time. In the design of Rainwater Water Harvesting system, catchment (i.e. roof) area and depth of rainfall are important parameters for estimation of optimal storage size. Taking note of the fact that rainfall amounts vary on a year to year basis, the computed rainfall magnitude that is exceeded 90% of the time, is taken as the value of annual rainfall depth that can be expected to occur with some degree of certainty and thus used in the design. The exceedance probability is determined by ranking the observed annual rainfall in ascending order (from the minimum to the maximum value) and then calculating non-exceedance probability (P) as follows:

$$P = 100\left(\frac{m}{(n+1)}\right) \quad \text{.......} (3.10)$$

Where,

P is the probability that a given rainfall will be equalled or not exceeded (% of time),

m is the ranked position of a given rainfall value on the list,

n is the length of the sample.

The probability of dependable rainfall is obtained by calculating the value of exceedence probability (1 – P).

**Step 4: Flood flow estimation for intakes and small dams' spillways**

The need to estimate flood peaks or design floods arises where it is required to design a spillway of a dam proposed for water storage and also the design of water intake structures. Water intakes and spillways of small dams are designed to accommodate the 100-year flood.

Frequency analysis of observed Annual Maximum streamflow records from a gauging station enables the estimation of flood peaks. The statistical distribution namely the Gumbel distribution or other statistical distribution used in Tanzania such as Pearson type 3, Log-Pearson type 3, Log-Normal and General Extreme Value can be used to carry out frequency analysis in order to determine the magnitude of flood peak of 100-year required for the design. The estimation of design flood peak magnitudes for specified return periods using the Gumbel Distribution is illustrated below.

**Prediction equation**

$$QT = \mu + \alpha KT \quad \text{.......} (3.11)$$

Where,

$Q_T$ = Flood peak magnitude

**T** = Return period of one failure in T years

$\mu$ and $\alpha$ = Gumbel Parameters

$K_T$ = Frequency factor, obtained as:

$$K_T = -\ln\left(-\ln\left(1-\frac{1}{T}\right)\right) \qquad (3.12)$$

**Estimation of Gumbel parameters by Method of Moments (MoM)**

Mean, $\mu = u + 0.5772\alpha$ ........................................................................ (3.13)

Standard deviation, $\sigma = 1.28\alpha$ ........................................................................ (3.14)

The mean,m and Standard Deviation values,s are computed from observed annual maximum streamflow records. Note that other frequency distributions (Pearson type 3, Log-Pearson type 3, Log-Normal and General Extreme Value) have different expressions for estimating the distribution parameters.

**Step 5: Application for water permit**

Water abstraction for water supply from a river or spring requires a permit from the respective water basin office. The planner of the water supply project must apply for the water permit abstraction early in the project design because it can affect the viability of a project. The design engineer must seek the water permit if the project involves a new, replacement, increased withdrawal from a source or an increase in the water system's physical capacity.

**Step 6: Environmental flow considerations**

Environmental flow may be computed in terms of magnitude, timing of low flow in the dry month, duration of low flow in days, frequency of occurrence of the low flow event (return period) and rate of change of low flow over time ($m^3$/day of flow recession). The recommended environmental flow varies for individual rivers and streams and therefore to determine its flow value, a comprehensive Environmental Impact Assessment (EIA) should be conducted and approved by NEMC. Also, there are some guidelines and procedures for environmental flow assessment for specific catchments in Tanzania developed by NEMC. Accordingly, designers need to consult NEMC for environmental flow information in their project areas (https://www.nemc.or.tz/).

**Step 7: Data to support hydrological analysis**

Hydrological data is invaluable for planning of water supply systems. For example, water source adequacy and reliability can be determined from analysis of streamflow data which is important hydrological data. Hydrological data should be collected by water basin offices in Tanzania to support the planning of water supply systems, specifically to answer questions related to the following:

- Water availability in terms of quantity and quality

- Frequency of occurrence of low flows and flood flows
- Variability of flow regime in terms of quantity and quality

Important data to be collected include the following:

(i) Streamflow – required to quantify available water and estimate flood peaks and low flow magnitudes,

(ii) Rainfall – required to determine 90% dependable rainfall,

(iii) Sediment - Sediment deposition affects the water carrying capacity of rivers and the useful life of reservoirs. Sediment data is required to determine the useful reservoir capacity and the life span of the reservoir.

(iv) Climate data - (Evaporation, Temperature, Wind speed, sunshine hours, radiation and humidity) – required to estimate water loss from reservoirs.

During the feasibility and preliminary design stage, the design engineer must look for streamflow records from stream gauging stations located at or near water intakes and dam sites to support the design work. In a situation where there are no gauging stations at or near water intakes or dam sites, two options may be considered to get flow data to be used in the design.

**Option 1:** Transfer data from adjacent or neighbouring drainage areas that have comparable or similar characteristics. The same applies to a situation where rainfall data is missing, rainfall data from adjacent or similar catchments is used to derive flow frequency/probability curves required in the design of water supply system.

**Option 2**: Install permanent or temporary gauging stations and start recording flow data at the earliest possible time during the planning steps of the water supply project.

## 3.6.3 Hydrogeological Analysis of Groundwater

The safe or long-term yield of a borehole or well can be defined as the maximum quantity of water that can be obtained permanently from the borehole or well. The safe yield must be estimated to see whether the planned abstractions for water supply purposes can be sustained in the long term. The long-term yield evaluation of a water supply borehole relies on the following factors: Estimations of recharge; Calculation of hydrogeological parameters such as:

- Transmissivity (T),
- Storage Coefficient (S),
- Skin Factor and others; and
- Analysis of aquifer boundary conditions

### 3.6.3.1 Pumping Tests

A pumping test is a practical method of estimating well performance, well capacity, the zone of influence of the well and aquifer characteristics (e.g., the aquifer ability to store and transmit water, anisotropy, aquifer extent, presence of boundary conditions and possible hydraulic connection to surface water). It consists of pumping groundwater from a well, usually at a constant rate, and measuring the change in the water level(drawdown) in the pumping well and any nearby wells (observation wells) during and after pumping (see Figure 3.5).

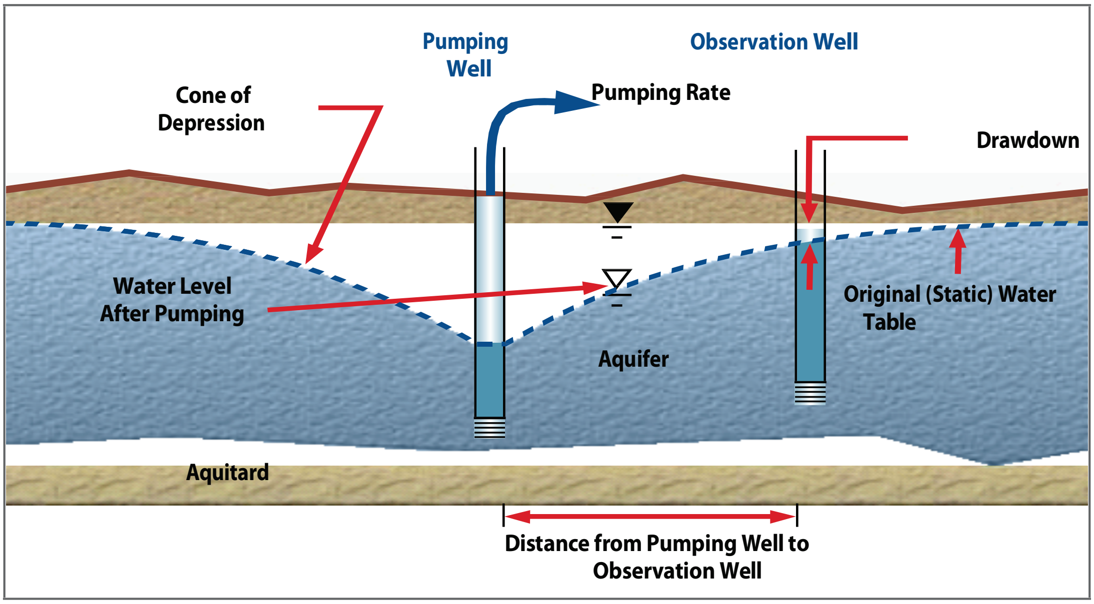



**Figure 3.5: Impact of a Pumping Well on the Water Table and Observation Well in an Unconfined Aquifer**
(Source: British Columbia Publications, A guide to conducting pumping Test, 2009).

Pumping tests can last from hours to days or even weeks, depending on the purpose of the pumping test and the intended use of the borehole. Traditional pumping tests last for 24 to 72 hours.

## (a) Purpose of conducting pumping tests

Pumping tests may be conducted solely to provide a greater confidence in the well driller's estimated well capacity. These pumping tests are typically shorter in duration (4 to 12 hours) and are commonly done on domestic or single-residence wells. Longer duration pumping tests are commonly required to:

- Provide proof of water availability under local government bylaws for new residential developments or regulatory requirements;
- Determine the maximum yield from a well;
- Assess impacts on neighboring wells or water bodies, such as streams, from the proposed use of the well; and/or
- Obtain hydraulic properties of the aquifer such as permeability, specific yield, transmissivity;

- Reveal the presence of any hydraulic boundaries;
- Obtain the general aquifer responses to water withdrawal from wells in the catchment;
- Provide information on water quality(Is the water quality suitable for the intended use? Are there likely to be any problems such as drawing saline or polluted water after extended periods of pumping?);
- Optimize operational pumping regimes;
- Help determine the correct depth at which the permanent pump should be installed in the borehole.

## (b) Pumping tests considerations

The preliminary studies before carrying out the pumping test assignment will require the knowledge of the following:

- Basic geology of the area (are the rocks crystalline basement, volcanic, consolidated or unconsolidated sediments)- Groundwater occurs in these rocks in different ways and behaves in different ways.
- Aquifer configuration (Is the aquifer confined, unconfined or leaky).
- Borehole construction (How deep is the borehole, its diameter, type of casings and screens installed, gravel pack material, size and shape)

Designing and planning a pumping test is critical and should be done first, before any fieldwork is done or equipment set up on the site. Lack of planning can result in delays, increased costs, technical difficulties and poor or unusable data. Things to consider in the pre-planning stage are:

- Time of year the pumping test will be done
- Natural variations in the groundwater levels
- Informing on who may be affected
- Depth of pump setting and type of pump
- Pumping duration
- Pumping rate
- Control and measurement of the pumping rate
- Frequency of changes in the water levels
- Measuring water levels in neighboring wells and/ or
- Streams
- Disposal of pumped water
- Collection of water samples for analysis
- Special circumstances to be aware of
- Accessibility of the well e.g., clearance from power lines, confined spaces, small pump houses, or nearby traffic.

## (c) Common types of pumping tests

The common types of pumping tests conducted include the following:

### Constant-rate tests

In this test, it is necessary to maintain pumping at the control well at a constant rate. This is the most commonly used pumping test method for obtaining estimates of aquifer properties.

### Step-drawdown tests

These tests proceed the sequence of constant-rate steps at the control well to determine performance characteristics such as well loss and well efficiency. They are designed to establish the short-term relationship yield and drawdown for the borehole being tested. It consists of pumping the borehole in the series of steps, each at a different discharge rate, usually with the rate increasing with each step. The final step should approach the estimated maximum yield of the borehole.

### Recovery tests

These tests use water-level (residual drawdown) measurements after the termination of the pumping. They are carried out by monitoring the recovery of water levels on cessation of pumping at the end of constant rate test. It provides a useful check on the aquifer characteristics derived from the other tests.

## (d) Acceptable borehole yields

Borehole yield is the volume of water that can be abstracted from a borehole. It is important not to over pump the borehole in order to prevent saline intrusion, encrustation, and excess lowering of the water table or piezometric surface or causing borehole failure. Acceptable borehole yields can be calculated using data obtained from pumping test applying several well flow equations as follows depending on the type of aquifer in consideration. The well-flow equations underlying the analysis methods were developed under the following common assumptions and conditions:

- The aquifer has a seemingly infinite areal extent;
- The aquifer is homogeneous, isotropic, and of uniform thickness over the area influenced by the test;
- Prior to pumping, the water table and/or the piezometric level is horizontal (or nearly so) over the area that will be influenced by the test;
- The aquifer is pumped at a constant-discharge rate;
- The water removed from storage is discharged instantaneous with decline of head.

Depending on whether the aquifer is confined, leaky, or unconfined, one can apply the following groundwater flow equations to analyze the borehole yields:

- Theis equation
- Theis-Jacob method

- Cooper and Jacob method
- Theim equation
- Hantush equation

In each case, data obtained from pumping tests (both during pumping and during recovery) are plotted on a semi-logarithmic paper (Figure 3.6).

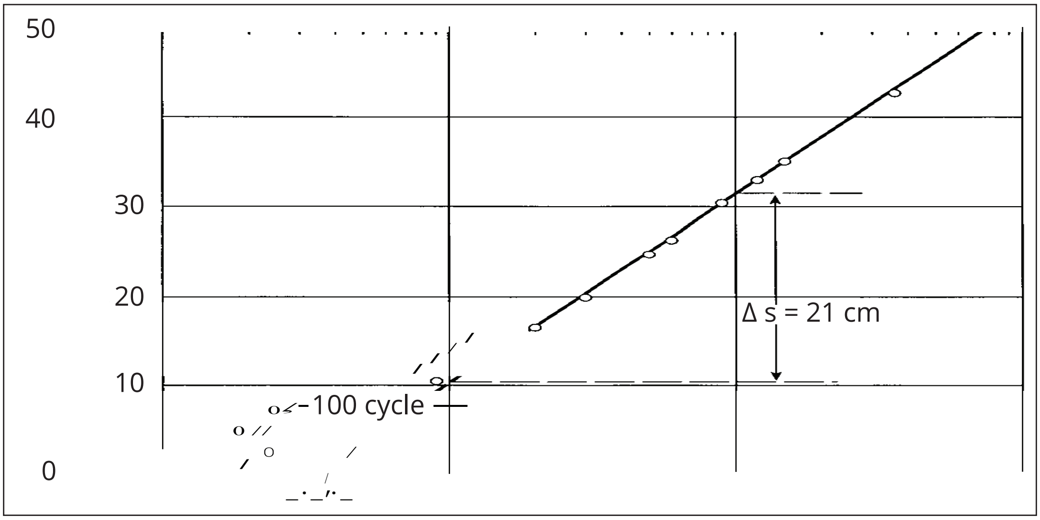



**Figure 3.6: A Plot of Drawdown Values s(r,t) Against the Corresponding Time t, on Semi-log Paper (with t on the Logarithmic Scale)**
Source: David K. Todd, Groundwater Hydrology, Year 2009.

**Table 3.3: Recommended Test and Duration to Estimate Sub-Surface Water Yield**

| Use of Water | Test | Duration | Recovery Test |
|---|---|---|---|
| Stock or domestic | Extended step | Total 6 hours | Up to 3 hours |
| Hand pump | Extended step | Total 6 hours | Up to 3 hours |
| Town water supply<br>Low-yield borehole | Step Constant<br>discharge | 4 x 1 hour<br>24 hours | -<br>Complete |
| Town water supply<br>High-yield or main borehole | Step Constant<br>discharge | 4 x 1 hour<br>72 hours or more | -<br>Complete |

(Source: MoW manual Uganda)

## (e) Software for analysing pumping test data

In the field, several groundwater software responsible for analysing pumping test data are available. Groundwater professionals chose different software to suit their need. The more preferred pumping test analysis software is AQTESSOLV because it features the most comprehensive set of solution methods for confined, unconfined, leaky confined and fractured aquifers.

Others include;

- Aquifer Test Pro 3.5
- SATEM 2002
- BGSPT
- AquiferWin32
- Schlumberger Aquifer Test Pro

More information on boreholes and detailed hydrogeological analysis is presented in Section 9.1.4.

## 3.7 OTHER CONSIDERATIONS FOR VARIOUS WATER SOURCES

### 3.7.1 Water Permits Considerations

During the course of implementation of water supply projects, designers will need to work with the relevant Water Basin Authorities and relevant catchment and sub-catchment committees to ensure all water users with water withdrawal permits are considered during the course of sizing the projects to ensure no developmental constraints are faced as a result of ignoring other users. It will be necessary to consult the updated water permits registers maintained by each Basin Water Board prior to planning expansion of any new water supply project.

### 3.7.2 Conservation of Water Sources

In line with NAWAPO, the protection and conservation of water sources is one of the main duties of all the Basin Water Boards. Intuitively, for national water resources the MoW also has the responsibility to deal with resolution of all water use conflicts. It will ensure that the WRM Act No.11 of 2009 as well as the associated regulations are fully observed by all parties. Other relevant laws such as those associated with pollution coordinated by other agencies or bodies like the National Environment Management Council (NEMC) are observed with respect to water resources. Where transboundary water resources are involved, the MoW has to ensure the protection roles that are expected of Tanzania are properly fulfilled in line with the relevant international laws, agreements or conventions stated in section 1.1.4.

## REFERENCES

Amit Kohli and Karen Frenken (2015). Evaporation from artificial lakes and reservoirs: AQUASTAT Programme, FAO 1:http://www.fao.org/3/a-bc814e.pdf

Dr. Sharafaddin Abdullah Saleh, Prof. Dr. Taha Taher and Prof. Dr. Abdulla Noaman (2017). Manual for Rooftop Rainwater Harvesting Systems. Water and Environment Center (WEC) – Sana'a University, Yemen

Chapter 4

# WATER DEMAND ASSESSMENT

## 4.1 WATER DEMAND ASSESSMENT

Water demand is the quantity of water that a source must produce to meet all the water requirements of a project. These include water delivered to the system to meet the needs of consumers, water supply for firefighting, system flushing, water required for the operation of treatment facilities and amount of water lost due to leakages in the infrastructure.

In planning and designing any water supply project, water demand assessments for current and future needs are of prime importance. Engineering decisions are required to determine the area and the population, industries, institutions and other existing and emerging consumers to be served, design period, the per capita consumption of various categories of consumers' pressure zones, amount of water likely not to be charged (NRW) and other needs of water in the area.

In addition, demand assessment may assist in determining the nature and location of various facilities to be provided such as source of water and capacity of water storage facilities (*FAO AQUASTAT, 2011 and MoW Design Manual, 2009*). For effective determination of water demand, designers need to critically assess the components of water demand for the planned water supply system. The following are the main components that should be considered when conducting water project demand assessment:

- Water demand for domestic use
- Institutional Water Demands
- Industrial Water Demands
- Water Requirements for Energy Cooling Systems
- Commercial Water Demand
- Livestock Water Demand
- Water for Fire Fighting
- Operational Demands
- System water losses
- Non-Revenue Water
- Net Water Demand (revenue water)

## 4.2 GENERAL FACTORS AFFECTING WATER DEMAND ASSESSMENT

Demand assessment is the most critical element in project planning for short, medium or long-term water projects (*Water Mission Technical Handbook of 2019 and MoW Design Manual of 2009)*. The complexity of water demand assessment to meet various socio-economic needs in a community may be brought about by many factors influencing individual water consumption patterns which include:

- Religion,
- Social economic status -cooking and health practices,
- Climatic conditions,
- Cultural, habits of a community,
- Age and education,
- Availability of alternative water sources,
- Level of service, technological process.

The combination of these factors may contribute to over or under estimation of the water demand in the given project area. Over-estimation of demand may justify a project that should not have been built. This then leads to unnecessary costs, over-estimation of intended revenue and premature implementation of a project. It is recommended that a thorough community survey be conducted for the project area to determine the magnitude of factors influencing water consumption pattern as described above.

The fundamental components of calculation of water demand are by determining the total population and the average daily consumption of individuals.

Apart from individual based water consumption pattern, institutions surrounding the community to be served have greater influence on the determination of the overall water demand in the project area. In view of this, designers of the water supply project must take into consideration the different water consumption patterns of institutions when calculating the total water demand in the project area. For each type of institution, specific requirements should be obtained from the institutions concerned.

Water requirements for each institution must be computed separately and aggregated into the overall water demand assessment. In the rural areas, allowance should also be given for connection to schools, dispensaries, health centres, offices etc. Losses should always be determined from the gross total water requirements and not from the net. For the best conceptualization of water demand, water requirement of each sector or sub-sector must first be computed separately and later aggregated with other computed segmental demands to obtain the overall project water demand. The formula below provides general consumers whose water demand may be required:

Total water demand = Domestic + Institutional +Industrial +Commercial +Livestock + Firefighting + other uses (Operational demand) + NRW ........................................................ (4.1)

It is worth noting that, the above formula is just a guide and should be adjusted according to the area in question. Additionally, in any assessments, demand assessment should aim at obtaining the design and operational information to optimise the net benefits to the community.

## 4.3 DETERMINATION OF WATER DEMAND FOR DIFFERENT USES

The foregoing section presented the components of water demand in a project. However, before establishing the project water demand, there is need to establish the *water consumption* of each consumer either individually or as an institution. Water consumption is the quantity of water that is directly utilized by the consumer. Water consumption is initially split into domestic and non-domestic components, non-domestic use includes Commercial Use, Institutional Use and Industrial Use, Fire Fighting and System Water Consumption.

To establish a project water demand, it is important to determine the prevailing water project consumption categories and their water requirements. The following steps detail a procedure to be followed when determining the project water demand:

Step1: Establish Domestic water consumption

Step 2: Establish Institutional Water Consumption

Step 3: Establish Industrial Water consumption

Step 4: Establish Water Requirements for Energy Cooling Systems (if they exist)

Step5: Establish Commercial Water consumption

Step 6: Establish Livestock Water consumption

Step 7: Establish Net Water Demand

Step 8: Establish Fire Fighting Water consumption

Step 9: Establish Operational water consumption

Step 10: Establish System water losses

Step 11: Establish Non-Revenue Water

Step 12: Establish Overall Water Demand

**Step 1: Establish Domestic water consumption**

Domestic water consumption is water utilised for household chores, such as bathing, cooking, washing, drinking, laundry, dishwashing, gardening, car washing and other less water intensive or less frequent purposes. Individual water use in rural, peri-urban or urban setting tends to differ considerably depending on levels of service.

**Levels of Service for domestic consumers:**

The amount of water consumed depends on the level of service provided, the following are five categories of level of service in domestic water use categories:

| | |
|---|---|
| Category one : | Low income using public taps or kiosks, |
| Category two : | Low income multiple houses with yard tap, multiple houses served from a single yard tap, |
| Category three : | Low income single household with yard tap, |
| Category four : | Medium Income Household, Medium income group housing, with sewer or septic tank. |

**High Income**

**Household:** High income group housing, with sewer or septic tank

Water use is at its lowest when water is distributed through water points (category one level of service-public taps or kiosks) with some walking distance from the home. When water is brought into the house by piping (regardless of whether the location is rural or urban), the consumption increases considerably. It is necessary to also note that there is no incentive for the consumer to utilize less water when water is supplied at a flat monthly rate. With waterborne sanitation and high standard of in-house installations (category five-bath, washing machine etc) the per capita consumption may be ten times more than from a public tap.

**Considerations when estimating domestic water consumption:**

The proposed figures for water consumption rates as given in Table 4.1 are a guide for designers. However to make an accurate estimate of the water requirements it is necessary to consider the following:

- Decide upon the use of different per capita consumption rate if water requirement survey data from the project area justifies,
- Divide the population to be served into consumption groups as per levels of service obtained through social economic survey data of the project area (combination of percentage of the population to be served from each service category),
- Use population density of residential plots if the data is available,
- Adjust the community water requirement if survey of alternative sources of water are available.

Establish the domestic water requirement by summation of individual domestic consumer category population multiplied by respective per capita consumption.

$$\textit{Domestic consumption} = \sum_{i=1}^{n} (\text{population consumer category})i \ x \ (\textit{per capita consumption consumer category})i$$

.................................................................................................................. (4.2)

**Table 4.1: Minimum Water Requirements for Various Categorised Domestic Users**

| Consumer Category | Consumption (l/ca/d) | | | Remarks |
|---|---|---|---|---|
| | FR | M-UT | M- PBT | |
| Low income using kiosks or public taps | 20 | 20 | 20 | Squatter areas, to be taken as the minimum. |
| Low income multiple household with Yard Tap | 50 | 45 | 40 | Low income group housing with pit latrine but no inside installation. |
| Low income, single household with Yard Tap | 70 | 60 | 50 | Low income group housing with pit latrine but No in-house installation. |
| Medium Income Household | 130 | 110 | 90 | Medium income group housing, with sewer or septic tank. |
| High Income Household | 250 | 200 | 150 | High income group housing, with sewer or septic tank. |

FR = Flat Rate; M-UT = Metered with Uniform Tariff; M-PBT = Metered with Progressive Block Tariff.

(Source: Modified from MoW 3rd Edition Design Manual 2009)

**Step 2: Establish Institutional Water Consumption**

When designing and planning for a water project, water requirements for present and anticipated institutions in the project area have to be computed using institutional water requirement data obtained in community survey. Public and private institutions include: Schools, Hospitals, Administration Offices, Army, Police, Missions, Churches and Mosques, Prisons, etc. In Table 4.2, some guiding figures for institutional water unit consumption are given. For water requirements for staff working in institutions see section 4.3.1. If large demand units are included in the scheme, such as Universities, major hospitals, boarding schools etc., a special study of their water requirements is recommended instead of using the average figures given in Table 4.2:

Additionally, water supply requirements for railway stations, bus stations, bus terminals, sea ports, airports including provisions for waiting rooms and waiting halls have to be considered separately in the planning and design of a water supply scheme. Table 4.3 provides general guidance for water requirements considerations in stations and ports. The number of persons should be determined by average number of passengers handled by the station daily and seasonal average peak requirements should be taken into consideration during design

**Table 4.2: Institutional Water Demands**

| Consumer | Unit | Consumption Litre/Person/Day | Remark |
|---|---|---|---|
| Schools<br>– Day Schools | 1/std/d | 10<br>25 | With pit latrine<br>With WC |
| – Boarding Schools | l/std/d | 70 | With WC |
| Universities and colleges | l/std/d | 60-80 | With WC |
| Health care Dispensaries | 1/visitor/d | 10 | Out patients only |
| Health | 1/bed/d | 50 | No modern facilities |
| Health | l/bed/d | 100 | With WC and sewer |
| Hospitals, District | l/bed/d | 200 | With WC and sewer |
| Hospitals, Regional | 1/bed/d | 400 | With surgery unit |
| Administrative Offices | 1/worker | 70 | With pit latrines<br>With WC |
| Prison | 1/Prisoner/day | 10-15 | Depending on climate and activities in prison |

(Source: Modified from MoW 3rd Edition Design Manual 2009)

**Table 4.3: Water Requirements for Ports and Stations**

| Nature of Station | Station Categorization | Where Bathing Facilities are Provided Litres /Capita | Where Bathing Facilities are not Provided Litres/Capita |
|---|---|---|---|
| Railways, bus stations and seaports | i. Intermediate stations (excluding mail and express stops ) | 45 | 25 |
| | ii. Junction stations and intermediate stations where mail or express stoppage is provided | 70 | 45 |
| | iii. Terminal stations | 45 | 45 |
| Airports | International and domestic airports | 70 | 70 |

(Source: National Building Code of India, 2016)

Establish the institutional water requirement by summation of individual institutional consumer category population multiplied by respective per capita consumption.

$$Institution\ consumption = \sum_{i=1}^{n}(\text{population Institution category})i\ x(per\ capita\ consumption\ Institution\ category)i$$

.............................................................................................................................. (4.3)

**Step 3: Establish Industrial Water consumption**

The water consumption in the industry varies considerably depending on the kind and size of the-industry. There are dry industries, which consume virtually no water in their processes, and the only water consumption is that for staff and cleaning of the premises. On the other hand, the water requirements for wet industries such as for a paper or cotton-processing factory can be a great deal. Table 4.4: gives unit water consumption in different kinds of industries. For an existing industry, the water consumption can be found out by checking their metered consumption or if there are-no records available by estimating according to the kind and size of production. The consumption figures for larger units must always be based on proper measurements and not on estimates.

**Table 4.4: Specific Industrial Water Requirements**

| Industry | Product or Raw Material Unit | Water Consumption in $m^3$ per Unit of Raw Material |
|---|---|---|
| **Food Industry** | | |
| Dairy | Milk received (1000 L) | 2 – 5 |
| Abattoir | Animal slaughtered (gross weight) | 4 – 10 |
| Brewery | Beer (1000 L) | 10 – 20 |
| Sugar | Cane (tonne) | 10 – 20 |
| Fish processing | Tonne | 7-9 |
| **Wood processing industry** | | |
| Pulp mill | Bleached pulp (tonne) | 100 – 800 |
| Paper mill | High quality paper (tonne) | 300 – 450 |
| Chipboard factory | Chipboard (tonne) | 50 – 150 |
| **Others** | | |
| Tannery | Raw skins (tonne) | 50 – 120 |
| Cotton mill | Cotton thread tufi | 50 – 150 |

(Source: Modified from MoW3rd Edition Design Manual 2009)

Future projections of industrial water requirements have to be established by direct interviews with the technical management of existing industries based on their production flow sheets and by contacts with the local planning officers and local Government officials, e.g. Municipal or Council officers. For future industries to be established, the Ministry of Industry and Trade, Regional Planning Officer or organizations such as the National Development Corporation (NDC), Small Industries Development Organization (SIDO) and owners of private industries should be consulted. Where there is only a reservation for an industrial area in the town/city plan but without any specifications; estimates of the future water requirements can be based on the figures given in Table 4.5.

**Table 4.5: Industrial Water Demand ($m^3$/ha/d) for Future Industries**

| Industry Type | Water Demand $m^3$/ha/d |
|---|---|
| Medium Scale (water intensive) | 50 |
| Medium scale (medium water intensive) | 20 |
| Small scale (dry) | 5 |

(Source: MoW, 3rd Design Manual 2009)

If the requirement of a particular industry is large compared to the existing or planned water supply system, then it will be necessary to establish the total demand and to consider identifying a separate local water source that must be examined whilst part of the requirement can be supplemented from the town or city water supply system.

Establish the industrial water requirement by summation of individual industry consumer category product volume multiplied by respective per capita consumption

$$Industry\ consumption = \sum_{i=1}^{n} (\text{Product Volume category})i\ x(per\ capita\ consumption\ Industry\ category)i \quad \text{.......... (4.4)}$$

**Step 4: Establish Water Requirements for Energy Cooling Systems**

For any water project planned in areas with high potential for thermoelectric investments, water intended for energy cooling system should be separated from the industrial demand (*Kohli and Frenken, 2011*). This is due to the fact that such plants require huge amounts of water, which in some occasions may be beyond the capacity of the water supply system. Table 4.6: provides guidance on water requirements for energy cooling systems.

**Table 4.6: Approximate Withdrawals and Consumptions, not Accounting for Ambient Temperature or Plant Efficiency**

| Plant and cooling system Type Water | Withdrawal (litres/MWh) | Consumption (litres/MWh) |
|---|---|---|
| Fossil fuel/biomass/waste \| once-through cooling | 76,000 – 190,000 | 1,000 |
| Fossil fuel/biomass/waste \| closed-loop cooling | 2,000 – 2,300 | 2,000 |
| Nuclear steam \| once-through cooling | 95,000 – 230,000 | 1,500 |
| Nuclear steam \| closed-loop cooling | 3,000 – 4,000 | 3,000 |

(Source: Adapted from FAO AQUASTAT Report, 2011)

Establish the plant water requirement by summation of individual plant cooling consumer category water withdrawal multiplied by respective per capita consumption

$$\textit{Plant consumption} = \sum_{i=1}^{n} (\text{Water withdrawal plant category})i \; x(\textit{per capita consumption plant category})i$$

.......................................................................................................................(4.5)

**Step 5: Establish Commercial Water consumption**

Commercial water consumption is sometime considered under institutional or industrial demands. The augmentation of such demands can cause technical errors in the process of design and projection of water demand. Commercial water consumption occurs in hotels, restaurants, bars, shops, small workshops, car wash, service stations, etc. The present water demand should be known by their metered water consumption, and at least, the bigger hotels, restaurants and services stations must be checked. Future water requirements can be based on the estimated development of this sector. Table 4.7: gives water consumption figures for hotels and restaurants.

If there is a reservation in the town plan for the future business area without any specification, the estimates must be based on per hectare demand. As a guide, a water demand of 10 - 15 $m^3$/ha/d for a non-specified commercial area in a new town plan can be adopted.

**Table 4.7: Commercial Water Requirements**

| Consumer | Unit | Consumption (L/pd) | Remarks |
|---|---|---|---|
| Hotels | L/bed/d | 70<br>200<br>400 | Low class<br>Medium class<br>High class |
| Bars | | 70<br>100<br>300 | Low class<br>Medium class<br>High class |
| Shopping Malls | | 70 -130 | |

(Source: Modified from MoW 3rd Edition Design Manual 2009)

Establish the commercial water consumption by summation of individual commercial consumer category population multiplied by respective per capita consumption

$$\textit{Commercial consumption} = \sum_{i=1}^{n} (\text{population consumer category})i \; x\,(\textit{per capita consumption consumer category})i$$

.......................................................................................................................(4.6)

### Step 6: Establish Livestock Water consumption

The water demand for livestock can be calculated using 25 L/stock unit per day. Water requirement for livestock should be included in water supply designs where feasible. However, emphasis should be placed on the use of dams, charcoal and water wells for livestock.

### Applicable definitions

For the sake of assessment of water demand for livestock the following grading of domestic animals in terms of a stock unit is to be used where one stock unit is equivalent to:

One head of cattle, or two donkeys, or five goats or five sheep (sometimes referred to collectively as shoats), or thirty head of poultry (hens, ducks, geese). Special cases include high grade dairy cows, where one cow is equal to 2 (or 3) stock units. Present livestock numbers can be found either by counting or inquiry and then converting into stock units.

Future populations of livestock may be taken at 25% growth in 10 years and 50% growth in 20 years, respectively provided the carrying capacity of the land allows it or otherwise the present figures can be taken for the future also. This decision should be reached in consultation with the competent authorities. What is of prime importance when considering both livestock watering and horticultural irrigation is that the costs of construction and O & M of such a scheme should still be within the affordable limits of the community.

Establish the Livestock water consumption by summation of individual livestock category population multiplied by respective per capita consumption

$$Livestock\ Water\ \ consumption$$

$$= \sum_{i=1}^{n} (\text{population consumer category})i\ x\ (per\ capita\ \ consumption\ consumer\ category)i$$

.......................................................................................................................... (4.7)

### Step 7. Establish Net Water Demand

Net Water Demand (revenue water) is obtained by summation of all the water demands as discussed in the foregoing sections (Domestic, institutional, industrial, and commercial demands).

**Net water demand** = Domestic consumption+ Institutional consumption +Industrial consumption+ Commercial consumption + Livestock consumption + Energy cooling

This is then the potential billable water consumption or revenue water

**Step 8: Establish Fire Fighting Water consumption**

The determination of fire-fighting requirements is an extremely complex issue due to the socio-economic dynamics, which might be taking place in the areas. Fire fighting requirements are necessary in urban areas and commercial rural centres, which are fast growing including airports and dry ports constructed in peripheral areas. Under normal design and operation 2% of the water demand should be set for fire fighting. It should be noted that the water supplied here normally forms part of the unbilled authorised consumption as in the IWA recommended water balance (Table 4.8).

The fire-fighting water requirements for industrial areas must be estimated separately. The principle should be that if the fire-fighting water demand is bigger than what is normal capacity of the distribution system, the industry in question must provide its own water reserve for fire fighting. Fire fighting reserve and storage requirement for private hydrant should be in accordance with the number of people served by a reservoir as presented in Appendix B and in the Fire & Rescue Force Regulations website available at *http://frf.go.tz/pakua/*

**Step 9: Establish Operational water consumption**

With the exception of water for fire-fighting discussed above, the operational water demands is required for operation of the water treatment processes such as clarifier de-sludging, rapid sand filter backwashing and chemical mixing, and operational activities such as the flushing out of reservoirs and the pipe work system through washouts and when cleaning bulk meter screens. Best practise is to get a good estimation of the actual amount, however, the estimate of five per cent (5%) is suggested for water treatment where it takes place and a further 2% for other operational demands be allowed for this water use also forms part of the unbilled authorised consumption as explained in the IWA recommended water balance (Table 4.8).

**Step 10: Establish System water losses**

In any design it is necessary to allow for water losses that are likely to occur. These will tend to increase over time and they depend on several conditions. Water loss in the system may take different forms including water determined for operations so care should be taken to avoid double counting for operational demands discussed in the foregoing sections. Technical water losses occur due to leakages and overflow from reservoirs, treatment units, break pressure tanks, valves, mains and distributions piping. Traditionally, it is suggested that overall, this can be taken at between 20 to 25% of the gross water demand (gross supply). However, experience shows that in urban areas except under the best situations, this can grossly underestimate what actually occurs. Other losses result from third party damage, usually resulting from either successful or unsuccessful attempts at illegally obtaining water for consumption. Others temper with the bills collection software to block or delay registration of funds received as experienced by some DAWASA customers.

In the case of pipe work, loss will relate to inadequacy of design, poor pipe selection, poor quality of manufacture and installation, operating pressure and in urban areas in particular, risk of third party damage including vandalism. In a zero-failures cost approach, the loss due to these elements can be equated to zero. Losses of water due to negligence of water consumers, unauthorized abstractions from the network, third-party damage including vandalism etc., have in the past rarely been considered for design purposes. However, to ignore them, passing them off as an operation and maintenance problem that should be controlled by those in authority is but to pretend the problem does not exist. Not only in estimating revenue should this element be considered but also in designing, specifying and implementing projects. Again, where the zero-failures cost approach has been adopted the losses attributable to vandalism should be minimal and can also be equated to zero.

Vandalism and illegal connections vary enormously. In small Tanzanian towns with a continuous water supply, a relatively small proportion of urban poor and with good controls, it can be quite small. In larger towns, with large populations of urban poor and irregular or rationed supply the same can be very significant.

**Step 11 Establish Non-Revenue Water**

Non-Revenue Water (NRW) in the supply system can originate from different causes including expected and un-expected causes. The designer for a water supply project has the responsibility to determine the possible amount of unaccounted for water likely to be experienced in the system to be constructed. More often, during planning and designing stages, Non – Revenue Water is neglected or partially computed through water loss calculated in the treatment units. Table 4.8 (IWA water balance) provides guidelines, which may be employed to come up with possible NRW in the water supply system. In case the water loss has been determined as explained in the foregoing section, the designer will have to undertake analysis to determine the likely sources and percentage of unbilled authorised consumption and then incorporate it in the appropriate system component.

Establish the Non-Revenue Water (NRW) by summation of unbilled authorised consumption and water losses.

$$Non\ Revenue\ Water(NRW) = \text{Water Losses} + \text{Unbilled Authorised Consumption} \quad (4.8)$$

**Table 4.8: The IWA 'Best Practice' Standard Water Balance**

| System Input Volume (corrected for known errors) | Authorised consumption | Billed Authorised Consumption | Billed Metered Consumption (including any water exported) | Revenue Water |
|---|---|---|---|---|
| | | | Billed Unmetered Consumption | |
| | | Unbilled Authorised Consumption | Unbilled Metered Consumption | Non-Revenue Water (NRW) |
| | | | Unbilled Unmetered Consumption | |
| | Water losses | Apparent Losses | Unauthorised Consumption | |
| | | | Customer Metering Inaccuracies | |
| | | Real Losses | Leakage on Transmission and/or Distribution Mains Leakage and Overflows at Utility's Storage Tanks | |
| | | | Leakage on Service Connections up to point of Customer Metering | |

(Source: MoW 3rd Edition Design Manual 2009)

**Step 12: Establish Overall Water Demand**

Water demand is calculated by summation of all the consumption categories (Net Water Demand) explained in the preceding sections and making allowance to NRW (normally expressed as a percentage of whole consumption),

$$\textit{\textbf{Water Demand}} = \frac{\text{Net Water Demand}}{(1 - NRW)} \quad \text{.......} (4.9)$$

However, in estimating water demands discussed in the above sections, variations in water consumption must be taken into consideration. The following section discusses the variations in water consumption that the water supply designers should take into consideration

# 4.4 VARIATIONS IN WATER CONSUMPTION

## 4.4.1 Definitions

Though water demand is normally calculated according to the average requirements, actual consumption varies from hour to hour and from day to day. Due to this non-uniformity of water demand, provision needs to be made in different units of the water supply system to cater for these variations. To evaluate the importance of variation in water consumptions, the following definitions are relevant:

- **Average Annual Demand (Qaa)** - The total volume of water delivered to the system in a full year expressed in cubic meters. When demand fluctuates up and down over several years, an average is used.
- **Average Daily Demand (Qda)** - The total volume of water delivered to the system over a year divided by 365 days. The average use in a single day is expressed in cubic meter per day.
- **Maximum Month Demand (Qmmax)** - The cubic meter per day average during the month with the highest water demand. The highest monthly usage typically occurs during a summer month.
- **Peak Weekly Demand (Qwmax)** - The greatest 7-day average demand that occurs in a year expressed in cubic meter per day.
- **Maximum Day Demand (Qdmax) -** The largest volume of water delivered to the system in a single day expressed in cubic meters per day. The water supply, treatment plant and transmission lines should be designed to handle the maximum day demand.
- **Peak Hourly Demand (Qhmax)** - The maximum volume of water delivered to the system in a single hour expressed in cubic meter per day. Distribution systems should be designed to adequately handle the peak hourly demand or maximum day demand plus fire flows, whichever is greater. During peak hourly flows, storage reservoirs supply the demand in excess of the maximum day demand.

Summary of relationships between patterns

$K_d = Q_{dmax} / Q_{da}$ = peak day factor ........................................................ (4.10)

$K_h = Q_{hmax} / Q_{dmax}$ = peak hour factor ..................................................... (4.11)

For design purposes, the peak factor shall be selected by giving consideration to the size and kind of the scheme and services required. Generally, an intake and the supply main from the intake to the treatment works is dimensioned to meet the peak day demand. For gravity schemes this means a main designed for a flow during 24 hours, while for pumping main the design flow is according to the pumping hours decided upon. This gives the minimum size of pipe required and where the water is treated, the required ultimate dimensions of the treatment units.

## 4.4.2 Variation in the Rate of Consumption

The average-rate of supply per capita is in fact the mathematical average taken over an average year. Thus, if Q is the total Quantity of water supplied to a population 'P' for 365 days, then the average rate of daily consumption 'q' is given by the following equation:

q = Q/(P x 365) litres/capita/day ........................................................ (4.12)

The types and nature of variations of demand of which ‘q’ is an average are given below.

### 4.4.2.1 Diurnal Variation in Water Demand

The consumption of water is not uniform throughout the day. Generally, two peak periods of demand are observed, one in the morning and one in the evening. The maximum intensity of demand which occurs in the morning varies between about 1.5 and 2.4 times the average demand for the day depending upon the size of the population served.

Large impounding reservoirs are designed to cater for the variations in demand and to ensure a steady delivery of water for a period from one drought year up to *three* consecutive dry years. For the supply channels and mains for the conveyance of this water to the treatment works, a figure of between 1.35 and 1.50 times the average demand rate ‘q’ is used. The pumping stations and treatment units are usually designed by taking 1.35 to 1.5 times the average demand rate since they are to meet the peak seasonal requirements. If the pumping is for less than 24 hours, then the above rate must be multiplied by the ratio of 24 hours to the hours of pumping. Transmission mains are designed to meet peak daily demand although distribution mains in the supply area are required to meet peak period demand and in urban areas are designed for up to 2.5 times the average rate of demand.

### 4.4.2.2 Peak Factors

Though peak factors are necessary for calculating the actual peak period demand, application in individual cases is difficult. The population data such as the number of people using each water point, number of hours per day, people’s habits in collecting water is difficult to obtain and again may vary by season or by school calendar. Hence use of peak factors to calculate an individual’s demand is impracticable and the figures given in Tables 4.9 and 4.10 are simply rough guides. Peak factors however, must be adopted for the dimensioning of the various components in a water distribution network. For every scheme, a study is required to establish the appropriate water demand and for large schemes it must be done thoroughly.

For small rural water supply schemes, a simpler method may be adopted to derive peak flows beyond the last storage tank with the peak flow for human consumption being given by the expression:

(Average daily demand × 4) / 24 .................................................................. (4.13)

That assumes the peak demand is four times the average hourly demand or in other words the daily demand is drawn over 6 hours during the two peak periods of the day.

For livestock, the usual practice is to work on the basis of a ten-hour period such that the peak flow may be taken as:

Daily livestock demand / 10 ............................................................................ (4.14)

Thus in small rural water supply schemes providing water both for human consumption and livestock, the peak flow used for dimensioning of the distribution mains is the sum of the peak flow for human consumption and the peak flow for livestock consumption.

In many urban areas and particularly in those that have experienced past restrictions in water supply, a number of consumers and especially those in high cost areas and institutions etc., will have constructed their own ground level or below ground storage, often well in excess of their daily requirements. In such instances, the actual peak hour flow can be noticeably less than 2.5 indicated in Table 4.9. In the absence of information to the contrary, peak factors that may be used for different consumer categories are as given in Table 4.9:

**Table 4.9: Consumers' Peak Factors**

| Consumers | Peak Day Factor | Peak Hour Factor | Remarks |
|---|---|---|---|
| **1. Residential** | | | Should be based on study of population density housing type and water pressure tests in the area. |
| – Low class housing | 1.50 | 2.50 | |
| – Medium class housing | 1.30 | | |
| – High class housing | 1.10 | | |
| **2. Public Institution** | 1.10 | 4.00 | In particular cases of ablution blocks of police lines and field force unit quarters a peak hour factor of 3.0 shall be adopted. |
| 2.1. Prisons | 1.10 | | |
| 2.2. Primary schools | 1.10 | | |
| 2.3. Secondary schools | 1.10 | 2.00 | |
| 2.4. Colleges | 1.10 | 2.50 | |
| 2.5. Hospitals | 1.10 | | |
| 2.6. Dispensaries | 1.20 to 2.30 | 1.80 | |
| **3. Commercial** | | 1.80 to 2.50 | |
| **4. Industrial** | 1.00 | 3.40 | Appropriate peak factors should be used for specific industries depending on factory sizes, process involved number or employees working hours, natural of reticulation system, if any based on industrial survey |
| **5. Domestic Points** | 1.00 | 2.0 – 3.0 | |
| **6. Livestock** | | | Constant flow or flow control device 10n 1/min for 12hrs n = number of taps (10 x n)/60 1/s |

(Source: MoW 3rd Design Manual 2009)

When it is impossible to separate consumers into categories as given in the previous table, but the total population of the area is known Table 4.10, can be used as a guideline.

**Table 4.10: Peak Factors for Known Population in the Area**

| Population | Range of Peak Factors | |
|---|---|---|
| | Peak Day Factor | Peak Hour Factor |
| 10,000 | 1.80 - 1.50 | 2.40 - 2.0 |
| 10,000 - 30,000 | 1.50 - 1.40 | 2.0 - 1.70 |
| 30,000 - 100,000 | 1.50 - 1.30 | 1.70 - 1.60 |
| 100,000 | 1.30 | 1.60 - 1.50 |

(Source: MoW 3rd Design Manual 2009)

## 4.4.3 Predicting Water Demand

It is advisable to project demand for short, medium- and long- term periods and to update this at least every five years. The Water Mission Handbook, 2019 provides guidance on means of computing the water demand in various situations. It categorizes demand predictions into three levels namely:

(a) Maximum Demand After Commissioning
This is calculated by assuming that 100% of the members of the population will each use full individual consumption amount.
Therefore MDAC = Total population Served x Maximum individual consumption

(b) Anticipated Demand After Commissioning (ADAC)
This is computed based on the assumption that not all 100% of population in the community will use the maximum amount of water. So predictions of the percentage of the population that will use the water need to be estimated. Additionally, it may be decided that individual consumption may be less than full amount determined in the time period just after commissioning of the system.

$$ADAC = Total\ Pop.served\ x\ \frac{Aniticipated\ \%\ of\ Pop}{100}\ x\ Anticipated\ Individual\ Consumption \quad (4.15)$$

(c) Maximum Future Demand
This type of water demand is computed to determine the system sustainability and capacity to meet the future demand growth. It is worth to note that the future demand can be hard to predict. However, inputs like population growth, migration, trade, urbanization and any significant changes in the national policies can be used to facilitate determination of projected future demand in the project area using data from the National Bureau of Statistics (NBS).

The population growth rate (r) is among the factors, which are predominantly used to facilitate the computation of the future demand. The future population in a project area may be computed using the following formula.

$P_n = P_O (1 + r / l00)^n$ ............................................................................(4.16)

Where,

$P_n$ = population after n years,

$P_O$ = present population, and

r = annual growth rate (%)

Once the predicted future population is determined, it can be multiplied by the anticipated population to be served and individual consumption to determine the maximum future demand (MFD).

$$MFD = \text{Future Pop.} \times \frac{\text{Anticipated \% Pop.}}{100} \times \text{Anticipated individual consumption} \quad \ldots\ldots (4.17)$$

## 4.4.4 Design Water Demand

All three calculated demands will have an impact on the safe water system design. It is recommended that the system be initially designed to meet the Anticipated Demand After Commissioning. Once that design is completed it can be compared to the Maximum Demand After Commissioning and the Maximum Future Demand. If the design can meet all the three calculated demands, this increases the confidence level of the design. However, if the design does not meet the other two demands, this does not necessarily mean that the design should be changed.

Instead, the design engineer can establish what changes could be made to meet the other two demands. If meeting these would take a minimal amount of additional equipment and cost, then doing so would be appropriate. However, if meeting the other two demands would take a large amount of additional equipment and cost, then it may not be economical or practical. Under such situations, decisions will have to made after discussing with the authority or the client. The first consideration tobe considered should be to increase the storage capacity of tanks.

## REFERENCES

Amit Kohli and Karen Frenken (2011). Cooling water for energy generation and its impact on national-level water statistics: http://www.fao.org/3/a-bc822e.pdf

FAO AQUASTAT Report, 2011

MoW(2009). The 3rd Edition Design Manual, 2009

Marco, Farina et al (2011). Water consumptions in public schools. Available online at www.sciencedirect.com

Reynolds, C., Steedman, J.C. and Threfall, A.J. (2008). Reinforced concrete Designers Handbook. Francis and Taylors.

Water Mission Technical Handbook of 2019: Version 1.2

WaterSustainabilityinPrisons:WaterEfficiency:TheJournalforWaterResourceManagement. http://digital.waterefficiency.net/display_article.php?id=2349297&view=284543. Retrieved on 29th December, 2019

Chapter 5

# PIPELINES DESIGN

The quantity of water to be supplied to a particular community has to be conveyed from a source to consumers. Normally, the medium through which water has to be conveyed is in pipelines. This chapter describes the design of pipelines. It presents the calculation of pipelines hydraulics and associated design approaches.

## 5.1 DESIGN REQUIREMENTS OF PIPELINES

There are five principle operational requirements for a pipeline. The requirements are:

- It must convey the quantity of water required at the design pressure,
- It must be capable of resisting all external and internal forces,
- It must be durable and meet the design working life,
- It must be properly laid and embedded,
- The material from which it is made should not adversely affect the quality of the water being conveyed.

## 5.2 TYPES OF PIPELINES

Broadly, there are two types of pipelines which should be considered for design. They are transmission and distribution systems. Transmission and distribution systems vary in size and complexity but they all have the same basic purpose, which is to convey water from the source(s) to the consumer.

## 5.3 RIGHT OF WAY FOR WATER PIPELINES

When designing a water supply project, the pipeline route should be carefully located. It should be accomplished by ensuring pipeline way-leaves. For security reasons marker posts should be provided for the boundaries of the way-leave. For all pipelines it is important to obtain and secure a way-leave so as to avoid problems later on. Even in road reserves the alignment should be agreed with the road authority in advance and officially recorded so that even many years later there can be no argument when it comes to any dispute or compensation claim.

### 5.3.1 Methods of Water Transmission and Distribution

There are three (3) methods which should be considered when transporting water from the source to the treatment plant, if any, and the distribution system thereafter to eventually to consumers. The methods are:

- Through gravity flow,
- Through pumping with storage,
- Through direct pumping to the distribution system.

### 5.3.2 Gravity Flow

This is the ideal set-up when the location of the water source is at a considerably higher elevation than the area to be served. The operation cost of a gravity system is very low, as it does not require energy cost.

### 5.3.3 Pumping with Storage

Water is either (a) pumped to a distribution pipe network, then to consumers, with excess water going to a storage tank, or (b) pumped to a storage tank first, then distributed by gravity from the tank to the consumers. The maintenance and operation cost of this system is higher than the gravity system.

### 5.3.4 Direct Pumping to the Distribution System

In this system, water is pumped directly from the source to the distribution system to the consumers. Where capital cost for a reservoir is not affordable at the initial stage of the water system, direct pumping to the distribution is usually resorted to. Variable speed or variable frequency drive pumps are most ideal for direct pumping operations, but the capital costs for such equipment are higher than for conventional water pumps.

## 5.4 PIPELINE HYDRAULICS ASSESSMENT

### 5.4.1 Pressure

Pressure is generally expressed in $N/m^2$, also called Pascal. Because of the level or amount of pressure in a water supply system, pressure is commonly expressed in kilopascals (kPa) or simply in meters (m).

Pressure increases linearly with the depth of water. For water at rest, the variation of pressure over depth is linear. The pressure exerted by a column of water is called pressure head (h) and can be calculated using the formula below:

$$h = \frac{p}{\gamma} \qquad \text{(5.1)}$$

Where, $p$ = pressure; $\gamma$ = specific weight of water

### 5.4.2 Determination of Head Losses

The commonly used formulae for computation of head loss due to friction (also called friction loss) are:

- Darcy-Weisbach formula
- Hazen-Williams formula
- Manning's formula
- Combined Darcy-Weisbach and Colebrook-White equation

This Manual recommends the use of Hazen Williams among the above formulae. The formula, which is the most widely used, relates the velocity of the flow, hydraulic mean radius and hydraulic gradient. In terms of head loss due to friction, the formula is:

$$h_L = \frac{10.7LQ^{1.852}}{C^{1.852}\,D^{4.87}} \qquad (5.2)$$

Where,

$h_L$ = head loss due to friction,

L = distance between sections or length of pipeline (m),

C = Hazen – Williams C-Value; D = internal diameter (m),

Q = pipeline flow rate ($m^3$/s).

The C-value is a carrying capacity factor that is sometimes referred to as the roughness coefficient, which varies depending on the pipe material being considered. Higher C-values represent smoother pipes and lower C-values are for rougher pipes. Higher C-values indicate higher carrying capacities. C-values increase with pipe size but decrease with pipe age. Although C-values are affected by changes in flow rates, the effect is negligible. Thus, network designers usually assume uniform C-value for different flow rates. Table 5.1 presents the recommended C-values for various pipe materials.

**Table 5.1: Recommended Pipe C-values (New Pipes)**

| Pipe Material | Diameter | Recommended C-Values |
|---|---|---|
| Plastic | 300 mm | 150 |
| | < 300 mm | 140 |
| Iron | 300 mm | 140 |
| | < 300 mm | 130 |

(Source: Philippines Rural Water Supply Design Manual, 2012)

Another contributing component of total head loss is the head loss from turbulence due to pipe fittings and appurtenances. This category of losses is sometimes called minor losses. The total minor losses in a distribution network is usually insignificant compared to the total head loss of the system, thus, the designer may ignore this component in network analysis computation.

## 5.5 WATER SUPPLY TRANSMISSION SYSTEM

The transmission system's function is to transport water from source to the reservoir, if any, and to the distribution point. Water conduits for the transmission system may be canals, aqueducts or tunnels, free-flow pipelines, or pressure pipelines. The transmission of water will either be by gravity or pumping. Pressure pipeline is generally the type of water conduit used for water supply transmission systems.

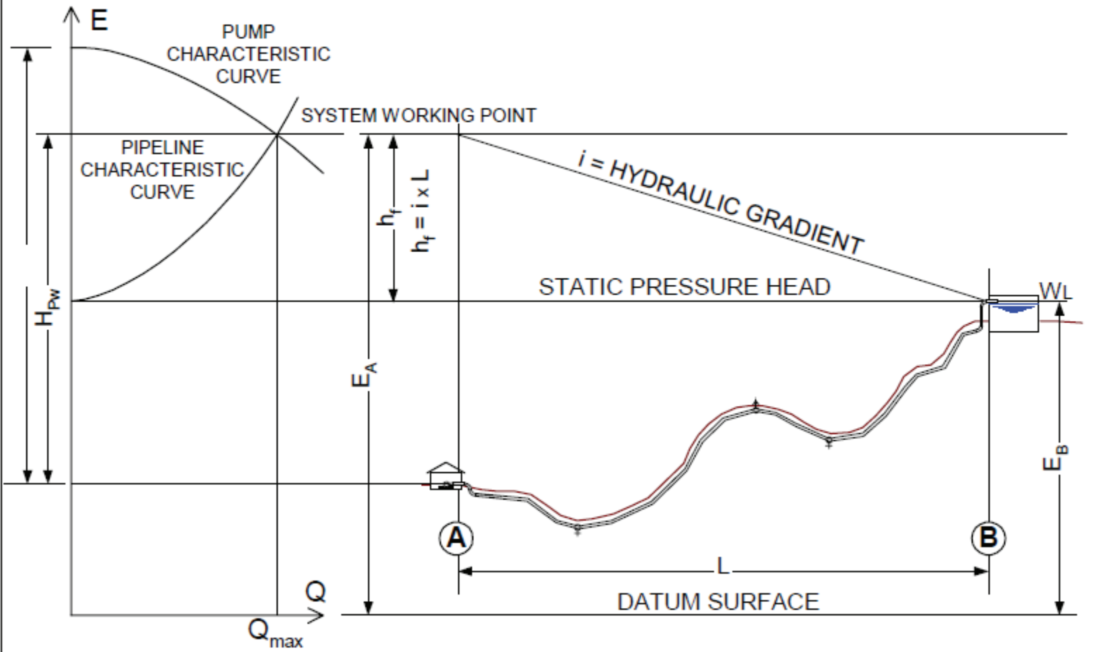
### 5.5.1 Determination of Transmission Pipe Size

Normally, the sizing of the transmission main is dependent on the total storage capacity and the way the supply is transmitted to the distribution system. The main should have at least the carrying capacity to supply water at a rate equivalent to the maximum day demand of the system for a given design year.

***As a rule of the thumb, for transmission by pumping, it is advisable to assume a preliminary head loss (hL) of about 5.0 m/km of pipeline. (As much as possible, head loss should be limited to 10.0 m/km of pipeline for transmission by pumping)***. For a gravity system with a considerably elevated source (e.g. highland springs), the transmission line could afford to have higher head losses as long as the remaining pressure head at the downstream end is sufficient for the distribution system's needs. For a gravity system with source elevation that is not much higher than the distribution system, the head losses are lowered to attain just sufficient pressure head in the distribution system.

### 5.5.2 Maximum Pressure

The pipe material should be selected to withstand the highest possible pressure that can occur in the pipeline. For a gravity system, the worst-case scenario is for pressure to be at its maximum during shut-off at the downstream end when the static pressure is too high. For the transmission line design, a maximum computed HGL based on a minimum supply rate equivalent to 0.3 times the average day demand should be examined. However, practical experience from the ground suggests that there is no need to limit the maximum allowable pressure to 60m head. Instead, considerations should be given on the basis of the economic grounds. However, in any circumstances where the maximum allowable pressure be exceeded, ,break pressure tanks should be installed along the main. The break pressure tank will limit the static pressure by providing an open water surface at certain points of the transmission line. Design of transmission line should be undertaken with the use of hydraulic computer software, which is discussed in the succeeding sections. Figure 5.1 below shows the longitudinal section of pumping main showing hydraulic gradient and characteristic curve




**Longitudinal Section of Pumping Main showing Hydraulic Gradient & Characteristic Curve**

**Figure 5.1: Longitudinal Section of Pumping Main Showing Hydraulic Gradient and Characteristic Curve**

## 5.6 DISTRIBUTION SYSTEM

For purposes of designing pipelines, the distribution systems are considered in terms of the topology or layout that is used. There are two types:

- Branched system and
- Looped System

### 5.6.1 Branched System

Also referred to as a Dead-end System, the size of the main line in this distribution system decreases as its distance from the source increases, in consideration that the further pipes have to carry less water. The design of a branched system is generally straightforward, where the direction of water flow in all pipes and the flow rate can be readily determined.

One of the advantages of a branched system is generally lower costs.The disadvantages are:

- A main break will cause all downstream consumers to be out of service.
- It results in poor chlorine residuals and aging of water in low demand areas.
- During high demands, the velocities are faster, hence head losses are higher.

## 5.6.2 Looped System

A distribution network is looped when there are only a few or no pipe dead-ends, such that water can move through the system freely. The advantages of a looped system are:

- The lower water velocities in the main reduce head losses, resulting in greater capacity,
- Main breaks can be isolated, minimizing service interruptions to consumers,
- Usually better chlorine residual content is achieved.

The disadvantage is generally more costs because of the need for more pipes to create the loops.

A major transmission design consideration is to ensure that if any section of the distribution main fails or needs repair, that section can be isolated without disrupting service to all or a great number of users in the network. Figure 5.2 illustrates the dead end (A) and the looped system (B).

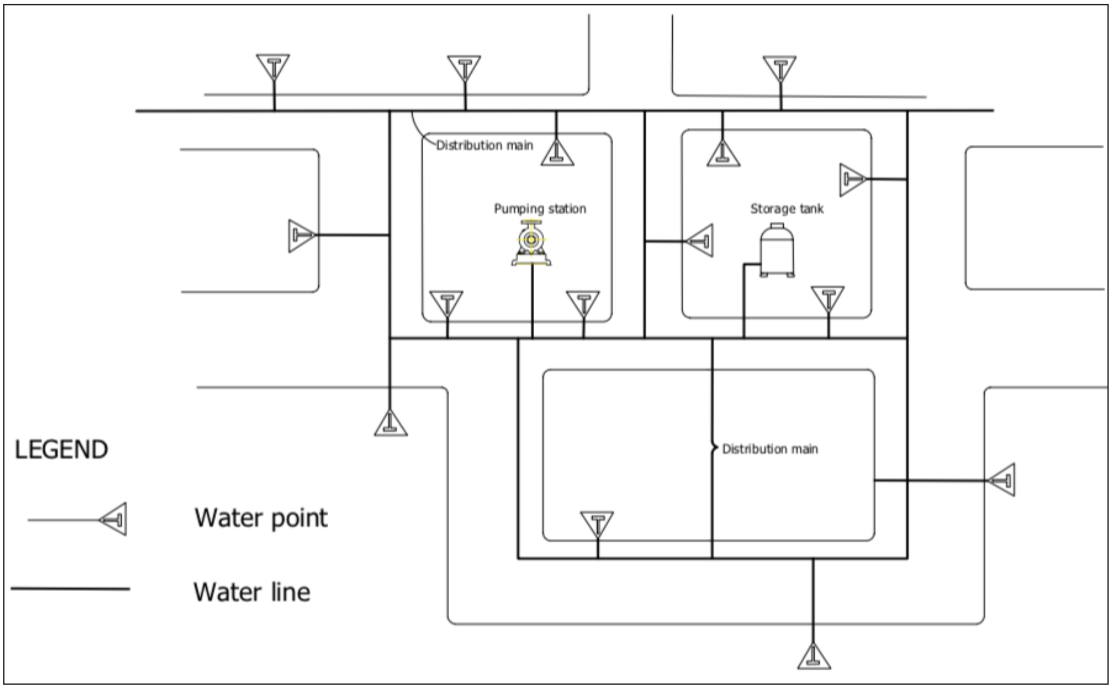



**Figure 5.2: Distribution System Basic Layouts**

## 5.7 PIPE NETWORK ANALYSIS

Pipe network analysis involves a detailed and careful scrutiny of the fluid flow through a hydraulic network containing several interconnected branches and loops. In the design of a distribution system, a pipe network analysis must be done to determine the flow rates and pressure drops in the individual sections of the network, thus giving a basis for selecting pipe diameters.

### 5.7.1 Network Analysis by Conventional Method (Hardy Cross)

The most common conventional method (not using computers) that is used in designing hydraulic networks is the Hardy Cross algorithm method. This involves iterative trial and error. One approach of Hardy Cross is the method of balancing the heads on the nodes by adjusting assumed flows in the pipe network.

Nowadays, manual computation for hydraulic network analysis is only acceptable when applied to systems with only a single pipeline or branched network with no loop. For networks with loops, it is highly recommended to use one of the more accurate, fast and convenient network modelling computer software, which are discussed in the following section.

### 5.7.2 Network Analysis by Computer Software

There are several pipe network analysis software (also called network simulation software, or hydraulic network modelling software) that mathematically solve hydraulic equations for all interconnections, branches and loops of the pipe network. With the advent of such powerful software, the conventional methods of water distribution design have been mostly discarded. The computer software requires the designer to create a water supply system model by inputting in the computer programme information that includes pipe lengths, junctions or node elevations, connectivity of the pipes and nodes, demand in each node, information on pumps, elevations of reservoirs, elevations and yield of sources.

Among the current software available on the weband from proprietary sources, the EPANET is highly recommended. EPANET is public domain software developed by Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency that can be downloaded for free on the internet. The software tracks the flow of water in each pipe, the pressure at each node, and the height of water in each tank.

The important features of EPANET for distribution network design is its ability to:

- Handle systems of any size;
- Compute friction head loss using the Hazen-Williams, the Darcy Weisback, or the Chezy-Manning head loss formula;
- Includes minor head losses for bends, fittings, etc.;
- Models constant or variable speed pumps;
- Allows storage tanks to have any shape.

The design process using EPANET usually involves the (a) layout of the system configuration including locations of sources and storage facilities, (b) determination of the distribution of demands to the nodes, input of network data, running hydraulic simulation, viewing results in any of the variety of formats, modifying the model by editing the network data, and modifying the model until the design criteria are met or results are acceptable.

### (a) Steps in Distribution System Design using Computer Software

#### Step 1: Base Mapping

Detailed maps of the project area should be gathered as a basis for pipeline alignment, distance and elevations. These should be obtained from field topographical surveys or certified suppliers of map products. For preliminary analyses, the Google Earth (http://earth.google.com) internet site which makes it possible to view and print aerial images of the area being designed may be used. However, accuracy of elevations from open source products is discouraged.

The information in the maps should be correlated to produce a base map, on which the proposed system layout will be drawn. The designer should conduct an ocular inspection of the whole project area to verify, validate and update the information on the source maps. The resulting base map must include positions and information on roads, streets, rivers, creeks, elevations, topographic contours and locations of built-up areas. It should also provide relevant information like potential large consumers, e.g., piggeries and poultry farms. Ideally, the base map should be scaled.

#### Step 2: Water Demand Projection

The average day demand for the design year will be the basis of the hydraulic network analysis. The demand condition will be varied by adjusting the demand factor; that is 1 for the average day demand condition, 1.3 for the maximum day demand and 2.5 – 3 for the peak-hour demand. More details on demand projections are given in Chapter Four.

#### Step 3: Tentative Layout

Using the base map, the designer should next develop a tentative layout of the pipe network, which should also show the positions of the source(s) and reservoir(s).

Pipelines should be laid in the mandated wayleave or utilities corridor preferably along roads, and the network should cover the target consumers. Nodes are placed at locations for pipe junctions, street or road junctions and intersections, locations for water points, demand centres, and not more than 100 metres from the nearest node. In systems where it is expected that pressure will be generally low or fluctuating, nodes are placed at the highest points of the service area.

#### Step 4: Distribution of Demands

It is important to plot the town or community boundaries and the service area delineation on the base map. Once the tentative layout (with nodes) is plotted on the base map, the service area should be sub-divided into node areas. This will give the designer a working idea of the respective number of houses within the area covered by each node.

The projected average day demand for the design year is distributed to all the nodes within its delineated service area. The distribution of demands should take into consideration the relative number of houses for the different node areas.

**Step 5: Encoding of Input Data**

Most of the hydraulic analysis software have common input data requirements. These data are grouped into pipe data and node data. Pipe data are the assigned pipe number, pipe diameter (mm), C-value, the pipe nodes, and length (m). Node data are node number, elevation (m), and water demand (lps). Usually, the values of the design criteria are required by computer software.

**Step 6: Hydraulic Network Simulation:**

This step is done by the computer software. If all the data required has been input by the designer, the software could proceed with its hydraulic run. The software computes the head losses (m) in each pipe, the rate of head loss (m/km) in each pipe, the flow velocities (m/s), and the pressure in each node (m).

The model is run for: (a) its peak-hour demand condition, to check for the possible value of the minimum systems pressure; and (b) its minimum demand condition, to check for the value of the possible maximum pressure in the network.

**Step 7: Examination of Hydraulic Run Results**

Usually all possible hydraulic parameters can be shown from the computer run results. Of these parameters, the designer must examine two important results very closely: (a) The low system pressure points that are below the 7 m pressure and the affected nodes, and (b) the pipes that have high head loss per km in excess of the 10 m/1,000 m pipeline criteria.

The designer must also examine the balancing flows of the reservoir and analyse if the reservoir discharge or inflow are reasonable for its storage size.

**Step 8: Adjusting Assumed Parameters of the Elements:**

Based on the results of the computer simulation, the designer will improve the network model by adjusting the pipe and node data for specific elements, particularly for those that did not meet the design criteria. For example, for pipes that have high resulting head losses, the designer will have to increase the pipe size to the next larger diameter. If there is a system pressure that is below 7 m, the designer could replace some of the pipes leading to the affected node with a larger diameter. The height of the reservoir could be adjusted if needed to achieve a good system pressure.

The adjusted model is run again in the software. After the run, the results are examined and the model readjusted. The above cycle is repeated until an acceptable hydraulic model is achieved.

**Step 9: Finalizing the Network Configuration**

The model is subjected to repeated simulation and data adjustments until an acceptable network configuration is reached.

## 5.7.3 Pipeline Design Criteria

The pipeline should be designed to withstand the following:

- Internal test pressure of water,
- Water hammer (positive surge),
- Vacuum and negative surge,
- External pressures when laid below ground (overburden and surcharge),
- Conveyance water temperature (thermoplastic pipes),
- Maximum working temperature (ferrous pipe coatings),
- Temperature stresses when laid above ground,
- Flexural stresses when laid over supports, constructed at intervals or on bridges,
- Longitudinal stresses due to flow at tees, tapers and bends,
- Foundation reaction depending upon the nature of support,
- Handling stresses,

For flexible pipes (thermoplastic and steel) the following criteria should be met;

- The pipe deflection (out-or-roundness) must not exceed the allowable limit;
- The combined stress or stain in the pipe wall must not exceed the allowable limit, and
- The factor of safety against buckling must be adequate;

For semi-rigid pipes (ductile iron) the following criteria should be met:

- The pipe deflection (out-or-roundness) should not exceed the allowable limit;
- The pipe wall bending stress should not exceed the allowable limit.

The distribution pipelines should be designed to handle the peak hour demand of the system under the following criteria;

**Table 5.2 Design Criteria for Water Supply Projects**

| No | Criteria | Unit | Value |
|---|---|---|---|
| 1 | Minimum pressure at the remotest end of the system | m | 5 |
| 2 | Maximum velocity of flow in pipes-Transmission Lines | m/s | 3 |
| 3 | Maximum velocity of flow in pipes in Distribution Lines | m/s | 1.5 |
| 4 | Minimum velocity of flow in pipes | m/s | 0.4 |
| 5 | Minimum value of Demand Factor | - | 0.3 |
| 6 | Maximum value of Demand Factor –Peak Demand | - | 3 |
| 7 | Minimum allowable head loss in pipes | m/Km | 0.005 |
| 8 | Maximum allowable head loss in pipes | m/Km | 0.01 |
| 9 | Minimum allowable pressure | m | 5 |

| No | Criteria | Unit | Value |
|---|---|---|---|
| 10 | Maximum allowable pressure | m | 60 |
| 11 | Minimum Demand | factor | 0.3 ADD* |
| 12 | Maximum Day Demand | factor | 1.3 ADD |
| 13 | Peak Hour Demand for served population <1000 | $m^3$ | 3 ADD |
| 14 | Peak Hour Demand for served population >1000 | $m^3$ | 2.5 ADD |
| 15 | Allowable Non-Revenue Water for a new system (NRW) | % | 15 |
| 16 | Household per water point | Households | 50 |

NB: ADD*-Average Day Demand

$$Average\ Day\ Demand = \frac{Design\ Population\ {*}per\ capita\ conumption}{1-NRW} \quad \ldots\ldots\ldots\ldots\ldots (5.3)$$

# 5.8 PIPELINE MATERIALS SELECTION

## 5.8.1 Considerations in Selecting Pipeline Materials

### 5.8.1.1 Flow Characteristics

The friction head loss is dependent on the flow characteristics of pipes. Friction loss is a power loss and thus may affect the operating costs of the system if a pump is used.

### 5.8.1.2 Pipe Strength

Select the pipe with a working pressure and bursting pressure rating adequate to meet the operating conditions of the system. Standard water pipes are satisfactory usually only in low pressure water supply systems.

### 5.8.1.3 Durability

Select the type of pipe with good life expectancy given the operating conditions and the soil conditions of the system. It should have an expected life of 30 years or more.

### 5.8.1.4 Type of Soil

Select the type of pipe that is suited to the type of soil in the area under consideration. For instance, acidic soil can easily corrode G.I. pipes and very rocky soil can damage plastic pipes unless they are properly bedded in sand or other type of material.

### 5.8.1.5 Availability

Select locally manufactured and/or fabricated pipes whenever available.

### 5.8.1.6 Cost of Pipes

Aside from the initial cost of pipes, the cost of installation should be considered. This is affected by the type of joint (such as screwed, solvent weld, slip joint, fusion welding, etc.), weight of pipe (for ease of handling), depth of bury required, and width of trench and depth of cover required.

## 5.8.2 Types of Pipe Materials Available

### 5.8.2.1 Galvanized Iron (GI) Pipes

GI pipes are available in sizes of 13, 19, 25, 31, 38, 50, 63 and 75 mm and in lengths of 6 m. They are joined by means of threaded couplings.

Advantages

- Strong against internal and external pressure.
- Can be laid below or above ground.
- People in rural areas know how to install these kinds of pipes.

Disadvantages

- GI Pipes can easily be corroded, thus the service life is short.
- These have rougher internal surface compared to plastic pipes, hence, have higher friction head losses.

### 5.8.2.2 Plastic Pipes

Polyvinyl Chloride (PVC) and Polyethylene (PE) are commercial plastic pipes. They are available in different pressure ratings and sizes of 13, 19, 25, 31, 38, 50, 63, 75, 100 up to 200 mm. PVC is supplied in lengths of 3 m and 6 m while PE is available in rolls and, for diameters greater than 100 mm, in straight lengths. Suppliers have to be consulted with respect to the pressure ratings to be used. PE pipes are joined by butt-welding. PVC pipes can be joined either through solvent cement welding or through the use of special sockets with rubber rings.

Advantages

- Smooth internal surface
- Resistant to corrosion
- Extremely light and easy to handle
- Do not form encrustation

Disadvantages

- Lose strength at high temperatures (500° C+)
- Not suitable for laying above the ground
- Can deform during storage
- Require good and carefully prepared bedding materials
- Rubber rings can be eaten by some termites if appropriate pipes lubricant is not used in jointing. Thus, the use of edible oil should be avoided
- When joints of fusion welding are opted for, local expertise is scarce

**Table 5.3 Characteristics of Different Pipe Materials**

| Parameters | GI | PVC | PE |
|---|---|---|---|
| Crushing strength versus superimposed loads in trench | Excellent | Fair | Poor |
| Burst strength versus internal pressure | Excellent | Good | Good |
| Durability | Fair | Excellent | Excellent |
| Resistance to corrosion | Poor | Excellent | Excellent |
| Flow capacity | Fair | Excellent | Excellent |
| Resistance to external mechanical injury | Excellent | Fair | Fair |
| Ease of installation | Easy | Must be handled gently. Must be buried | |
| Pipe Cost | High | Low | Low |
| Cost per fitting | Low | High | High |
| Number of fittings | High | High | High |

(Source: Philippines Rural Water Supply Design Manual, 2012)

## 5.9 APPURTENANCES FOR TRANSMISSION AND DISTRIBUTION MAINS

Pipe fittings are those specially manufactured fittings used to facilitate changes in direction, changes in diameter, the making of branches etc. to the pipeline. Further, fittings are needed to install valves, meters and other mechanical devices and to allow for the change from one pipe material to another or diameter changes.

### 5.9.1 Valves

One of the most important types of appurtenances is the valve. A valve is a device that can be opened and closed to different extents (called throttling) to vary its resistance to flow, thereby controlling the movement of water through a pipeline. Valves can be classified into five general categories as follows:

#### 5.9.1.1 Isolation Valves

Perhaps the most common valve in the water distribution system is the isolation valve, which can be manually closed to block the flow of water. Isolation valves include gate valves (the most popular type), butterfly valves, globe valves, and plug valves.

#### 5.9.1.2 Check Valves

Check valves, also called directional valves, are used to ensure that water can flow only in one direction through a pipeline.

### 5.9.1.3 Float Valves

Many water utilities employ devices called float valves at the point where a pipeline enters a tank. When tank level rises to a specified upper limit, the valve closes to prevent any further flow from entering, thus eliminating overflow.

### 5.9.1.4 Air Release Valves

These valves are provided at high points, where trapped air settles, and at changes in grade, where pressures are most likely to drop below ambient or atmospheric conditions.

### 5.9.1.5 Pressure Reducing Valves

Pressure reducing valves (PRVs) throttle automatically to prevent the downstream hydraulic grade from exceeding a set value, and are used in situations where high downstream pressures could cause damage.

### 5.9.1.6 Washout Valves

To be provided at lowest points for the purpose of flushing the pipeline.

## 5.9.2 Fittings

Fittings are installed in the pipelines for the following purposes;

- To connect the same type and size of pipe - *Union*: Unions are provided in the pipeline for ease of repair. Unions are usually installed at 60-metres intervals on straight pipelines. *Coupling*: Used in jointing 2 pipes of the same diameter. It is cheaper than unions.
- To connect two pipes of different sizes- *Reducers* are used when there is a reduction of pipe size and include bushes and elbows for galvanized iron pipes. Also available are reducing elbows, tees and crosses.
- To change the direction of flow - *Elbow*; to divide the flow into two - *Tee*; to divide the flow into three - *Cross*.
- To stop the flow - caps, plugs and blind flanges.

## REFERENCES

MoW 3rd Edition Design Manual, 2009

Philipines Rural Water Supply Design Manual, 2012

Chapter 6

# PUMPING SYSTEMS

## 6.1 INTRODUCTION

This section describes the water supply pumping systems. A brief presentation of types of pumps is provided. It describes how to design and select pumps for a water supply system. Lastly, it gives key considerations of their installation. It is important to understand the different types of pumps, design procedures, sources of pumping power, motor starting, machines protection and economics of electric power systems. More details on pump types and their functioning are given in Appendix E.

## 6.2 RATIONALE

The main goal of any water pumping plant and pumping system is to lift water from a lower to a higher level.

## 6.3 COMMON TYPES OF PUMPS USED IN WATER SUPPLY

There are two main types of pumps used in water supply projects. The pumps are different in design and application. Table 6.1 shows the most commonly used pump types. Further details of each type of pump can be seen in Appendix E of this DCOM manual.

**Table 6.1: Most Commonly Used Pump Types**

| Main Types | Sub-types | Specific types |
|---|---|---|
| Rotordynamic | Centrifugal | Single-stage<br>Multi-stage shaft driven<br>Multi-stage submersible |
| | Peripheral | Axial flow<br>Mixed flow<br>Turbine<br>Submersible |
| Positive displacement | Reciprocating | Suction (shallow well)<br>Lift (deep well) |
| | Rotary | Helical Rotor |

(Source: Modified from Uganda water design manual 2013)

## 6.4 PUMPING SYSTEM SETUP

When setting up a pumping system, carefully calculate the driver Horsepower (HP) required based on the flow data, pressure and efficiency of the pump. Check the pump RPM and drive RPM and select the proper size pulleys to achieve the desired flow. Review the maximum horsepower per belt to assure that the pump receives adequate power to deliver the desired flow. The correct belt length and centre distance must be established to achieve the proper HP. If in doubt, consult your pump and/or drive supplier for their recommendations.

## 6.5 SOURCE OF PUMPING POWER

The different types of power sources commonly used for water supply pumps include:

- Electrical grid power
- Diesel/gasoline generators and engines
- Natural gas/biogas generators
- Solar Energy
- Wind Energy

The choice of pumping energy depends on several factors namely:

- Availability of and proximity to grid power,
- Capital costs of the alternatives,
- Operational costs of the alternatives.

In Tanzania when deciding on water pumping energy, grid power is considered as the basic source in the sense that when available it becomes the 1st choice. It is only when the grid source is too far from the pumping point that other sources of power are considered. The three alternatives to grid power namely diesel/petrol/ natural gas/biogas generators or engines have both positives and negatives. In the following subchapters each alternative shall be discussed.

## 6.6 PUMP SELECTION

Pump selection involves choosing the type of pump that fits the application and sizing the pump to be able to deliver the required pressure and flow to the point of delivery.  The factors which should be considered in the selection and sizing of a pump include:

- Depth to the water level and seasonal variations of the water source;
- Pressure ranges needed for adequate water supply;
- Heights through which water has to be lifted, both below and above the pump;
- Pump location; and
- Pump durability and efficiency.

The type of pump selected for a particular project should be determined on the basis of the following fundamental considerations:

- Yield of the well or water source;
- Daily needs and instantaneous demand of the users;
- The “usable water” in the pressure or storage tank;
- Size and alignment of the well casing;
- Total operating head pressure of the pump at normal delivery rates, including lift and all friction losses;
- Difference in elevation between ground level and water level in the well during pumping;
- Availability of power;
- Ease of maintenance and availability of replacement parts;
- First cost and economy of operation;
- Reliability of pumping equipment; and
- Pump start-up problem and time.

Tables 6.2(a) and (b) shows the operation ranges of different types of pumps according to the head required and the flow rate needed. For water supply pumps the two must be considered for a good pump. Examples of pump duty calculations are provided in Appendix E.

**Table 6.2(a): Selection of Types of Pumps for Water Supply by the Head Needed[6]**

| Under 10 m | From 10 to 100 m | From 100 to 1,000 m | From 1,000 to 10,000 m | From 10,000 m and over |
|---|---|---|---|---|
| One-stage centrifugal pumps | | | | |
| | | Multistage centrifugal pumps | | |
| Axial flow pumps (head is up to 20-30 m) | | | | |
| | Piston pumps | | | |
| | Screw pumps | | | |
| | | | Plunger pumps | |
| Vortex pumps | | | | |

**Table 6.2(b): Selection of Types of Pumps for Water Supply by Flow Rate Needed[7]**

| Under 10 $m^3/h$ | From 10 to 100 $m^3/h$ | From 100 to 1,000 $m^3/h$ | From 1,000 to 10,000 $m^3/h$ | From 10,000 $m^3/h$ and over |
|---|---|---|---|---|
| One-stage centrifugal pumps | | | | |
| | | Multistage centrifugal pumps | | |
| | | Axial flow pumps | | |
| Piston pumps | | | | |
| Screw pumps | | | | |
| Plunger pumps | | | | |
| | Vortex pumps | | | |

6 https://ence-pumps

7 https://ence-pumps

## 6.7 PUMP PROTECTION

In order to ensure acceptable technical soundness of plants and water pumping systems the following protective measures need to be considered at the design stage and adopted wherever desirable. For the purpose of avoiding unnecessary sophistication and yet establish reliable protection of plants and pumping systems the protective systems enumerated hereunder shall be adopted in the fashion recommended herein:

1. Protection against dry running
2. Protection against water hammer
3. Protection against cavitation's
4. Protection against overloads/over current
5. Protection against cathodic corrosion.

The most important protection is water hammer which can be analysed through number of software. Of these, the one recommended for consideration is Surge 2000 produced as part of the KY pipe software package, and its details can be found by logging on to their website. A 250 pipe solution costs US$ 3,000 in 2006.

Another alternative, especially if the designer is interested in is to pursue a controlled air transient technology (CATT) approach for the use of air-valves. This is available by contacting Vent-O-Mat of South Africa by logging on to their website.

## REFERENCES

MoW 3rd Edition Design Manual, 2009

Uganda Water Design Manual, 2013

Chapter 7

# WATER TREATMENT

## 7.1 INTRODUCTION

In this chapter, different categories of water treatment units that are utilized to achieve different water treatment levels are described. These are followed by a description of the recommended approach in the design of treatment plant components. Emphasis should be given to potential water sources that have undergone investigations on the variability of both quality and quantity for at least two years. The data gathered should be used for the selection of appropriate treatment flow sheets and relevant designs.

### 7.1.1 Classification of Water Sources in Tanzania by Quality According to the Complexity of Desired Treatment

Water treatment refers to any process that improves the quality of water to make it more acceptable for human consumption. The production of drinking water involves the removal of contaminants from raw water to produce water that is pure enough for human consumption without any short-term or long-term risk of any adverse health effects.

The processes involved in removing contaminants from water includes physical processes such as settling and filtration, chemical processes such as disinfection and coagulation and biological processes such as slow sand filtration. Table7.1 present the recommended treatment process flow for the most common water sources in Tanzania. The above water contaminants removal processes are best presented in water treatment flow sheets (Figures 7.1–7.8).

**Table 7.1: Recommended Water Treatment Flow Sheets for the Most Common Water Sources in Tanzania**

| S/No. | Nature of the Water Source | Recommended treatment process flow (Minimum) | Remarks |
|---|---|---|---|
| 1. | Rainwater harvesting | Periodic disinfection | Assuming design allows flushing of first rains |
| 2. | Deep well with no minerals | Residual chlorination | Assuming water is supplied through a distribution line and storage tank. |
| 3. | Shallow well with no pollution | Periodic chlorination | Well used by many families |
| 4. | Highland stream with no soil erosion | Screens, grit chamber or sand trap, sedimentation with lamella plates, Slow Sand Filtration (SSF), chlorination | Water supplied via distribution pipes and storage tanks. |
| 5. | Deep well with Iron and Manganese | Aeration, sedimentation, SSF or RSF, chlorination | Assume water is supplied via distribution pipe and storage tanks |
| 6. | Deep well with Fluoride | Sedimentation tank, Bone char, chlorination or Sedimentation, UF/MF, NF, disinfection. | Energy for pumps needed & chlorine not used to avoid corrosion of Polyamide fibres. |
| 7. | Surface water with low pH | Screens, Lime, sedimentation with lamella plates, RSF, disinfection | |
| 8. | Surface water with Nitrate & Sulphate | Screens, Coagulation, Flocculation, Sedimentation with lamella plates, RSF, disinfection. Or Screens, UF/MF, NF, disinfection. | |
| 9. | Surface water with Cadmium, Selenium, Arsenic, Mercury, Lead, Copper, Uranium, Chromium, Cyanide, Nickel | Screens, Coagulation, Flocculation, Clarifier, RSF, UF/MF, NF. | Areas near mining sites with metal recovery possibility. |
| 10 | Surface water with pesticides | Screens, Lamella settlers, RSF, UF/MF, NF | Areas with intense large scale agriculture |

Note: UF-Ultra filtration, MF-Micro filtration, NF-Nano filtration, RSF-Rapid sand filtration and SSF-Slow sand filtration.

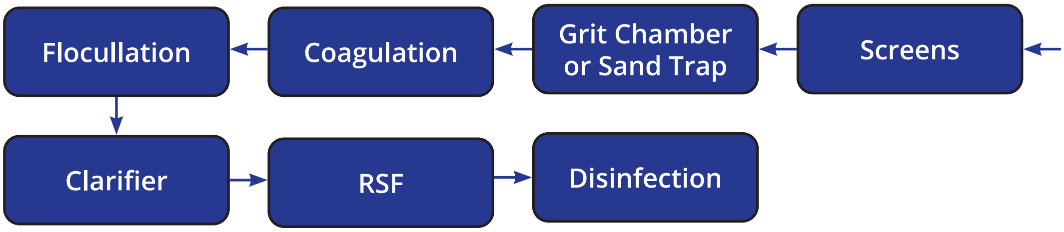



**Figure 7.1: Flow Sheets for Treatment of Lake water, Dam, Reservoir (Multiple Water Intakes)**


**Figure 7.2: Flow Sheets for Treatment of Surface Water with Pesticides**

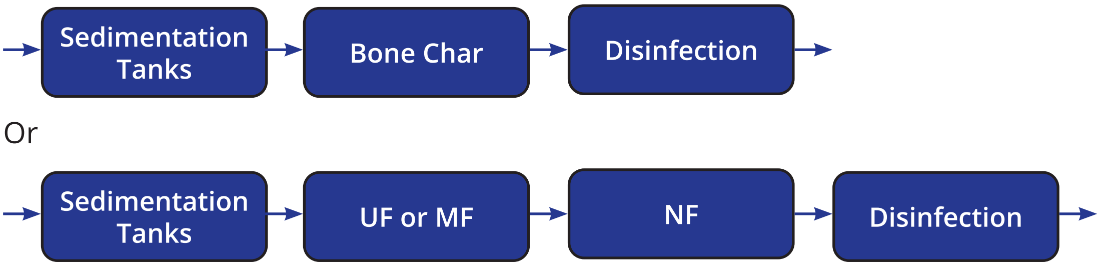



**Figure 7.3: Flow Sheets for Treatment of Deep Well Water with Fluoride**

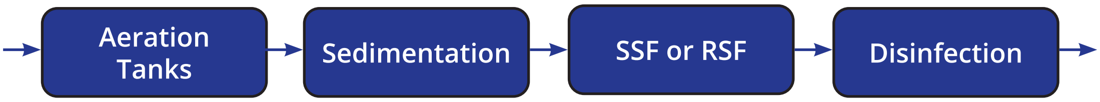



**Figure 7.4: Flow Sheets for Treatment of Deep Well Water with Iron and Manganese**


**Figure 7.5: Flow Sheers for Treatment of Upland Stream with no Soil Erosion**

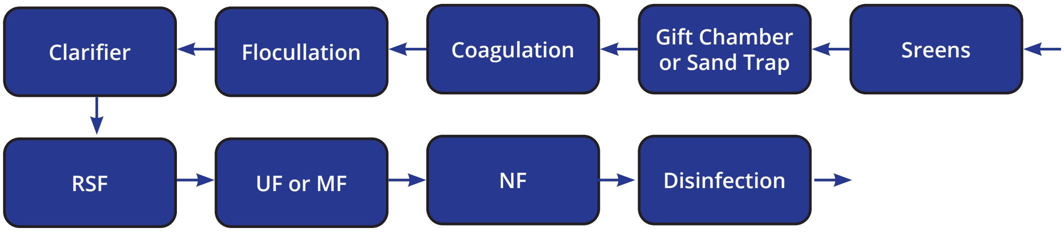



**Figure 7.6: Flow Sheets for Treatment of Imported Water with High Natural Organic Matters**

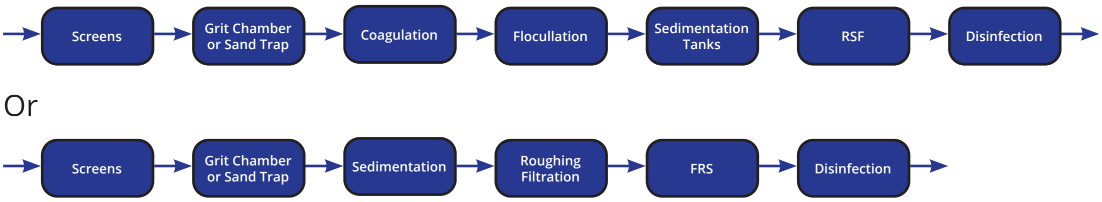



**Figure 7.7: Flow Sheets for Treatment of ery Turbid Surface Water or Silted Lowland Stream**


**Figure 7.8: Flow Sheets for Treatment of Water with Cadmium, Selenium, Arsenic, Mercury, Lead, Copper, Uranium, Chromium, Cyanide and Nickel**

### 7.1.2 Classification of Unit Operations to Achieve Water Treatment Levels

The categories for water treatment levels are; pre-treatment, primary treatment, secondary treatment and tertiary treatment.

(a) *Pre-treatment* includes units for removal of Scum and floating matters, Screening (fine and coarse), Sand trap, Grit removal, Pre-chlorination, Water conditioning (pH correction).

(b) Primary treatment comprises Sedimentation, Primary filtration, Floatation, Aeration.

(c) Secondary treatment includes Coagulation, Flocculation, Clarification, Filtration, Softening, Reverse Osmosis, Capacitive De-Ionisation (CDI), Ion Exchanger, Adsorption, Constructed wetlands.

(d) (iv)Tertiary treatment includes Disinfection, Softening, Water conditioning, Water polishing.

## 7.2 RECOMMENDED OVERALL DESIGN APPROACH FOR COMPONENTS OF WATER TREATMENT PLANTS

The design of the main conveyance units for the treatment plants including pipes and channels which should be designed for a period of 20 years (design life). On other hand, individual unit operations should be designed for a design period of 10 years in an approach that allows for phased implementation. Intentionally, to allow for adoption of the latest technologies and to avoid tying substantial capital in the treatment plants. It is recommended that a potential water source be closely and intensely monitored for a period ranging from 2 to 3 years. This period can include the time when the feasibility study for the water supply project is being undertaken including the environmental impact assessments. Water source quality and quantity data should be used for the determination of the most suitable treatment flow sheet.

## 7.3 DOCUMENTS AND WEBSITES CONSULTED AND WHICH ARE HYPER-LINKED TO THE DCOM MANUAL

The following documents were consulted for purposes of making reference for the design of water treatment plants in Tanzania:

- URT, 2009. Design Manual for Water Supply and Wastewater Disposal.
- Ministry of Drinking Water and Sanitation, May 2013. Operation and maintenance manual for rural water supplies. India.
- The Republic of Uganda, Ministry of Water and Environment, 2013. Water Supply Design Manual, 2nd edition.
- World Bank Phillipines, February 2012. Water Partnership Program. Rural Water Supply Vol. I Design Manual.
- Washington State Dept. of Health USA, October 2019. Water System Design Manual.
- URT, July 1997. Design Manual for Water Supply and Wastewater Disposal.

Throughout this manual, on a number of occasions, the designers are referred to the websites of the Ministry of Water (https://www.maji.go.tz) or RUWASA (https://www.ruwasa.go.tz) vide the various hyperlinks inserted in order to access the standard drawings for various appurtenances.

## 7.4 WATER TREATMENT DESIGN CONSIDERATIONS

The manual has made reference to a number of design guidelines that are relevant for the design of various unit operations and wherever necessary the full guidelines have been hyper-linked to the manual. For each unit operation, a few critical design criteria have been provided.

Before proposing or designing any water treatment plant for any planned water supply project, water quality of the anticipated water sources to be treated has to be known by the designer. Knowing the historical and current water quality trends of the water sources will help in designing a treatment plant that can address the localized water quality challenges of concern in the given area in addition to the general water quality parameters.

The sizing and selection of water treatment technology and different units to be installed should always aim at meeting the established national and international water quality standards and associated health criteria which are often updated from time to time. There are different criteria and standards across the world. However, in Tanzania, the most recent water quality standards of Tanzania Bureau of Standards (TBS) and the World Health Organization (WHO) guidelines are recommended for reference when designing a water supply project in the country.

## 7.5 WATER TREATMENT LEVELS AND UNITS

### 7.5.1 Pre-treatment

#### 7.5.1.2 Scum and Floating Materials Skimmer

This is the unit operation that enables the manual or automated removal of scum and floating matter ahead of the screening units. These are designed to skim the entire width of the approach area ahead of the screens. In view of the variability of flow of water from the sources, skimmers ought to be designed such that they can be adjusted up or down depending on the quantity variation that is established during the feasibility study. The width of the channel or any open conduit delivering the raw water will determine its design. Figure 7.9 shows the design of such a skimmer.

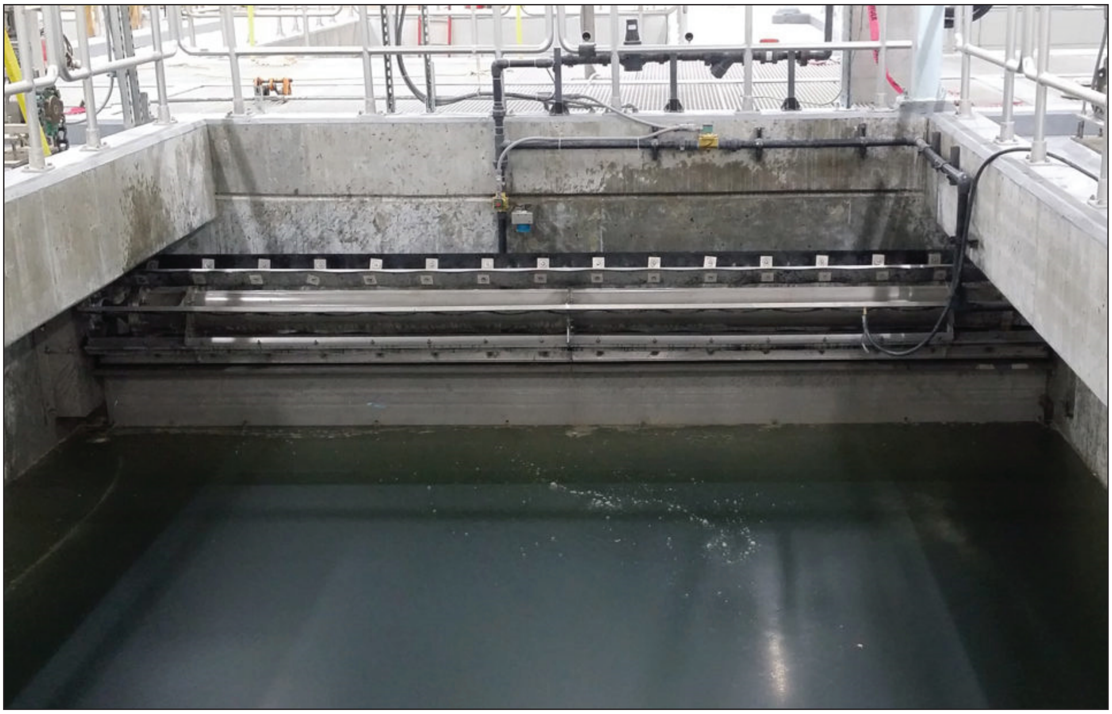


**Figure 7.9: A Typical Skimmer**

(Source: https:www.jmsequipment.com>skimming systems/vis.06/01/2020)

#### 7.5.1.2 Screening or Straining

This unit operation consists of fine screens and coarse screens which perform the task of removing all fine and coarse matters that may block the screen or damage downstream appurtenances or machines. This is a physical, pre-treatment process used to remove weeds, grass, twigs, bilharzial snails and other freshwater crustaceans as well as coarser particles including plastics, tins and other hard matter so that they do not enter the pumping, treatment, or supply system. Screens are placed at the entrance to the intake of a water supply project.

The design considerations for surface water screens are;

- They should be easily accessible, at least during medium and low flows and inclined downstream of the river or stream as well as during cleaning (if manually done) as indicated in Fig.7.10.
- Distance between bars should be between 10 and 30 cm. for coarse screens and between 0.5 and 5 cm. for fine screens. The shape of the screen bars is either round or rectangular.
- Approach velocity entering the screen ($V_a$) from upstream should not exceed 0.3 to 0.5 m/sec. to limit sedimentation.
- Velocity through the screens ($V_s$) should not exceed 0.7 to 1.0 m/sec. to prevent soft deformable materials from being forced through the screens.
- The ratio of the width of the screens (Ø) and the space between the bars (b) determines the ratio between the two velocities ($V_a$) and ($V_s$).
- Small screens are made removable for cleaning, medium sized can be hand raked in-situ whilst large screens will need in-situ mechanical or electrically operated rakes.

Figure 7.10 presents the formula for the calculation of the headloss through the screens as well as the screen bars coefficient (β).

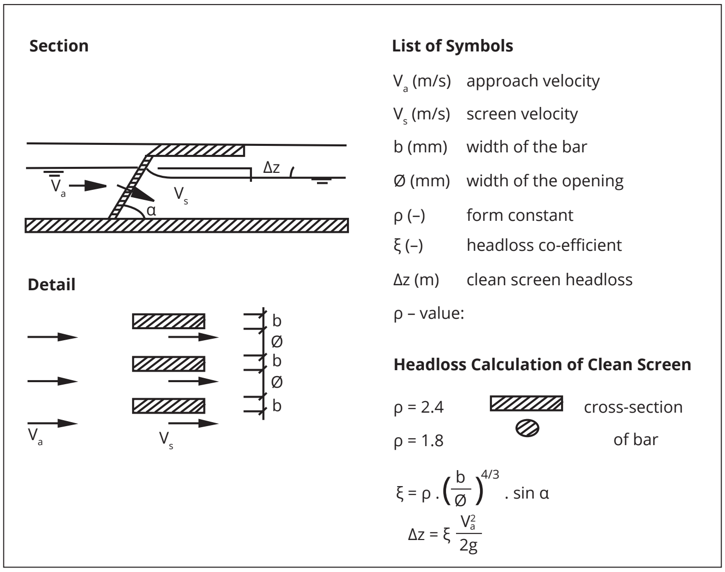



**Figure 7.10: The Headloss Calculation Formula Through Screens**
(Source: Poppel, 1982)

## 7.5.1.3 Grit Removal

Grit consists of the heavy inorganic fraction of solids that include road grit, sand, eggshells, broken glass, coconut shells and metal pieces. The purposes of including grit channels in the design are as follows:

- To protect pumps and other mechanical parts from excessive wear and tear,
- To avoid undue clogging/filling up of subsequent unit operations,
- To differentially remove grit but not the organic particulates in water.

The average specific gravity of grit is 2.5 with an average settling velocity S = 30 mm/sec. In comparison, while sand grit has an average solids density $\beta_s$ = 2,650 kg/m³ organics have a density $\beta_o$ ranging from 1,020 to 1,200 kg/m³.

**Design approach**

To exploit the differential sedimentation rates of the particles by providing channels that ensure removal of grit rather than any other lighter particles and to maintain the horizontal flow velocity $V_h$ has to be maintained at about 0.3m/sec. Provision of a parabolic or near parabolic cross section of the channel guarantees that the constant velocity is maintained at all flows. In practise, due to the difficulty of construction of parabolic sections, trapezoidal sections are used.

### *7.5.1.3.1 Design Criteria*

Length of the channel L = 20 (maximum depth of flow)

$L/d = V_h/V_s$ ............................................................................................... (7.1)

Where,

$V_h$ = Horizontal flow velocity

$V_s$ = Vertical settling velocity

L = Length of the grit channel

D = Depth of flow in the channel

There are three types of grit channels that can be designed, these include horizontal flow, Rotational flow and Vertical flow.

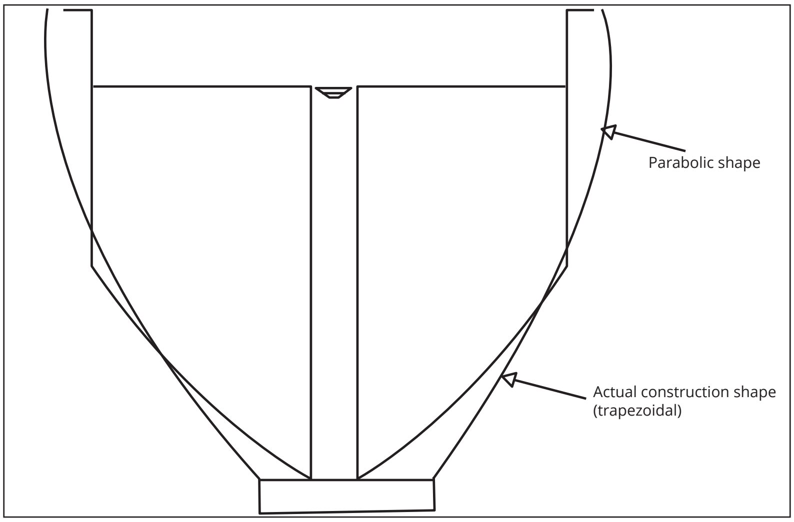



**Figure 7.11: Typical Trapezoidal Section of a Grit Channel**

(Source: Poppel, 1982)

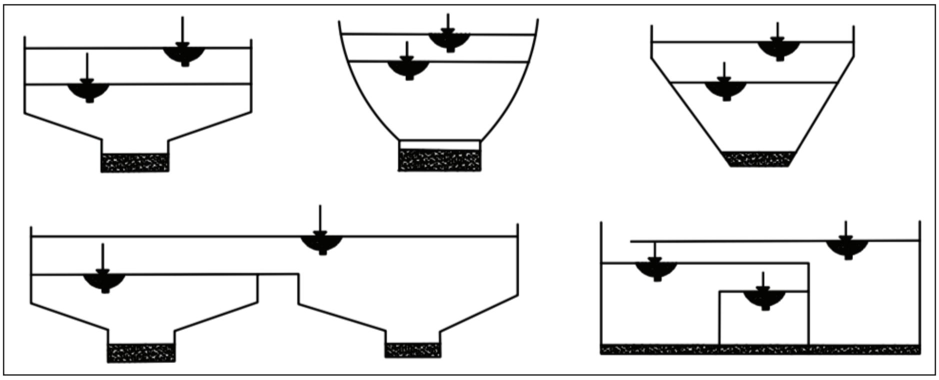


**Figure 7.12: Single and Common Section Type of Grit Channels**
(Source: Poppel, 1982)

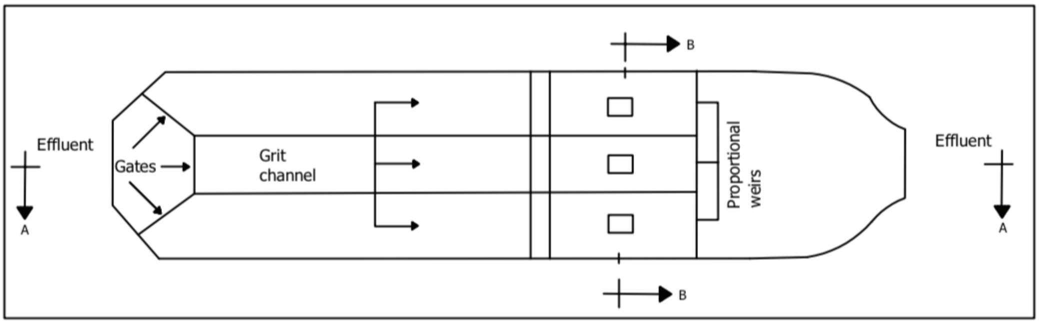



**Figure 7.13: The Schematic Layout Plan of a Grit Chamber with Proportional Sutro Weir**

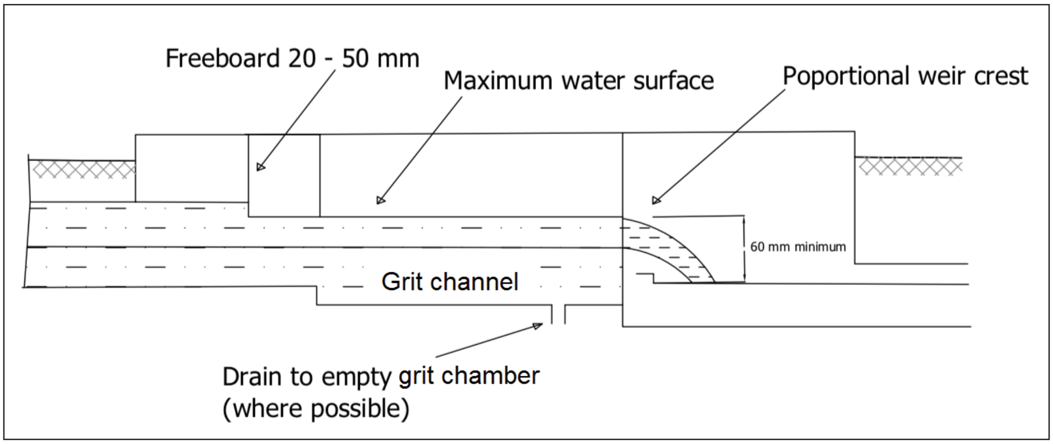



**Figure 7.14: Cross Section Through the Grit Channel**
(Source: Poppel, 1982)

### 7.5.1.4 Sand Traps

This pre-treatment unit is designed to trap sand after water has been guided into the intake chamber in order to reduce wear and tear as well as silting up the unit operations that are located downstream of the intake structure. The minimum diameter of washout pipes of such sand traps is 75 mm and the bigger the main intake pipe, the bigger is the flushing pipe for sand. Figures 7.15, 7.16 and 7.17 show the layout plan and two sections of such a sand trap for small streams intake.

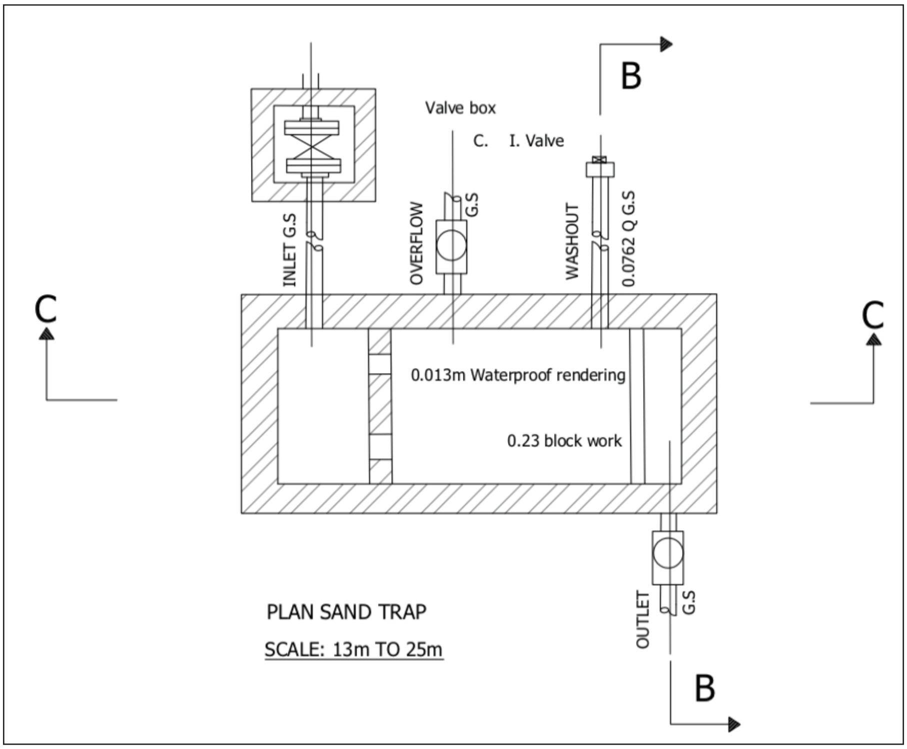



**Figure 7.15: Typical Layout Plan of a Sand Trap for a Small Scheme**

(Source: adapted from Philippines Rural Water Supply Design Manual, Volume I, 2012)

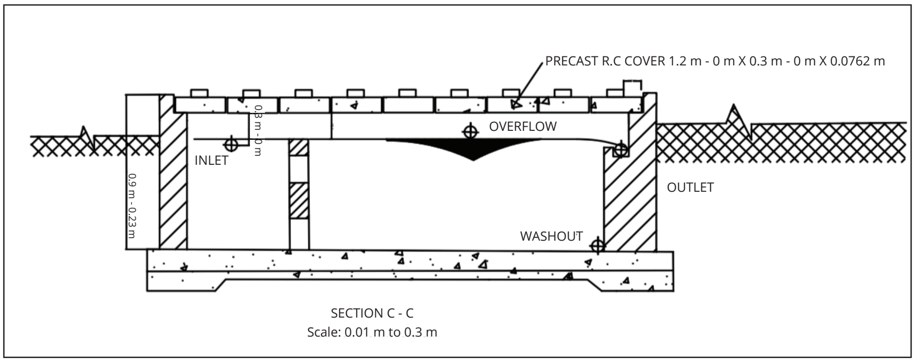



**Figure 7.16: Longitudinal Cross Section**

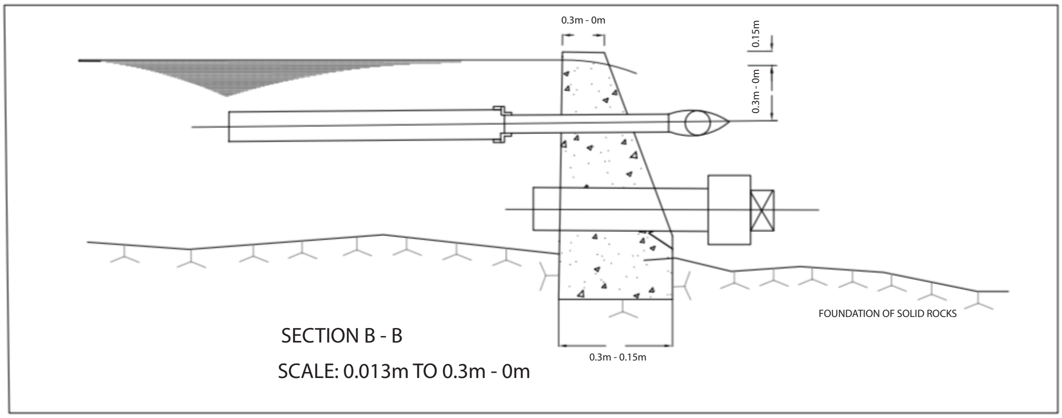



**Figure 7.17: Transverse Cross Section**

### 7.5.1.5 Pre-chlorination

This is a unit operation that is used for purposes of controlling algae growth in raw water and the process of preparation of the chemical to facilitate dosing will include the standard preparation of the aqueous solution as done during disinfection. This is often added upon the establishment of occurrence of algal blooms during certain periods of the year as confirmed by laboratory tests undertaken daily. The amount of the dose will be established daily in order to pre-determine the pre-chlorination dose that has to ensure no interference with downstream unit operations in case the flow sheet involves biological treatment processes like slow sand filter or others. The location of dosing of the chemical must be secured against direct sunlight and must ensure intense mixing. Where chlorine gas is used for pre-chlorination, the usual safety measures that are taken when dosing chlorine in gaseous form must be taken including leakage detection. To minimise chlorine consumption, pre-chlorination has to be done downstream of the fine screens.

Other design features can be seen in the Appendix H of this DCOM Manual. It should be noted that pre-chlorination also controls the growth of bacteria in pipes and tanks (BRITANICA, 2020). Lanfair et al (2020) cautions that if pre-chlorination is applied to water with a high concentration of natural organic matter (nom), the latter is suspected to react with chorine to form Disinfection by-products (DBPs) that include trihalomethanes or haloacetic acids that are suspected to contribute towards stomach and bladder cancer.

### 7.5.1.6 Water Pre-conditioning (pH Adjustment)

Water pre-conditioning can entail a number of pre-treatments undertaken prior to pre-chlorination which is often the final step in pre-treatment. This unit operation involves adjustment of the pH upstream in order to ensure the chemicals used during further treatment processes are dosed to water that has the correct pH range for maximum efficiency. A good example is when the treatment flowsheet includes coagulation with Ferrous Sulphate that requires an optimum pH range of 7 to 8.5. As a matter of fact, the application of this unit operation guarantees that the daily fluctuations in quality are also reflected in the chemical that is used for pH correction whether an acid or an alkali is used. It is usual to apply lime for increasing pH and use acids like dilute sulphuric acid or hydrochloric acid to lower pH. Other pre-conditioning measures include the use of sodium carbonate (soda ash) to remove water hardness (i.e. calcium carbonate).

## 7.5.2 Primary Treatment

Primary treatment of water removes material that will either float or readily settle out by gravity. This water treatment level includes the physical processes of screening, grit removal and sedimentation.

### 7.5.2.1 Sedimentation

In designing sedimentation tanks, the required detention time determines the dimensions of the tank. A rectangular tank is the simplest design to use. Detention time is calculated as Volume/Flow rate (Q). The detention time based on the average daily flows usually ranges from about 45 minutes to 3 hours depending on water turbidity. The ideal inlet reduces the entry velocity and distributes the water as uniformly as possible across the depth and width of the tank. Outlets are usually weirs which are sufficiently long to reduce the flow velocity, and so avoid the re-suspension of the solids in the water. Plain sedimentation tanks should be designed for a surface loading in the range of 0.1 - 0.5 $m^3/m^2/h$. The exact surface loading to be adopted should be determined after carrying out settlement tests on samples of raw water, typical of all regimes of the water source. The settling properties of water will depend on the soil and vegetation conditions in the catchment area, and they will vary considerably between different locations and regimes of the water source. Figures 7.18 (a and b) show a cross-section through a circular sedimentation tank while Figures 7.19 (a) and (b) show a cross section through a rectangular sedimentation tank.

### 7.5.2.2 Lamella Plate Settlers (Inclined Plate Settlers)

Settling efficiency of a basin depends upon the design surface loading. Lamella plates and small diameter tubes having a large wetted perimeter, relative to wetted area providing laminar flow conditions and low surface loading rates have shown good results in terms of settling efficiency and economy in space as well as cost. The plate or tube configuration can be horizontal or steeply inclined. In inclined plates or tubes ($55^o$ – $60^o$) continuous gravity drainage of the settleable material onto the floor below can be achieved, without impairment of effluent quality. However, to work effectively an efficient flocculation stage is critical.

In purpose built lamella plate settlers, the water enters at the base of the lamella plates and travels upwards between the lamellas. Each space between the lamella plates tends to act as a semi -independent settling module with the lamella plates extending from near the base of the tank to about 125 mm above the

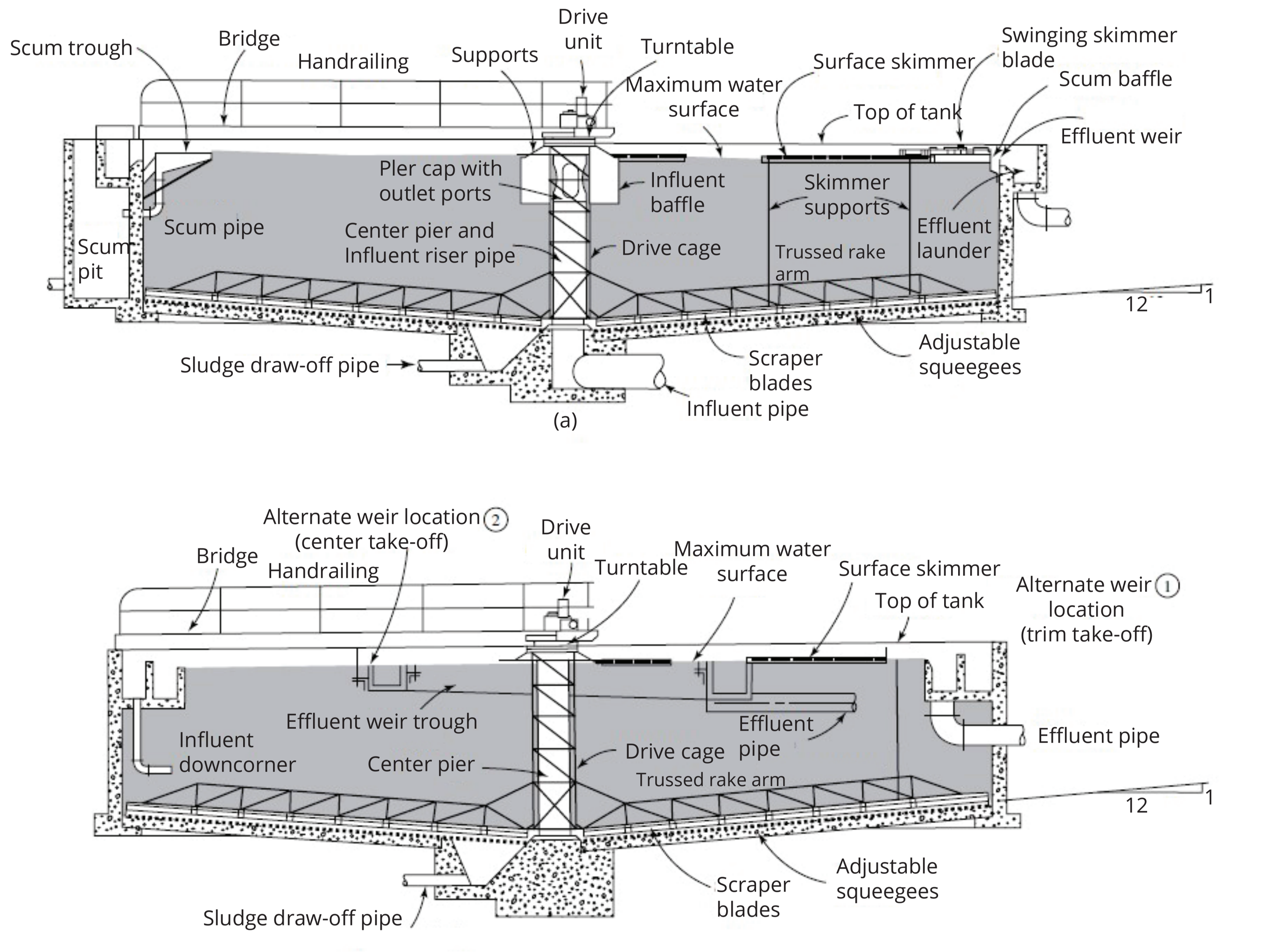



**Figure 7.18: Cylindrical Sedimentation Tank**

(Source: Wastewater Engineering: Treatment and Reuse, by Inc. Metcalf & Eddy, George Tchobanoglous)

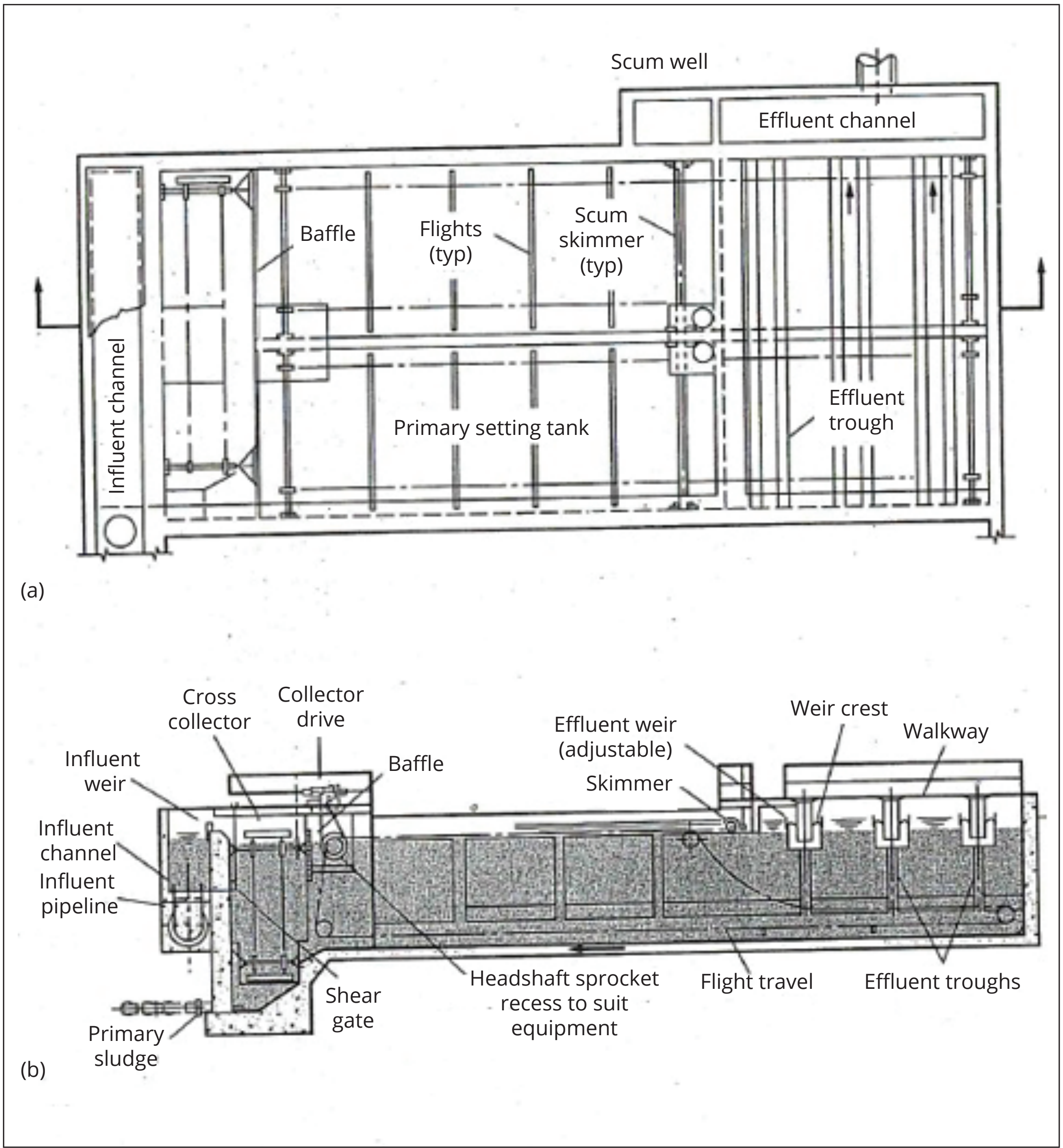



**Figure 7.19: Rectangular Sedimentation Tank**

(Source: Wastewater Engineering: Treatment and Reuse, by Inc. Metcalf & Eddy, George Tchobanoglous, 2002) top water level. The clarified water is collected by saw-toothed notched launders running along each side of the plate. Unless the sludge is removed mechanically by a scraper, sufficient depth beneath the plates is required for access during cleaning, although this can be aided by pressurised water. A typical arrangement is illustrated in Figure 7.20.

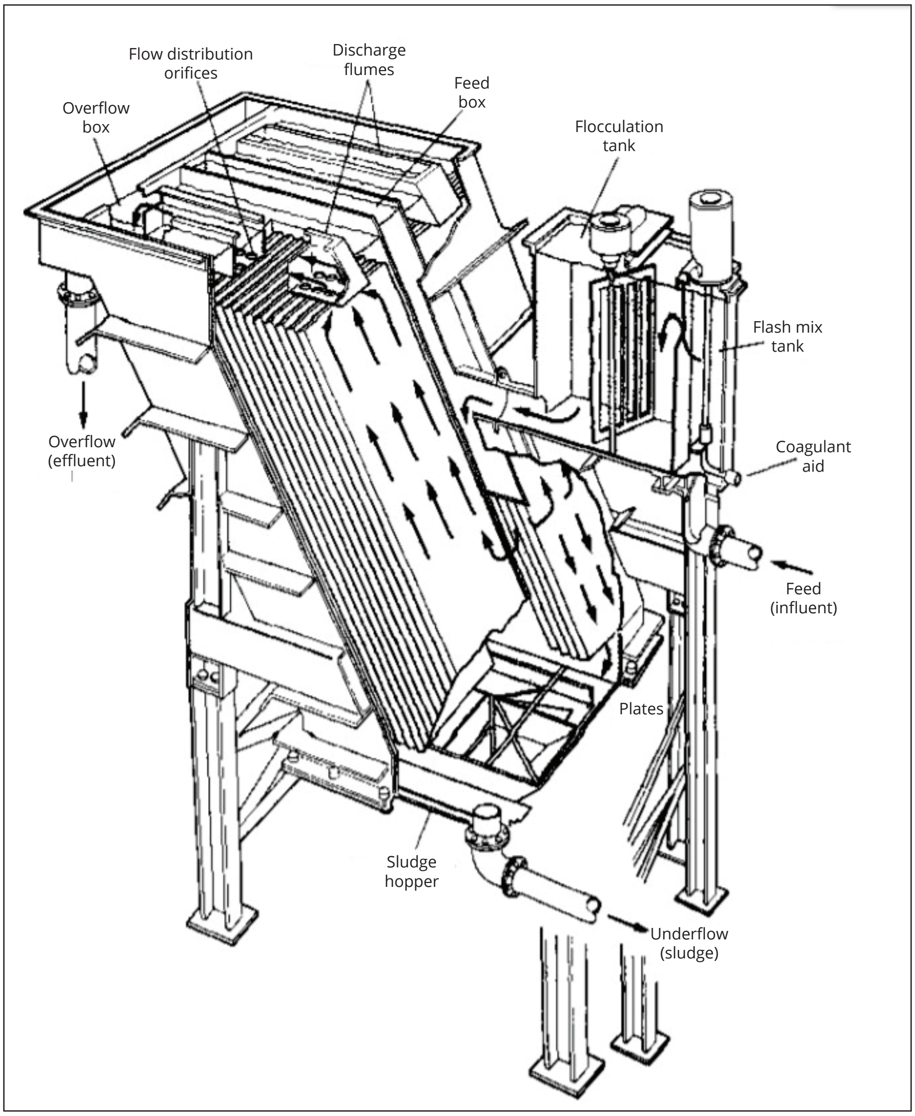



**Figure 7.20: Typical Purpose Built Lamella Settler**

(Source: Wastewater Engineering: Treatment and Reuse, By Inc. Metcalf & Eddy, George Tchobanoglous)

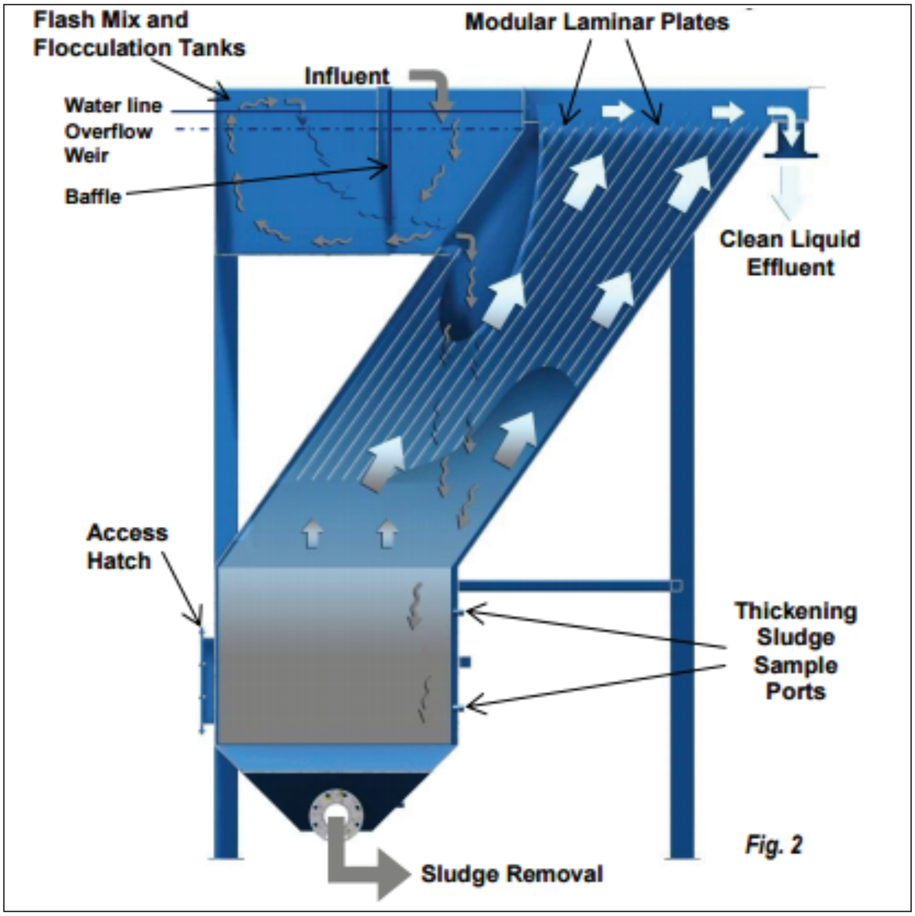



**Figure 7.21: Typical Purpose Built Lamella Settle**

Plates are made of stainless steel or plastics with a width of 1.25 to 1.5 m and a length of 2.5 to 3.25 m including the length above water. *It is advisable to use standard lengths of metal sheets and plastic sheets available in the market. In Tanzania such sheets are in dimensions of 120 cm x 240 cm.* Plate thickness is usually about 0.7 mm for stainless steel whilst the horizontal spacing between plates is varied according to the nature of the raw water but within the range 50 – 80 mm.

Total settled area is then:

$$A = (n-1) \times L \times W \times \cos\theta \qquad (7.2)$$

Where;

n = number of plates

L = plate length in water (m), less the transition length W = the plate width (mm), and

θ = the angle of inclination of the plates to the horizontal (55° – 60°)

A = Settled area ($m^2$)

**Table 7.2: Important Parameters for the Design of Lamella Plate Settler**

| S/No. | Parameter | Units | Range |
|---|---|---|---|
| 1. | Surface[8] loading rate | m/h | 5 to 10 m/h[9] |
| 2. | Hydraulic Loading Ratio | - | 0.25 -0.5 |
| 3. | Angle of inclination of the plates, θ | - | 45-70°<br>Typical: 60° |
| 4. | Plate spacing, w | mm | Min 20 |

**Figure 7.22: Pilot Lamella Settler Installed at Nadosoito, in Monduli District, Tanzania for Dam Water Treatment**

[8] Calculated over horizontally projected area

[9] https://e-ht.com/wp-content/uploads/2015/08/PlateSettlerPresentation140415.pdf

### 7.5.2.3 Primary Filtration

Primary filtration of water removes material whose particle size is greater than the opening size. The target particles are those which are not removed by sedimentation tanks.

#### *7.5.2.3.1 Slow Sand Filtration*

A Slow Sand Filter (SSF) is basically a large tank containing the sand bed. A distinguishing feature of a slow sand filter is the presence of a thin layer, called the *schmutzdecke,* which forms on the surface of the sand bed and includes a large variety of biologically active micro-organisms.

Water is introduced at the top and trickles down through the sand bed to the under-drains and goes to the storage tank. The impurities in the water are retained at the upper layers of the sand bed. In the process, the schmutzdecke consisting of bacteria and microscopic plants which grow. The schmutzdecke removes the organic matter and most of the pathogenic micro-organisms in water which might be smaller than the pores of the sand.

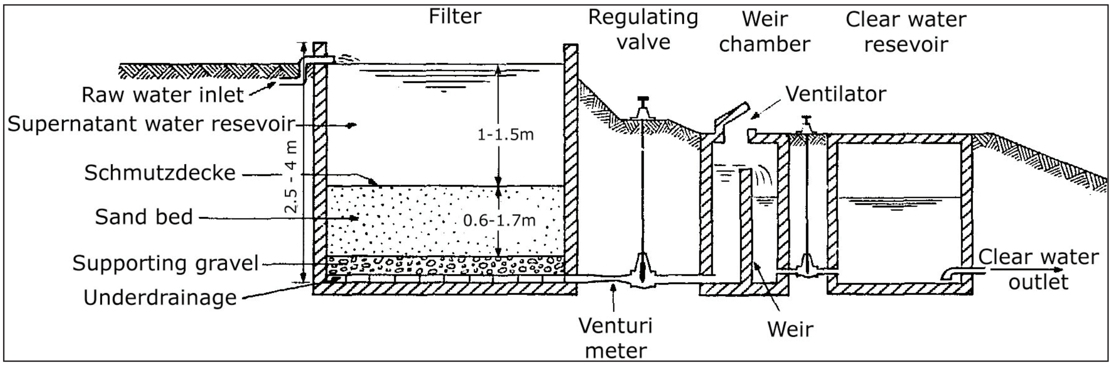



**Figure 7.23: A cross Section of the Slow Sand Filter (SSF)**

(Source: *Huisman (1974)*

### (a) Elements of a Slow Sand filter

Figure 7.23 presents, the various elements that make up a Slow Sand Filter. Essentially the SSF elements consist of:

- (i) a supernatant (raw) water reservoir, the principal function of which is to maintain a constant head of water above the filter medium. This head provides the pressure that carries the water through the filter;
- (ii) a bed of filter medium (nearly always sand), within and upon which the various purification processes take place
- (iii) an under-drainage system, which fills the dual purpose of supporting the filter medium while presenting the minimum possible obstruction to the treated water as if it emerges from the underside of the filter-bed; and

(iv) a system of control valves to regulate the velocity of flow through the bed, to prevent the level in the raw water reservoir from dropping below a predetermined minimum during operation, and to permit water levels to be adjusted and backfilling to take place when the filter is put back into operation after cleaning

(v) It comprises approximately 1.2 m depth of fine sand supported on two or three gravel layers. The effective size of the sand used in slow sand filters is about 0.2 mm, but may range between 0.15 mm and 0.35 mm, and with a coefficient of uniformity of between 1.5 and 3.0.

(vi) It is a very simple and effective technique for purifying surface water. It will remove practically all of the turbidity from the water as well as most of the pathogens without the addition of chemicals. Slow sand filters can frequently be constructed largely from locally-available materials.

In an SSF the water is purified by slow percolation through a bed of fine sand. Pre-treatment is necessary with raw waters having an average turbidity of 25 NTU or more, but should be considered also for less turbid raw waters 5 to 25 NTU) to improve effluent quality and reduce the frequency of cleaning. The SSF is also useful for treating groundwater containing solids in suspension, e.g. ferric and manganese compounds converted by aeration from the soluble state of the salts. Table 7.3shows the typical performance of a Slow Sand Filter.

**Table 7.3: Typical Performance of Slow Sand Filters**

| Parameter of Water Quality | Purification Effect of Slow Sand Filtration |
|---|---|
| Colour | 30% to 100% reduction |
| Turbidity | Turbidity is generally reduced to less than 1 NTU |
| Faecal Coliforms | 95% to 100%, and often 99% to 100%, reduction in the level of faecal coliforms |
| Cercariae | Virtual removal of cercariae of schistosomes, cysts and ova |
| Viruses | Virtually complete removal |
| Organic matter | 60% to 75% reduction in COD |
| Iron and manganese | Largely removed |
| Heavy metals | 30% to 95% reduction |

(Source: 3rd Design Manual for Water Supply and Wastewater Disposal, 2009)

## (b) Design Considerations of Slow Sand Filter (SSF)

- Raw water quality and necessity for pre—treatment and/or aeration;
- Necessity for chlorination room and clear water storage, pumping and distribution;
- Site location, foundation conditions, space for expansion and pre-treatment;
- Availability of source of filter media, and construction materials;
- Fencing and security; and

- The filter sand must be free from any clay or silt content and preferably of a well- rounded quartz material. Organic matter should be avoided.

## (c) Design Criteria

- When choosing the filter sand, the grain size distribution should meet the effective size 0.15 to 0.35mm and the coefficient of uniformity should be less than 3.
- In order to calculate for the total area of filter beds, a working rate of 0.1 - 0.2, $m^3/m^2/hr$ is recommended. When one filter is not operational, the working rate of the remaining filter should not exceed 0.2 $m^3/m^2/h$.
- The turbidity in the incoming water should not exceed an average of 5 - 10 NTU. In cases of higher turbidity, preliminary treatment such as roughing filters is necessary
- The inlet structure should be designed in such a way that the raw water is equally distributed over the filter bed area. To achieve this, the inlet velocity should be around 0.1 m/s and the width of the inlet structure should be at least (0.05 x Q) metres, where Q is the design flow in $m^3/h$.
- The minimum size of a filter unit should be 15 to 20 $m^2$;
- The height of the supernatant water should be 1 to 1.5 m,
- The oxygen content of the water after filtration should not be less than 3mg/l.
- Calculate filter surface area = flow capacity ($m^3$) / rate of filtration.

## (d) Main Water Underdrain

- Calculate the Diameter of the underdrain = $(22 \times d^2) / (28 \times r)$
- Area of holes or slots to be 1.5% of area of the filter
- As a check, the oxygen content of the water after filtration should not be less than 3 mg/l.
- Freeboard above water level = 0.2 - 0.3 m
- Height of walls above ground surface = > 0.8 m
- Gravel filter support = 0.3 - 0.5 m
- Depth of under—drainage system = 0.3 - 0.5 m
- Area (A) per filter bed = 10 - 100 $m^2$

### *7.5.2.3.1 Rapid Gravity Sand Filtration*

This is a process in which water flows onto the top of the filter media and is driven through it by gravity. In passing through the small spaces between the filter sand grains, impurities are removed. The water continues its way through the support gravel, enters the under-drain system, and then flows to the reservoir. It is the filter media which actually removes the particles from the water. The filter media is routinely cleaned by means of a backwashing process.

Rapid Sand Filtration (RSF) is a technique commonly used for treating large quantities of drinking water. It is a relatively sophisticated process usually requiring power operated pumps for backwashing or cleaning the filter bed, and some designs require flow control of the filter outlet. A continuously operating filter will usually require backwashing about every two days or so when raw water is of relatively low turbidity and at least daily during periods of high turbidity. Because of the higher filtration rates, the area requirement for a rapid gravity filtration plant is about 20% of that required for slow sand filters.

Surface loading should be between 4 and 7 $m^3/h.m^2$, and the filter structure should be designed with a minimum height between the top of the filter media and the bottom of the wash water channel of at least 30% of the height of the filter media as this expands during backwashing. It may be necessary to include for air-scour as well as backwashing, or both combined in a single operation.

Normally a sufficient distribution of the wash water will be achieved if the:

- Ratio of area of orifice to area of bed served is (1.5 to 5) × ($10^3$) : 1
- Ratio of area of the main to laterals served is (1.5 to 3) : 1
- Diameter of orifices is 5 - 20 mm
- Spacing of orifices is between 100 - 300 mm centre to centre
- Spacing of laterals approximates to the spacing of orifices.

The filter bed should be approximately 1.0 m thick and preferably consist of well-rounded quartz sand with an effect size of 0.7 - 1.0 mm and uniformity coefficient in the range of 1.3 - 1. 5. The available hydraulic head above the top of the filter bed should be 1.3 - 1.5 m. The following stratification of the sub structure should be used to support a filter of an effective grain size as suggested earlier (finest strata at the top).

- 0.15 m of grain size 2 - 2.8 mm
- 0.10 m of grain size 5.6 - 8 mm
- 0.10 m of grain size 10 - 20 mm
- 0.10 m of grain size 20 - 40 mm
- 0.10 m of grain size 40 - 60 mm

The washing velocity should be in the range of 35 – 55 $m^3/m^2$/hour. However, care must be taken to ensure that sand carry over into the wash-water channel does not occur and the actual wash-water rate is adjusted accordingly.

In order to achieve proper washing of the filter, a storage volume sufficient for continuous washing for an 8 to 10 minute period should be made available.

### (b) Design Steps

The following design steps should be followed;

(i) Filter Units

- Rate of filtration should be 4 - 6 $m^3/m^2$/hr
- Determine flow capacity in the filter ($m^3$/day. demand)
- Calculate filter surface area = flow capacity ($m^3$) / rate of filtration

(ii) Main Water Under-drain

- Select flow capacity and flow velocity
- Calculate area required = (low capacity)/ (low velocity) ($m^2$)
- Calculate diameter of the under drain, d= Square Root of (4A/Π)
- Area of holes = 0.2 - 0.4 % of the area of filter
- Use five layers of sand particles in the filter as indicated above:
- Water depth 1 – 1.2 m
- Sand depth 1 – 1.2 m

## (c) Filter Backwashing

A major cause of poor performance by rapid sand/gravity filters is a result of either inadequate or excessive backwashing rates. Backwashing is sometimes carried out by water alone but more often by air and water usually applied one after the other by reverse flow to the filter bed. The first operation however is to allow the filter to drain down until the water lies a few centimetres above the top of the bed. Air is then introduced through the collector system at a rate of about 6.5 to 7.5 mm/s.

Where air and water are applied separately, air scour normally lasts about 3 – 4 minutes and the water wash about 4 – 6 minutes. Where applied concurrently, air is first introduced and after about 1.5 – 2 minutes when it is fully established water is introduced and the combined backwash lasts for about 6 – 8 minutes. Air is stopped first and the water run for several more minutes to rinse the bed. Generally, total water consumption per wash amounts to about 2.5 bed volumes, but should normally not exceed 2% of the treated water output in well run plants.

### *7.5.2.3.3 Comparison between Slow Sand Filters and Rapid Sand Filters*

(a) Base material: In SSF it varies from 3 to 65 mm in size and 30 to 75 cm in depth while for RSF it varies from 3 to 40 mm in size and its depth is slightly more, i.e. about 60 to 90 cm.

(b) Filter sand: In SSF the effective size ranges between 0.2 to 0.4 mm and uniformity coefficient between 1.8 to 2.5 or 3.0. In RSF the effective size ranges between 0.35 to 0.55 and uniformity coefficient between 1.2 to1.8.

(c) Rate of filtration: In SSF it is small, such as 100 to 200 L/h/sq.m. of filter area while in RSF it is large, such as 3000 to 6000 L/h/sq.m. of filter area.

(d) Flexibility: SSF are not flexible for meeting variation in demand whereas RSF are quite flexible for meeting reasonable variations in demand.

(e) Post treatment required: Almost pure water is obtained from SSF. However, water may be disinfected slightly to make it completely safe. Disinfection is a must after RSF.

(f) Method of cleaning: Scrapping and removing of the top 1.5 to 3cm thick layer is done to clean SSF. To clean RSF, sand is agitated and backwashed with or without compressed air.

(g) Loss of head: In case of SSF approx. 10 cm is the initial loss, and 0.8 to 1.2m is the final limit when cleaning is required. For RSF 0.3 m is the initial loss, and 2.5 to 3.5 m is the final limit when cleaning is required.

### *7.5.2.3.4 Other Types of Filters*

### (a) Pressure Filters

Pressure filters are circular pressurised vessels containing the filter media and usually designed for vertical flow. They work on the same principle as rapid gravity filters differing in that the filter medium is enclosed in a steel vessel and the water is forced through it under pressure.

### (b) Upward Flow Filters

Upward flow filters are theoretically more efficient than gravity filters where the water to be filtered flows upwards through the naturally desegregated, progressively finer and finer media so that coarser particles are trapped first in the coarser bottom layers. This tends to extend the period between backwashing. Several sand grades, getting progressively finer upwards have also been used in which case some restraining means such as a grid is required often located about 0.1 m below the surface where upward backwashing is used.

The major reason why RSF is a preferred option of water filtration to SSF is in the rate of filtration where RSF is higher per filter area flexibility, while SSF are not flexible for meeting variation in demand method of cleaning where in the case of RSF no sand is lost. Also, RSF requires minimal land size (Source: Ugandan Water Supply Design Manual (2013).

### (c) Roughing Filters

Roughing filters have their place as a form of primary treatment, especially for turbid or highly changeable river water. This technique of primary treatment has been greatly under-utilised in Tanzania in the past. Although much research has been undertaken since the late 1970s (Mbwette & Wegelin, 1984). It is used primarily to remove solids from high turbidity source waters prior to treatment with such unit operations as slow sand filters.

In a typical roughing filter, there are series of tanks which are filled with progressively smaller diameter media in the direction of the flow which can either be horizontal or vertical. With respect to the vertical flow direction, up-flow or down-flow roughing filters can be designed. The media in the tank which may include gravel, rice husks or any other suitable local material, plays an important role of reducing the vertical settling distance of the particles to a distance of a few millimetres.

### (i) Advantages of Roughing Filters

- They can considerably reduce the number of pathogens in the water, as well as the amount of iron and manganese.
- They can be considered a major pre-treatment process for turbid surface water since they efficiently separate fine solid particles over prolonged periods.
- Long filters (10 m) at low filtration rates (0.5 m/h) are capable of reducing high suspended solids concentrations (1000 mg/l TSS down to less than 3 mg/l).
- They are capable of reducing peak turbidity by 80 to 90 percent and faecal coliforms by 77 to 89 percent.
- They are placed at the treatment plant site and operated in combination with other pre-treatment units such as dynamic filters or sedimentation tanks.

**NB:** For detailed information, designers are recommended to download the comprehensive SANDTEC design report entitled '*Surface Water Treatment by Roughing Filters, A Design, Construction and Operation Manual*' available at http//sandec.ch/WaterTreatment/Documents/Surface%20Water%20Treatment.pdf

Features of Roughing Filters (RF).

The main part of the filter is the section containing the filter material. However, a Roughing filter comprises the following six elements:-

- Inlet Flow Control
- Raw Water Distribution
- Actual Filter
- Treated Water Collection
- Outlet Flow Control
- Drainage System

### (ii) Parameters for the Design of Roughing Filter

- Filter media size
- Filtration Rates
- Filter Length
- Filter media materials

### (iii) Filter Media Size

- The size of filter media decreases successively in the direction of water flow and ideally the uniformity of filter media fractions is maximized to increase filter pore space (storage capacity) and aid in filter cleaning (Boller, 1993).
- The use of multiple grades of filter media in a roughing filter promotes the penetration of particles throughout the filter bed and takes advantage of the large storage capacities offered by larger media and high removal efficiencies offered by small media

- The effect of surface porosity and roughness of filter media on particle removal efficiency in roughing filtration is insignificant compared to the size and shape of macro-pores in the filter (Wegelin, 1986)
- Media types commonly used in roughing filtration are quartz, sand and gravels but can be replaced by any clean, insoluble and mechanically resistant material (Graham, 1988)
- Common grades of media used in roughing filters are provided in the Table7.4

**Table 7.4: Different Sizes of Media in a Roughing Filter**

| Roughing Filter Description | 1st Compartment | 2nd Compartment | 3rd Compartment |
|---|---|---|---|
| **Coarse** | 24-16 | 18-12 | 12=8 |
| **Normal** | 18-12 | 12-8 | 8-4 |
| **Fine** | 12-8 | 8-4 | 4-2 |

(Source: *Onyeka Nkwonta and George Ochieng (1996)*)

### (iv) Filtration Rates

- Filtration rate has a significant influence on the treatment removal.
- Good removal in roughing filters are best achieved with low filtration rate (Boller, 1993), because low filtration rates are critical in retaining particles that are gravitationally deposited to the surface of the media.
- While used as pretreatment for iron and manganese removal, it can operate at filtration rates of 1.5 - 3 m/h (Hatva, 1988). It is reported that horizontal flow roughing filter is capable of removing metals like iron, manganese, turbidity and color at a filtration rate of 1.8 m/h (Dastanaie, 2007)
- At increased filtration rate (2 m/h), coarse particles penetrate deeper into the bed and they cause decrease in filter efficiency (Wegelin, et al. (1986), whereas at 1 m/h there is good distribution of solids loading throughout the bed. Hendricks (1991) also suggested that normal filtration rate of horizontal roughing filters is between 0.3 and 1.5 m/h.

### (v) Filter Media Length

- Improved cumulative removal efficiencies are typically correlated to longer filter lengths (Collins, 1994; Wegelin, 1986).
- Incremental removal efficiency decreases with increasing filter length due to the preferential removal of larger particles early in the filter (Wegelin, 1996).
- The rate of decline is dependent on filter design variables and the size and nature of particles in suspension. The use of different media sizes may allow for treatment targets to be met by a shorter filter with multiple media sizes compared to with long filter packed with one media size.

### (vi) Filter Media Materials

The following material could, therefore, be used as filter media:

- Gravel from a river bed or from the ground.
- Broken stones or rocks from a quarry.
- Broken burnt clay bricks.
- Plastic material either as chips or modules (e.g. used for trickling filters), these may be used if the material is locally available.
- Burnt charcoal, although there is a risk of disintegration when cleaning the filter material, should only be considered in special cases (e.g. for removal of dissolved organic matter).
- Coconut fibre, however, due to the risk of flavoring the water during long filter operation, should be used with care.
- Broken burnt bricks and improved agricultural waste (e.g. charcoal, maize cobs), can also be effectively used as pretreatment media (Ochieng (2006) and therefore could serve as alternatives where natural gravel is not readily available.

### (vii) Types of Roughing Filters

There are many types of roughing filters with different flow directions and with different types of filter medium (e.g. sand, gravel, coconut husk fibre). However, the common types are:

- Horizontal Flow Filters (HRF)
- Vertical (VRF)
- Dynamic (DRF)

## (d) Bank Filtration (BF)

Bank filtration (BF) is the infiltration of surface water, mostly from a river system into a groundwater system induced by water abstraction close to the surface water (e.g. river bank). This water abstraction is commonly done by operating wells.

As the water flows through the soil, it is filtered and hence its quality is improved. In the context of developing or newly-industrialised countries, bank filtration may contribute to a more sustainable water cycle by recharging stressed groundwater bodies with filtered surface water. (Sharma & Amy 2009; Huelshoff et al.)

Bank filtration is a water treatment technology that consists of extracting water from rivers by pumping wells located in the adjacent alluvial aquifer. During the underground passage, a series of physical, chemical, and biological processes take place, improving the quality of the surface water, substituting or reducing conventional drinking water treatment.

Bank filtration works by pumping pressure in the alluvial aquifer adjacent to the river that forces the water to percolate from the river into the aquifer. In this path, a series of physical and biogeochemical processes take place, including physical filtration, adsorption, absorption, biodegradation, and dilution. Thus, riverbank-filtrate often shows better quality than river water, making its treatment for human consumption easier and less expensive.

The removal of sediment, organic and inorganic compounds, and pathogens takes place during the first metres from the river in what is known as the hyporheic zone, which usually presents reducing conditions due to the high microbial activity which consumes the oxygen in the water. Within this zone there are important biochemical processes and Redox reactions that affect groundwater quality.

The efficiency of BF depends on the local conditions including the hydrology and hydrogeology of the site, the geochemistry of water (from both the river and the aquifer), the geochemistry of microbial populations, and associated metabolic activity. This is the reason why it is difficult to define general procedures for identifying appropriate sites to implement the BF technique, as well as the expected efficiency of the process.

One limitation on the efficiency of BF is the clogging of the bed and the banks of the river, which decreases the hydraulic conductivity in the hyporheic *zone*. This clogging can be caused by infiltration of the fine sediments, gas entrapment, bio-film formation related to microbiological activity, or the precipitation and co-precipitation of inorganic compounds, being the first of these the most influential factor in clogging formation.

**Siting and Design Parameters**

(i) The most important parameters for success during BF are the flow path length, the thickness of the aquifer, and the infiltration area in the river (Grischek *et al*. (2002).
(ii) The siting and design of a BF system depend on hydrogeological, technical, economical, regulatory, and land-use factors.
(iii) Riverbank filtration wells can be designed either vertically (as the most common practice especially for the extraction of low water quantities) or horizontally (for higher extraction rates).
(iv) Local factors such as river hydrology, hydrogeological site conditions (i.e., aquifer thickness and hydraulic conductivity), and the aims of water withdrawal are used to determine the capacity of the wells, the travel time of the bank filtrate, distance between the river and the well.
(v) Horizontal wells (sometimes with a radial pattern), also known as collector wells, are directed toward the river and extract water from beneath the riverbed, whereas vertical wells extract water along the riverbed.
(vi) The BF wells can also be distributed parallel to the riverbank in galleries or groups.

### 7.5.2.4 Floatation

Floatation may be defined as the transfer of a suspended phase from the bulk of a dispersion medium to the atmosphere/liquid interface by means of bubble attachment. There are three basic processes involved which are: bubble generation, bubble attachment and solids separation.

Floatation is described as a gravity separation process, in which gas bubbles attach to solid particles to cause the apparent density of the bubble-solid agglomerates to be less than that of the water thereby allowing the agglomerates to float to the surface. The different methods of producing the gas bubbles give rise to different types of Floatation processes which are Dissolved-air Floatation, Electrolytic Floatation and Dispersed-air Floatation.

### *7.5.2.4.1 Dissolved-Air Floatation*

This is a relatively new solution for the clarification of surface and ground waters. The process as shown in Fig 7.24is relatively simple and can be very effective. After flocculation, the produced floc attaches to micro-bubbles and rises to the water surface. The floated solids are periodically evacuated either hydraulically or mechanically, depending on the sludge concentration.

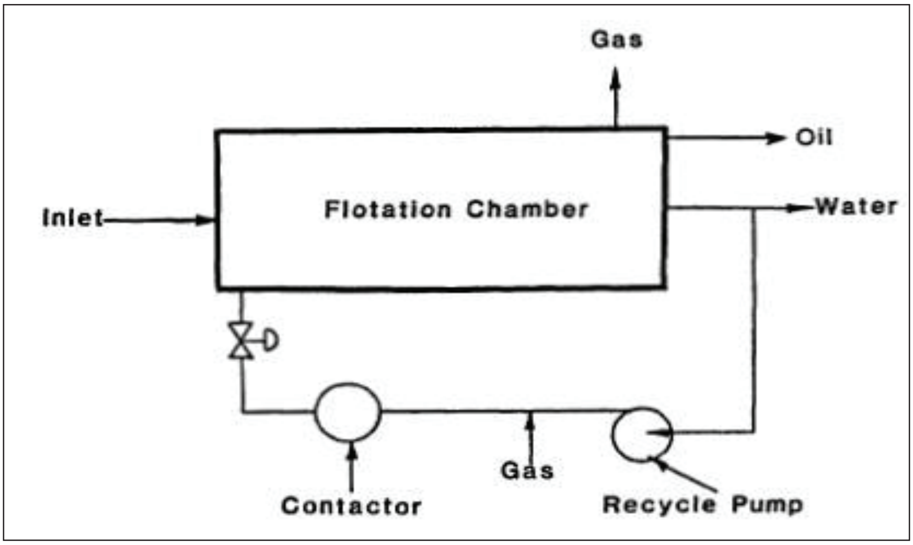



**Figure 7.24: A Diagram of Dissolved –air Flotation Mechanism**

(Source: https://www.ircwash.org/sites/default/files/259-72WA-1.pdf

The dissolved air flotation process can be a good solution for the treatment of water with a high concentration of algae or other low density particles. The system is said to have a number of advantages including:

(a) Reliable removal of algae, *cryptosporidium* and *giardia cysts*
(b) Removal of colour and taste compounds
(c) Removal of low-density solids
(d) No polymer required
(e) Concentrated sludge
(f) Rapid start-up after shutdown

(g) Few mechanical components
(h) Low operating costs

The removal of suspended matter solids is achieved by dissolving air in the water under pressure and then releasing the air at atmospheric pressure in a floatation tank basin. The released air forms tiny bubbles which adhere to the suspended matter causing the suspended matter to float to the surface of the water where it may then be removed by a skimming device. Drinking water supplies that are particularly vulnerable to unicellular algal blooms, and supplies with low turbidity and high colour often employ DAF.

### *7.5.2.4.2 Electrolytic Floatation*

The basis of electrolytic or electro-floatation is the generation of bubbles of hydrogen and oxygen in a dilute aqueous solution by passing a direct current between two electrodes. Electrical power is supplied to the electrodes at a low voltage potential of 5 to 10 VDC by means of a transformer rectifier. The energy required for electro-Floatation depends largely on the conductivity of the liquid and the distance between the electrodes.

### *7.5.2.4.3 Dispersed-Air Floatation*

Both foam and froth dispersed air floatation are unsuitable for water treatment applications because the bubble size tends to be large (>1 mm. compared to 20-111 for dissolved-air Floatation and electro-Floatation) and either high turbulence (froth Floatation) which would break up the fragile flocs formed during the chemical pre-treatment, or undesirable chemicals (foam Floatation) are required to produce the air bubbles required for Floatation. Appendix B.4 (iii) illustrate the Performance of the Dispersed-Air Floatation.

## 7.5.2.5 Aeration

Aeration is the process whereby water is brought into intimate contact with air. Aeration has a large number of uses in water treatment for the following purposes:

(a) Increasing dissolved oxygen content in the water;
(b) Reducing taste and odour caused by dissolved gases in the water, such as hydrogen sulphide, which are then released; and also to oxidise and remove organic matter;
(c) Decreasing carbon dioxide content of water and thereby reducing its corrosiveness and raise its pH value;
(d) Oxidizing iron and manganese from their soluble states to their insoluble states and thereby causing them to precipitate so that they can be removed by clarification and filtration processes;
(e) Reduction of radon; and
(f) Removing certain volatile organic compounds.

Aeration is widely used for the treatment of ground water having unacceptably high contents of dissolved iron and/ or manganese. The atmospheric oxygen brought into the water through aeration, reacts with the dissolved ferrous and manganese compounds, changing them into insoluble ferric and manganic oxide hydrates. The hydrates can then be removed by the subsequent processes of sedimentation and/or filtration.

Chemicals removed or oxidized by aeration:

- Ammonia
- Chlorine
- Carbon dioxide
- Hydrogen sulphide
- Methane
- Iron and Manganese
- Volatile organic chemicals, such as benzene (found in gasoline), or trichloroethylene, dichloroethylene, and perchloroethylene (used in dry-cleaning or industrial processes)

**Types of Aerators**

Aerators fall into two categories:

- Falling water aerators
- Injection aerators

### *7.5.2.5.1 Falling Water Aerators*

In the falling water aerators, water is dropped through air and in the second group air is introduced into the water as small bubbles. Falling water aerators can be divided into:

- Spray Aerators
- Multiple Tray Aerators
- Cascade Aerators

## (a) Spray Aerators

Water is sprayed through nozzles upward into the atmosphere and broken up into either a mist or droplets. Water is directed vertically or at a slight inclination to the vertical. The installation consists of trays and fixed nozzles on a pipe grid with necessary outlet arrangement.

## Design details of Spray Aerators

- Nozzles usually have diameters varying from 10 to 40 mm, spaced along the pipe at intervals of 0.5 to 1m or more. Special (patented) types of corrosion resistant nozzles and sometimes plain openings in pipes, serving as orifices are used.

- The pressure required at the nozzle head is usually 7 m of water but in practice, varies from 2 – 9 m and the discharge rating per nozzle varies form 30 - 600 l/min.
- Aerator areas are usually 30 – 90 $m^2$ per 1,000 $m^3$/hr.
- The 'Dresden' type of nozzles gives very good results in removing $CO_2$ and in adding $O_2$ but is poor for radon removal.

## (b) Multiple Tray Aerators

These aerators consist of a series of trays with perforated bottoms. The trays are filled with coke, stone or ceramic balls, limestone, or other materials having a catalytic effect on iron removal. The primary purpose of the materials is providing additional surface contact area between the air and water. Through perforated pipes, the water is divided evenly over the upper tray, from which it trickles down, the droplets being dispersed and re-collected at each successive tray. Appendix B.5 (iii) illustrates the Multiple Tray Aerators.

## Design details of Multiple Tray Aerators

- 3-5 trays are normally used at the intervals of 0.3 - 0.7 m which means that the head needed is 1.5 – 3 m.
- The area required is 40 $m^2$ per 1,000 $m^3$/hr. These aerators have good $CO_2$ removal and good $O_2$ increases *(3rd edition Design Manual, 2009).*
- The design surface loading of a multiple tray aerator should be of the order of 70 $m^3$/hour/$m^2$ (*Water supply Design Manual, Uganda (2013).*

However, disadvantages of *Multiple Tray Aerators* are:

- risk of clogging
- difficult cleaning and breeding places for worms

## (c) Cascade Aerators (Gravity Aerators)

The Cascade Aerators are the simplest type of free-fall aerators and will take large quantities of water in a comparatively small area and at low head. They are simple to keep clean and can be made of robust durable material such as reinforced concrete and are best in the open air. Turbulence is secured by allowing the water to pass through a series of steps or baffles (*3rd Edition Design Manual, 2009).*

## Features of Cascade Aerator

(i) A cascade aerator consists of a flight of 4 - 6 steps, each about 300 mm high**,** to produce turbulence and thus enhance the aeration efficiency, obstacles are often sat at the edge of each step as illustrated in Figure 7.25.

(ii) The design capacity of a cascade aerator should be of the order of 35 $m^3$/hour per meter of width.

(iii) Exposure time can be increased by increasing the number of steps which is normally between 3 - 10.
(iv) The fall in each step is usually between 0.15 - 0.6 m. The area required is about 40 $m^2$ per 1,000 $m^3$/hr.
(v) The efficiency for raising $O_2$ content is good and can reach 2.5 kg $O_2$/kWh, but that for $CO_2$ removal rarely better than 60 – 70 %, whilst radon reduction can exceed 50%.
(vi) The principle is to spread the water as much as possible and let it flow over obstructions to produce turbulence.
(vii) These are similar to tray aerators, but with a series of steps or platforms over which the water cascades. Obstacles may be placed on the edge of each step.
(viii) Cascades aerators generally take up more space than tray aerators, but the overall head loss is lower, and maintenance is minimal.
(ix) Where space permits, Cascade aerators are the preferred type of aerators.

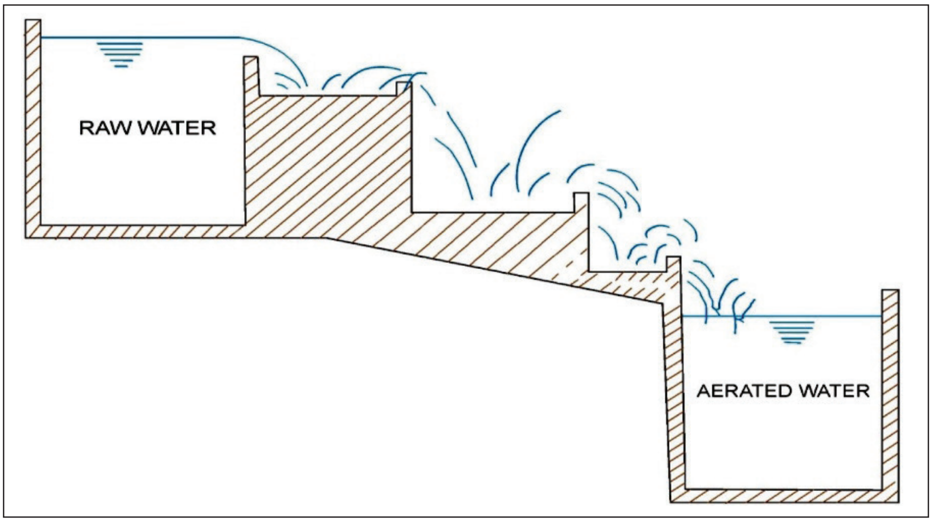



**Figure 7.25: Diagram of a Cascade Aerator**
(Source: Water Supply Design Manual, 2013, Uganda)

## Design details of Cascade Aerators

- Number of drops = 4-6
- Height of drops = 30-60 cm
- Overflow rate = 0.01 $m^3$/s over meter width of step.
- Height of aerator = 2-3 m
- Cascade area = 1.5-2.0 $m^2/m^3$/min of flow

### *7.5.2.5.2 Injection Aerators*

These aerators have a good efficiency in raising $O_2$ content but poor $CO_2$ removal. They can be categorised as:

- Bubble Aerators
- Venturi Aerators
- Brush Aerators
- Inka Aerators

### (a) Bubble Aerators

In these aerators, air is blown to the bottom of the tank through porous filters. Bubble aerators are often applied to existing treatment plants where no spare hydraulic head is available

### Design details of Bubble Aerators

- The depth of the tank is 3 - 4.5 m and the width about 2 times the depth.
- Detention time in the tank is 10 - 20 minutes.
- Air required is 40 – 120 m$^2$ per 1,000 m$^3$/hr, being 40% to 80% of water capacity.
- The air bubbles should be as small as possible but the clogging of the filters interfere with that aspect.
- Good mixing in an aeration tank improves efficiency.

### (b) Venturi Aerators

In venturi aerators air is not blown into the water but is drawn in by a venturi. The $O_2$ improvements is good but $CO_2$ removal is poor.

### (c) Brush Aerator

This type of aerator consists of a revolving drum, diameter about 0.5 m, submerged about 0.4 m of the diameter, and rotating about 100 rpm. However, this type is not commonly used for water treatment.

### (d) Inka Aerators

This aerator consists of a perforated stainless steel plate under which air is blown. Water flows over the plate. The air water ratio is very high. It can be as high as 100:1. This amount of air causes heavy turbulence and so $O_2$ raising efficiency and $CO_2$ removal are good. Energy consumption is rather high corresponding to a hydraulic head of 7 m if air water ratio is 100. A disadvantage is the clogging of the perforated plate.

## 7.5.3 Secondary Treatment

In this DCOM Manual, the following unit operations have been described in detail as secondary treatment. However, constructed wetlands are simply mentioned because they are described in detail in volume II of the manual.

- Coagulation

- Flocculation
- Clarification
- Filtration (SSF, RSF and other types)
- Reverse Osmosis
- Membrane Filtration (ultrafiltration (UF), Microfiltration (MF), nanofiltration (NF)
- Ion Exchange
- Adsorption
- Softening

### 7.5.3.1 Clarification

Clarification is the process of removing all kinds of particles, sediments, oil, natural organic matter and colour from the water to make it clear. A clarification step is the first part of conventional treatment for water and wastewater treatment. It usually consists of physical and/or chemical treatment. Coagulation is normally followed by flocculation in a clarifier, which could be circular or rectangular in shape. After clarification water is then ready for filtration.

### 7.5.3.2 Coagulation

Coagulation is the process of adding a chemical (coagulant) to the raw water containing colloidal matter to form small gelatinous precipitated masses, which can readily settle out in sedimentation tanks within the normal range of surface loading. The coagulation stage occurs when a coagulant is added to the water to neutralise the charges on the colloidal particles in the raw water, thus bringing the particles closer together to allow a floc to begin to form. Rapid, high energy mixing (e.g. mechanical mixers, in-line static mixers, jet sparge mixing) is necessary to ensure the coagulant is fully mixed into the process flow to maximise its effectiveness. The coagulation process occurs very quickly, in a matter of fractions of a second. Poor mixing can result in a poorly developed floc. The most common coagulant in use in Tanzania is Aluminium Sulphate (Alum), which at times is supplemented with coagulant aids.To determine the correct chemical dosage for Aluminium sulphate solution and for water disinfection, jar testing is recommended.

### 7.5.3.3 Flocculation

The flocculation process, following coagulation, allows smaller particles formed during the rapid coagulation stage to agglomerate into larger particles to form settleable and/or filterable floc particles. After coagulant addition, the process water is mixed slowly for a defined flocculation period, commonly 10 - 30 minutes. However, the optimum flocculation time will vary depending on the raw water quality and downstream clarification process. Gentle mixing during this stage provides maximum particle contact for floc formation, whilst minimising turbulence and shear which may damage the flocs. The effectiveness of flocculation depends on the delay (or contact) time and mixing conditions prior to any flocculants being added, the rate of treatment, water temperature and the mixing conditions within the flocculation chamber. Flocculation takes place in a flocculator. There are two types of flocculators namely hydraulic and mechanical.

### 7.5.3.4 Filtration

Filtration is the process in which organisms, bacteria and particles of size less than $10^{-8}$ cm are removed. There are four main types:

- slow sand filters
- rapid (gravity) sand filters
- pressure filters
- upflow sand filters

Sand filters become clogged with floc after a period in use and they are then backwashed or pressure washed to remove the floc. This backwash water is usually run into settling tanks so that the floc can settle out and it is then disposed of as waste material. The supernatant water is sometimes run back into the treatment process, although this can bring some problems with it, or disposed of as a wastewater stream.
The criteria for designing filters are:

- Flow Rate
- Size of Media
- Depth of Media
- Type of Media
- Arrangement of gradation of Media
- Fluid characteristics
- Head loss
- Length of run
- Method of cleaning

The detailed description of filtration design considerations, criteria and steps have been given in primary treatment section.

## 7.5.4 Tertiary Treatment

Tertiary treatment has considered the following unit operations:

- Disinfection,
- Ozonation,
- Water softening and
- Water conditioning.

The single most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease or illness to the consumer. Processes such as storage, sedimentation, coagulation and flocculation and rapid filtration reduce to varying degrees the bacterial content of water. However, these processes cannot assure that the water they produce is bacteriologically safe, therefore disinfection is finally needed. Disinfection is carried out by observing the following criteria:

- The nature and number of organisms to be destroyed
- The type and concentration of the disinfectant used
- The temperature of water to be disinfected
- The time of contact needed
- The nature of water to be disinfected
- The pH, acidity/alkalinity of the water

### *7.5.4.1.1 Disinfection Methods*

There are two principle methods for disinfecting water; one is physical and the other chemical. Further details about disinfection methods are given in Appendix H.

### *7.5.4.1.2 Chlorinators*

Chlorine is the most common used disinfectant for drinking water in Tanzania. A chlorinator is a device designed for feeding chlorine into a water supply. Its functions are:

- to regulate the flow of gas from the chlorine container at the desired rate of flow,
- to indicate the flow rate of gas feeding,
- to provide means or properly mixing the gas either with an auxiliary supply of water or with the main body of the liquid to be disinfected.

The usual fittings and parts of a chlorination system are:

- Chlorine cylinder or drum supplied with its own main valve and filled with liquid and gaseous chlorine, under pressure,
- Fusible plug, a safety device provided on all cylinders and containers designed to meet temperatures often between 70 $^0$C to 75 $^0$C,
- Reducing valve / vacuum regulator to bring the pressure of the gas down to between 70 to 30 kPa so that the pressure is below atmospheric (approx 100 kPa). This should be located in the storage room so that any leakage in the dosing room is into the feed pipes rather than into the room itself,
- Pressure gauges one to indicate the cylinder pressure and the other the delivery pressure,
- A measurement device consisting of an orifice to measure upstream or downstream pressure of gas with manometer containing liquid of carbon tetrachloride,
- A “desiccator valve” or non-return valve containing concentrated sulphuric acid or calcium chloride through which the chlorine must pass to free it from moisture so that any corrosive action of the moist chlorine on the fitting is prevented.

### *7.5.4.1.3 Design Considerations for Chlorinators*

The following should be considered while designing chlorinators:

- Access to storage and dosing rooms should separate and be from the open air and doors should always open outwards,
- External windows should be avoided where possible with artificial illumination being provided throughout,
- Both storage and dosing rooms should be provided with low level outlet venting fans that either come on automatically when the door is opened or are activated from outside the room so that any leakage is purged to the outside before entering such rooms,
- High level fresh air inlets should be provided, especially to the storage room.

## 7.5.4.2 Ozonation

Ozone has been proved to be one of the most effective disinfectants and is widely used to inactivate pathogens in drinking water (Xu, 2002). Transferred ozone dose is the critical parameter for the design of wastewater disinfection by ozonation. The process should have an efficient filtration step to meet stringent standards. A properly designed ozonation process can deactivate viruses and bacteria that may be contained in wastewater.

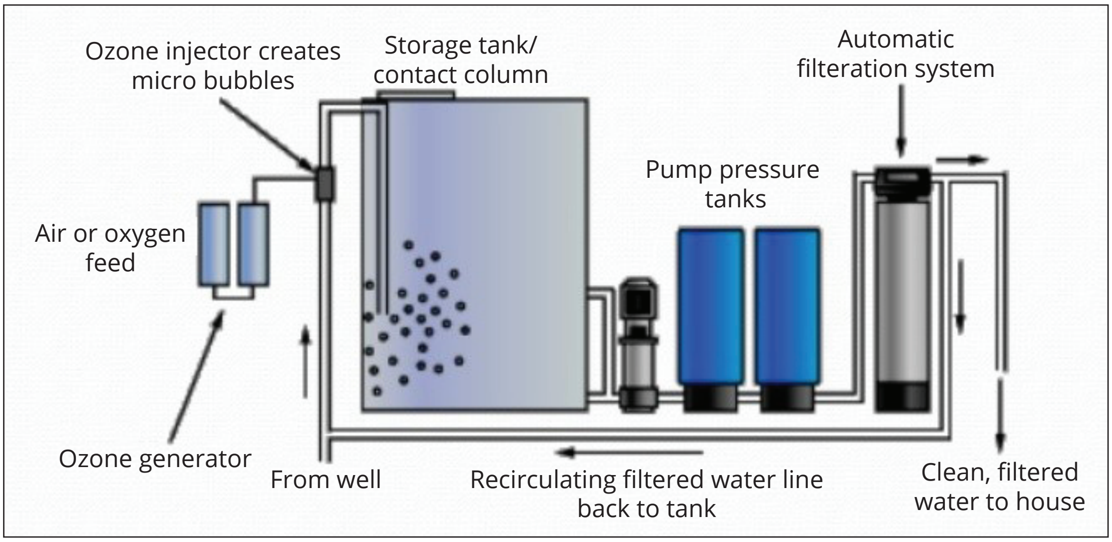



**Figure 7.26: Ozonation Process[10]**

## 7.5.4.3 Water softening

Softening is the process of removing the dissolved calcium and magnesium salts that cause hardness in water. The hardness or soap consuming power of water is due to the presence of bicarbonates, carbonates, sulphates, chlorides, and

[10] https://www.slideshare.net/gowrivprabhu/water-treatmentwater-treatment

nitrates of calcium and magnesium. The dissolved compounds have the following negative effects:

(a) Soap destroying or increased soap consumption in laundries,
(b) Deposition of scale in boilers and engine jackets,
(c) Corrosion and incrustation of pipelines, joints valves and plumbing fixtures; and
(d) Serious difficulties and detrimental effects in the manufacturing processes, e.g. textile finishing, dyeing, canning, paper making, ice manufacturing, tanning etc.

When water is hard, it can clog pipes and soap will dissolve in it less easily. In industrial scale water softening plants, the effluent flow from the re-generation process can precipitate scale that can interfere with sewage systems. Hard water leads to the build-up of lime scale, which can foul plumbing, and promote galvanic corrosion. Water softening is the removal of calcium, magnesium, and certain other metal cations in hard water. It is achieved either by adding chemicals that form insoluble precipitates or by ion exchange. Water softening is usually achieved using lime softening or ion-exchange resins but is increasingly being accomplished using nanofiltration or reverse osmosis membranes.

Chemicals used for water softening include calcium hydroxide (slaked lime) and sodium carbonate. The resulting soft water requires less soap for the same cleaning effort, as soap is not wasted bonding with calcium ions. Soft water also extends the lifetime of plumbing by reducing or eliminating scale build-up in pipes and fittings. Detailed information about hardness is provided in Appendix I

### *7.5.4.3.1 Methods of Softening Water*

The most common means for removing water hardness (calcium and/or magnesium) and hence achieve softening are:

- Chemical precipitation;
- Ion-exchange resin; and
- Reverse Osmosis (RO).

### (a) Softening By Chemical Precipitation

Chemical precipitation is among the most common methods used to soften water. Chemicals used are lime (calcium hydroxide, $Ca(OH)_2$) and soda ash (sodium carbonate, $Na_2CO_3$). Softening by chemical precipitation is accomplished by adding lime or lime and soda ash. Softening with these chemical is used particularly for water with high initial hardness greater than 500 mg/l and suitable for water containing turbidity, colour, and iron salts because these have a tendency to inactivate the ion exchange bed, by a coating on the granules. Lime-soda softening cannot, however, reduce the hardness to values less than 40 mg/l and this should not be attempted.

Ion-exchange softening can produce zero-hardness water but such water should always be blended with water to leave a residual hardness of not less than 70 mg/l because apart from the risk of cardiovascular problems, very soft drinking water may be corrosive and result in feelings of sickness.

**(i) Lime Treatment**

Lime softening is the process in which lime is added to hard water to make it softer. It has several advantages over the ion-exchange method but requires full-time, trained personnel to run the equipment. Addition of lime to hard water only removes the carbonate hardness. Insoluble carbonates of calcium and magnesium are precipitated out and removed in sedimentation tanks. (An overdose of lime is usually used and the excess lime is neutralized by re-carbonation before filtration). This treatment is good when the bulk of the hardness is due to calcium and magnesium is insignificant. When the water contains more than 40 mg/l of magnesium warranting its removal, excess lime treatment must be done.

**(ii) Lime – Soda Treatment**

Lime is used to remove chemicals that cause carbonate hardness, while Soda-ash is used to remove chemicals that cause non-carbonate hardness. In lime treatment only the carbonate hardness is removed but by addition of soda, the non-carbonate hardness is also removed, thus the removal of both carbonate as well as non-carbonate hardness is possible in the lime-soda process. This happens because in lime-soda ash softening process $Ca^{2+}$ is removed from water in the form of calcium carbonate, $CaCO_3$ (s) and $Mg^{2+}$ is removed in the form of magnesium hydroxide, $Mg(OH)_2$ (s). These precipitates are then removed by conventional processes of coagulation/flocculation, sedimentation, and filtration. Because precipitates are very slightly soluble, some hardness remains in the water usually about 50 to 85 mg/l (as $CaCO_3$). This hardness level is desirable to prevent corrosion problems associated with water being too soft and having little or no hardness. Precipitation of these salts is affected by the available Carbonate species and pH of the system

- For calculating the theoretical amount of lime and soda required for softening, an analysis of the following constituents in the water is necessary:
- free carbon dioxide dissolved in the water bicarbonate (total alkalinity)
- total hardness
- total magnesium

Chemical requirement (mg/l) are computed by the sum of the following factors:

- **Lime requirements as $Ca(OH)_2$ (100% purity)**
  - 56/44 of concentration of $CO_2$ (mg/l as $CO_2$)
  - 56/24 of concentration of Mg (mg/l as Mg)
  - 56/100 of concentration of alkalinity (mg/l as $CaCO_3$)

Additional lime required for raising the pH to the range of 10 to 10.5 for precipitation of $Mg(OH)_2$ is about 30 - 50 mg/l as CaO (*Quick lime*).

- **Soda requirements as $Na_2CO_3$**
  - 106/100 of difference between total hardness and bicarbonate alkalinity both expressed as $CaCO_3$.
  - For neutralizing excess lime at 30 mg/l, additional soda required is (30/56) x 106 mg/l as $Na_2CO_3$.

Plant conditions like temperature; time of detention and agitation influence the completeness of reactions and dosage of chemicals may have to be increased to provide for the inadequacies.

Alternatively, caustic soda can be used instead of lime. Liquid caustic soda should be used since it can be handled and fed easily. The amount calcium carbonate sludge formed in this case is theoretically half that formed by use of lime. However, using caustic soda is costlier than soda ash which is more expensive than lime.

### (iii) Excess Lime Treatment

When water contains more than 40 mg/l of magnesium, excess lime treatment has to be done since magnesium has to be removed as magnesium hydroxide whose solubility decreases with increasing pH values. The water treated thus is highly caustic and must be neutralised after precipitation either by re-carbonation or by split treatment. In split treatment, the total flow is divided into two parts, one part being treated with excess lime and the settled effluent then mixed with un-softened water. The final residual hardness in the water will depend on the percentage flow by-passed and the levels of or hardness in both the portions (treated and by passed).

### (iv) Hot Lime-Soda Treatment

This process is used for boiler feed water treatment. It is similar to the cold process already discussed except that the raw water is heated to about 95° - 100°C before being taken' to the reaction tank. Reactions take place rapidly due to decreased viscosity hastening the settling of the precipitates. A greater degree of softening is accomplished than that in the conventional cold processes.

### (v) Re-carbonation

After lime and/or soda ash treatment is applied, the treated water will generally have a pH greater than 10. In addition, after softening, water becomes supersaturated with calcium carbonate. If this water is allowed to enter the distribution system in this state, the low pH would cause corrosion of pipes and the excess calcium carbonate would precipitate out, causing scale. So, the water must be re-carbonated, which is the process of stabilizing the water by lowering its pH and precipitating out excess lime and calcium carbonate.

Therefore, the goal of re-carbonation is to produce stable water. Stable water has a calcium carbonate level, which will neither tend to precipitate out of the water (causing scale) nor dissolve into the water (causing corrosion). This goal is usually achieved by pumping $CO_2$ into the water. Enough $CO_2$ is added to reduce the pH of the water to less than 8.7. When $CO_2$ is added, the excess lime will react with $CO_2$ producing $CaCO_3$ (s). Recarbonation also lowers the water pH.

**(b)Water Softening through Ion-Exchange**

The ion-exchange process is the reversible inter-change of ions between a solid ion exchange medium and a solution and is used extensively in industrial water/ softening. The hardness producing ions preferentially replace the cations in the exchangers and hence this process is also known as base or cation exchange softening.

The ion exchange works on the hydrogen or sodium cycle. The hydrogen ions are released into the water in the former case and the sodium ions in the latter. There is only a temporary change in the structure of the exchange material. The exchange material can be re-generated using an acid and sodium chloride respectively.

In general, ion exchange materials used in water softening, also called zeolites, are hydrated silicates of sodium and aluminium. There are inorganic and organic zeolites:

**(i) Inorganic Zeolites**

Natural inorganic zeolite is available as 'green sand' while the synthetic or gel type is obtained through the reaction of either sodium aluminates or aluminium and is graded to suitable sizes by the reaction of either sodium aluminate or aluminium sulphate with sodium silicate which, after drying, is graded to suitable sizes by screening. For regeneration, 3.5 to 7 kg of salt is required for every kilogram of water hardness removed.

**(ii) Organic Zeolites**

These consist of carbonaceous materials and sulphonated stayrone type resins which have excellent cation exchange properties, requiring for regeneration, 2-4 kg salt for every kilogram of hardness removed. These are resistant to attack by acid solutions and hence can be regenerated by acid. They can be used for waters with a wide pH range, whilst the loss due to attrition is negligible compared to the synthetic inorganic zeolites.

**(iii) Raw water characteristics**

Raw water to be treated by ion-exchange should be relatively free from turbidity otherwise the exchange material gets a coating which affects the exchange capacity of the bed. The desirability of using filters prior to zeolite beds or resorting to more frequent regeneration would depend upon the level of turbidity. Metal ions like iron and manganese, if present, are likely to be oxidised and can coat zeolites, thus deteriorating the exchange capacity steadily since the regeneration cannot remove the coats.

Oxidizing chemicals like chlorine and carbon dioxide as well as low pH in the water will have a tendency to attack the exchange materials particularly the inorganic type, the effect being more pronounced on the synthetic inorganic zeolites. Waters low in silica inorganic zeolites, are to be avoided in boiler feed water. The organic zeolites, operating on a brine regeneration cycle do not add any silica to the water and consequently are ideally suited for boiler feed water.

**Caution**

The ion exchange process is both costly and delicate and should not be adopted without advice from a competent authority. In case the need arises for using this type of process for water softening then the details of the process design should be obtained from a standard textbook or plant manufacturer.

### 7.5.4.4 Defluoridation of Water

#### *7.5.4.4.1 Fluorides*

Fluoride is the ionic form of fluorine. Fluorides are organic and inorganic compounds containing the element fluorine. As a halogen, fluorine forms a monovalent ion (−1 charge). Fluoride forms a binary compound with another element or radical. Examples of fluoride compounds include hydrofluoric acid (HF), sodium fluoride (NaF) and calcium fluoride ($CaF_2$), and uranium hexafluoride ($UF_6$).

Fluoride compounds, usually calcium fluoride, are naturally found, usually in low concentration in water. However, water from underground sources can have higher levels of fluoride to the level that it becomes a health hazard.

Excessive fluorides in drinking water may cause mottling of teeth or dental fluorosis, a condition resulting in the coloration of the tooth enamel, with chipping of the teeth in severe cases, particularly in children. With even higher levels of fluorides, there are cases of fluorosis of the bony structure.

The chief sources of fluorides in nature are:

- fluorapatite (phosphate rock)
- fluorspar
- crylite and
- igneous rocks containing fluorosilicates

A designer, in deciding on whether or not to include de-fluoridation in a water supply scheme should, therefore, consider both the number of potential consumers, alternative sources, the financial consequences both in capital and running and whether or not there is a possibility to dilute the water containing the fluoride as a means of reducing the concentration. Defluoridation technology opted by the designer should ensure that the product (treated water) meets relevant Tanzania Standards and other standards/guideline which may have been adopted by the country.

### *7.5.4.4.2 Defluoridation*

Defluoridation is necessary when the fluoride concentration is higher than the acceptable limits. The following methods may be considered for attaining defluoridised water standards.

- Desalination
- Additive methods
- Absorption methods

**Desalination**

Desalination effectively removes all dissolved impurities from water. This can be accomplished in one of several ways, by freezing, by distillation, by electrolysis or by reverse osmosis. The cost of desalination is high although the costs of reverse osmosis have fallen considerably in recent years. Nevertheless, this method is more appropriate to deal with brackish or sea water although it should not be entirely ruled out where there a few, if any, alternative sources and there is a good supply of electricity.

**Additive method**

In this method, one or more chemicals are added to water. The fluoride is then absorbed and both the additive and the fluoride are consequently removed by using conventional treatment processes such as sedimentation and filtration. A wide variety of materials have been tried including lime, magnesium sulphate, magnesium oxide, calcium phosphate, aluminium sulphate, various natural earths, bauxite, sodium silicate and sodium aluminate.

Excessive lime treatment for water softening affects the removal of fluoride due to its absorption by the magnesium hydroxide floc. However, sizeable fluoride removal is possible only when magnesium is present in large quantities which may not always be the case and magnesium may have to be supplemented in the form of salts.

The initial cost and cost of chemicals is very high and the resultant sludge is environmentally difficult to dispose of.

**Absorption methods**

Absorption methods employ a bed or filter of generally insoluble material through which the water is allowed to percolate periodically. As it becomes saturated, with the fluoride ‘removed, the absorptive media is either replaced or appropriately regenerated.

(a) Materials used have included charred bone, activated alumina, activated carbon, tricalcium phosphate, natural and synthetic ion exchange materials and aluminium sulphate.

(b) Studies elsewhere have revealed that activated carbon has a good capacity to remove fluoride where the concentration is less than 10 mg/l and the water is low in salinity.

(c) An activated carbon for fluoride removal has been developed in India by carbonising paddy husk or saw dust, digesting under pressure with alkali and quenching it in a 2% alum solution. The spent material can be regenerated by soaking it in a 2% alum solution.

(d) Also a granular ion exchange material, 'Defluoron 2', which is sulphonated coal operating on the aluminium cycle has been developed in India.

(e) The water treatment specialist M/S Degrémont recommended the activated alumina in the case of the Arusha urban water supply project.

However, defluoridation must be regarded as a sophisticated process and to determine suitability and quantity of chemical, pilot plant trials should be conducted first.

#### 7.5.4.5 Water Conditioning

Water conditioning entails ensuring that at the end of the treatment process but just before the water is pumped or gravitated into the clear water reservoir, storage tanks or the distribution network, the treated water should be neither precipitative nor corrosive. Hence this may require a pH correction in either direction depending on the outcome of the daily laboratory analysis results. Moreover, if there will be a need to increase the pH, lime will have to be prepared as a slurry for the sake of controlling the dosage and hence dosed at the point of dosing the coagulants and other chemical additives. In case the pH of water has to be increased, a dose of an acid would suffice. Care has to be taken against introducing major shifts of pH due to the risk of making it limiting as for example when Ferrous chemicals are used in coagulation of water.

### 7.5.5 Management of Water Treatment Sludge

The conventional water treatment plant involves the process of coagulation, flocculation, sedimentation, filtration and disinfection. Large volumes of water treatment sludge (WTS) or residues thereof are generated during the processing of raw water to make it fit for drinking purposes. A typical water treatment plant produces about 100,000 ton/year of sludge (Bourgeois et al., 2004). Experiences elsewhere have shown that, due to the lack of sludge management strategies, most of the WTPs discharge their filter backwash water and sludge into nearby drains which ultimately meet the water source. Aluminium salts (e.g. $Al_2(SO4)_3.18H_2O$) or Iron salts (e.g. $FeCl_3.6H_2O$, $FeCl_2$, $FeSO_4.7H_2O$) are commonly used as coagulants (Sales et al., 2011). These salts get hydrolysed in water to form their respective hydroxide precipitates. Colloidal and suspended impurities such as sand, silt, clay, humic particles present in the crude water are removed by charge neutralization, sweep floc mechanism and adsorption onto hydroxide precipitates (Trinh and Kang, 2011). The hydroxide precipitate along with sand, silt, clay and humic particles removed from the raw water mainly constitute the solids present in the sludge. The moisture content of the wet sludge is generally above 80 wt% (Tantawy et al., 2015). In general, this sludge is discharged directly into nearby water bodies or dumped in the landfills after dewatering.

## 7.5.5.1 Treatment of Water Treatment Sludge

### *7.5.5.1.1 Sludge Thickening*

Sludge thickening is defined as the removal of water from the sludge with the aim of substantially reducing sludge volume. For example if sludge with 0.8% dry solids (DS) can be thickened to 4% DS; a fivefold decrease in sludge volume is achieved. The objective of sludge thickening is to produce a sludge that is as thick as possible which can be pumped without difficulty and includes a relatively solid free liquid supernatant.

### (a) Gravity Thickening

#### (i) Description of Unit

Gravity sludge thickening is the method commonly adopted. The slope of the bottom of the gravity thickeners should be carefully selected in order to facilitate the flow of thickened sludge towards the centre/collection pit. Gravity thickeners, usually circular in shape and provided with pickets or rakes to improve dewatering of sludge. A dry solids content of 2-2.5% can be expected from gravity thickening.

The gravity thickening design is similar to a clarifier. Thickeners are usually circular-shaped; and the feed is carried out through a pipe to a central hood serving as distribution and still area, with a height that has no effect on compaction or compression bottom area. Except for small thickeners, static and with hopper floor, these units have a system of very strong bottom scrapers, which carry the sludge to a central tank and on which pickets are installed. These vertical bars, that move smoothly, enhance mass homogeneity and create preferential channels enabling the disposal of interstitial water and occluded gases generated by fermentation phenomena and facilitating the thickening. The supernatant liquid is collected by a perimeter weir and sent to the plant head or primary treatment.

#### (ii) Design considerations and procedures

Hydraulic loading rate and solids load are the main design parameters.

The hydraulic loading rate is based on the real flow through the unit, that is, which goes by the perimetral discharge weir(s) (outflow).

$$HLR = \frac{Q}{A} \qquad (7.3)$$

Where,

HLR= Hydraulic loading rate (m/h),

Q=sludge flow sent to the thickening unit ($m^3$/h),

A= horizontal thickener surface ($m^2$)

Solid load: This defines the required surface for an appropriate sludge thickening in the unit bottom part (compression area)

$$CS = \frac{Q.X}{A} \qquad (7.4)$$

Where,

CS = solids load (kg SS/$m^2$/h)

Q = sludge flow to the thickening unit ($m^3$/h)

X = solids concentration (mg/L)

A = horizontal thickener surface ($m^2$)

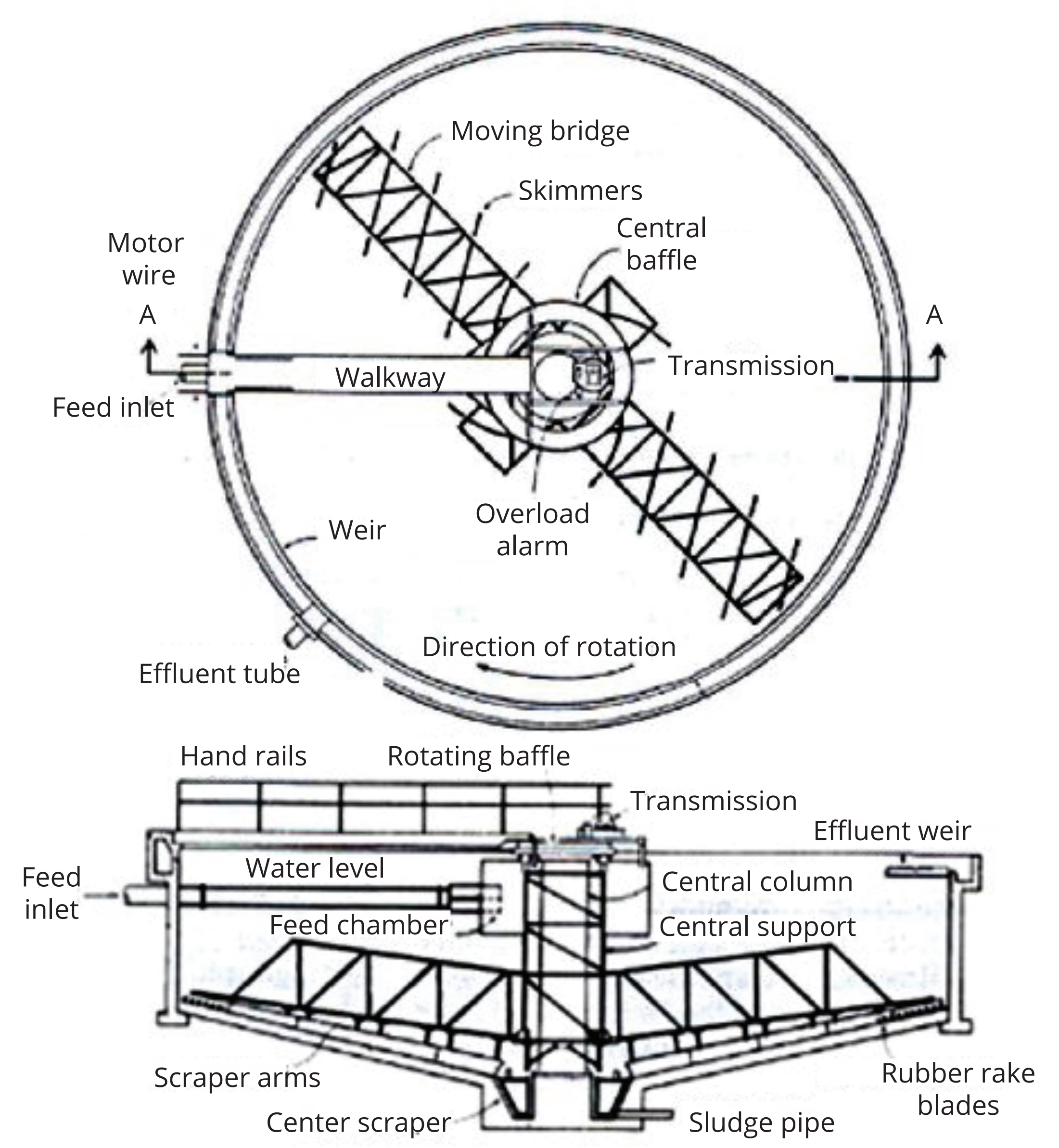



**Figure 7.27: Gravity Thickener Plan and Height**

### *7.5.5.1.2 Sludge Dewatering*

The alternatives for sludge dewatering systems are described below. Guidance for the selection of an appropriate system is given in Table 7.5.

**Table 7.5: Comparison of Sludge Dewatering Systems**

| Treatment Process | Advantages | Disadvantages | Land Requirement | Cost Per Unit or Treated Water |
|---|---|---|---|---|
| Sludge Drying Beds | Low cost | Leteral Clogging Rain can impede the drying process | High | Low |
| Solar Sludge Drying Beds | DS content of 80–85% possible | Capital Cost is slightly increased due to UV Protected Polythene | Considerably lower than conventional drying beds | High |
| Sludge Lagoons | Low cost | Rain can impede the drying process | Low | Low |
| Filter Press | DS content of 20–30% possible | High capital & O&M cost Polymer dosing is required Sand and grit can damage the belt | Low | High |
| Centrifuge | DS content of 15–20% possible | High capital and O&M cost Polymer dosing is required | Low | High |

The filtrate from the selected dewatering system should be returned to the sludge regulation tank.

### *7.5.5.1.3 Sludge Drying Beds*

Sludge drying beds are most favoured when lands are available in close proximity to the water treatment works. Areas where strong sunlight is available with average annual rainfall lower than 2,200 mm are appropriate. The filtrate of the sludge drying beds can be directed under gravity to sludge regulation tank for subsequent thickening and should not be discharged to the environment. However in areas having higher rainfall (average annual rainfall between 3000 to 6000 mm) in order to achieve higher dry solid contents solar sludge drying beds having a roof cover of UV protected polythene can be utilized.

### *7.5.5.1.4 Sludge Lagoons*

Lagoons may be the cheapest method of sludge dewatering but large land area are required compared to mechanical dewatering techniques. However, compared to conventional sludge drying beds, lagoons require considerably lesser land. Lagoons can be lined or unlined or can be provided with under drain arrangement for better dewatering requirements. Unlined earthen sludge lagoons are more effective in dealing with large volumes of sludge from higher capacity water treatment plants. However due consideration should be given to the following during planning of the water treatment plant layout to locate sludge lagoons:

- Fluctuation of groundwater table (seasonal high groundwater table should be- sufficiently below the bottom of the lagoon preferable below 2.5 m),
- Underlying soil characteristics should be investigated to see its suitability, soil- percolation rate between 25 to 150 mm/hr being preferred,
- Annual precipitation preferably to be below 3,000 mm,
- Access ramps to be provided for dumper/tractor/mini loader to collect dried-sludge from the lagoon and final disposal.

### *7.5.5.1.5 Mechanical Sludge Dewatering*

Important operational parameters to be considered in evaluation of these systems are;

- energy consumption,
- required polymer dosage and
- separation efficiency

Further, mechanical dewatering systems such as;

- Belt Filter Presses,
- Filter Presses and
- Centrifuges should operate continuously as far as possible in order to reduce usage of treated water for the cleaning operation required at the end of each operation cycle and to optimize utilization of equipment.

These equipment require

- high skilled maintenance staff
- suitable polyelectrolyte with dosing arrangement
- electrical power for the operation of the equipment

Therefore, the process is usually attractive only in large sludge dewatering facilities with incoming flow > 0.3 $m^3/s$ (25,920 $m^3/day$). As such, correct assessment of sludge generation and selection of the capacity of each unit is very important. After mechanical dewatering, the sludge is generally directed through a conveyer system into a skip or a hopper. The filtrate from mechanical dewatering facility can be directed back to the sludge regulation tank. Most mechanical dewatering equipment can achieve 15-20% DS content but the actual performance needs to be verified from the manufacturers. Mechanical sludge dewatering is only recommended for:

- major water treatment plants that generate large quantities of sludge,
- treatment plants that do not have adequate land or areas that experience average annual rainfall in excess of 3,000 mm.

### *7.5.5.1.6 Backwash Water Recovery*

Backwash recovery aims at utilizing water resources to maximum potential, to minimize energy consumption and thereby optimize production costs. The backwash recovery process should not cause any adverse impacts on the treated water quality. The possible health implications could be trace amounts of heavy metals that may be present in raw water or in water treatment chemicals as impurities, pathogenic microorganisms, algal toxins/THM which may be produced during physical/chemical processes of the treatment works or present in raw water. In this context, reuse of thickener supernatant is not recommended as about 95% of the contaminants in raw water are removed from the sedimentation process and hence may contain pathogenic organisms such as *Cryptosporidium and Giardia Lamblia which are resistant to chlorination.* It is advantageous to reuse the backwash water in order to conserve energy by minimizing the utilization of low lift pumps and to recover 2-5% of water. Backwash water recovery has many advantages. However, the introduction of backwash water recovery needs careful evaluation of the raw water quality, the proposed treatment process and the cost-benefits.

Therefore, the recommended unit processes for backwash water recovery to be incorporated in the sludge treatment stream are as follows:

- Backwashed water is first directed to backwash recovery tank (minimum two tanksü each having capacity at least to hold two backwashes) where it is allowed to settle for a selected time,
- After allowing for sedimentation, supernatant is gravity fed to backwash recirculation tank which is constructed with common wall to the backwash recovery tank to minimize construction cost,
- The gravity feeding system should comprise pontoon attachment at the surface of the inlet pipe (which needs to be covered with suitable mesh to prevent escaping of floating debris) and the bottom end supported by a hinged bend in order to facilitate rotation of the pontoon attached inlet pipe as water level drops up to the sludge thickening zone of the backwash recovery tank.

### *7.5.5.1.7 Waste from Slow Sand Filters*

During the scraping operation of the biological layer called “schmutzdecke” of the slow sand filters, the removed sand can be washed using “hydro cyclone” and reused when “re-sanding” of the slow sand filters is required. The dirty water resulting from the hydro cyclone needs to be treated appropriately in line with the disposal standards. A sludge thickener can be used if the number of filters is higher and continuous type of treatment is needed as in larger scale water treatment plants. If the treatment plant is of small scale, it may be sufficient to have a roughing filter together with natural or constructed wetland to polish further to meet the discharge standards as this type of system may be suitable for intermittent operation as generally slow sand filters need to be cleaned once in two to three months depending on the raw water quality.

### 7.5.5.2 Disposal of Sludge

The dried sludge resulting from different dewatering methods discussed above should be disposed in compliance to environmental regulations. The options available for disposal of sludge are:

(a) Land Disposal
  (i) Forest
  (ii) Land Reclamation
  (iii) Landfill
(b) Incineration
(c) Melting
(d) Brick and roof tile construction (after mixing with other constituents to obtain the desired consistency)

## REFERENCES


Eliakimu, N., R. Machunda and K. Njau (2018). "Water quality in earthen dams and potential health impacts: case of Nadosoito Dam, Tanzania." Water Practice and Technology 13(3): 712-723.

Jordan K. Lanfair, Stephen T. Schroth, Archis Ambulkar (2020). Desalination. Retrieved from: https://www.britannica.com/technology/prechlorination

*Huisman (1974).https://sswm.info/sswm-university-course/module-6-disaster-situations-planning-and-preparedness/further-resources-0/slow-sand-filtration)*

Mbwette, T.S.A and Wegelin 1984. Field Experience with HRF-SSF Systems in Treatment of Turbid Surface Waters in Tanzania. Proc. IWSA Congress, Monathis, Tunisia pp 10-12 (SS6)

https://www.slideshare.net/gowrivprabhu/water-treatmentwater-treatment

Chapter 8

# TREATMENT OF WATERS WITH SPECIAL CONTAMINANTS

In recent times, there has been an emergence of water contaminants which in the past, were either of less concern or their health impacts not known. These special contaminants are bringing new dimension in the manners in which they are supposed to be dealt with. This chapter presents special water contaminants that have emerged to be of major health concern. Treatment techniques and methods for these special contaminants have also been presented.

## 8.1 NATURAL ORGANIC MATTER

Over the past 10–20 years, the amount of natural organic matter (NOM) has increased in raw water supplies in several areas. The presence of NOM causes many problems in drinking water treatment processes, including: (i) having a negative effect on water quality by colour, taste and odour, (ii) increased requirement of coagulant and disinfectant dose, (which in turn results into increased sludge and potential harmful disinfection by-product formation), (iii) increased biological growth in the water distribution system, and (iv) increased levels of complex heavy metals and adsorbed organic pollutants.

In Tanzania, this problem of organic matter in water is more pronounced at the Igombe dam, the source of water supply for Tabora Municipality. A study on water quality monitoring at the Dam conducted between October 2009 – November 2010, revealed that colour and turbidity of raw water is very high due its very high humic content and algal blooms. These substances are mainly the result of organic matter from the surrounding dry forest.

From the conclusions of the first report (February 2, 2010), it is now known that the current treatment of water in the area is not sufficient to guarantee a good water quality in the network, mainly due to the risk of bacterial re-growth and also due to acidic pH, causing pipe corrosion. Igombe Dam water is characterized by high concentrations of organic matter. The organic matter is mainly of natural origin, that is, natural organic matter (NOM). The collected water quality data confirms the high amount of natural organic matter (NOM) in the raw water of the Igombe Dam.

The high concentration of NOM in this water may cause different problems, including excessive chlorine demand, formation of disinfection by-products

(DBPs), bad taste and odour problems and bacterial re-growth in the distribution system. The low concentration of the different nitrogen compounds (nitrate, ammonia) in the raw water of the Igombe Dam indicates moderate to little agricultural and human activities in the watershed, confirming that the source of the organic matter in the raw water is principally of natural origin. Therefore, one of the principal objectives of the future water treatment plant at Igombe will involve the reduction of NOM (measured as DOC and $UV_{254}$) through different treatment processes - principally coagulation/sedimentation and (biological) filtration.

## 8.2 ARSENIC

The occurrence of arsenic in groundwater is increasingly becoming a serious pollution problem to human health worldwide. The occurrence of such pollutants can either be geogenic or anthropogenic. Recently, elevated concentrations of arsenic (>10 μg/L) has been reported in the northern parts of Tanzania and are associated with gold mining activities within or near the Lake Victoria Basin (LVB). The geology of LVB is dominated by greenstone belts in the North-Western region up to Mpanda and Lupa Mineral fields in the Ubendian system of western Tanzania. These regions are characterized by small, medium and large scale mining, using various technologies. Several adverse human health effects are noticed among the communities living around the relevant mining areas. Based on the developed geo-statistical model using past study results on total arsenic in 9 districts, a spatial trend ($R^2$ =0.19) in the east-west transect was also noticed. In addition, a spatial trend ($R^2$ =0.009) was observed along the north-south transect. The reported study results are a potential to watch in engaging in improved drinking water supplies targeting groundwater sources.

Thus, every water treatment must consider removing this pollutant. One way of completely removing this pollutant is to consider a number of different units and modules such as to include oxidation, coagulation, flocculation, and membrane techniques[11].

## 8.3 RADIOACTIVE CONTAMINANTS

Radioactive contamination, also called radiological contamination, is the deposition or presence of radioactive substances on surfaces or within solids, liquids or gases, where their presence is unintended or undesirable. Such contamination presents a health hazard because of the radioactive decay of the contaminants, which produces such harmful effects as ionizing radiation (namely α, β, and γ rays) and free neutrons. The sources of radioactive pollution can be classified into two groups: natural and man-made. In Tanzania such radioactive materials have been reported in geographical areas of Chemba and Bahi.

[11] Nina Ricci Nicomel, Karen Leus, [...], and Gijs Du Laing. Technologies for Arsenic Removal from Water: Current Status and Future Perspectives. Int. J Environ Res Public Health. 2016. 13(1):62.

Water sources with radioactive contamination should not be exploited by any water authority or project.

## 8.4 FLUORIDE

In Tanzania, fluorides are distributed in the regions of Arusha, Moshi, Singida, and Shinyanga, with a severely affected area being on the foothills of Mount Meru and Kilimanjaro. De-fluoridation of water is necessary when fluoride concentration is higher than the acceptable limits as per Tanzania Standards. Excessive fluorides in drinking water may cause mottling of teeth or dental fluorosis, a condition resulting in the coloration of the tooth enamel, with chipping of the teeth in severe cases, particularly in children. With even higher levels of fluorides, there are cases of fluorosis of the bony structure. The chief sources of fluorides in nature are fluorapatite (phosphate rock), fluorspar, crylite and igneous rocks containing fluorosilicates.

In deciding whether or not to include de-fluoridation in a water supply scheme, the designer should consider both the number of potential consumers, alternative sources of water, financial consequences both capital and running costs of the de-fluoridation plant and whether or not there is a possibility to dilute the water containing fluoride in the effort to reduce fluoride concentration.

## 8.5 TOXIC CYANOBACTERIA IN DRINKING WATER

Concern about the effects of cyanobacteria on human health has also grown in many countries in recent years for a variety of reasons. These include cases of poisoning attributed to toxic cyanobacteria and awareness of contamination of water sources (especially lakes) resulting in increased cyanobacterial growth. Cyanobacteria also continue to attract attention in part because of well-publicised incidents of animal poisoning. The problems associated with cyanobacteria are likely to increase in areas experiencing population growth with a lack of concomitant sewage treatment and in regions with agricultural practices causing nutrient losses to water bodies through over-fertilization and erosion. This problem shall also increase in areas depending on dams for water provision especially where the dams are shared with livestock.

Outbreaks of human poisoning attributed to toxic cyanobacteria have been reported in Australia, following the exposure of individuals to contaminated drinking water, and in the UK, where army recruits were exposed to the danger through swimming and/or canoeing. However, the only known human fatalities associated with cyanobacteria and their toxins occurred in Caruaru, Brazil, where exposure through renal dialysis led to the death of over 50 patients. Fortunately, such severe effects on human health appear to be rare, but little is known of the scale and nature of either long-term effects (such as growth of tumours and liver damage) or milder short-term effects, such as contact irritation. Eutrophication is the main contributor to the occurrence of cyanobacteria.

In Tanzania, eutrophication is one of the most common water quality pollution challenges facing Lake Victoria today. Despite numerous and intensive research efforts to alleviate the problem, the present extent and magnitude of the problem is unprecedented (Myanza *et al.,* 2014).High phycocyanin (PC) pigment values (19.9 to 495 μg/l) unique to cyanobacteria have also been reported. The PC values are well above the WHO alert level of 30 μg/l in earthen dams in Arusha (Eliakimu, Machunda et al. 2018) and is associated with a variety of toxins affecting humans and animals. These dams are shared between animals (wild and domesticated) and humans.

Human influences such as population growth, urbanization and agriculture contribute to the cause of algal blooms which disrupt natural food webs. Furthermore, some algae are toxic e.g. blue-green algae, also known as cyanobacteria. Algae are responsible for water odours and tastes, as well as toxins, though attempts to use odour and taste as potential surrogates for the presence of toxins are inconclusive. Because of concern about the potential detrimental water quality impacts on human and animal health, eutrophication necessitates active water source control and monitoring of water quality. Eutrophic water increases the costs of water treatment and water supply to consumers. Climate change amplifies eutrophication as rising temperatures favour cyanobacteria over other phytoplankton species (e.g. diatoms). Warming of surface waters also enhances vertical stratification of lakes, again favouring the growth of cyanobacteria.

Eutrophication is the enhancement of the natural process of biological production in rivers, lakes and reservoirs, caused by increases in levels of nutrients, usually through phosphorus and nitrogen compounds. Eutrophication can result in visible cyanobacterial or algal blooms, surface scums, floating plant mats and benthic macrophyte aggregations. The decay of this organic matter may lead to the depletion of dissolved oxygen in the water, which in turn can cause secondary problems such as fish mortality from lack of oxygen and liberation of toxic substances or phosphates that were previously bound to oxidized sediments. Phosphates released from sediments accelerate eutrophication, thus closing a positive feedback cycle. Some lakes are naturally eutrophic but, in many the excess nutrient input is of anthropogenic origin, resulting from municipal wastewater discharges or run-off from fertilizers and manure spread on agricultural areas. Losses of nutrients due to erosion and run-off from soils may be low in relation to agricultural input and yet high in relation to the eutrophication they cause, since even low concentrations of phosphorus of less than 0.1 $mgl^{-1}$ are sufficient to induce a cyanobacterial bloom.

**Design considerations and procedures for cyanobacterial removal from water sources**

It is important to consider the removal of cyanobacteria when designing a water treatment plant. The design consideration for cyanobacterial removal should be centred more in lakes and water reservoirs. The areas of concern should be those which will be using Lake Victoria, various types of dams where water is likely to be polluted by organic matter and nutrients and reservoirs as their source of water supply. A combination of water treatment units and modules as presented in the preceding section is recommended for the effective and efficiency toxic cyanobacterial removal.

For dams, provision should be included in the design to limit intrusion of animals into the water for domestic purposes by providing cattle troughs where animals can access drinking water without polluting other areas of the resource.

## 8.6 AVAILABLE METHODS FOR THE REMOVAL OF SPECIAL WATER CONTAMINANTS

The following methods may be considered for attaining ensuring that water is free from contaminants.

- Desalination
- Additive methods
- Adsorption methods
- Reverse Osmosis
- Membrane filtration
- Capacitive De-Ionization (CDI)

### 8.6.1 Desalination

Desalination effectively removes all dissolved impurities from water. This can be accomplished in one of several ways, such as by freezing, by distillation, by electrolysis, Capacitive Deionization (CDI) or by reverse osmosis (RO). There are some research efforts to use biomass[12] based materials for de-fluoridation. The cost of desalination is high although the costs of reverse osmosis have fallen considerably in recent years. Nevertheless, this method is more appropriate in dealing with brackish or sea water although it should not be entirely ruled out where there a few, if any, alternative sources and there is a good supply of electricity.

### 8.6.2 Additive Method

In this method, one or more chemicals are added to water. The fluoride is then absorbed and both the additive and the fluoride are consequently removed by using conventional treatment processes such as sedimentation and filtration. A wide variety of materials have been tried for the purpose including lime, magnesium sulphate, magnesium oxide, calcium phosphate, aluminium sulphate, various natural earths, bauxite, sodium silicate and sodium aluminate.

[12] Hezron T. Mwakabona, Mateso Said, Revocatus L. Machunda and Karoli N. Njau, (2015). Plant biomasses for defluoridation appropriateness: Unlocking their potentials. Research Journal in Engineering and Applied Sciences 3(3) 167-174.

Excessive lime treatment for softening affect removal of fluoride is used due to its absorption by the magnesium hydroxide floc. However, sizeable fluoride removal is possible only when magnesium is present in large quantities which may not always be the case, and magnesium may have to be supplemented in the form of salts.

The initial cost and actual cost of chemicals is very high and the resultant sludge is environmentally difficult to dispose of.

### 8.6.3 Adsorption Methods

Adsorption methods employ a bed or filter of generally insoluble material through which the water is allowed to percolate periodically. As the filter becomes saturated with the fluoride 'removed, the absorptive media is either replaced or appropriately regenerated.

The materials used have included charred bone, activated alumina, activated carbon, tri-calcium phosphate, natural and synthetic ion exchange materials and aluminium sulphate. Studies elsewhere have revealed that activated carbon has a good capacity to remove fluoride where the concentration is less than 10 mg/l. and the water is low in salinity. An activated carbon for fluoride removal has been developed in India by carbonising paddy husk or saw dust, digesting under pressure with alkali and quenching it in a 2% alum solution. The spent material can be regenerated by soaking it in a 2% alum solution. Also a granular ion exchange material, 'Defluoron 2', which is coal operating on the aluminium cycle has been developed in India.

The bone char method developed and promoted by the Ngurdo to De-fluoridation Research Station (NDRS) of the MOW has been widely applied for domestic and institutional point of de-fluoridation. In recent times Reverse Osmosis (RO) has been applied for desalination at Gairo small town, in Morogoro and in Arusha for de-fluoridation.

### 8.6.4 Capacitive Deionization (CDI)

Capacitive deionization (CDI) is a technology to de-ionize water by applying an electrical potential difference over two electrodes, which are often made of porous carbon. Anions, that is ions with a negative charge, are removed from the water and are stored in the positively polarized electrode. Likewise, cations (positive charge) are stored in the cathode, which is the negatively polarized electrode. Today, CDI is mainly used to remove ion materials such as Fluoride, Nitrate, Lime, and salinity water. Other technologies for the deionization of water are distillation, reverse osmosis and electro-dialysis. Compared to reverse osmosis and distillation, CDI is an energy-efficient technology for removal of ions.

A Korean company has produced CDI which remove fluoride and dissolved salinity in water without chemicals or clogging membranes. This technology is called MCDI and it removes ionized materials in water by using state of the art electrodes running with very small amount of electricity of just 1.5 volts. (Figure 8.1) Its operational cost is lower than any other known de-fluoridation and desalination technologies.

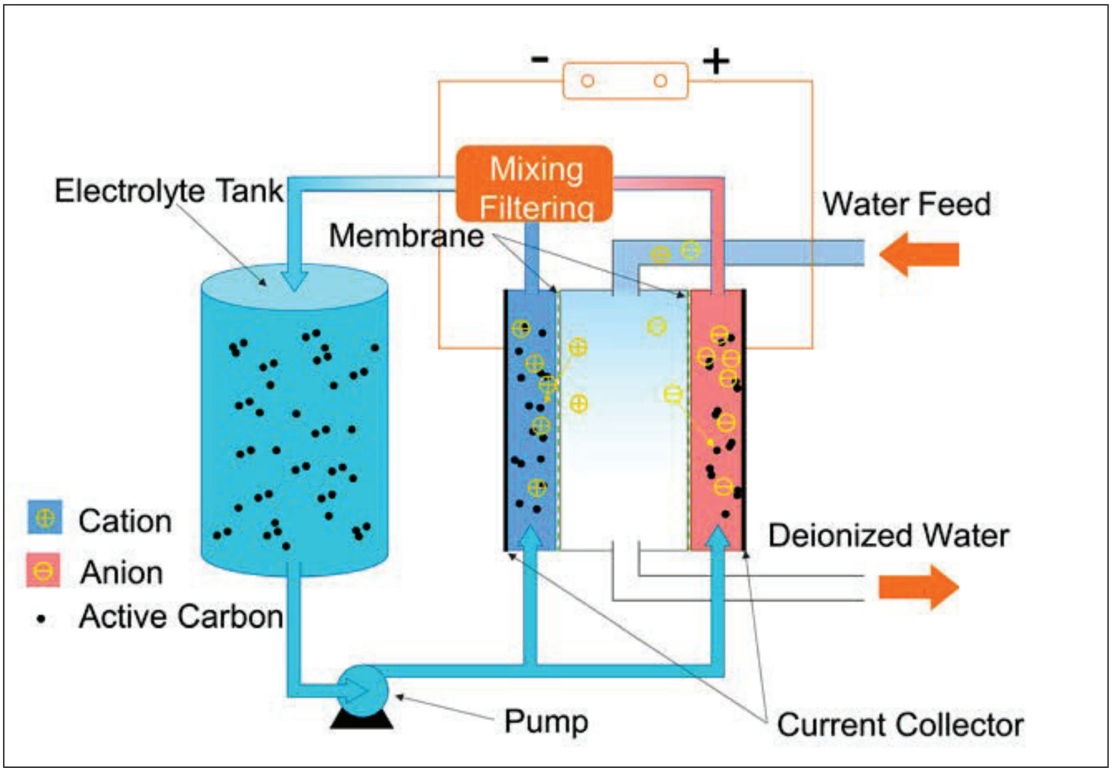



**Figure 8.1: Configuration of Capacitive Deionization**[13]

## 8.6.5 Membrane Filtration

Membrane filtration is a process of physical separation of solid particles that uses a semi-permeable membrane to remove suspended solids from a liquid stream. Microfiltration (MF), Ultrafiltration (UF), Nanofiltration (NF) and Reverse Osmosis (RO) are all examples of various pressure-driven membrane filtration techniques that are used in the treatment of drinking water and sometimes in recovery of wastewater. Membrane filters are semi-permeable materials that allow various particle sizes to either flow through or to be trapped. These filters can remove fine particles that are smaller than 2 μm as opposed to porous media filters that can remove particles that range from 1 – 2 μm (SAMCO, 2020). Membrane Filters can remove certain ions and particles from water. In this section, further discussions will concentrate on NF, UF and MF.

[13] https://www.google.com/url?sa=i&url=https%3A%2F%2Fwww.wikiwand.com%2Fen%2FCapacitive_deionization&psig

*Nano-filtration* (NF) is one of the most important recent developments in drinking water treatment as well as in process industries. NF shows performance characteristics that fall somewhere in between that of UF and RO membranes (Agboola *et al*, 2014). NF is capable of removing finer contaminants than MF and UF. In comparison to RO, NF entails slightly coarser filtration than RO and can remove particles as small as 0.002 – 0.005 µm diameter including pesticide components and organic micro-molecular particles. NF can remove larger divalent ions like calcium sulphate but it allows sodium chloride to pass through. They can remove contaminants based on their particle size as well as their electrical charge. NF can remove the following: bacteria, calcium, colloidal particles, fluoride, iron, manganese, organic materials, salt, viruses, hardness, heavy metals, nitrates, sulphates and total dissolved solids. In water and wastewater treatment, NF is applied to produce potable drinking water, to soften water, to remove nitrates, to remove pesticides from surface or groundwater and to demineralise valuable by-products like metals from wastewater. The design of the NF is based on the flux which is the volume of water passing through a membrane in a given time (litres/$cm^2$). The designs are given by the manufacturers but are dependent on the water quality, temperature and the salinity. Another design parameter is the flow rate as given by the manufacturers in (litres per minute/$cm^2$) [10-14 GPM/$ft^2$]. NF are recommended for filtration of highland streams of fresh groundwater that may be hard.

*Microfiltration* (MF) and *Ultrafiltration* (UF) are membrane filtration techniques that are, in principal, similar to NF but have pore sizes that are bigger than NF and these are 0.1 - 10µm and 0.1 – 0.01µ, respectively. Hence, MF membranes have larger pore sizes and allow monovalent and multivalent ions and viruses through but block certain bacteria and suspended solids. MF/UF can remove particulates, bacteria, viruses, organic materials, certain dyes, colour, improves taste and odour. They are both used as pre-treatments for NF and RO in order to reduce the chances of RO fouling. In treatment plants that use porous media filters, these membrane filters should be located downstream of such filters.

The following are common features of the three membrane filters:

- The membrane filters are made as Hollow fibres, Plate and frame, Spiral-wound or Tubular shaped.
- The pertinent components include; a cartridge filter, membrane module, pressure pump, water supply, clean in place system (CIP) and a holding tank.
- Membranes are manufactured as composite from; cellulose acetate, polyamides, polysulfone, polyethylene terephthalate, Aluminium.

**Figure 8.2: Sketches Showing Installation of a Nanofiltration Unit**

## 8.6.6 Reverse Osmosis

Reverse osmosis is the process of forcing a solvent from a region of high solute concentration through a membrane to a region of low solute concentration by applying a pressure in excess of the osmotic pressure. It is a technology that is used to remove the majority of contaminants from water by pushing the water under pressure through a semi-permeable membrane. It occurs when the water is moved across the membrane against the concentration gradient, i.e. from lower concentration to higher concentration.

### Detail Features of RO

- The RO semi-permeable membrane has pores large enough to admit water molecules; Hardness ions such as $Ca^{2+}$ and $Mg^{2+}$ remain behind and are flushed away by excess water into a drain. The process involves the reversal of flow through a membrane from a high salinity, or concentrated, solution to the high purity, or permeate, stream on the opposite side of the membrane.
- RO takes advantage of hydrostatic pressure gradients across the membrane. Pressure is used as the driving force for the separation. The applied pressure must be in excess of the osmotic pressure of the dissolved contaminants to allow the flow across the membrane. For example, the membrane may allow the passage of water molecules, but blocks molecules of dissolved salt.
- The membrane retains unwanted molecules while the ultra-pure water continues on for use or further treatment. This process takes any unwanted molecules retained by the membrane and sweeps them away to the drain.
- The resulting soft water supply is free of hardness ions without any other ions being added. Membranes have a limited capacity, therefore, require regular replacement as recommended by their manufacturers.

### Impurities that are removed by RO

- Heavy metals in the water
- Minerals/saline
- Fluoride

### Desalination of Water by Reverse Osmosis

Desalination effectively removes all dissolved impurities from water. This can be accomplished in one of several ways, including by freezing, by distillation, by electrolysis or by reverse osmosis. The cost of desalination is high although the costs of reverse osmosis have fallen considerably in recent years. Nevertheless, this method is more appropriate in dealing with brackish or sea water although it should not be entirely ruled out where there a few, if any, alternative sources and there is a good supply of electricity. Therefore, of the several techniques available for desalination and in the absence of large quantities of low grade heat from power stations, the most promising method of desalination of both brackish and sea water remains reverse osmosis (RO) whether it be for brackish water or seawater.

Reverse osmosis is the reverse of the normal osmosis process, which is the natural movement of solvent from an area of low solute concentration, through a membrane, to an area of high solute concentration when no external pressure is applied. The membrane here is semi-permeable, meaning it allows the passage of the solvent but not of solute.

Membranes used for reverse osmosis have a dense polymer barrier layer in which separation takes place. In most cases the membrane is designed to allow only water to pass through this dense layer while preventing the passage of the solute (e.g. salt). This process requires that a high pressure be exerted on the high concentration side of the membrane, usually 2-14 bar for fresh and brackish water, and 40-70 bar for seawater, which has around a 24 bar natural osmotic pressure which must be overcome.

### Features of RO plants

- Package plants are available from small single membrane units producing 20 l/hr of potable water to large units producing 20m3/hr and greater providing there is a good and reliable electricity supply;
- a competent workforce; and
- a potential area where these can prove competitive in coastal areas to deal with seasonal peaks such as occurs in tourist beach areas and where the alternative is the duplication of pipelines just to meet those peak demands as water can be readily blended with water from a surface source.

A diagram of Reverse Osmosis process can be found at: https://softeningwater.com/4-best-methods-of-water-softening/

### The Process of Reverse Osmosis

Reverse Osmosis systems typically include a number of stages (Figure 8.2), including:

- a chlorine disinfectant stage;
- a sediment filter to trap particles including rust and calcium carbonate;
- a second filter stage often using sand/anthracite pressure filters;

- an activated carbon filter to trap organic chemicals and residual chlorine;
- one or two stages of high pressure pumps with an energy recovery turbine after the first stage;
- one or two stages of a reverse osmosis (RO) filter with thin film composite membranes;
- optionally a second carbon filter to capture those chemicals not removed by the RO membrane; and
- Disinfection of the remaining microbes.

## REFERENCES

Hezron T. Mwakabona, Mateso Said, Revocatus L. Machunda and Karoli N. Njau, (2015). Plant biomasses for de-fluoridation appropriateness: Unlocking their potentials. Research Journal in Engineering and Applied Sciences 3(3) 167-174.

https://www.google.com/url?sa=i&url=https%3A%2F%2Fwww.wikiwand.com%2Fen%2FCapacitive_deionization&psig

# Chapter 9

# DESIGN OF WATER STRUCTURES

The main components of a water project include water intakes, break pressure tanks, water points, valve chambers and storage/sedimentation tanks. The following sections describe the design procedures for various water structures.

## 9.1 SIZING AND LOCATING WATER STRUCTURES

### 9.1.1 Tanks

#### 9.1.1.1 Storage tanks

The primary purpose of water storage tank is to balance supply during peak hour demand. It is typical to have two peak times during the day, one in the morning the other in the evening when large amounts of water is collected. Water storage tanks should be positioned on higher ground relative to the supply area so as to command pressure.

The following design points should be considered as procedures when estimating the water tank volume/capacity

- Estimate tank capacity by calculating the water demand at various times of the day and comparing that to the yield of the water supply scheme,
- Establish the demand and supply patterns for a typical day during the assessment phase of the project,
- The supply yield pattern of the project depends on the design operation period for pumping systems,
- Consider the providing tanks for solar powered projects as they have limited pumping hours,
- Establish the tank volume based on the amount of water needed from the time when there is more water leaving the tank than entering the tank (demand> supply) until the time when there is more water entering the tank than leaving it (Supply > Demand),
- Size the water tank volume so that it is able to meet the deficit during these hours,
- Calculate the volume of the tank by comparing the supply with demand at incremental time periods and balance to the existing storage,

- The balance should start at zero, then calculated for each time period (iteration) by adding the surplus or deficiency to the balance of previous iteration.

This is represented below, with n representing the iteration.

Balance $_n$ = Surplus / Deficiency $_n$ + Balance $_{n-1}$ ............................................ (9.1)

The necessary tank capacity ($V_{tank}$) is then calculated as the maximum balance ($V_{max}$) minus the minimum balance ($V_{min}$) minus the final volume ($V_{final}$).

$V_{tank} = V_{max} - V_{min} - V_{final}$ ........................................................................................ (9.2)

Refer to Appendix C example of sizing of tanks.

### 9.1.1.2 Sedimentation/Settling Tanks

Settling Tanks and clarifiers should be sized based on the settling velocity of the smallest particle to be theoretically 100% removed. Settling velocity of particle may be determined using Stokes law:

$$V_t = \frac{D^2 g\left(\frac{\rho_s}{\rho} - 1\right)}{18\nu}$$ ........................................................................................ (9.3)

Where;

$V_t$ = Particle settling velocity

D = Diameter of particles (grain)

g = Gravitation acceleration

$\rho_s$ = Particle density

$\rho$ = Water density

v = kinematic viscosity of water ($m^2/s$)

Particle characteristics (diameter and density) may be determined in the laboratory through standard methods. Particle settling is governed by the condition that the settling velocity of a particle ($W_t$) should be less than the settling tank overflow rate ($V_o$). The settling tank overflow rate ($V_o$) is defined as:

Overflow rate ($V_o$) = Flow of water (Q ($m^3/s$)) /

(Surface area of settling basin (A)($m^2$))......................................................... (9.4)

Settling tanks and clarifiers may be designed as long rectangles after calculating the surface area, rectangular shapes are hydraulically more stable and easier to control for large volumes. Factors such as flow surges, wind shear, scour, and turbulence may reduce the effectiveness of particle settling. To compensate for these less than ideal conditions, it is recommended to double the area determined from theoretical calculations. It is also important to equalize the water flow distribution at each point across the section of the basin. Poor inlet and outlet designs can produce extremely poor flow characteristics for sedimentation.

Sedimentation efficiency does not depend on the tank depth. If the forward velocity is low enough so that the settled material does not re-suspend from the tank floor, the area is still the main parameter when designing a settling basin or clarifier, taking care that the depth is not too low.

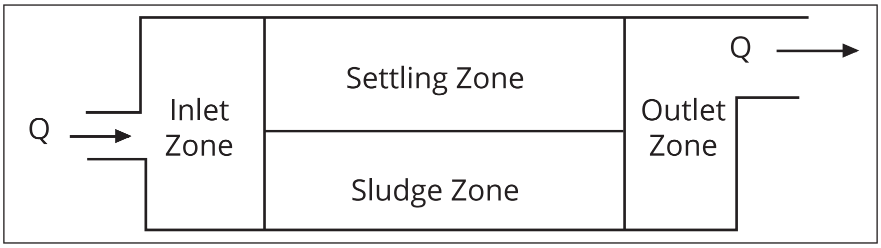



**Figure 9.1: The Four Functional Zones of a Continuous Flow Settling Basin**

### 9.1.1.3 Break Pressure Tanks

Break pressure tank is a structure that is located between a water reservoir and supply point with the aim of reducing the pressure in the system to zero (atmospheric pressure). Conventional break pressure tank is constructed of concrete in rectangular shape with the depth of the tank about 1.2m. The design criteria for the break pressure tank is to find such that the minimum diameter of the outlet pipe can convey the design flow without causing the overflow in the tank.

Example:

Consider:

Free flow in the tank at depth $d$ conveyed to outlet pipe at velocity $V_2$:

$$V_2 = \sqrt{2gd} \quad \text{.......} \quad (9.5)$$

Continuity equation to control overflow from the tank:

$$A_1V_1 = A_2V_2 \quad \text{.......} \quad (9.6)$$

The minimum area of the outlet pipe is calculated as:

$$A_2 = \frac{A_1V_1}{\sqrt{2gd}} \text{ or } A_2 = \frac{Q}{\sqrt{2gd}} \quad \text{.......} \quad (9.7)$$

Where,

$A_1$ = Area of inflow pipe,

$V_1$ = Inflow velocity equals to the design velocity,

$g$ = gravitation force,

$d$ = depth of the tank.

Choose the dimension of the tank at *d* values ranging from 1.0 m -1.5 m, length and width of the tank between 1 m to 3 m. The sizing of break pressure tank may be calculated based on the following specifications:

**Table 9.1: Standard Break Pressure Tank of 5 l/s – 25 l/s**

| Length (L) in m | Width (W) in m | Height (H) in m | Volume in $m^3$ | Retention time in min | Flows l/s | Minimum diameter of outlet pipe (mm) |
|---|---|---|---|---|---|---|
| 1.0 | 0.6 | 1.3 | 0.78 | 2.6 | 0 - 5 | 50 |
| 1.4 | 0.8 | 1.3 | 1.456 | 2.43 | 5.1 - 10 | 60 |
| 1.6 | 0.9 | 1.3 | 1.872 | 2.08 | 10.1 - 15 | 65 |
| 1.8 | 1.0 | 1.3 | 2.34 | 1.95 | 15.1 - 20 | 75 |
| 1.8 | 1.2 | 1.3 | 2.808 | 1.97 | 20.1 - 25 | 90 |

## 9.1.2 Water Intakes

This is a structure built in the body of water to draw water from the water source. The source may be a canal, river, spring, lake or dam usually built as an integral part of the source. The intake consists of an opening, a strainer or grating through which water enters and conduit conveying the water usually by gravity to a well. These structures are masonry or concrete structures and provide relatively clean water, free from pollution by sand and objectionable floating material.

### (a) Function of Intakes

The main function of an intake is to provide the highest quality of water from the source and to protect pipes and pumps from being damaged or clogged by wave action, floating objects/debris and submerged marine objects or creatures.

### (b) Consideration for locating water intake structures

The following general considerations should be taken while designing and locating water intake structures:

(i) The source of supply should not have wide fluctuation in water level,
(ii) The site selected should be able to admit water even under lowest flow conditions in the river. Generally, it is preferred that intake should be sufficiently below the shoreline,
(iii) Proximity of the intake structure to the treatment plant,
(iv) Good foundation, away from navigation requirements, safe from effect of floods, storms and scouring in the bottom,
(v) It should be placed at a location that is free from pollution. It is better to provide the intake at a location reasonably far from settlements so that the water is not contaminated,
(vi) Away from locations with high frequency of floating materials such as ice, vegetation and logging,

(vii) Intake capacity must be large enough to meet the requirement of design discharge,
(viii) It should not interfere with river traffic, if any.
(ix) It should be located where good foundation conditions are available,
(x) It should be located so that it admits relatively pure water free from mud, sand and pollutants. This implies protection from rapid currents,
(xi) It should be created on the erosion side of a river bend and not the deposition side.

## (c) Considerations for design of various types of water intakes

### (i) Reservoir intake

The water in a reservoir is likely to vary in quality at different levels. This feature makes it usually desirable to take water from about 1 m below the surface. Due to fluctuations in water level, it is desirable to have entry ports at various heights with gate valves. These gate valves are used to regulate water supply. When the water level goes down, a gate valve of a lower port is opened. Access to the ports is through an operating room.

### (ii) River intakes

A river intake should consist of a port (conduit) provided with a grating and a sump or gravity well. The conduit should be supported on pillars 1-2 m above the bottom to prevent entry of silt. Also it is kept 1m below the top surface to avoid entry of floating debris. Velocity should be kept less than 0.15 m/s to prevent entry of small fish. River intake structure should be constructed upstream the point of sewage disposal or industrial wastewater disposal. River intakes may need screens to exclude large floating matter. The bottom of the river intake must be sufficiently stable.

### (iii) Lake intakes

If the lake shore is inhabited, the intake should be constructed such as to minimize the danger of pollution. The intake opening should be 2.5 m or more above the bottom so that the entry of silt with water is minimized. The water entry velocity must be low to exclude floating matter, sediment, fish or ice. Entering velocity of 0.15 m/s is usually used. Offshore winds tend to stir up sediments which will be carried for long distances. So, intakes must be located at a distance of not less than 600meters from the shoreline. Such intakes are recommended to be of multiple level intakes to ensure the source can cope with climate change impacts.

### (iv) Intake conduit

Intakes located at long distances from the pumps usually deliver their water to the pump well at the shore end by gravity. This requires a large pipe or conduit so that the velocity is low. But velocity should not be too low to allow for occurrence of sedimentation. The conduit may be a submerged pipe or tunnel. A submerged pipe should be protected by burying it in a trench, surrounding it with rock or held in place with piling.

## 9.1.3 Dams

**A dam** is a structure built across a stream, a river, or an estuary to retain water. Dams are built to provide water for human consumption, for irrigating arid and semiarid lands, or for use in industrial processes.

### 9.1.3.1 Engineering Classification of Dams

Dams are classified according to their function, shape, construction materials and design. This is termed as engineering classification. Figure 9.2 presents a typical dam engineering classification:

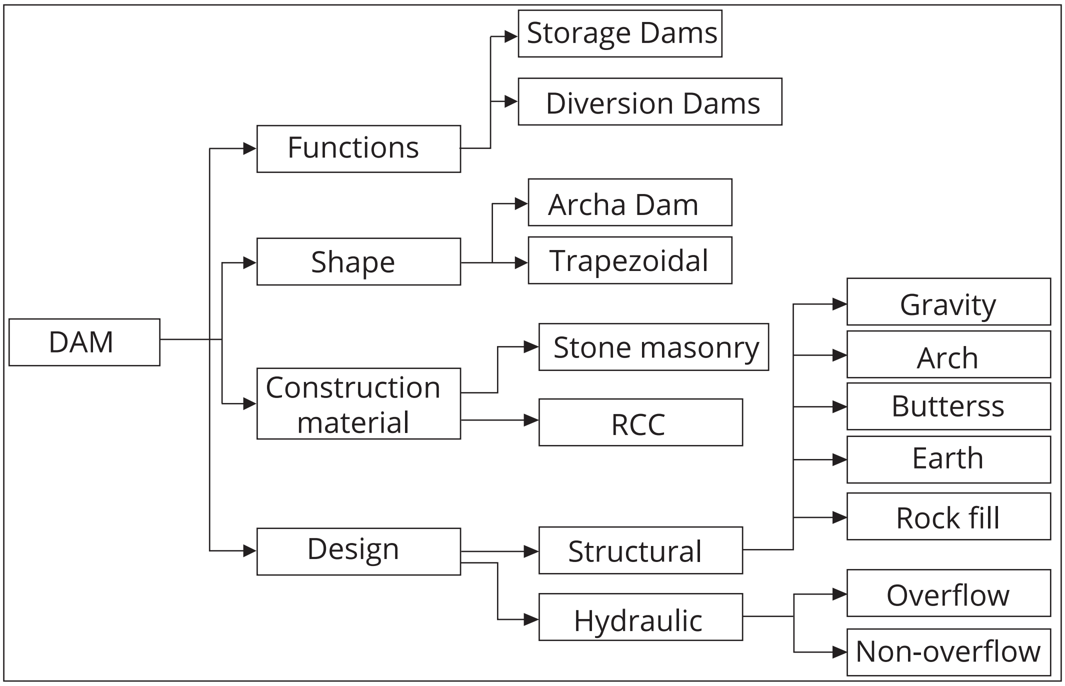



**Figure 9.2: Engineering Classification of Dams**

Under structural classification, there are several dam type namely; gravity, arch, buttress, earth fill and rock fill. The Ministry of Water has inventoried about 710 dams whose information has been recorded in a database. As per the inventory, more than 99% of these dams are earth fill and rock fill dams and the rest fall into other categories. Due to this fact, the emphasis of this manual is on earth fill and rock fill dams to guide their design, construction and Operation and Maintenance.

### 9.1.3.2 Size classification of dams

#### (a) Large dams

According to International Commission on Large Dams (ICOLD), large dams are those having capacity of 3million cubic meters or more and a height of more than 5 meter as summarized by the expression below:

5<H<15m and V>3Million cubic meters.

Where:

> H is height in meters above riverbed level to maximum crest level,
> V is storage volume in million at Maximum Operating Level=Full Supply Level in most cases.

#### (b) Small Dams

Small dams have height of less than 2.5m and the product of $H^2\sqrt{V}$ and should be less than 200 as indicated here below:

2.5<H<15m and $H^2\sqrt{V} < 200$

Note: (*) Minimum dam height can be changed to 2 or 3 m in the case of dams in residential or very populated areas.

Also, note: For flood retention dams holding no water the storage volume at crest of spillway level (design storage volume) should be used.

In Figure 9.3, the ICOLD classification (www.icold-cigb.net) of dams based on size is presented:

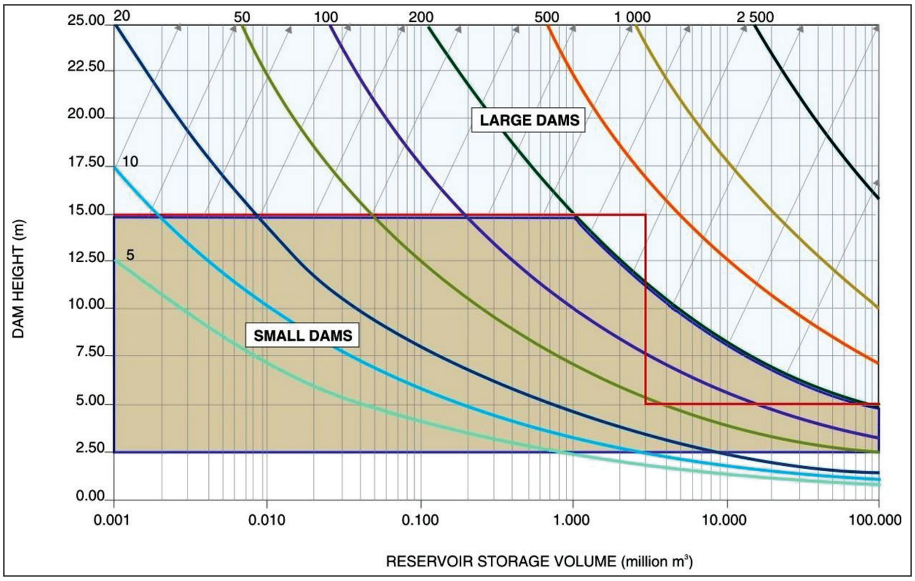



**Figure 9.3 Classification of Dams According to Size**

(Source: www.icold-cigb.net)

The focus of this manual is on large dams. Small and sand dams are covered in the rainwater harvesting guidelines prepared by the Ministry of Water and thus a designer should consult the relevant guidelines as appropriate.

### 9.1.3.3 Feasibility investigations

During the feasibility study all data and related information of the project should be collected and analysed. The following are the detailed guidelines for conducting feasibility studies and preliminary designs of a dam project. Note that, in this manual, the emphasis is on earth fill dam as the dominant dam type in the country.

## (a) Topographical Survey

Once all preliminary investigations have been made and a suitable dam axis has been selected, the next step is to carry out a detailed survey of the valley and reservoir area to allow for more accurate estimates of quantities and to provide the necessary data for appropriate design work to be undertaken. The aim of such a survey is to present, on paper, contour map of the reservoir up to and exceeding the maximum flood level, and to provide details for the location of the embankment, spillway and outlet works. From the contour map, the capacity of the reservoir can be assessed by varying dam heights.

The topographical survey works can be conducted by using different techniques depending on the site condition, size of the catchment, availability of resources and level of accuracy required. Light Detection and Ranging (LiDAR) survey, drone (Aerial), Differentials GPS (Real Time Kinematics-RTK), Totals station and ordinary levelling survey may be deployed during the topographical survey. The required contour map should range from 0.25m to 1.0m contour interval depending on the topographical characteristics of the area. Note that the contour interval is directly proportional to accuracy of the estimated volume of reservoir. The elaboration of the survey techniques used for topographical data collection and analysis are as provided here below.

### (i) Grid survey

This is a simple and straightforward but time-consuming method. It may not be appropriate if the area is heavily vegetated and/or physically inaccessible.

### (ii) Cross-sections

Cross-section surveys are taken along various lines within the river valley(s) from benchmarks previously established. Levels are observed at set intervals and outstanding features (changes of slope in particular) are also noted.

### (iii) Spot heights

This is especially suited for larger areas. A circuit of benchmarks is established and spot height observations with bearing, distance and elevation are made from each station. For smaller dams, and if a theodolite or electronic instrument is used, it may be possible to take all the readings from one station. Alternatively, reasonably accurate GPS surveys can be used to establish a network of elevation readings across the site.

**(iv) Aerial survey**

This does not have much difference from the spot heights. However, this technique deploys plane (LiDAR) and drone to conduct topographical surveys of the proposed dam catchment area. The major advantage of this technique is that it is time saving and the major disadvantage is its high cost compared to other techniques. Therefore, this technique is more suitable for large projects.

The output of the topographical map is a depth-capacity curve and can then be drawn up to provide a quick and easy method for the dam designer to choose the optimum full supply level. Figure 9.4 represents the area capacity depth curve:

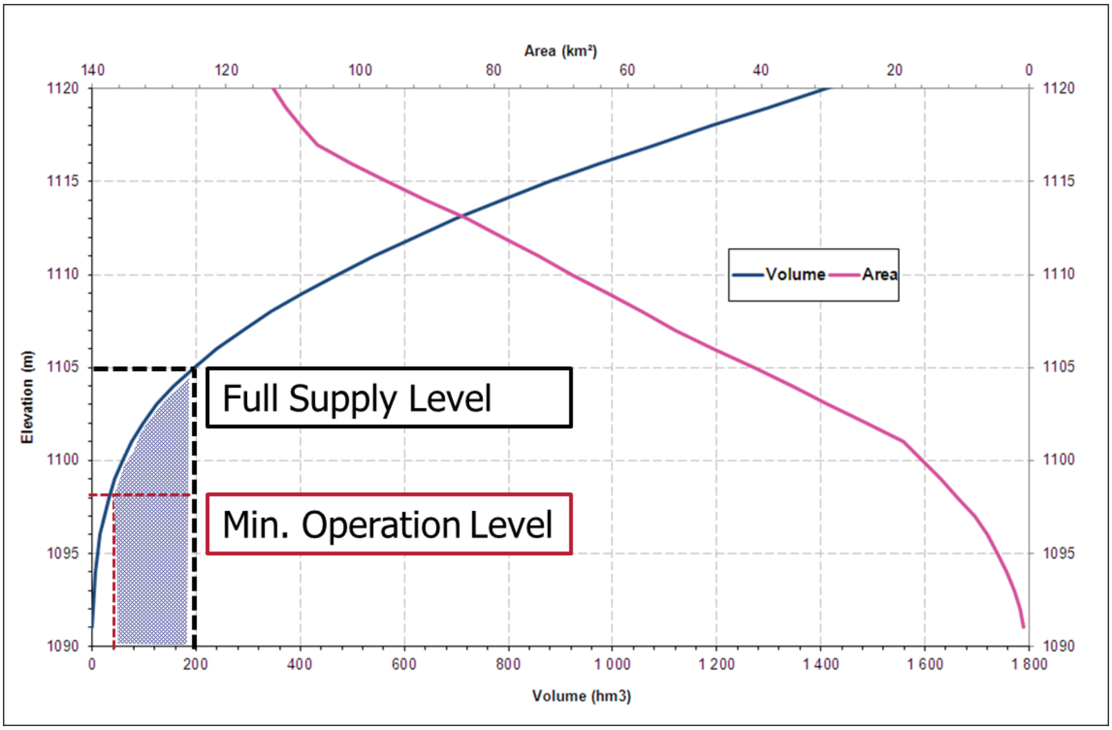



**Figure 9.4: Typical A-V-H Curve of the Reservoir**

(Source: www.icold-cigb.net)

## 9.1.3.4 Hydrological Analysis of the Water Catchment Area

Hydrological analysis is a very important aspect to be considered when designing any hydraulic structure. During a hydrological analysis of a catchment, three design issues should be considered which are:

- Water yield of the catchment,

- Design flood and
- Sediment accumulation in the reservoir.

In Tanzania, some of the rivers are un-gauged which calls for the use of empirical formulae in the estimation of reservoir inflows and spillway capacity of a dam. In the absence of direct measured flows of rivers, the catchment yield is estimated based on rainfall run off models.

**Steps to be followed in conducting hydrological analysis of a catchment**

**Step 1: Determine the amount of data required for the analysis**

Rainfall data is required for the probability rainfall analysis, flood analysis and water availability analysis. For this reason, existing rainfall data from several sources have to be collected and reviewed, including: daily rainfall, monthly rainfall, monthly discharges, daily mean discharges and other hydro-meteorological data which are used as part of the input   rainfall runoff modeling.

**Step 2: Assessment of Watershed Rainfall Records and gap filling of rainfall data**

In watershed work it is often necessary to know the average depth of rainfall over an area. The rainfall data used includes daily amounts as measured at rain gauges. Rainfall data is required for probability rainfall analysis, flood analysis and water availability analysis. Missing data may be interpolated using several methods and in this manual Inverse Distance Squared Method is suggested to be used and compared with other methods available in different references as indicated below.

$$P_x = \frac{\sum_{i=1}^{n}\left[\left(\frac{1}{D_i^2}\right)P_i\right]}{\sum_{i=1}^{n}\left[\frac{1}{D_i^2}\right]} \quad \text{.........................................................(9.8)}$$

Where,

$P_x$ = estimate of rainfall for the un-gauged station

$P_i$ = rainfall values of rain gauges used for estimation

$D_i$ = distance from each location the point being estimated

N = No. of surrounding stations

In this method, weights for each sample are inversely proportional to its distance from the point being estimated (Lam, 1983)

**Step 3: Estimating Average rainfall depth**

Beside other methods, Thiessen is user friendly for estimating watershed Average rainfall depths using a rain gauge network. In the Thiessen method, the watershed area is divided in sub areas, using rain gauges as hubs of polygons. This method and other related approaches are explained in hydrological literature.

**Step 4: Assessment of Runoff flow records**

Daily river flow records collected within the catchment area and contributing to the proposed reservoir should be used for runoff estimation.

**Step 5: Rating curve**

In order to transform the hydrometric levels into flow values, it is necessary to process a rating curve for each of the stations being considered. The calculation of the rating curve consists of estimating the parameters of a fitting function on known pairs of level-flow values. The fitting function is specifically chosen so that it can mathematically adapt to the experimental points. The look up function in spread sheet should be deployed to estimate the discharge from the known pair of level-flow. The following is the basic equation used to estimate the discharge from water level data measured in the river.

$H \rightarrow Q \qquad Q = a(H - b)^c$ ........................................................................ (9.9)

## 9.1.3.5 Geotechnical Investigation

### (a) The purpose of geotechnical investigation

The following are the purposes of geotechnical investigation for dam design and construction:

(i) To evaluate the parameters of soil/rock at the proposed site,
(ii) To assess the engineering parameters and to estimate the safe bearing capacity of dam foundation,
(iii) Drilling borehole in order to know the stratification and conducting necessary field tests and to collect samples for laboratory testing,
(iv) Testing soil samples in the laboratory to determine the physical and engineering properties of the collected samples, and
(v) Analyzing all field and laboratory data to evaluate safe bearing capacity for given foundation sizes and necessary recommendations for foundation design and construction materials.

### (b) In-situ Tests

The level of geotechnical investigation depends much on the size of the dam project, surface geological characteristics, topographical and geomorphological characteristics of the site. The geotechnical investigation mainly is subdivided into main three categories.

To obtain the geotechnical properties of the foundations of major structures including dam, spillway and weir, the in-situ tests are carried out in the boreholes and trench pit field test including standard penetration test and plate load test in soil and weathered zone dilatometer test to identify strength a different stratification of the geological formation, as well aswater pressure test in the bedrock (Hydraulic Conductivity of the Geological formation).

**Investigation of the underlying geological formation of the dam site**

In order to analyze the formation of the underlying rock of the dam site, different techniques are deployed. For small scale investigation which is conducted in soft formation, augering is used while in hard formations deployment of heavy equipment is highly recommended through coring. The core samples are analyzed to determine faults and stratification of the underlying geology of the dam and reservoir sites.

**Soil Sampling for Laboratory Test**

Soil sampling is conducted in order to determine mechanical and chemical properties of the geological formation and construction materials collected from boreholes. Recommended soil tests for design of the embankment dam are detailed on section 12.4.4.

## 9.1.4 Design of Dams

### (a) Design (Spillway Dimensioning) Flood by rainfall runoff model

The aim is to evaluate the extreme floods to be adopted for the design of a dam and the pertaining hydraulic structures. More in detail, the hydrological elaborations estimated are expected to satisfactorily assess the flood associated to several return periods. Two methodologies may be adopted for the hydrological study for the purpose of constructing hydraulic structure such as dam namely:

- Probable rainfall,
- Rainfall-runoff model.

Different distributions, including two parameters log-normal, three parameters log-normal, Gumbel and Log Pearson type-III have to be compared. It is noteworthy that since the historical runoff time sets are quite limited in some catchments, the prediction of the extreme flood with high return periods with statistical methods can lead to results on significant uncertainties.

The transformation of rainfall into runoff to estimate the peak flood and the entire hydrograph at the section of interest should be developed. The hydrological models can allow the estimating of the Probable Maximum Flood (PMF) i.e. the greatest flood physically possible over the catchment of interest.

Probable rainfall by frequency is calculated by statistical analysis on annual extreme series data. The daily maximum rainfall according to various return periods through two parameters log-normal, three parameters log-normal, Gumbel and Log Pearson type-III distribution.More information on statistical approaches can be found in appendix J.

The return period (sometimes called the recurrence interval) is often specified rather than the exceedence probability. For example, the annual maximum flood flow exceeded with a 1 percent probability in any year, or chance of 1 in 100, is called the 100-year flood. In general, $X_p$ in the T-year flood is computed as follows

T = 1/ (1-p) .......................................................................................................(9.10)

After geotechnical investigation and soil testing, hydrological analysis, sediment analysis, detailed topographical survey and Environmental and Social Impact Assessment the following stage is to conduct a detailed design and cost estimate of the project. The cost of the project should form a basis for economic analysis of the entire project. During economic analysis of the project the cash flow is prepared which defines the payback period of the project. This chapter is not prepared to guide the users on project economic analysis since the matter is covered in other chapters of the manual.

The design section focuses on earth fill and rock fill dams and other types of dams may be designed using international guidelines.

## (b) Reservoirs, Dams and their Spillways

*Reservoirs*

*A reservoir is required when the minimum daily design flow at the intake site plus any* minimum compensation release requirement is less than the average daily demand. It is a basin constructed in a valley of the stream. For storage of water, weirs may be constructed if the difference is small but if the difference is appreciable, an earthen dam, masonry, rock fill, concrete dam may be constructed across the stream for storage

*Site Selection*

Site selection criteria have been defined in the pre-feasibility study and more elaborated in feasibility study section. Therefore, the design section deploys the information gathered from other studies of the project and are highlighted below.

The site for an upstream or mainstream Reservoir and its Dam should be selected such that it may command the entire area without pumping.

The following main features should be considered for the selection of a site of such an impounding reservoir:

(i) It should be on impermeable strata, i.e. permeability coefficient (k) should be less than $10^{-7}$ m/sec.

(ii) A narrow opening in the valley should be selected to reduce the length of the dam.

(iii) Objectionable salts and minerals soluble in water should be absent from the reservoir site so that the water is of acceptable quality.

(iv) The land to be submerged should be relatively unoccupied and cheap, i.e. involve least compensation and observe laws of environment conservation.

(v) The land should be free of marshy layers and vegetation which could affect the colour, odour and taste of water.

(vi) The slopes of the basin should be steep so to reduce the surface area per unit of volume in such a way that both undesirable shallow water and surface evaporation can be minimized.

(vii) The valley of the stream in which the dam is to be constructed should be rapidly widening upstream of the dam so that it may afford the greater average volume per metre height and length of the dam.

(viii) The capacity of the reservoir should be such that it will ensure the total demand of water for the town to be delivered continuously. The capacity depends upon the climatic conditions of locality and the period of dry weather.

(ix) The dam site should not be located over a fault line parallel to the stream bed.

(x) The storage capacity in regions of dry climate, long droughts and infrequent rains will be more than that of the regions of moist climate and of frequent rains, and an initial estimate can be made using Hawksley's rule.

$$D = 1600/\sqrt{F} \quad \text{.......} \quad (9.11)$$

Where:

D = the number of days for which the supply is to be stored,

F = the mean annual rainfall of three consecutive dry years expressed in cm.

(xi) If the stream brings large quantities of silt, debris, etc., the storing capacity will be reduced due to considerable accumulation. It should be taken into account by providing extra storage (called dead storage) which is normally 1/4- 1/5 of the total storage. The actual amount of dead storage depends on the silt load and design period adopted. As far as practicable, silting should be controlled. The sediment analysis of the reservoir should be computed based on the available information from the manual. Details of other design steps are presented in Appendix K.

## 9.1.5 Boreholes

### 9.1.5.1 Groundwater Analysis

#### (a) Geology

Geological information is of great value to every hydrogeologist, as it indicates the extent of water bearing layers and less pervious layers, both in a horizontal and in vertical direction. A simplified geological map of Tanzania is shown in Fig. 9.5, together with its key.

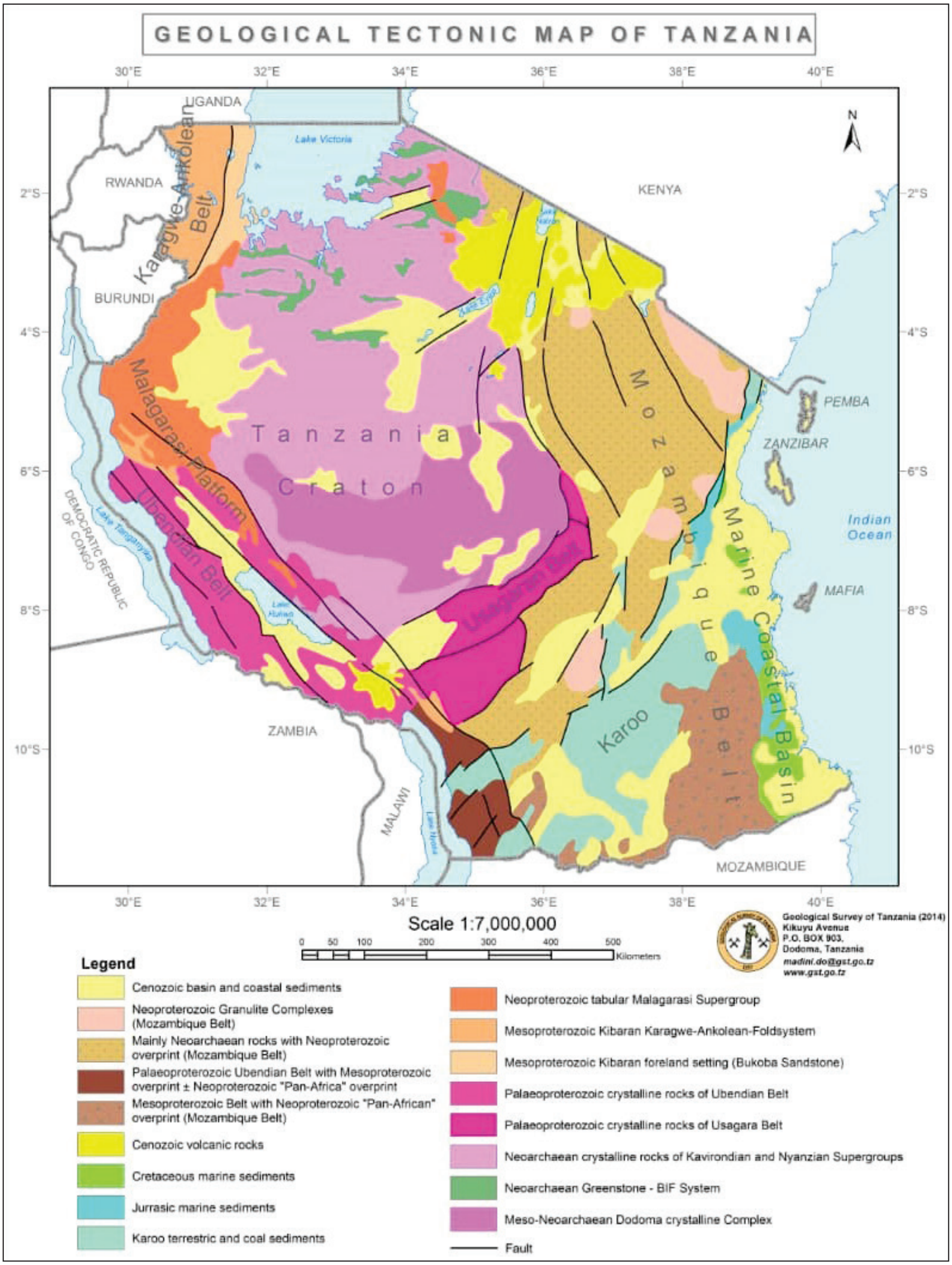



**Figure 9.5: Geological Map of Tanzania**

(Source: GST, 2014)

Hydrogeologically, rocks may be considered as falling into one of the two groups as follows:

**(i) Sedimentary rocks**

These rocks are the result of eroded sediments from source rocks. Sedimentary rocks are composed of silicate, carbonate and clay minerals. The rocks rarely occur as single units and younger beds are usually laid down upon older ones. However, the sequence may be disturbed or extensively folded and faulted.

The occurrence of groundwater in sedimentary formations is generally found in sandstones, carbonates and unconsolidated materials. The sandstone formations are rated as highly productive aquifers due to both their primary and secondary porosity and permeability. The Calcareous formations are also good aquifers due to solution process taking place at later stages (karstification). Unconsolidated formations are regarded as good aquifers due to both weathering and depositional processes resulting in the formation of sand dunes, alluvial fans, floodplains, buried river channels, etc.

**(ii) Igneous and metamorphic rocks**

Igneous rocks may be classified into two types: extrusive and intrusive. In extrusive rocks (volcanic) such as tuff, andesite, basalt, etc., water occurs in fractures and pyroclastic materials. The intrusive and metamorphic rocks termed as hard rocks are mainly granites and gneisses, respectively. They are solid and non-porous but can hold water in networks of cracks, joints, fractures or faults or along contacts between rocks of various types, as in the case of dikes and sills.

Fine grained igneous rocks, e.g. aplites can be good aquifers because they have short and narrowly spaced fractures. While coarse grained rocks like granites, have long and widely spaced fractures forming a mosaic of splinths. The storativity is reasonably good between the splinths. Basic intrusive rocks, e.g. diorites and gabbros have low and tend to be poor aquifers.

## (b) Hydrogeological considerations

The water resources study of the hydro-geological conditions of the project area will indicate the viable sites for well exploration in terms of supply capacity and water quality. Hydro-geological studies should be conducted by knowledgeable professionals or drillers, who assess available information on existing wells. Professionals should examine well data such as

- water quality,
- well yield,
- seasonal fluctuations,
- water table depth, and
- well drilling logs showing geologic layers.

A geo-resistivity survey of the areas being considered for possible well sites will indicate the depth and thickness of aquifers.

Wells can be designed and constructed in a number of ways depending on the geological condition, budget for the construction and the desired capacity of the well. The following are the types of wells that can be identified based on the construction methods employed.

### (i) Dug wells

Dug wells are holes or pits dug manually into the ground to tap the water table. The dug well may be up to 15 meters deep, with diameter usually ranging from 1 meter to 1.5 meters. The well is usually lined with concrete masonry, bricks, stones, or reinforced concrete to prevent the wall from caving in. At depths of the aquifer layer, the wall is embedded with slots; or prefabricated concrete caisson rings are installed for the passage of groundwater into the dug well. Dug wells are normally circular in shape (Figure 9.6). This type of well is sometimes capable of drawing sufficient supplies of water from shallow sources but is easily polluted by surface water.

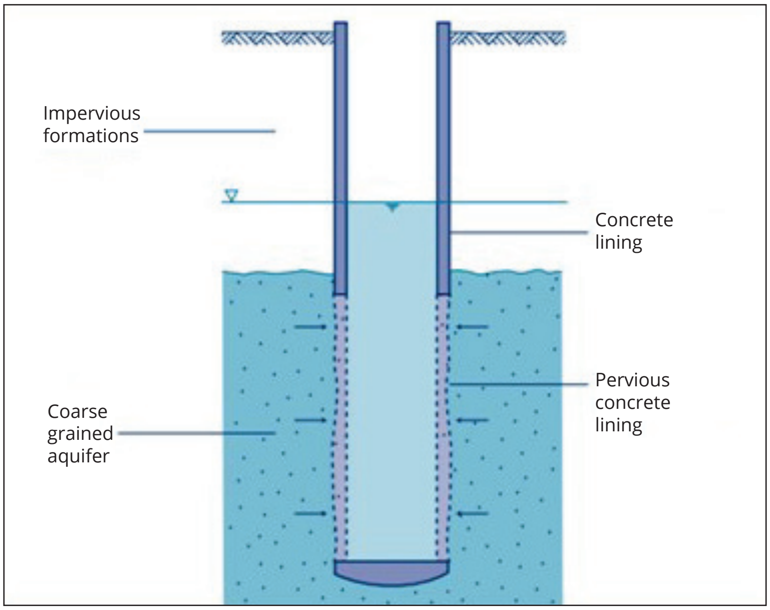



**Figure 9.6: Dug Well in Coarse Granular Material**

*(Source: SMET & WIJK (2002))*

### (ii) Driven wells

Driven wells are like dug wells, in the sense that they tap the shallow portion of the unconfined aquifers (Figure 9.7). They are easy and relatively inexpensive to construct in locations with unconsolidated formations that are relatively free of cobbles or boulders. The wells are constructed by driving to the ground an assembly of Galvanized Iron (G.I) pipe and a pointed metal tube called a "well point". The pointed end of the well point, which is the penetrating end, has screens or holes to allow the passage of water.

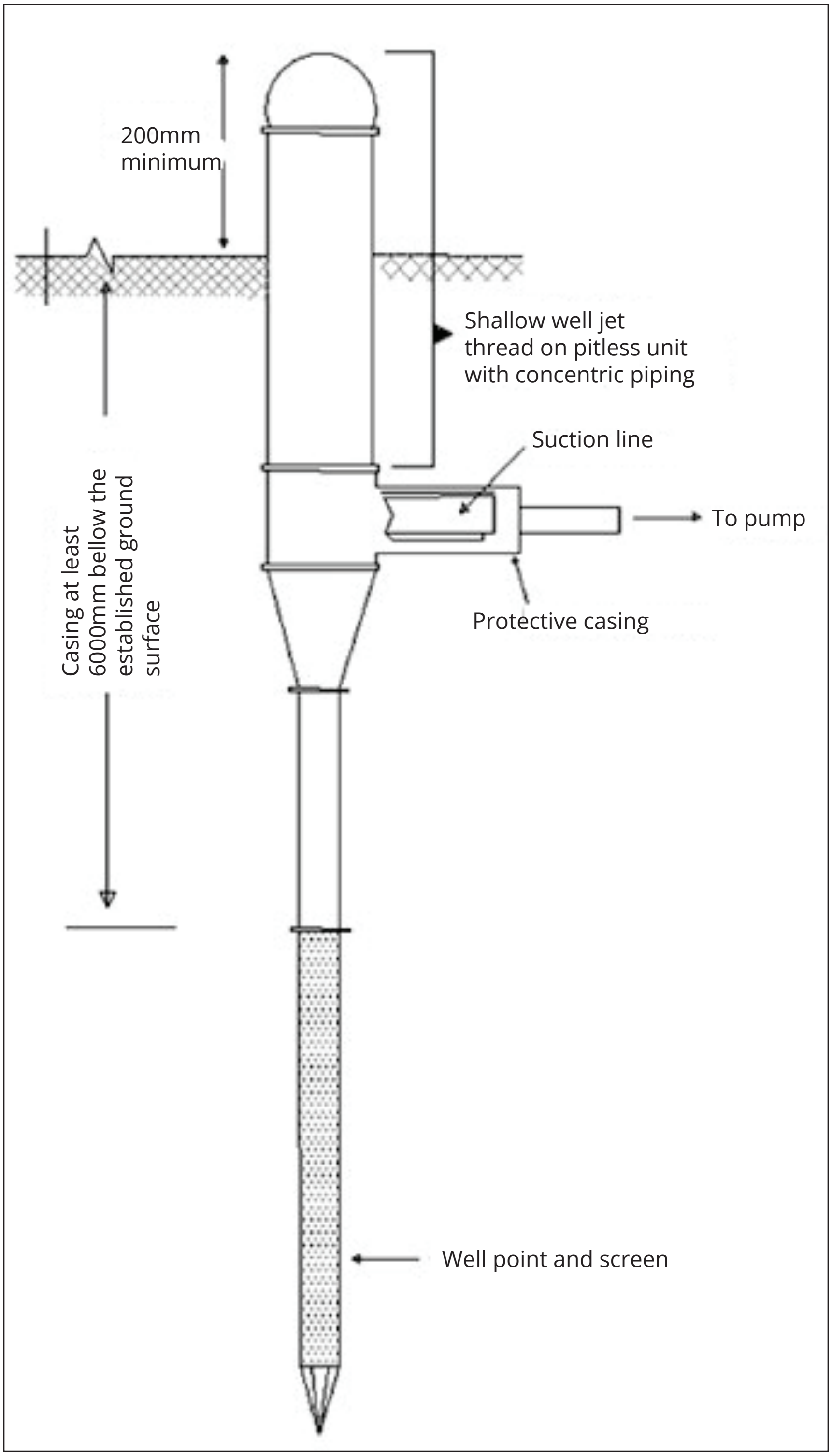



**Figure 9.7: Driven Wells/IDPH**
(Source: http://www.dph.illinois.gov)

### (iii) Bored wells

Bored wells are constructed with hand or power augers, usually into soft cohesive or non-caving formations that contain enough clay to support the boreholes. The depth of bored wells could be up to 15 metres. Bored wells are very prone to surface contamination. The well construction method is not applicable on hard consolidated materials and is not advisable on predominantly boulder formations. Figure 9.8 shows a properly bore well.

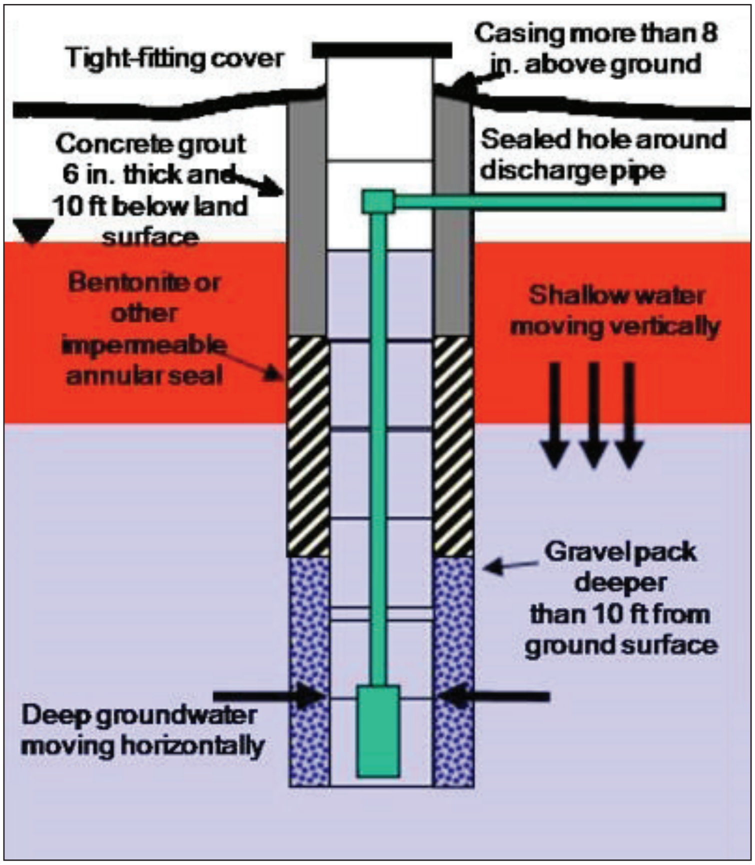



**Figure 9.8: A Properly Bored Well Construction, not to Scale**

(Source: https://water.ca.uky.edu/boredwellconstruction, Environmental and Natural resources issues)

### (iv) Deep drilled wells

Deep Wells drilled by professional drillers with the appropriate experience and equipment can extract groundwater from a much deeper level than the other types of wells. Various well drilling methods have been developed because geological conditions range from hard rock such as granite and dolomite to completely unconsolidated sediments such as alluvial sand and gravel. Particular drilling methods have become dominant in certain areas because they are most effective in penetrating the local aquifers, thus offer cost advantages.

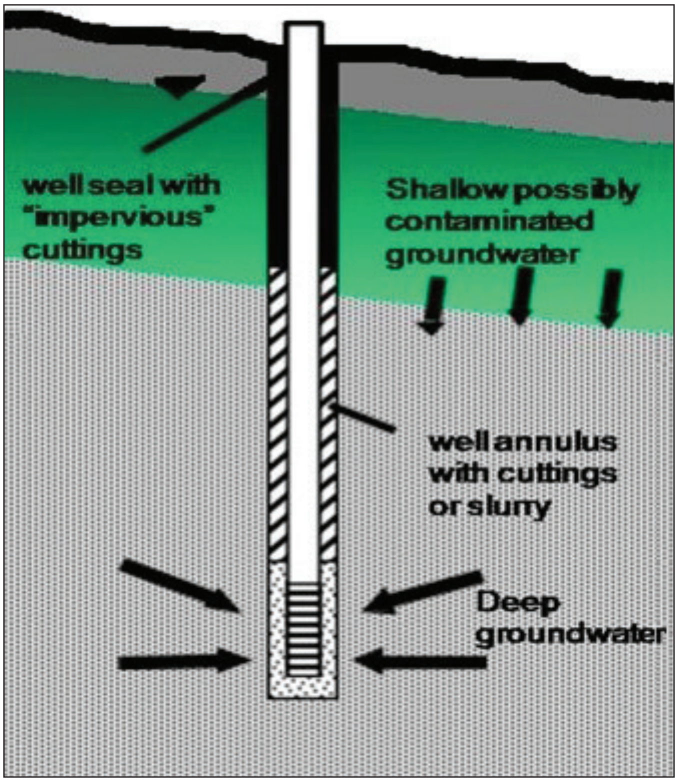



**Figure 9.9: Deep Drilled Well Construction**

(Source: http://www.water.ca.uky.edu, Environmental and Natural Resources Issues)

## 9.1.5.2 Groundwater Prospecting

### (a) Required data

Prior to carrying out hydrogeological and geophysical investigation a thorough analysis of previous work should be carried out at the survey area. The activities will include the use and interpretation of available data of the survey area. Available data can be present in the form of maps, aerial photographs, satellite imagery and drilled boreholes.

#### (i) Topographical maps

Topographical maps are used to show the physical features of the land.

#### (ii) Geological maps

This is an important tool normally used to give in some representative vertical sections an indication about the lithology of the layers present. It is of great help for the hydrogeological investigation.

#### (iii) Hydrogeological maps

These maps should show all the hydrogeological features of the area concerned.

#### (iv) Archived Satellite Imagery

Archived satellite imagery, much of which is increasingly available at little or no cost is another important source of useful information. There are other maps such as Soil Maps, Aerial Photographs as well as some others with special features which may give valuable hydrogeological information.

#### Other data

#### (v) Water quality data

Information regarding the quality of the groundwater should be collected.

#### (vi) Data on different hydrological parameters

Records on discharges of rivers and streams, others on meteorological data such as rainfall, evaporation, etc., as far as they could be of help for the hydrogeological investigations envisaged, should be collected.

#### (vii) Information on wells and springs

Information about any previous drilled boreholes is of great help in knowing groundwater potential areas. Also springs in the area should be measured and the records be kept in an archive.

#### (viii) Data on borings

Information about the composition of the ground and water obtained from samples taken from a borehole should be collected.

### (b) Reconnaissance survey

The hydrogeologists should check geological structures and extent of the catchment areas. These evidences are the key elements focus on while selecting the best site recommended for drilling.

#### (i) Geological survey

Geological survey aims at identifying the visible surface geology of the area and its surrounding, including identification of topography, physical features, soil type, vegetation and drainage, rock outcrops, strike/dip of rocks etc. Geological survey helps to provide a model of historical development of an area with respect to geological time.

#### (ii) Hydrogeological survey

Generally, the occurrence of groundwater in hard rocks i.e. igneous and metamorphic formation is largely limited to secondary features such as fractured / weathered/shear zones or faults. The potentiality of weathered zones depends on the degree and depth of saturation and associated fracturing are normally the groundwater controls. Recharge in this formation is by rain water infiltration through the superficial formation, shear or weathered rocks and fractures.

Shear/ weathered or fractured zones are the expected aquifer points in the surveyed area.

## (c) Geophysical methods

Geophysical surveys are a relatively cheaper and faster way in groundwater investigation compared to drilling of exploratory wells. This is the reason why geophysical surveys are applied in the preliminary stages of investigations. However, geophysical surveys require specialized professionals (Geophysists or Hydrogeologists) to execute the surveys and to interpret the results. Several methods are used in hydrogeology depending on the geological formation as described below:

### (i) Geo-electrical resistivity soundings

With the resistivity method normally an electrical current, generated by an artificial source, is sent through the ground by means of two current electrodes at the land surface. The resulting electrical potentials are measured with two other electrodes, also at the land surface. Since rock types have different electrical properties depending on their density, pore fluids, porosity etc., the strength of the current applied gives a measure of the apparent resistivity of the rock. By varying the distances between either the current electrodes (Schlumberger arrangement,), or both types of electrodes (Wenner arrangement) apparent resistivity at different depths and layer thicknesses are obtained. The wider the spacing distance, the deeper the layers involved in transmitting the electrical current.

The major advantage with the resistivity method as far as ground water investigations are concerned is that it gives direct indications of the presence of water as resistivity is a function of water content of the rocks. Other geophysical methods tend to provide indirect indications.

### (ii) Electromagnetic (EM) methods

This method is cheap and fast, as it does not involve extended walking with electrodes as for the case of resistivity. The methods rely on the measurement of secondary magnetic fields generated by conducting bodies in the ground when subjected to primary signal. The principle is that a time varying low frequency electromagnetic field is generated by a transmitter at the land surface, which later is transformed by an electrical conductor underground. The method is useful in detecting buried conducting bodies such as fractured fault zones, dikes etc. This method may also be used in detecting polluted groundwater or, fresh water/brackish water interface.

### (iii) Magnetic method

Magnetc profiling is used to measure the variation of the total magnetic field in relation to geology along the profiles. The magnetic field of fractures/features, dykes system and deeply weathered zone differs considerably from surrounding rocks. The changes are increase or decrease in the normal field of the area.

#### (iv) Seismic methods

The method has limited direct use in groundwater exploration. However, it is useful in determining the depth to the bed rock. The methods makes use of the elastic property of rocks. The seismic waves are generated on the ground by explosion or any other instantaneous release of energy into the earth.

## 9.1.5.3 Drilling

After the survey, the drilling work in areas that look promising follows. It is the duty of the hydrogeologist to supervise all drilling activities and provide technical advice in the course of drilling. Another responsibility is the analysis of rock cuttings, identification of the productive zones and recommending the borehole depth. During drilling, drillers must keep a detailed log of the drill cuttings obtained from the advancing borehole. In addition, after the drilling has been completed but before the well is installed, it is often desirable to obtain more detailed data on the subsurface geology by taking geophysical measurements in the borehole.

### (a) Drilling Methods

There are several different types of rigs available for drilling water boreholes. They vary in size, capacity and capability depending on the type of formation expected and the depth required. There are rigs which do not perform well in hard rock formations and there are those that are multipurpose. Percussion and rotary-percussion drilling methods are generally the most applicable techniques for drilling in igneous and metamorphic rocks. If a significant thickness of granular or other overburden materials is present, a combination of methods can be effective, although not very practical. Cable-tool, hydraulic-rotary percussion and air-rotary percussion (down-the-hole air hammer) and foam drilling modifications are the most common types of equipment in use today for such rock types (Referred to web: resvol.design).

### (b) Borehole Logging

#### (i) Rock sampling

Rock cuttings are collected, normally at a 2 meters interval while drilling is in progress and a proper borehole log is kept. The samples are analysed to identify the water bearing zones, and the driller keeps a record of penetration rate to assist in identifying the hard formations and their thickness as well as recording the water struck levels.

#### (ii) Down-hole logging

For accurate description of the penetrated strata, application of geophysical logging methods assist in the determination of thickness of formations, the zones of highest porosity and water quality.

### 9.1.5.4 Siting of Well/Borehole

The borehole siting methodology should be adjusted to the hydrogeological conditions and the local experience and should be done by an experienced hydrogeologist. It should include the following steps:

- Identification of fracture zone on aerial photographs, satellite images and maps;
- Identification of fracture zones in the field using resistivity profiling, Electrical profiling, and
- Vertical Electrical Sounding (VES).

### 9.1.5.5 Well Design

Well design is done in two stages, the preliminary design and the final well design. Designing consists primarily of deciding the well depth, casing diameter, screen type and slot size and its position in the well. Once the well site is determined, a preliminary well design is prepared by an experienced hydrogeologist or driller based on hydro-geologic information gathered before the drilling. This preliminary design is the basis of the well drilling contract and the cost estimates.

During the drilling period, the preliminary well design will be adjusted based on actual observations and findings on the specific site. This adjusted design will then become the final well design. During this stage, the design assumptions used are verified and become actual design parameters, such as water table level, drawdown, depth and thickness of the geologic layers, types of material of each geologic layer encountered, and other relevant information.

The main objective of the design is to construct a well that:

(a) Is structurally stable;
(b) Is able to extract groundwater at the desired volume and quality;
(c) Has the proper and correctly placed screens or slots to tap the productive aquifers as well as to allow effortless flow of ground water into the well;
(d) Has enough space to house pumps;
(e) Has appropriate gravel packing that minimizes entry of sediments and sand particles.

## 9.1.6 Structures for Rainwater Harvesting

### 9.1.6.1 Roof Catchment

Following details are available:

Catchment: Rooftop

Area of the catchment (A) = 100 sq. m.
Average annual rainfall (R) = 611 mm (0.61 m)
Runoff coefficient (K) = 0.85

Family size = 5

Per capita household water requirement = 25 litre/person/day

a) Calculate the maximum amount of rainfall that can be harvested from the rooftop:

Annual water harvesting potential, S = 0.85 x 100 x 0.61 = 51 cu. m (51,000 litres)

b) Determine the tank capacity: This is based on the dry period, i.e., the period between the two consecutive rainy seasons. For example, with a rain period extending over four months, the dry season is of 245 days.

c) Calculate drinking water requirement for the family for the dry season,

C = 5 x 25 x 245 = 30,625 litres

As a safety factor, the tank should be built 20 per cent larger than required, i.e., 36,750 litres. This tank can meet the basic drinking water requirement of a 5-member family for the dry period. A typical size of a rectangular tank constructed in the basement will be about 5.0 m x 5.0 m x 1.5 m.

Note that in this case the analysis of the water availability/harvested against satisfaction of the demand need to be done. Alternatively, other detailed analysis such as the cumulative mass curve should be carried out for storage determination. Figure 9.21 shows a typical rooftop water harvesting configuration.

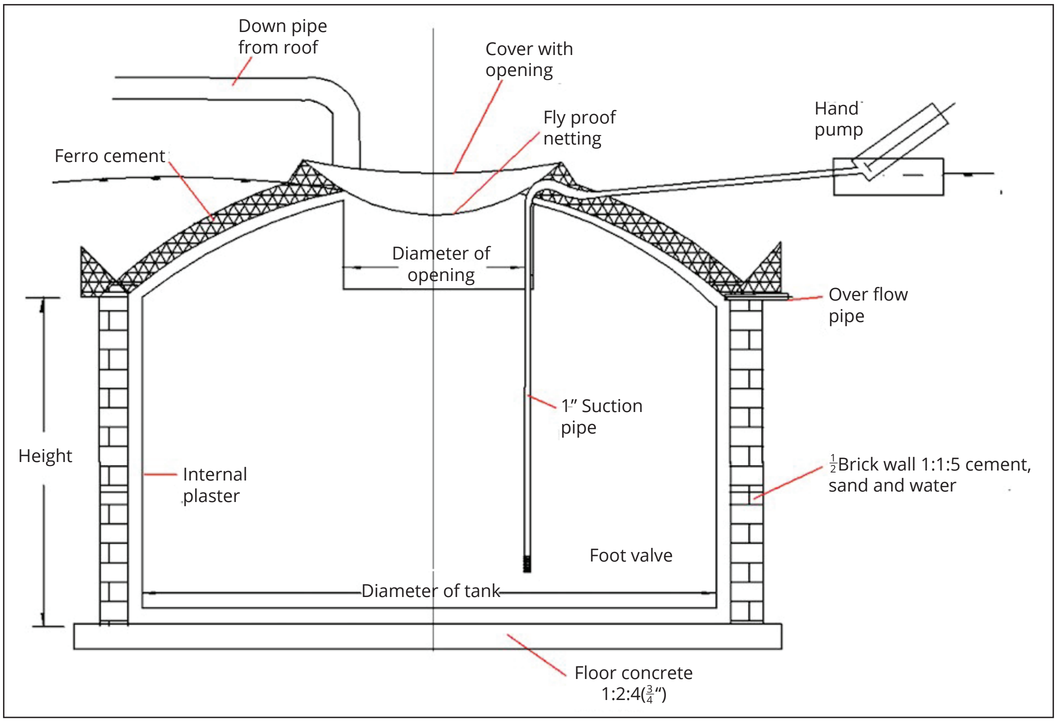



**Figure 9.10: Underground Dome Shaped Ferro-cement Tank**

Storage tanks should be located as close to supply and demand points as possible and should be protected from direct sunlight. Storage tanks can be elevated, on the ground or underground. They can be made of plastic materials, concrete or galvanised steel. Configuration depends on the size of the tank. For large catchment, underground tanks fitted with a pump to draw water to elevated distribution tank are preferable. Underground tanks can be rectangular or cylindrical, reinforced or plain concrete. Overflow pipes must be installed in the top of the tank to allow the safe disposal of excess rainwater. The size of the overflow pipe should be the same as that of the inlet pipe, with mesh at the bottom to prevent rats, squirrels, cockroaches, and other pests from entering.

The most common materials for gutters and downspouts are half-round PVC, vinyl, pipe, seamless aluminium, and galvanized, steel. Regardless of material, other necessary components in addition to horizontal gutters are drop outlets, which route water from the gutters downward, and at least two 45-degree elbows, which allow the downspout pipe to sit snugly against the side of the house. Additional components include the hardware, brackets, and straps to fasten the gutters and downspout to the fascia and the wall. First flush system separates first rainwater contaminants namely debris, dirt, and dust that collect on roofs during dry periods and prevent it from entering the storage tank.

### 9.1.6.2 Surface Runoff Harvesting

Surface runoff water harvesting is the collection, accumulation, treatment or purification, and storing of storm water for its eventual reuse. It can also include other catchment areas from man-made surfaces, such as roads, or other urban environments such as parks, gardens and playing fields. The harvested surface runoff water is an excellent alternative to replace treated water for use in other purposes. If properly designed, surface runoff catchment systems can collect large quantities of rainwater.

**Figure 9.11: Runoff Water Collection Tank[14]**

[14] https://www.appropedia.org/Run-off_rainwater

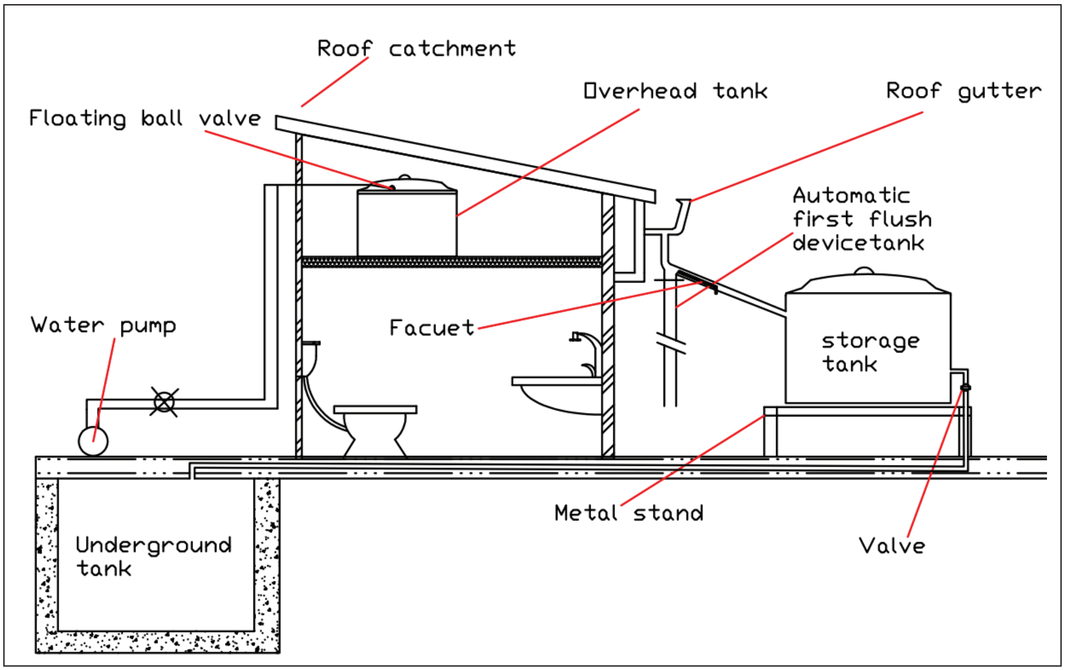



**Figure 9.12: Roof Rainwater Harvesting System Including the Storage Tank(s)**
(Source: Nnaji 2019)

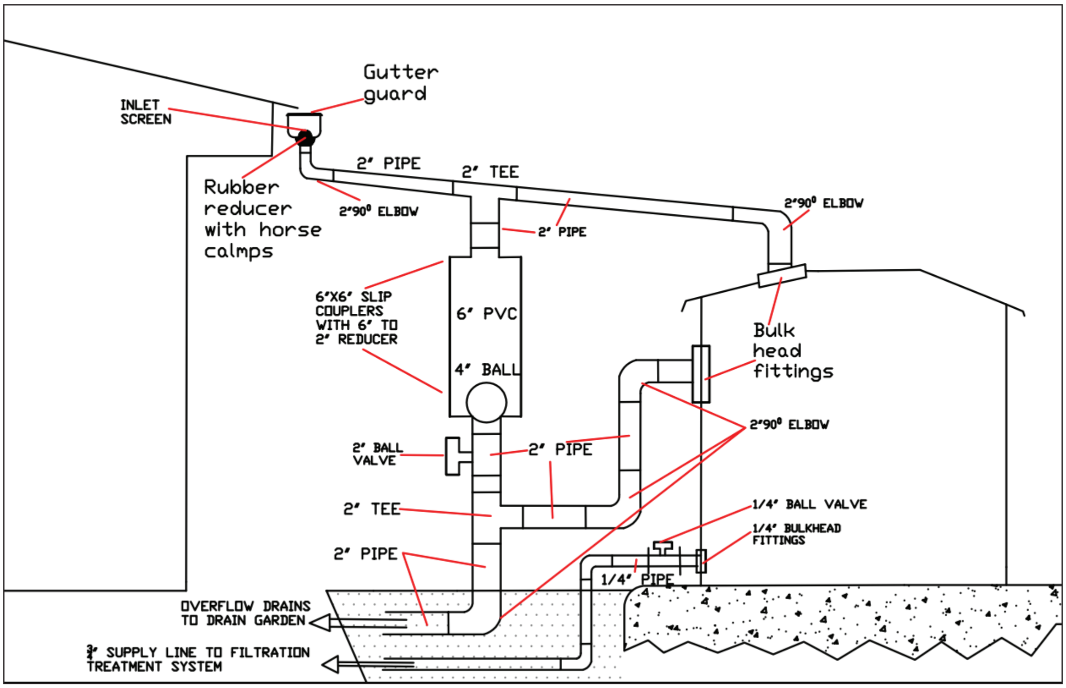



**Figure 9.13: Arrangement of the Coarse Mesh and First Flush Pipe**
(Source: Saleh, Taher and Noaman, (2017))

### 9.1.6.3 Treatment of Harvested Rainwater

Effective and reliable water treatment is essential in protecting public health and promoting consumers' confidence in the water they receive. Rainwater collection systems had historically been thought to provide safe drinking water without treatment, because its collection surfaces (roofs) are isolated from typical contamination sources (e.g. soil materials, rocks and sanitation systems). However, contamination of waters from such sources may originate from materials used to manufacture the roofing sheets and drainpipes (gutters). Other sources of contaminants may include domestic and industrial dust, leaves and other debris blown onto roofs. In addition, birds and climbing animals defecate upon such roofs and gutters presenting risks associated with bacteriological and metal contamination.

These are among the potential sources of rainwater contaminants and thus preventing them from entering the storage tanks can significantly enhance the quality of rainwater collected. The reduction of contaminants in rainwater tanks may be brought about through different ways including:

(a) Takinginto consideration several environmental conditions to improve rainwater quality, such as proper design, operation, and periodic maintenance of collection systems, cleanliness of the catchment area and water storage tank, and protection of collection systems from pollutants,
(b) Discarding the first spill of rain (for at least 15 minutes) before collecting the water,
(c) Installation of screeners in drainpipe inlets or gutters,
(d) Avoiding roof painting with materials likely to present health risks,
(e) Adopting closed tanks option rather than open tanks which are prone to contamination emanating from atmospheric deposition,
(f) Disinfecting the harvested rainwater with appropriate water disinfectants such as chlorine.

## 9.1.7 Water Points and Service Connections

To achieve the Water Sector Sustainable Development Goals (SDGs), Water Points (WPs), or community taps, are essential components of any new or expanded water supply schemes. In peri - urban or informal urban housing areas, this predominant service should as far as possible be water points. In rural areas the decision on whether to install a community tap or water point (WP) should be taken by the community but only after they have been fully informed on what this means. In either case, water should be charged for on some volumetric basis. Individuals who wish to have yard or in-house connection should be allowed in return for higher payment.

Because WPs play a key role in the supply of water in rural areas and to the urban poor and will continue to do so for many years to come, the most appropriate ways of supplying water through them are under review worldwide. The following guidelines should not, therefore, be rigidly followed but should be used as a basis from which designers can start.

Regardless of the detail, involving the community in finding proper sites for the water points has proven to be an efficient way to improve psychological ownership, prevent vandalism and make treated water accessible. As the design of water point is vital for its sustainability; therefore, it has to be well adapted to the local context and the requirements of operators and customers. The water point design has also a significant implication on the required funds for construction. The user community should be consulted before deciding on where to locate WPs. Use of the criteria of time to walk to WPs should be adopted with an indicative time of 30 minutes (maximum).

### 9.1.7.1 Common Types of Water Points

- Water taps
- Public standpipe with super structure/kiosk
- Public standpipe without super structure but has a simple shade with one or two tap points

### 9.1.7.2 Minimum Technical Requirements of Water Points

The utilities must ensure that the following minimum requirement for technical aspects must be adhered to in order to guarantee the ergonomic and hygienic aspects of water supply outlets as well as their user-friendliness in terms of the movements and practices of outlet users (and of women and children in particular) and operators:

(a) No one should have to carry the water for more than 20 m in an urban area and 400 m in rural areas whilst the time spent on collecting water should not exceed 30 minutes elsewhere in Tanzania,
(b) Users should be able to fill their container safely,
(c) For water points with long operating hours, a shade, roof or shelter for sun protection should be provided;
(d) A solid slab should be installed to ensure a hygienic and safe water collection
(e) Sufficient slope to allow natural drainage and efficient cleaning;
(f) An elevated fetching bucket bay as lifting aid;
(g) A soak away into the ground should be provided at the water point site to ensure that the water point site is adequately drained;
(h) A water meter for an accurate measurement of the water delivered and sold at the water point site
(i) High quality water taps to be used
(j) Where water is only available intermittently or supply pressures are low, provision of storage should be considered.

### 9.1.7.3 Criteria for the Design of Water Point/Kiosks

(a) Water point should be designed for a water consumption of 25 l/p/d,
(b) Design periods to be adopted: short term - 5 years; medium term - 10 years,
(c) Satisfactory pressure at the WP/water points should be provided for: The minimum static head should be not less than 6m and the maximum 25m. The flow from each tap at a WP should not be less than 10 l/min, and where appropriate, a constant flow valve should be installed,
(d) A single WP/Water point should in general not supply more than 40-50 households, or 200 to 250 persons, which number is likely to be the minimum if the supply is to be financially viable for an operator or a community,
(e) The water quality must comply with the latest edition of Tanzanian Standards (TBS) and contamination at the source should be avoided,
(f) Acceptability: The design of the water point should be user friendly and according to the minimum technical standards specified in Figures 5.4, 5.5 and 5.6,
(g) Water points structures (pre-paid taps, yard taps, etc.) should be designed to be vandalism proof, e.g. by putting a concrete slab on top of the iron roofing sheets or even a concrete roof,
(h) Water at the source should be provided in sufficient quantity and the supply should be regular and continuous,
(i) Design of a water point should take into a consideration upgrading to yard and house connection later,
(j) Design should consider the most appropriate technology, which can serve a high number of poor people in urban settlements at low per capita cost and takes shorter time to construct,
(k) Potential users should participate in planning and designing of the water source and access to information must be guaranteed,
(l) A water point with a superstructure is often a more convenient place to fetch water since it is more hygienic; gender sensitive and allows customers to interact at the water point.

### 9.1.7.4 Location Considerations for Water Points

(a) The water points should be accessible to all users. It is easier to guarantee free and non-discriminatory access to water points for all customers if the water points are placed on public land, such as a municipal land or on a land that belongs to a water utility. Private landowners are potentially in a position to deny access to water points which are placed on their premises,
(b) Public participation in decisions about a water point location is an important requirement. Potential water point customers know their own surrounding areas best and can easily locate appropriate water point sites. This, in turn, will develop a sense of ownership for the water point,
(c) The placement of WPs must take the preferences of the operator or attendant and of future customers into account, as well as the technical and commercial constraints and objectives identified by the designer. In other words, a water

point should be placed in such a way that it can serve a maximum number of customers in an efficient and customer friendly manner,

(d) The location of water points in relation to one another should ensure adequate coverage,

(e) Water points shall be constructed on public places or private owned land with written permission from the landowner, witnessed by the LGA (Street Leadership),

(f) The water point should be located strategically starting with densely populated areas to less populated areas,

(g) Notwithstanding criteria (a) to (f), the location of a water point should be confirmed by the communities to be served through their Government,

(h) The potential beneficiaries of water point services should be involved in the selection of the water point location. If the location is not conducive to its customers, they might consider other sources and thus abandon the water point.

### 9.1.7.5 Technical Tips to Improve Water Points

The following are practical tips concerning the design and construction of water points:

- The soak-away: This should have a pit filled with stones to avoid breeding of infectious insects. Also, the drains should have a slope that allows for regular cleaning and unblocking,
- Paintwork: Only washable paints should be used; for the exterior and the interior walls of the water point,
- The water meter: The water meter should be placed inside the water point. A vertical meter should be installed vertically and a horizontal meter horizontally,
- Door, window and locks: it is important that water point doors and windows (and frames) are made of steel; strong enough to withstand vandalism or attempts of theft. Water point windows should be installed in such a way that if they are fully opened, they are parallel to the front wall of the water point. This allows for the full use of the lifting slabs,
- Cement works: It is important that the Utility's supervisor is present during the mixing of cement. This is important in order to have the right mix and to prevent "sandy water points",
- Making the roof water-proof: In order to make the roof water proof, it should be tarred with a sufficient quantity of Roof Compound. Experience shows that many water point roofs are often leaking,
- Quality Water taps: Only high quality taps and valves should be used. Do not forget that a water point tap is used hundreds of times each day. The quality requirements of a water point tap, therefore, are much higher than of a normal domestic tap,
- The siting of WPs: should be so that the area can be kept clean with minimum effort and there must be sufficient scope for natural drainage of

wastewater. Where cloth washing slab is provided this should be not more than 20 metres from the water point,

- Water taps: Install the high water taps to avoid wastage of water caused by leaking taps and unhygienic conditions around the water point surroundings,
- The operator: or attendant should be made responsible for keeping the area clean and drained without altering any stagnant water pools,
- A charge: should be made for the supply of water from a WP. This should preferably be based on a coin still in wide circulation per container of 20 litres (e.g. 50/- or 100/-),
- In urban areas: the operator should be selected or appointed by the household representatives and the street executive officer. In urban areas, the supply is best provided through a formal structure/water point, and the operator should be encouraged to sell other non-contaminant dry goods items both to augment income and extend operating hours.

Figure 9.14, 9.15 and 9.16 illustrate details of water points and service connections.

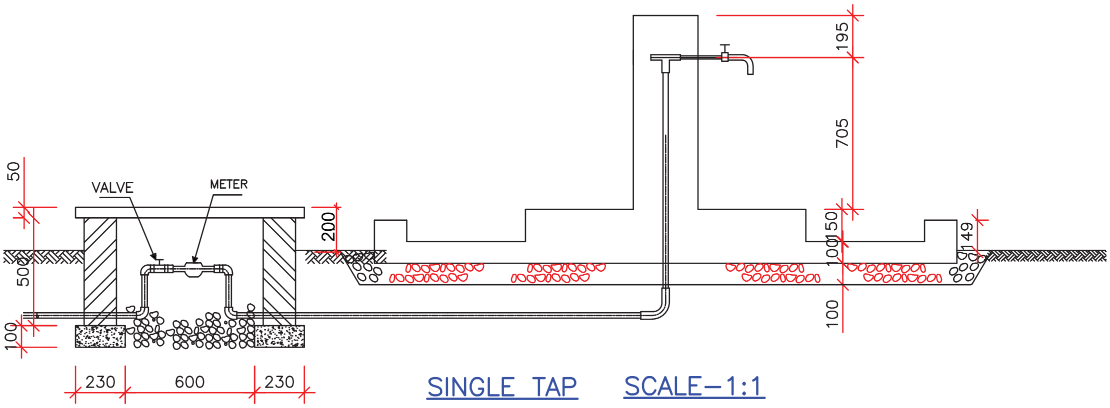



**Figure 9.14: Typical Detail of a Single Tap Domestic Point**

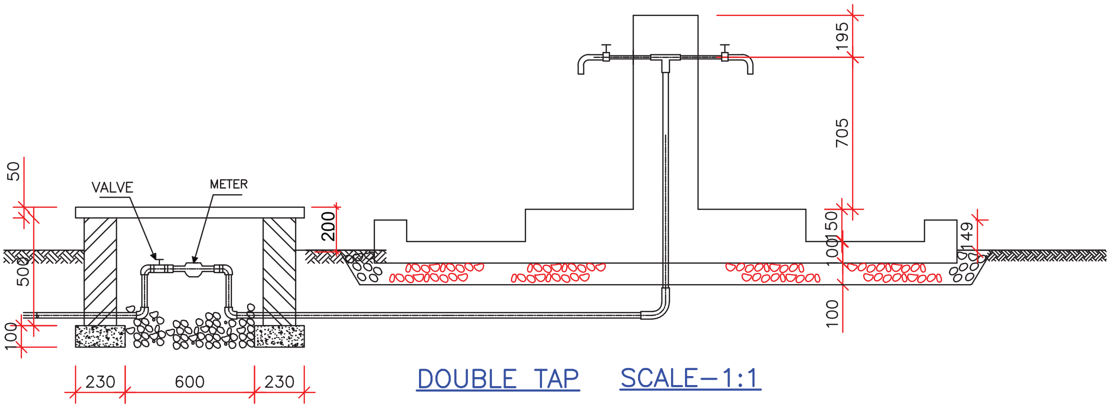



**Figure 9.15: Typical Detail of a Double Tap Domestic Point**

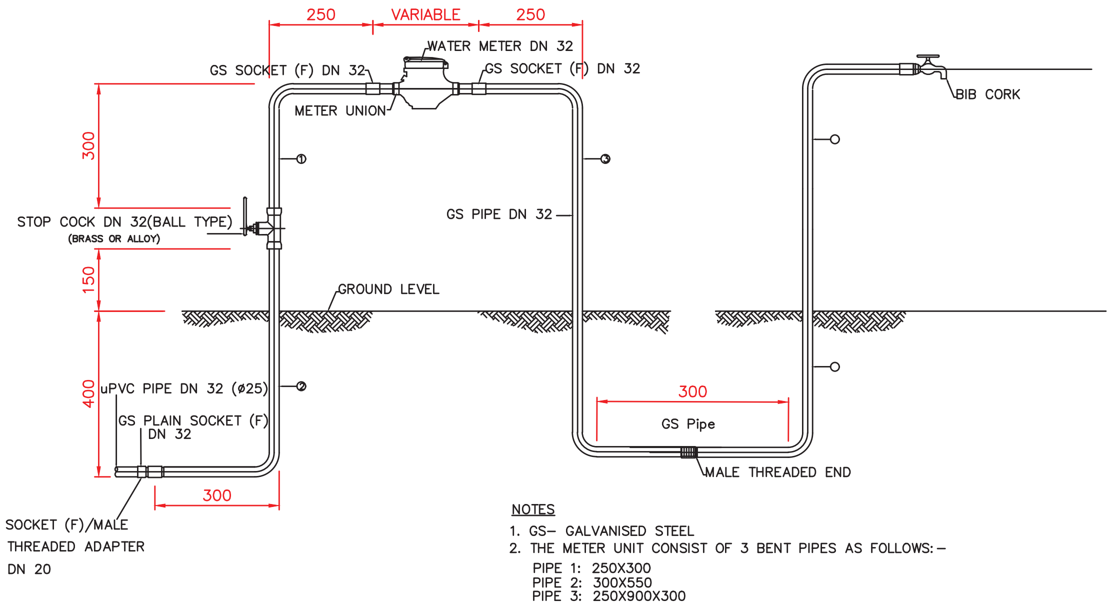



**Figure 9.16: Typical House Connection**

# 9.2 STRUCTURAL DESIGN OF CONCRETE

## 9.2.1 Structural Requirements

Concrete members used in water structures range from walls, slabs, roofs and floors. Design of Reinforced Concrete (RC) should ensure these structures have sufficient resistance to cracking, adequate strength and does not allow leakage.

## 9.2.2 Methods

Three methods or approaches are available for structural design of concrete, these are listed below:

### 9.2.2.1 Working Stress Method

- Produces uneconomical section,
- Produces stable section

### 9.2.2.2 Ultimate Load Method

- Produces cheaper section,
- Produces unstable section.

### 9.2.2.3 Limit State Method

- Produces economical section,

- Produces stable section.

Detailed design steps for concrete structures are presented in Appendix D to this manual.

## REFERENCES

American Ground Water Trust. (2019). Retrieved from American Ground Water Trust Web Site: http://www.agwt.org

Amit Kohli and Karen Frenken (2011). Cooling water for energy generation and its impact on national-level water statistics: http://www.fao.org/3/a-bc822e.pdf

BS 8110 (1997): Structural Use of concrete, Part1: Code of Practice for design and construction, British Standards Institution, London 1997.

Harter, T. (2016 (University of California, Davis) Retrieved from UC Groundwater Cooperative Extension Program Web Site: http://groundwater.ucdavis.edu

Manual for Rooftop Rainwater Harvesting Systems in the Republic of Yemen. November, 2017

Ministry of Water, The 3rd Edition Design Manual for Water supply and Wastewater Disposal, 2009.

Nibedita Sahoo (2008): Design of Water Tank, National Institute of Technology Rourkela, India.

Nnaji, C.C. (2019)July. Sustainable Water Supply in Buildings through Rooftop Rainwater Harvesting. In Construction Industry Development Board Postgraduate Research Conference (pp. 390-400). Springer, Cham

Reynolds et al (2008): Reinforced concrete designer’s handbook, Eleventh edition, Taylor & Francis, New York.USA

S. A. Saleh, T. Taher and A. Noaman (2017), Nov. Manual for Rooftop Rainwater Harvesting Systems in the Republic of Yemen

Reynolds et al, (2008): Reinforced concrete designer’s handbook, Eleventh edition, Taylor & Francis, New York.USA

Water Mission (2019): The Technical Handbook Version 1.2`,North Charleston, USA

Chapter 10

# APPLICATION SOFTWARE

## 10.1 APPLICATION SOFTWARE CONTEXTS

Application software[15], or simply applications, are often called productivity programs or end-user programs because they enable the user to complete tasks, such as creating documents, spreadsheets, designing, analysis and modelling, doing online research, sending email, etc. The application software can be applied for many essential tasks like designing huge and small structures, for example, water supply systems, sanitation systems, water treatment plants, buildings, etc. Also, they can also be applied for virtual reality, predicting behaviour of engineering structures, solving equations for optimization of resources tender bidding, earth-work estimation, cost estimation, project management, structural drawing, predictive model making, aid in satellite surveying, data transfer; its interpretation, analysis and others.

The designers should use genuine application software with valid license or make use of open source/freely available software for Water supply and Sanitation project Designs, Analysis and Modelling. Basic design software are especially important because they facilitate tasks and enhance the quality of design, modelling and analysis process for engineering infrastructure. In addition, use of software make the work easier and faster, more accurate, time saving, as well as reducing the total cost, the workload and manpower compared to the work that is done manually. Recently, many commonly-used software are available on the market.

## 10.2 RECOMMENDED APPLICATION SOFTWARE

Common software being used in Water Supply and Sanitation Projects are:

[15] Study.com, Application Software. Retrieved at 17:43, January 6, 2020 from https://study.com/academy/lesson/what-is-application-software-definition-examples-types.html

## 10.2.1 Distribution Network Design Software

### 10.2.1.1 Epanet

Epanet - is a software application used throughout the world to model water distribution systems. It was developed as a tool for understanding the movement and fate of drinking water constituents within distribution systems, and can be used for many different types of applications in distribution systems analysis. EPANET is public domain software that can be freely copied and distributed. It is a Windows®-based program that will work with all versions of Windows. For more details, visit its website at https://www.epa.gov/water-research/epanet.

### 10.2.1.2 AutoCAD

AutoCAD - is a computer-aided drafting tool that allows many different types of designers to create diverse kinds of drawings and designs. This program helps designers to create their designs much more quickly than by hand and offers many quick, easy, and useful features, such as copy and paste. AutoCAD can create any 2D drawing and 3D model or construction that can be drawn by hand. For more details, visit its website at https://autocad.en.softonic.com/download

### 10.2.1.3 WaterCAD

WaterCAD - is an easy-to-use hydraulic and water quality modelling application for water distribution systems. Utilities, municipalities, and engineering firms trust WaterCAD as a reliable, resource-saving, decision-support application for their water infrastructure. From fire flow and constituent concentration analyses, to energy cost management and pump modelling, WaterCAD helps engineers and utilities to analyze, design, and optimize water distribution systems. - https://watercad.software.informer.com/

### 10.2.1.4 WaterGEMS

(d) WaterGEMS – is a Water Distribution Analysis and Design Software. WaterGEMS can also enable the running of a model from within ArcGIS and for optimization modules (calibration, design, pump scheduling, pipe assessment, SCADA integration, and network simplification). - https://bentley-watergems.software.informer.com/

### 10.2.1.5 KY PIPES

KY PIPES - Full-featured hydraulic analysis software package with the ability to design for rural and urban contexts. It includes full design capabilities, including pipe diameter, demand, fire flow, temperature variation, and pump optimization. It is compatible with AutoCAD and GIS. It uses EPANET for its water quality analysis. http://kypipe.com/downloads/

#### 10.2.1.6 GeoNode

GeoNode - GeoNode is a web-based application and platform for developing geospatial information systems (GIS) and for deploying spatial data infrastructures (SDI). It is designed to be extended and modified, and can be integrated into existing platforms. http://geonode.org/

#### 10.2.1.7 InfoWater

InfoWater **-** InfoWater Pro modelling and management software enables users to design, plan and operate their water network with an easy-to-use, sophisticated solution. Hydraulic models in InfoWater bridge the gap between network modelling and ArcGIS, ensuring you get sound, cost-effective engineering solutions for designing, planning, and operating your systems. InfoWater is completely integrated within the ESRI ArcGIS environment.

https://docs.google.com/document/d/1P8fE0vaDhU5ul4SdKAg3qh1z7if16Q1u-xXuH-VpPAc/edit

#### 10.2.1.8 GIS Software

GIS Software - A Geographic Information System (GIS) is a system designed to capture, store, manipulate, analyze, manage, and present spatial or geographic data. GIS applications are tools that allow users to create interactive queries (user-created searches), analyze spatial information, edit data in maps, and present the results of all these operations. https://www.qgis.org/en/site/

### 10.2.2 Operation and Maintenance Software

#### 10.2.2.1 MS Project

MS Project - is a project management software product, developed and sold by Microsoft. It is designed to assist a project manager in developing a schedule, assigning resources to tasks, tracking progress, managing the budget, and analyzing workloads. For more details visit https://products.office.com/en-us/project/project-management-software

#### 10.2.2.2 Excel

Excel - Microsoft Excel is a spreadsheet program included in the Microsoft Office suite of applications. Spreadsheets present tables of values arranged in rows and columns that can be manipulated mathematically using both basic and complex arithmetic operations and functions. For more details visit website https://www.microsoft.com/en-us/p/excel/cfq7ttc0k7dx?activetab=pivot%3aoverviewtab

#### 10.2.2.3 EDAMS

EDAMS is comprehensive technical information system that deals with Billing, Asset Maintenance of Utility, Municipal and Government assets, including Water, Sewer, Drainage, Groundwater (boreholes, aquifers), Meteorology (stations, etc.), surface water (catchments, rivers, dams, weirs, man-made structures, etc), Waste management & Recycling. For more details visit its website. https://www.edams.com/edams-products/edams-maintenance-management-systems/

## 10.2.3 Water Quality

### 10.2.3.1 WaterCAD

WaterCAD - is an easy-to-use water quality modelling application for water distribution systems. Utilities, municipalities, and engineering firms trust WaterCAD as a reliable, resource-saving, decision-support application for their water infrastructure. From fire flow and constituent concentration analyses, to energy cost management and pump modeling, WaterCAD helps engineers and utilities analyze, design, and optimize water distribution systems. For more details visit website https://watercad.software.informer.com/

**Epanet-MSX (Multi-species Extension)**

Epanet-MSX is a model that analyses water quality in distribution networks using a dynamic link library of EPANET 2, the hydraulic simulation model described in 10.2.1.1 above. It analyses multiple chemical substances simultaneously, accounts for any differences in the quality of water from different sources and provides a framework that permits the simulation of any chemical or biological constituent. For more details, visit its website at https://www.epa.gov/water-research/epanet

## 10.3 SUPERVISORY, CONTROL AND DATA ACQUISITION (SCADA) SYSTEMS

SCADA is a monitoring and control system architecture comprising computers networked data communications and graphical user interfaces for high-level process supervisory management[16]. SCADA can control large-scale processes that can include multiple sites, and work over large distances as well as small distance.

To monitor a variety of water supply data like flows, pressures, temperatures, water levels (intakes, reservoirs and water tanks) etc, in various water supply and sanitation projects, SCADA can be used for data acquisitions, data communication, information/data presentation, monitoring and controlling. Principally, SCADA system gathers data (like leakage on a pipeline) from sensors and instruments located at remote area and sends to the computer, which processes this data and presents the processed information in logical and in a timely manner.

SCADA systems can be supplied in Tanzania. For example, in October 2019, was in the news that AREVA's Transmission and Distribution division will deliver and implement a Supervisory Control and Data Acquisition Electricity Management

[16] Wikipedia contributors. (2019, December 21). SCADA. In *Wikipedia, The Free Encyclopedia*. Retrieved 08:37, January 6, 2020, from https://en.wikipedia.org/w/index.php?title=SCADA&oldid=931783794

System, or SCADA/EMS, to Tanzania Electric Supply Co. Ltd. (TANESCO).Honeywell Company supplied their SCADA system named Technofab & Garmon Toddy Engineering to Bukoba Urban Water Supply and Sanitation Authority. Other known companies that are good in supplying SCADA systems include Edibon.

## REFERENCES

Wikipedia (2019). SCADA. In *Wikipedia, The Free Encyclopedia*. Retrieved 08:37, January 6, 2020, from https://en.wikipedia.org/w/index.php?title=SCADA&oldid=931783794

Chapter 11

# METERING

## 11.1 INTRODUCTION

Water meters are used to measure the volume of water used in residential and commercial buildings that are supplied with water by a public water supply system. Water meters can also be used at the water source, well, or throughout a water system to determine its flow through a particular portion of the system. In most of the world water meters measure water flow in cubic metres ($m^3$).

## 11.2 TYPES OF WATER METERS

There are several types of water meters in common use. The choice depends on

- the flow measurement method,
- the type of end-user,
- the required flow rates, and
- accuracy requirements.

The following are the different types of water metres:

- Displacement water meters
- Velocity water meters
    - o Multi-jet meters
    - o Turbine meters
    - o Compound meters
- Electromagnetic meters
- Ultrasonic meters
- Prepaid water meters

Meters can be prepaid or post-paid, depending on the payment method. Most mechanical type water meters are of the post-paid type, as are electromagnetic and ultrasonic meters. With prepaid water meters, the user purchases and prepays for a given amount of water from a vending station. The amount of water credited is entered on media such as an IC or RF type card. The main difference is whether the card needs contact with the processing part of the prepaid water meter. In some areas, a prepaid water meter uses a keypad as the interface for inputting the water credit.

## 11.3 PREPAID METERS

Prepaid water systems including water meters have attracted significant attention in Tanzania and they are considered to be game changer in ensuring sustainability of urban as well as selected rural water services in the country located in Kishapu, Karatu, Babati and Arusha. The management of these prepaid systems has covered over 1,400 villages (Human Development Innovation Fund (HDIF) Tanzania, 2019) and 23 out of 70 urban utilities (MoW, 2019) that are currently using the prepaid water metering as a means to improve revenue collection processes. Examples of urban utilities that have introduced pre-paid meters include IRUWASA, DAWASA, MWAUWASA and KASHWASA).The urban utilities have prioritized installation of prepaid water meters for big public consumers of water who have large debts for extended durations. On the other hand, most of small scale users in rural areas collect their water from water points that are fitted with pre-paid water metres.

Prepaid water refers to the situation where a consumer purchases water credit in the form of a prepaid water token in advance of getting the service. When the token is entered into the user interface unit (located in the user or consumer's home).The token instructs the water management device to allow a certain amount of water through the meter before closing. Consumers can track usage, load credit remotely, and decrease the possibility of bill shock due to leakages or incorrect monitoring.

## 11.4 TYPES OF PREPAID WATER METERS COMMONLY USED IN TANZANIA

Different models of prepaid water meters are being piloted in various urban Water Supply and Sanitation Authorities (WSSAs) and in selected districts supported by Development Partners as well as in some CBWSO owned projects:

**Table 11.1: Different Models of Prepaid Water Meters Piloted in Various WSSAs and in Selected Districts**

| S/N | Water Utility / District | Prepaid water meter Model |
|---|---|---|
| 1 | Arusha WSSA | Calin |
| 2 | WSSAs such as Bunda, Geita, Mafinga, Singida, Dodoma, Shinyanga, Mbeya, Iringa, Sumbawanga, Morogoro, MANAWASA and Tanga | e –Joy |
| 3 | WSSAs such as Dar es Salaam and Coast (DAWASA), Kahama, Mwanza, Moshi, Tabora, Wanging'ombe and Makambako | Baylan |
| 4 | Kigoma WSSA | Shangai Xvnhvi Environment Technology |
| 5 | Babati and Arusha districts | EeWater system |
| 6 | Karatu district | Grundfos AQ taps |
| 7 | Kishapu district | Susteq |

Source: MoW report, 2019

## 11.5 IMPORTANCE OF PREPAID WATER METERING

- Prepaid water systems are an effective and efficient way of collecting water tariffs and they offer a high level of convenience to both the users and the local water supply authorities. They save time and do not require any paperwork. Moreover, the system eliminates cash transactions and therefore contributes to the transparency of tariff collection,
- Prepaid metering reduces administration costs to a minimum, while removing the risk and frustration of late or non-payment of water bills,
- Collecting data from prepaid meters is more efficient than the manual collection required for post-paid meters,
- Prepaid water systems generate real time data on water collection by users, tariffs collected from users, and water point functionality;
- Prepaid systems are cost-effective solutions to sustainable water management in that they have a low cost of acquisition and, by curbing water usage; capital recovery is possible within months,
- The water systems are able to distribute water equally, based on free water quotas, water balancing and fluctuating demand,
- Prepaid water metering gives consumers the opportunity to monitor their consumption and react immediately to possible leakages, thereby saving money.

## 11.6 DESIGN CONSIDERATIONS FOR PREPAID WATER METERS

It is important to understand two key aspects of prepaid water metering; prepaid water metering does not involve meters alone, but rather is a system; and three (3) major applications of prepaid technology which have different characteristics, impacts, and challenges. The notion of pre-payment metering obscures the complementary components of an integrated pre-payment system as illustrated by HDIF (2019), which includes:

- The prepaid metering is a system that comprises metering, dispensing, and credit-loading components. The prepaid dispensing device is the technology required to load and transfer credit, a database recording customer purchase and metered consumption and on-going engagement with customers.
- Prepaid water meters use a mechanical water meter, coupled to an electronics module with a credit meter and a water control valve. However, prepaid systems use rotating piston and Multijet water meters of which their accuracy can be easily affected by grit, sand, and air; and frequent supply interruptions raise the risk of malfunctioning.
- The presence of the rotating piston and Multijet is a significant vulnerability for its metering systems especially in urban areas, where there are ageing networks, discontinuous supplies, and low pressure fluctuations. Therefore, in such cases, the designers need to opt for electromagnetic and ultrasonic prepaid meters that are technically better suited to networks with supply

interruptions. These models are also highly accurate; resilient to pressure changes, air, and grit; and have no moving parts.

(a) Domestic prepaid water meter in DAWASA


(b) Calin prepaid water meter


(c)

(d)

**Figure 11.1: Water Kiosk/ Point with a Prepaid Meter in Kishapu E-Water Point and in Sangara Village Babati.**

(Source: http://www.hdif-tz.org/)

## 11.7 DESIGN CONSIDERATIONS FOR PREPAID PUBLIC STANDPIPES

Most of these facilities differ from one place to another. In the rural water utilities, the prepaid metering technology consists of the following (HDIF (2019):

(a) a water point, a kiosk (where the water point is,
(b) a shop,
(c) digital water tags,
(d) an application programming interface (API) for mobile money transfers,
(e) a database, and a dashboard for system monitoring and reports generation.

(f) In another case, the system is a simple technology through which users can purchase credit using the eWATERapp on smart phones, through mobile money, or by receiving a remote gift via PayPal.

(g) Customers use a standpipe, kiosk or water point loads credit bought from designated vendors using a programmed metal key, a smartcard, or a keypad.

(h) Some prepaid systems do not support multiple taps, nor do they operate well when the water pressure is low, and on the other hand, some technology can handle multiple taps.

(i) In the case as (h) above, the utility must develop partnerships with technology providers in sustaining and scaling up the prepaid water meter systems. Among other things, this means agreeing on and enforcing software licensing agreements, protocols on data management and sharing and the use of future warranties; as well as understanding the technology provider's role and responsibilities during and after the installation of prepaid water meters (including after-sale services by technology providers).

## 11.8 DESIGN CONSIDERATION FOR INDIVIDUAL DOMESTIC CUSTOMER

(a) As illustrated by HDIF (2019), customers use their own prepaid meters, and load credit using a tag, smartcard, or keypad.

(b) The tag, card, or code can only be used on the specific meter for which it is programmed.

(c) Once the credit is loaded into the meter memory, customers do not have to use the key each time they draw water.

## 11.9 DESIGN CONSIDERATION FOR COMMERCIAL AND INSTITUTIONAL CUSTOMERS

As illustrated by HDIF (2019), commercial and institutional customers usually use the Prepaid Bulk flow meters where wide variation in flow can be expected, such as in multi-story business buildings, hospitals, schools, offices and other places where both low and high flows can occur due to several consumptions users.

These wide flow ranges are measured by using a built-in change-over valve together with small residential meters and large bulk meter. All bulk meters should be tested to ensure that they meet approved standards. The meter needs to be designed for far higher volumes than domestic meters and far greater accuracy, given the volumes. The large volumes of water sold to commercial and institutional customers comprise a significant source of income for water service providers in most urban towns.

## 11.10 IMPORTANCE OF INTEGRATING PREPAID WITH POST-PAID REVENUE MANAGEMENT

Integration with post-paid revenue management is vital, supported by a database of meters and customers with records of consumption, credit purchases, and performance. This integration is more difficult and costly in terms of investment required (staffing and/or computer billing upgrades) or efficiencies foregone, than is often assumed. Regular monitoring is required to track faults, exceptions, and real-time consumption against prepaid sales. Finally, making prepaid meters work and ensuring their acceptability requires consistent and sustained interaction with customers.

## 11.11 SELECTION CRITERIA FOR PREPAID WATER METERS

For accurate water flow measurements, the characteristics of the water flow have to be known, before a suitable meter type with the right specifications can be chosen to fulfil this task. The following aspects should be considered when selecting a suitable prepaid water meter:

(a) Assess the water quality: The water quality must comply with the one specified for a meter. Metering accuracy is significantly affected by suspended solids and depositions. Dirty water will cause under-registration with Positive Displacement as well as with Velocity Meters. Growth of algae in the meter can lead to blockage.

(b) Determine the consumption pattern (minimum and peak flow rates): Prepaid Water meter measures accurately only in flow rates that lie within its range of ability (Prepaid Meter accuracy should comply with **ISO 4064:2014** Standards.

  (i) Because domestic customers are large in numbers, it is recommended to make use of empirical field studies to analyze entire groups of domestic customers, e.g. those with or without storage tanks.

  (ii) Commercial water user however, tend to follow a more individualized their consumption pattern and are worth to be assessed on an individual basis to determine his/her consumption pattern.

  (iii) Also, the water supply network's to be assessed in terms of pressure zones to determine whether the minimum and maximum are possible at all.

(c) Inaccuracy of wrongly sized meters

  (i) When the prepaid water meter is too large the flow rates might be lower than the minimum flow rate and cause under-registration that will result in high non-revenue water. Also oversized meters are more costly than rightly sized meters.

  (ii) Alternatively, when the prepaid water meter is too small it results in accelerated inaccuracy and high pressure loss.

(iii) Undersized prepaid water meters can cause excessive pressure loss, reduced flow, noise and will eventually shorten its life span through wear and tear if operated frequently at or above its allowable maximum flow.

(d) Minimum and maximum pressure drop: should be within the meter specifications.

(e) The ordering of prepaid meter: is complete when they have in-line strainers to block debris to enter the metering unit. However, if a strainer needs to be installed it is recommended to install a backflow preventer (i.e. non-return valve as well).

Also, the following aspects should be considered when selecting a prepaid water meters:

(i) The piping conditions: are they new or existing network?
(ii) Assess available service and calibration service providers and costs e.g. Weights and Measures Agencies (WMA) etc
(iii) Availability of spare parts– if easily accessible in the local markets
(iv) Meter life span
(v) Costs of the prepaid meter integrated system (cheap or expensive)
(vi) Simplicity of the technology (software) used for prepaid metering system

(f) The correct specifications of the batteries used to power the unit. During procurement, it will be necessary to specify the minimum battery size to ensure they last for a long time.

## REFERENCES

Ministry of Water, The 3rd Edition Design Manual for Water supply and Wastewater Disposal, 2009.

Chapter 12

# DESIGN STANDARDS AND SPECIFICATIONS

## 12.1 DESIGN STANDARDS

A Standard is a limit of a measure of quality of a product prepared for judgement and compliance by an authoritative agency, professional or a recognized body. According to (Business Dictionary, 2020) Standards can be classified as:

- Government or statutory agencystandards and specifications enforced by law,
- Proprietary standards developed by a firm or organization and placed in public domain to encourage their widespread use, and
- Voluntary standards established by consultation and consensus and available for use by any person, organization, or industry.

Once established, standards (like bureaucracies) are very difficult to change or dislodge. Standards that apply for water supply and sanitationprojects will be as set by theTanzania Bureau of Standards, and relate toconstruction works British standards .

The list of institutions whose standards are recommended to be used in designing water projects is shown below:

- Tanzania Bureau of Standards
- British Standards (BS)
- American Society for Testing and Materials (ASTM)
- Deutsches Institut für Normung (DIN)
- German institute for standardisation
- American Association of State Highway and Transportation Officials (AASHTO)
- European Standards(ES)

Table 12.1 describes standard codes of practise and relevant area for application construction works.

**Table 12.1 Common Standards Used in Water Projects**

| No | Name | Institution | Use of standard |
|---|---|---|---|
| 1 | BS 8110 of 1997 | British Standards | Code of practise for design and construction |
| 2 | BS 812 : Part 2 : 1995 | British Standards | Testing aggregates- Methods of determination of density |
| 3 | ISO 1167-1 Part 1 | International Standards (ISO) | Thermoplastics pipes, fittings and assemblies for the conveyance of fluids — Determination of the resistance to internal pressure |
| 4 | ISO 1452-2 Part 2 | International Standards | Thermoplastics pipes, fittings and assemblies for the conveyance of fluids — Determination of the resistance to internal pressure |
| 5 | BS 6399 : Part 1 : 1996 | British Standards | Code of practice for dead and imposed loads |
| 6 | BS 1377-9: Part 9: 1990 | British Standards | Methods of test for Soils for civil engineering purposes: In-situ tests |
| 7 | DIN 1048 | Deutsches Institut für Normung | Quality tests of concrete |
| 8 | DIN 4226 | Deutsches Institut für Normung | Concrete aggregates; definitions, sizes, quality requirements and testing |
| 9 | DIN 15018 | Deutsches Institut für Normung | Steel construction; basis for design and performance, calculations |

For projects with requirements that are not covered by the above recommended standards, the designer should seek approval from the Tanzania Bureau of Standards (TBS)

## 12.2 SPECIFICATIONS

Specifications is a detailed description of how work is to be performed or requirements achieved, dimensions met, materials used. These are standards to be followed and tests carried out for the related product to meet acceptance criteria.

Specifications are normally drafted by the client to suit the need for a particular work, for the purpose of construction of water projects standard specifications have been prepared for various woks as follows:

- Standard Specifications for Civil Works,
- Standard Specifications for Electrical works,
- Standard Specifications for Mechanical works and
- General Specifications.

These documents can be downloaded from Ministry's Website customized to fit the needs of particular works.

## 12.3 MATERIALS

### 12.3.1 Building Materials

Building materialis any material used for construction purposes such as wood, cement, aggregates, metals,sand, bricks, steel, concrete, and clay. These are the most common types of building materials used in construction.The choice of these is based on theircost effectiveness for building projects. Appropriate materials help to guard the designed facilities against wear and tear, corrosive compounds or chemicals.

### 12.3.2 Materials Testing

Before materials are used for the construction of works, it is imperative for the contractor or the PE to conduct appropriate tests as per applicable standards indicated above. The following are the minimum tests proposed which should be conducted on various construction materials.

#### 12.3.2.1 Aggregates

The test of aggregates explained below includes both fine and coarse aggregates.

**Flakiness indextest**

Flaky particles are those whose least dimension is 0.6 times lesser than the mean size. The thickness of these particles iscomparatively smaller than for the other two dimensions.

The maximum allowable limit of the flaky particles in amix is 30%. If it exceeds this value then the mix is considered unsuitable for construction purpose.

The flakiness index is the percentage by weight of flaky particles in a sample. The flakiness index is calculated by expressing the weight of flaky particles as a percentage of the total weight of the sample, test procedure is as outlined in BS – 812,1995.

**Elongation index test**

Elongated particles are particles having length considerably larger than the other two dimensions. One such dimension is 1.8 times greater than the other two dimensions. The maximum allowable limit of the flaky particles in the mix is 30%. If it exceeds this value then the mix is considered unsuitable for construction purposes.

An elongation index is the percentage by weight of elongated particles in a sample. The elongated Index is calculated by expressing the weight of elongated particles as a percentage of the total weight of the sample, test method is expalined in BS – 812,1995

Flaky and elongated particles lower the workability of concrete mixes due to the high ratio of surface area to volume. The presence of flaky and elongated particles may also cause inherent weakness in concrete with possibilities of breaking down under heavy loads.

**Abrasion (Los Angeles Abrasion Test)**

Abrasion test is the measure of aggregate toughness and abrasion resistance on crushing, degradation and disintegration. A test for abrasion is conducted based on BS 812: Part 113: 1990.

**Organic impurities test**

Sand should be checked for the presence of organic impurities such as decayed vegetation, humus, and coal dust as these affect the quality of concrete. Tests for organic impurities should be conducted as per. BS 812: Part 4: 1976.

**Aggregate crushing value (ACV) test**

Aggregate crushing value test on coarse aggregates is a relative measure of the resistance of an aggregate crushing under gradually applied compressive load. The method for determination of Aggregate Crushing Value (ACV) is the Code: BS 812 Part 110, 10% finer test.

The 10 per cent Fines Aggregate Crushing Value (**10 % FACT**) is determined by measuring the load required to crush a prepared aggregate sample to give 10 per cent material passing a specified sieve after crushing. Test procedure is outline as per code BS 812: 1990 Part III.

**Impact resistance value (AIC) test**

The aggregate impact resistance value is a measure of resistance to sudden impact or shock, this value may differ from resistance to gradually applied compressive load. The procedure of Aggregate impact resistance value is provided in code BS 812 : Part 112 : 1990.

**Grading – sieve analysis test**

This is classification of a coarse-grained soil based on the different particle sizes it contains. This aspect is important as it indicates the compressibility properties, shear strength and hydraulic conductivity. The standard gradation and sieve analysis test is: BS 812: Section 103.1: Sieve Analysis of Fine and Coarse Aggregates

**Absorption test**

Water absorption is a measure of the porosity of the aggregates. It gives an indication of the strength of the aggregates. When more water is absorbed, the aggregates is more porous in nature and generally considered unsuitable unless found to be acceptable based on strength, impact and hardness tests. The standard method for Testing aggregates to water absorption test is according toBS 812-120:1989.

**Specific gravity test**

The specific gravity of aggregate is the ratio of its mass to that of an equal volume of distilled water at a specified temperature. The standard method for Testing aggregates to determine the density is BS 812 : Part 2 : 1995.

**Chemical content (pH, Chloride and Sulphate)Test**

This test aims at establishing permissible levells of chlorides and sulfates in aggregate, high levels of chemicals may result in deterioration of concrete by corrosion ofsteel reinforcement.Corrosion of steel affects serviceabilityand strength of concrete structures. The test to determine the content of chemicals in aggregates is conducted as per BS 812-Part 117 & 118:1988.

## 12.3.2.2 Water

**Impurities test**

Water for washing aggregates and for mixing concrete shall be in accordance with DIN 4030 and DIN 1045 and shall be clean and free from objectionable quantities of organic matter, alkali, salts and other impurities. When TBS adopts the new standards that are under review as of now, the Tanzanian standards will be adopted.

**Chemical content such as chloride, ph values, sulphate**

Samples of the water being used or which is proposed to be used for mixing concrete shall undergo testing for quality to determine the concentration of sulphates and chlorides, which shall be such that the concrete mix as a whole complies with the specified limit for salt content. Chemical content in water may be determined through procedure explained in the code APHA 21st:2005 / ICP OES.

## 12.3.2.3 Cement

**Setting time test**

The settling time is the time required for cement to convert from a plastic paste to a non-plastic and rigid mass. The cement settling time is determined through procedures explained in the AASHTO T 131 and ASTM C 191: Time of Setting of Hydraulic Cement.

**Compressive strength test**

The compressive strength of cement is the measure of the strength it provides to the mix after it has hardened .The test enables one to identify the quantity of cement required and how much strength it will provide. The compressive strength of cement is a basic data needed for mix design. Cement, basically identified by its compressive strength as grade 53 grade, 43 grade, 33grade of cement. The test procedure to is as per code of practice BS EN 196-1:2005.

### 12.3.2.4 Concrete Works

Tests conducted for concrete includes:

**Slump test**

Concrete slump test or slump cone test is done to determine the workability or consistency of concrete mix prepared in the laboratory or at the construction site when undertakingconcreting. Concrete slump test should be carried out from batch to batch to check the uniform quality of concrete during construction. The slump test is carried out as per procedures mentioned in ASTM C143 in the United States, and EN 12350-2 in Europe.

**Compressive strenght test:**

Compressive strength of concrete is the measure of the Compressive strength which is the ability of the material or the structure to carry the loads on its surface without any cracks or deflection. Standard testing methods for Compressive Strength of Cylindrical Concrete Specimens is carried out using the procedure as stated in the American Society for Testing Materials ASTM C39/C39 M.

**Concrete voids test**

This test method is related to the susceptibility of the cement paste portion of the concrete to damage by freezing and thawing. The test estimates the likelihood of damage of concrete due to cyclic freezing and thawing. The parameters of the air-void system of hardened concrete can be determined by the procedures described in the code AASHTOT 269.

### 12.3.2.5 Steel

**Tensile strength**

The tensile strength of steel is the measure of maximum amount of stress that can be taken before failure. Tensile strength should be conducted as per standards methods as provided in code of practise DIN 15018 or BS449 (1978/1984).

## 12.4 SOIL TEST

### 12.4.1 Methodology of Conducting Soil Investigation for Borehole and Test Pit

There are several methods of soil investigation for engineering purposes.

**(a) Borehole drilling method**

This method used to collect soil materials from underneath by using Rig machine,heavy duty hand auger and light set hand auger.The diameter of the borehole ranges from 150 mm to 180 mm depending on the size of the bit used.During drilling, the following should be observed:

(i) Visual soil classification and texture should be recorded,
(ii) Water table should be observed and recorded,
(iii) Depth should include the collection of soil material at different depths dependingon changes in soil materials to be recorded.
(iv) Labelling of the soil collected at each depth and include its clasification
(v) Collection of soil materials and keep in plastic bags to avoid moisture content
(vi) Soil logging or profile should be shown as in the table below.

**Table 12.2: Soil Logging of Borehole or Test Pit**

| BH NO | SNO | Depth (M) | Soil Description | Remarks |
|---|---|---|---|---|
| **1** | 1 | 0.0-0.50 | Yellowish brown silt clay sand of medium plasticity | Co-ordinate |

**(b) Test pit excavation method**

This method includesthe digging of test pits with the following dimensions, 1.0 m x 1.5 m to the maximum depth of 2.0 m.The depth depends on hardness of the soil materials or hard rock.The equipment used for this method can be backhoe excavator, hoes and chisels.The collection of soil materials depends on the type of materials as mentioned for the borehole above including soil logging table.

## 12.4.2 Soil Investigation for Dams

In dam construction projects soil investigation, collection and clasification should be doneas in the following sections,

### a) Along dam axis

The bore holes or test pits should be conducted along the dam axis at intervals of 20 m or 30 m

### b) Reservor

The soil shoud be collected in left hand and right hand side towards the fetch area (windward side) at intervals of 30 m.The depth of these Boreholes (BH) and Test Pits (TP) depends on the soil strata and the presence of impervious materials (Clay) and hard rock.

### c) Spillway section

The Test pit or boreholes should be conducted along the spillway chanel to determine the soil materials if it is erodedmaterials or it is a rock (soft or hard rock) at the interval of 30 m.

### d) Borrow area section

The borehole or test pit should be conducted on borrow area to determine the suitable soil materials for dam construction and to quantify the amount of materials to be used for dam construction.

## 12.4.3 Suitability of Soil Materials for Dam Construction

The soil materials suitable for earthen dam construction are of two types:

- Silty clay sand of medium plasticity, and
- Silty clay sand of high plasticity.

If the materials at the site are predominantly found to have a lot of silty clay sand of medium plasticity, the dam will be homogeneous which means the same character of soil materials will be used. This should contain between 20 to 30 percent clay with balance made up of Silt, sand and some gravel. If a Unified Soil Classification has to be carried out, the selected clays should be in order of presence of, GC, CS, CL, and CH. Normally, homogeneous dams are confined to relatively small heights with maximum of 6m.For dams in excess of 6metres, a zoned embankment is recommended. But if the site has plenty of silty sand clay of high plasticity, the dam will be zoned and both type of soil materials will be used whereby the high clay materials will form the inner part (core) while the medium will be the outer part (shell materials).The above statement depends on the availability of the materials at the selected site. The zoned embankment is more stable.

## 12.4.4 Determination of In-Situ Bearing Capacity of the Soil

### (a) In-situ bearing capacity

The in-situ bearing capacity of the soil should be determined in order to know the bearing capacity of the soil before implementation of the civil structure and can be used for preliminary design of the structure.

There are several methods which are used to conduct in-situ bearing capacity of the soils

#### (i) Standard penetration test (SPT)

This method is done by drilling boreholes and conducting the SPT tests at the interval of 1.5 m downwards. The number of blows is applied by using a free-falling hammer of 63.5 kg at the height of 750 mm through the rod which is mounted with the cone of $60^{o}$ at the tip end.

By means of a drop hammer of 63.5 kg mass falling through a height of 750 mm at the rate of 30 blows per minute, the sampler is driven into the soil. This is as per IS -2131:1963 (Indian standards). The number of blows of hammer required to drive a depth of 150mm is counted. The main aim is to perform standard penetration in order to obtain the penetration resistance (N-value).The number or blows required for 300 mm penetration resistance of the soil. It is generally referred to as the 'N' value and is measured in blows/unit penetration. The Standard penetration test (SPT) is widely used to get the bearing capacity of the soil directly at a certain depth. The standard penetration test is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.

The main purpose of the test is to provide an indication of the relative density of granular deposits, such as sands and gravels from which it is virtually impossible to obtain undisturbed samples. The operation entails the operator counting the number of hammer strikes it takes to drive the sample tube 6 inches at a time. Each test drives the sample tube up to 450mm deep. It is then extracted and if desired a sample of the soil is pulled from the tube. The borehole is drilled deeper and the test is repeated'

**(ii) Dynamic cone penetration (DCP)**

This method is the same as above but the dropping hammer is 10 kg at a height of 1 m.

Table No 12.3 shows indicators for penetration resistance of different types of soils based on the standard penetration tests in relation to the number of blows per 30 cm of penetration (based on Terzaghi and Peck).The table shows relative densities of soils corresponding to the number of blows.

**Table 12.3: Number of Blows Comparison to Soil Type Based on SPT**

| Sand | | Clay | |
|---|---|---|---|
| **No. of blows per 30 cm (N)** | **Relative density** | **No. of blows per 30 cm (N)** | **Consistency** |
| 0 – 4 | Very loose | Below 2 | Very soft |
| 4 – 10 | Loose | 2 – 4 | Soft |
| 10 – 30 | Medium | 4 – 8 | Medium |
| 30 – 50 | Dense | 8 -15 | Stiff |
| 50 - over | Very dense | 15 – 30 | Very stiff |
| | | **Over 30** | Hard |

## (b) Laboratory Soil Test for Civil Engineering purposes

In order to determine the suitable classification of soil materials for dam construction, the following laboratory tests should be conducted in a soil laboratory.

## (c) The Tests to be conducted for dam construction:

There are many tests which should be done before project implementation which gives the soil character of the selected area. These are classified as soil mechanical and chemical test.

## (d) Indicator test or preliminary

In order to determine the soil classification for civil structures, one should know the type of soil and its strength where the structure will be built by conducting the following laboratory tests;

- Sieve analysis or particle size distribution of the soil materials by percentage. This will determine the percentage of Gravel, Sand, Silt and Clay.
- Atterberg limits this will determine the percentage of plasticity of the soil that is Liquid limit, Plastic limit and Plastic index.
- Bulk density this will determine the unit weight of the soil in $g/cm^3$
- Natural moisture content (NMC) this will determine the percentage of water content of soil before using the materials.
- Linear shrinkage this will determine the expansion and shrinking of the soil material in percentage.
- Free swell of the soil this is expressed in percentage to determine the swelling of the soil when subjected to the water.
- Specific gravity of soil particles this used for calculations of voids in soil
- Permeability test this indicate the water percolation in soil particle which depends the particle size of the soil particles arrangement and compaction.

## (e) Strength test or Secondary test.

Shear test, this will determine the cohesion and angle of response of the soil for ultimate bearing calculations

Consolidation test, this will determine the coefficient of volume compression (Mv) and compression index (Cc) of the soil for settlement calculation in soil foundation design.

- Triaxial test is more advanced than shear test, it is three force acting on soil sample during the test .This will determine the cohesion of the soil and an angle of friction for the drained and un-drained soil.
- Proctor test. This will determine the maximum dry density and Optimum moisture content- The parameters will be used for determination of percentage of compaction during the dam, road and Airfield construction. The filling of layer thickness should be 300 mm before compaction.

## (f) Chemical tests for dispersion of soil materials used for dam construction

The Total Exchangeable bases such as (Magnesium ($Mg^+$), Sodium ($Na^+$), Calcium ($Ca^+$), and potassium ($K^+$) is the one of the causes of soil dispersion. Generally, the causes of soil dispersion increase with the Sodium percentage. If $Na^+$ of the Total Exchangeable bases is more than 15%, the soil will be dispersed.

Soil Texture may contribute to formation of dispersive soils if silt % is greater than 60%.In this case, the hydrometer or sedimentation test should be conducted to determine the percentage of silt and clay separately.

### (g) The tests of soil chemical for concrete

The soil must be tested for Chemical concerns concrete works such as content of Sulphate, pH value, Sodium chloride and impurities which lower the strength of concrete during hardening.

## 12.5 OCCUPATIONAL HEALTH AND SAFETY

In all water supply projects implemented by any funding sources the working environment at the sites will have to comply with the Occupational Health and Safety(OHS) Act (2003), OSH Policy (2010), OSH rules of 2015, 2017, 2018 and any subsequent amendments. These can be accessed on the MoW Website - http://www.maji.go.tz/.

## REFERENCES

https://www.academia.edu/37842758/Determination_of_the_Flakiness_and_the_Elongation_Index_for_the_Given_Aggregate_Samplehttp://www.raklab.com/index.php/testing-services/chemicalhttp://www.concrete.org.uk/fingertips-

http://www.concrete.org.uk/fingertips-nuggets.asp?cmd=display&id=910

https://civiconcepts.com/2019/01/los-angeles-abrasion-test-on-aggregate/

Chapter 13

# ROLE OF STAKEHOLDERS IN THE DESIGN OF WATER SUPPLY PROJECTS

## 13.1 TYPES OF STAKEHOLDERS

Stakeholders (any person, group, or organization) in water supply and sanitation initiatives may take varying roles depending on the levels of an activity and envisioned interest of a given partner. Villagers/community, Water organizations, NGOs as well as governmental and international institutions have great responsibilities in the promotion and enhancement of the planning and designing the projects as well as supporting the construction and operation of the water supply and sanitation schemes. Table 5.1 illustrates different types and roles of stakeholders in the design of water supply and sanitation projects including guaranteeing sustainability of WASH services expected to be provided as guides by WASH guidelines for school and health (URT, 2016; URT, 2017) .

## 13.2 ROLES OF STAKEHOLDERS

**Table 13.1: Different Roles of Stakeholders in Project Planning and Design Stage**

| S/N | Stakeholder | Role Expected at Planning and Design Stages |
|---|---|---|
| 1. | Community and general public | • Participate in water sources identification, and collection of local based socio-economic data and information<br>• Participate in determining the factors likely to facilitate the establishment and administration of water and sanitation service prices/charges.<br>• Identify mechanism for establishing and enhancing accountability in water supply and sanitation schemes operation. |
| 2. | Urban and Local Government Authorities | • Coordinate physical planning with the water<br>• authorities and community organizations taking into consideration the community needs and local situations.<br>• Assist in choosing friendly water supply and sanitation technologies appropriate to local areas.<br>• Set strategies for the mobilization of resources for project investment. |

| S/N | Stakeholder | Role Expected at Planning and Design Stages |
|---|---|---|
| 3. | Basin Water Boards | • Provide water sources data and information including quantity and quality.<br>• Provide standards and guidance on the requirements pertaining infrastructures in different sources. |
| 4. | Water supply and sanitation organizations (including RUWASA and CBWSOs) | • Design strategies for effective involvement of the community members.<br>• Determine community water demands.<br>• Undertake engineering project design accordingly.<br>• Determine factors likely to influence or hinder revenue collection mechanism from the beneficiaries (community members).<br>• Prepare specifications for a given water supply and/ or sanitation project. |
| 5. | Ministry of Water | • Design strategies for effective involvement of the community members.<br>• Effective coordination and collaboration in cross-sectorial issues.<br>• Prepare specifications for a given water supply and/ or sanitation project.<br>• Provide guidelines and manuals on appropriate design of water and sanitation projects. |
| 6. | Service Providers | • Facilitate provision of data and information regarding availability of appropriate materials and technologies for a given water supply and sanitation project. |
| 7. | Academic and research institutions | • Facilitate the Ministry and its organizations in establishing strategies for effective involvement of the community members. |
| 8. | NGOs and CBOs | • Provide socio-economic and technological inputs during planning for water and sanitation projects.<br>• Liaison with Ministry and its organizations in establishing strategies for effective involvement of the community members. |
| 9 | President's Office Local Government and Regional Administration | • By-laws in favour of water utilities<br>• Community mobilization<br>• Land acquisition<br>• Compensation |
| 10 | Ministry of Land and Housing and Human Settlement | • Land acquisition<br>• Compensation for land acquired |
| 11 | Ministry of Transportation and Telecommunications | • Functioning GPS and GIS system in place |
| 12 | Ministry of Agriculture | • Water services protection against pesticides, herbicides and fertilizers |
| 13 | Neighbouring countries | • Water permits from shared water protocols |

## REFERENCES


Chumbula, J.J. (2016): Sustainability of water projects: A case of selected Projects in Iringa District, Tanzania: A dissertation submitted in partial fulfillment of the requirements for the Degree of Master of Arts in Rural Development of Sokoine University of Agriculture. Morogoro, Tanzania.

Kirenga, D. A.T (2019).Analysis of Determinants of Sustainability for Community Managed Rural Water Supply Projects in Tanzania -The case of Moshi District Council, Kilimanjaro Region: A Thesis Submitted in Fulfilment of Requirement for the Award of Doctor of Philosophy (PhD) Degree in Natural Resources Assessment and Management (NARAM) of The University of Dar es Salaam.

The Water Resources Act No. 11 of 2009

The Water Supply and Sanitation No.5 of 2019

Water, sanitation and hygiene (WASH) in schools: results from a process evaluation of the National Sanitation Campaign in Tanzania: Available at https://iwaponline.com/washdev/article/7/1/140/30542/Water-sanitation-and-hygiene-WASH-in-schools. Visited 10th January, 2019

# APPENDICES

# APPENDIX A: CLIMATE CHANGE AND RESILIENCE TO CLIMATE CHANGE

## A.1 Introduction

Climate change is now recognized as one of the defining challenges for the 21st century. More frequent and extreme weather events have resulted in higher incidences of floods and droughts around the planet. The ensuing adverse impacts of climate change on water and sanitation services constitute clear and present dangers for development and health. Ensuring optimal resilience of water and sanitation services in a globally changing climate context will be crucial for maintaining the momentum of making progress in health and development. Climate variability is already a threat to the sustainability of water supplies and sanitation infrastructure.

Floods are "normal" occurrences that continue to cause shocks for the affected population and to challenge water and sanitation managers. In many places they are likely to become more frequent with intensification of climate change, thus;

- Floods can have catastrophic consequences for water and sanitation infrastructure. Such damage can take years to repair.
- On a smaller scale, drinking-water infrastructure can be flooded and put out of commission for days, weeks or months.
- Where flooding of sanitation facilities occurs, there may not only be a break in services, but the resultant flooding may distribute human excreta and its attendant health risks across entire neighbourhoods and communities.

Droughts occur unpredictably, worldwide. In many places they are likely to become more frequent and more widespread with climate change. For example:

- Falling groundwater tables and reduced surface water flows can lead to wells drying up, extending distances that must be travelled to collect water, and increasing water source pollution. In response, drilling rigs – which would otherwise be used to increase access – may be redeployed to renew or replace out-of-service wells, slowing the actual progress in extending access

Since climate change is likely to affect water sources and infrastructure in Tanzania it must therefore be taken into consideration in design operation and maintenance of water and sanitation infrastructure or projects. Globally, climate change studies are coordinated by the United Nations Framework Convention on Climate Change (UNFCCC) and the Intergovernmental Panel on Climate Change (IPCC). Accordingly, designers should use the latest information, data and model predictions available and include a statement on what measures, if any, have been allowed for in order to cope up with the climate change within the time frame of their project design (i.e. design life).

## A.2 Potential Impacts of Climate Change on Water Supply Projects

It is emphasized that immediately after a project is conceived, hydrological, rainfall and other meteorological data collection must be initiated. In addition and given the long design life of such structures, consideration must be given to the possible impacts of climate change on the project.

IPCC (2014) indicated that human influence on the climate system is clear and recent anthropogenic emissions of greenhouse gases are the highest in history. Recent climate changes have had widespread impacts on human and natural systems. Climate change impacts the hydrological cycle resulting into changes in spatio-temporal distribution and magnitude of climatic variables such as temperature and precipitation. Changes in precipitation combined with rising temperatures, may adversely influence the availability of water, streamflow, soil moisture, the occurrence of droughts (Li et al., 2018; Asadieh and Krakauer, 2016) and flow regimes for freshwater ecosystems.

Climate models are needed in order to estimate future climate pattern. These models include a 3-dimensional representation of the atmosphere, land surface, sea, lakes and ice. The atmosphere is divided up into a 3-dimensional grid over and above the earth's surface. In order to obtain a good result, the models have to take the whole atmosphere into consideration, covering the entire surface of the earth as well as up into the air above it. These models are called global climate models (GCMs). The climate model calculations are based on emission scenarios or radiation scenarios. Emission scenarios are assumptions about future emission of greenhouse gases (GHGs), based on estimates of the development of the world economy, population growth, globalisation, increasing use of green technology, etc.

The amount of greenhouse gases that are emitted depends on global evolution. These scenarios are called SRES scenarios (Special Report on Emission Scenarios (Nakićenović, 2000)).Radiation scenarios are based on assumptions about how the greenhouse effect will increase in the future, known as radiative forcing (measured in $W/m^2$). If there is an increased emission of greenhouse gases, then there will be more radiative forcing. These scenarios are called RCP scenarios (Representative Concentration Pathways (Moss et al., 2010)).

The following are the sources of emissions with the energy sector/production remaining the primary driver of GHG emissions: energy sector (35%), agriculture, forests and other land uses (24%), industry (21%), transport (14%) and building sector (6.4%) as per 2010 GHG emissions (IPCC, 2014). Projections indicate that continued emissions of greenhouse gases will cause further warming and changes in the climate system. The following are among the potential impacts of climate change: food and water shortages, increased displacement of people, increased poverty and coastal flooding.

During climate negotiations in Cancún, Mexico 2010 there was an agreement in the ambition to limit increases in global average temperature to below 2 degrees compared to pre-industrial (1881-1910) levels. A temperature increase of more than 2 degrees is a limit that is considered too costly on society and environment (e.g. IPCC, 2007; UNFCC, 2010), but still possible to be below (IPCC, 2014). In 2015 the countries of the world agreed on the so called Paris Agreement. This states that the global temperature rise should be kept well under 2 degrees and that efforts should be made to limit the temperature increase to even a further low to 1.5 degrees above the pre-industrial levels.

Since the global temperature increase of 1.5 or 2 degrees are just averages it is interesting to look at the temperature increase at regional and local scales in Tanzania. Note that in the maps showing the patterns, future warming is compared to the period 1971-2000. Some of the warming occurred before 1971. To be precise, the global average temperature had already increased by 0.46°C from pre-industrial time until 1971. A warming of 2 degrees compared to pre-industrial levels corresponds to a warming of 1.54°C compared to 1971-2000 (Vautard et al., 2014).

Apart from temperature and precipitation, a number of climate indices are also calculated, with the help of the general meteorological parameters generated by the model. This could be the number of warm or cold days, accumulated weekly precipitation or the length of the vegetation period. This climate change analysis uses four scenarios generally as used by IPCC, AR5:

- RCP2.6: Powerful climate politics cause greenhouse gas emissions to peak in 2020. The radiative forcing will reach 2.6 W/m² by the year 2100. This scenario is closest to the ambition of the Paris Agreement.
- RCP4.5: Strategies for reducing greenhouse gas emissions cause radiative forcing to stabilise at 4.5 W/m² before the year 2100.
- RCP8.5: Increased greenhouse gas emissions mean that radiative forcing will reach 8.5 W/m² by the year 2100. This scenario is closest to the currently measured trends in greenhouse gas concentrations.

The reference period 1961-1990 was widely used to define the current climate. New observations are compared to the mean value for 1961-1990 to measure how they differ. For example, if a summer is warmer than normal, it means that it is warmer than the average value of the summers of 1961-1990. The World Meteorological Organization, WMO, defines the reference periods, and the next reference period will be 1991-2020 which will start being used in 2021.Climate scenarios are often presented as changes compared to the current climate. Often the reference period 1961-1990 is used, just as for observations. Since climate is changing, the period 1961-1990 is not fully representative of what we consider to be the current climate. Therefore, later reference periods have started to be used, and many projects are now using the years 1971-2000.

An ensemble is a collection of climate scenarios (estimates of the future climate) where the individual scenarios are different from each other. The climate scenarios can for example differ with respect to the climate model used, or the emission or radiation scenario. A climate scenario that is part of an ensemble is called a member. An ensemble gives a good overview of the spread of the difference between the members, and highlights some of the uncertainties associated with simulating the future climate. The ensemble is a measure of the reliability of the results. If many different climate scenarios give similar results, then the results are relatively more reliable than if they all pointed in different directions.

A global climate model can perform well in some parts of the world and less well in other areas. Another model describes temperature patterns but is not as good for precipitation. It can therefore, be worth using large ensembles since they are better at capturing the uncertainty of the results. In practice, the choice of the ensemble run depends very much on how many model simulations can feasibly be run. When an ensemble run has been carried out, the spread of the result gives an idea about the reliability of the results. Depending on the type of ensemble that has been produced, the significance of the choice of climate models and start values can be studied.

Future increases in precipitation extremes related to monsoons is very likely in Africa. Monsoons are the most important mode of seasonal climate variation in the tropics (i.e. tropical continents: Asia, Australia, the Americas and Africa), and are responsible for a large fraction of the annual rainfall in many regions. In Africa, monsoon circulation affects precipitation in West Africa where notable upper air flow reversals are observed. East and South African precipitation is generally described by variations in the tropical convergence zone rather than as a monsoon feature.

The African continent encompasses a variety of climatic zones. The continent is divided into four major sub-regions: Sahara (SAH), Western Africa (WAF), Eastern Africa (EAF) and Southern Africa (SAF).Tropical cyclones impact East African and Madagascan coastal regions and extra-tropical cyclones (ETCs) clearly impact Southern Africa. East Africa experiences a semi-annual rainfall cycle, driven by the Inter-tropical Convergence Zone (ITCZ) movement across the equator. Direct links between the region’s rainfall and El Nino-Southern Oscillation (ENSO) have been demonstrated (Giannini et al., 2008), but variations in Indian Ocean Sea Surface Temperature (SST, phases of the Indian Ocean Dipole - IOD) are recognized as the dominant driver of East African rainfall variability (Marchant et al., 2007). Indian Ocean Dipole (IOD) is defined by the difference in sea surface temperature between two areas (or poles, hence a dipole), leading to (i) a western pole in the Arabian Sea (Western Indian Ocean), (ii) an Eastern pole in the Eastern Indian Ocean South of Indonesia. Variability in Southern Africa’s climate is strongly influenced by its adjacent oceans (Rouault et al., 2003; Hansingo and Reason, 2008, 2009; Hermes and Reason, 2009) as well as by ENSO (Vigaud et al., 2009; Pohl et al., 2010).

The IPCC (2013) for CMIP5 projections under RCP4.5 indicate that for the East African region the predicted increase in temperature is between 0.5°C and 1.2°C, 1.0°C and 2.4°C and 1.0°C and 3.1°C for 2035, 2065 and 2100 respectively annually. The predicted increase in precipitation is between -5% and 10%, -6% and 17% and -7% and 21% for 2035, 2065 and 2100 respectively annually. This implies that the East African region will get more rain but become drier as temperatures rise and evapo-transpiration increases.

Therefore, water supply and sanitation designs should anticipate the potential impacts on water sources and infrastructure, and therefore inform on water resilience or water security aspects during the project design life. Also, seasonal precipitation change (mm) in East Africa for 2011-2040 (2020s), 2050s (2041–2070) and 2080s (2071–2100) with the baseline period 1961–1990 as predicted by the GCMs with Statistical Downscaling Model (SDSM) indicated that in Tanzania, heavy rains (*Masika*, in MAM) will get smaller, up to -500 mm and light rains (*Vuli*, in OND) will increase a little for all RCPs 2.6, 4.5 and 8.5 (Gebrechorkos, et al., 2019).

# APPENDIX B: MINIMUM WATER FLOW REQUIREMENT FOR FIRE FIGHTING

<table>
<tr><th>Category</th><th colspan="2">Description</th><th>Minimum Flow in L/s</th></tr>
<tr><td rowspan="2">Housing</td><td colspan="2">Housing developments with detached units or semidetached houses of not more than two floors should have a water supply capable of delivering a minimum through any single hydrant of.</td><td>8</td></tr>
<tr><td colspan="2">Multi occupied housing developments with units of more than two floors should have a water supply capable of delivering a minimum through any single hydrant.</td><td>20 - 35</td></tr>
<tr><td>Transportation</td><td>Lorry/coach parks, multi-storey car parks and service stations</td><td>All of these amenities should have a water supply capable of delivering a minimum through any single hydrant on the development or within a vehicular distance of 90 metres from the complex.</td><td>25</td></tr>
<tr><td rowspan="4">Industry</td><td rowspan="4">In order that an adequate supply of water is available for use by a Fire Authority in case of fire it is recommended that the water supply infrastructure to any industrial estate is as follows with the mains network on site being normally not less than DN150</td><td>Up to one hectare</td><td>20</td></tr>
<tr><td>One to two hectares</td><td>35</td></tr>
<tr><td>Two to three hectares</td><td>50</td></tr>
<tr><td>Over three hectares</td><td>75</td></tr>
<tr><td>Shopping, offices, recreation and tourism</td><td colspan="2">Commercial developments of this type should have a water supply capable of delivering a minimum flow to the development site of between:</td><td>20 - 75</td></tr>
<tr><td>Education, health and community facilities</td><td>Village and small community halls</td><td>Should have a water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 100 metres from the complex.</td><td>15</td></tr>
</table>

| Category | Description | | Minimum Flow in L/s |
|---|---|---|---|
| | Primary schools and single storey health centres | Should have a water supply capable of delivering a minimum flow of through any single hydrant on the development or within a vehicular distance of 70 metres from the complex. | 20 |
| | Secondary schools, colleges, large health and community facilities | Should have a water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 70 metres from the complex. | 35 |

# APPENDIX C: EXAMPLE ON ESTIMATING THE CAPACITY OF A STORAGE TANK

Estimate the minimum storage tank capacity for a solar powered system assuming 40L/Min for 6.5 hours per day, a daily water demand of 15,000L and the, morning evening peak pattern from shown on table below:

**Table C.1: Common Daily Percentage Consumption Patterns**

| Time Period | Morning /Evening Peak* | Mid-day Peak* | Water Mission |
|---|---|---|---|
| 5.00 | 0% | 5% | 5% |
| 6.00 | 15% | 5% | 5% |
| 7.00 | 15% | 10% | 10% |
| 8.00 | 5% | 5% | 10% |
| 9.00 | 5% | 5% | 5% |
| 10.00 | 5% | 5% | 10% |
| 11.00 | 5% | 20% | 5% |
| 12.00 | 5% | 15% | 5% |
| 13.00 | 5% | 5% | 5% |
| 14.00 | 5% | 5% | 5% |
| 15.00 | 5% | 5% | 10% |
| 16.00 | 15% | 5% | 10% |
| 17.00 | 15% | 5% | 5% |
| 18.00 | 0% | 5% | 5% |

* Data from A /Hand book of Gravity Flow Water systems (Jordan)

### Step 1: Determine the daily water demand pattern

The first step is to determine how much water will be used at different times during the day in a specific community. This can be accomplished by observing the community water collection habits. Water Demand for each time period can be calculated based on percentages obtained in the community water collecting habits. The balance is determined by subtracting the demand from the supply and adding it to the current surplus/deficiency balance.

**Table C.2: Determining Supply Deficiencies**

| Time period | Percentage of Daily Use | Supply (litres) | Demand (Litres) | Surplus/ Deficiency (Litres) | Balance (Litres) |
|---|---|---|---|---|---|
| 5.00 | 0% | 0 | 0 | 0 | 0 |
| 6.00 | 15% | 0 | 2,250 | -2,250 | -2,250 |
| 7.00 | 15% | 0 | 2,250 | -2,250 | -4,500 |
| 8.00 | 5% | 0 | 750 | -750 | -5,250 |
| 9.00 | 5% | 600 | 750 | -150 | -5,400 |
| 10.00 | 5% | 2,400 | 750 | 1,650 | -3,750 |
| 11.00 | 5% | 2,400 | 750 | 1,650 | -2,100 |
| 12.00 | 5% | 2,400 | 750 | 1,650 | -450 |
| 13.00 | 5% | 24,00 | 750 | 1,650 | 1,200 |
| 14.00 | 5% | 2,400 | 750 | 1,650 | 2,850 |
| 15.00 | 5% | 2,400 | 750 | 1,650 | 4,500 |
| 16.00 | 15% | 600 | 2,250 | -1,650 | 2,850 |
| 17.00 | 15% | 0 | 2,250 | -2,250 | 600 |
| 18.00 | 0% | 0 | 0 | 0 | 600 |

### Step 2: Determine the minimum tank capacity

To ensure sufficient water to a community, there must be enough water to last until the trough of the next day. This calculation is represented as;

$$V_{tank} = V_{max} - V_{min} - V_{final}$$

$$V_{tank} = 4{,}500L - (-5{,}400L) - 600L = 9{,}300L$$

The water storage tanks should be able to hold at least 9,300 L to suffice the need of the community. The designer should use 10,000 L storage tanks or use two 5,000 L storage tanks in sizing of the needed capacity.

# APPENDIX D: STRUCTURAL DESIGN OF CONCRETE

Concrete members used in water structures range from walls, slabs, roofs and floors. The design of reinforced concrete should ensure these structures have sufficient resistance to cracking, adequate strength and does not allow leakage. Three methods or approaches are available for structural design of concrete, these are described below:

**Working Stress Method:**

- Produces uneconomical sections,
- Produces stable sections.

**Ultimate Load Method:**

- Produces cheaper sections,
- Produces unstable sections.

**Limit State Method:**

- Produces economical sections,
- Produces stable sections.

For the design of water structures it is recommended to use Limit State Design Method, design of structural members should consider two design limits;

**Limit of Collapse Design:**

- Take care of safety of structure
- Deals with all types of forces, shear force, bending Moment, torsion moment,
- Design criteria refers to the resistance offered by structure which should not be less than the limit value set in design code,
- The appropriate loading value in the structure is based on loading combination of dead loads, live loads, wind load and earth quake load as provided in BS 8110 code.

**Limit of serviceability Design:**

- Take care of control, deflection, cracking, abrasion and corrosion,
- The calculated values of deflection shall be less than the permissible values of deflection,

## D.1 Design Requirements and Safety Factors

Design requirements for water structures should be according to BS 8110 but modified for the limits state of cracking to take care of crack width under the effect of applied loads, temperature and moisture content. The details of design requirements and partial safety factors are as per BS 8007 shown below.

## D.2 Criteria for Sizing of Concrete Slabs and Walls

**Slabs Resting on Firm Ground:**

- Concrete slabs casted to rest directly over firm ground should be designed with nominal percentage of reinforcement provided that it is certain that the ground will carry the load without appreciable subsidence in any part.
- Concrete slabs should be cast in panels with sides not more than 4.5 m with contraction or expansion joints between.
- A screed or concrete layer less than 75mm thick should first be placed on the ground and covered with a sliding layer of bitumen paper or other suitable material to destroy the bond between the screed and floor slab.
- In normal circumstances the screed layer should be of grade not weaker than grade 10, where injurious soils or aggressive water are expected, the screed layer should be of grade not weaker than grade 15 and if necessary a sulphate resisting or other special cement should be used.

**Slabs Resting on Support:**

- When structures are supported on walls or other similar supports the slabs should be designed as floor in buildings for bending moments due to water load and self-weight.
- When the slab is rigidly connected to the walls (as is generally the case) the bending moments at the junction between the walls and slab should be taken into account in the design of slab together with any direct forces transferred to the slab from the walls or from the slab to the wall due to suspension of the slab from the wall.
- If the walls are non-monolithic with the slab, such as in cases, where movement joints have been provided between the slabs and walls, the slab should be designed only for the vertical loads.
- In continuous T-beams and L-beams with ribs on the side remote from the liquid, the tension in concrete on the liquid side at the face of the supports should not exceed the permissible stresses for controlling cracks in concrete. The width of the slab is determined in usual manner for calculation of the resistance to cracking of T-beam, L-beam sections at supports as given in BS8110 design code.

**Circular Tanks with sliding joint at base:**

- If the wall of a cylindrical tank has a sliding joint at the base and is free at the top, then when the tank is full, no radial shear or vertical bending occurs, the tank wall will be subjected to pure circumferential tension with a varying magnitude whereby at bottom there is maximum value and at top there is zero value,
- The design of the tank wall should be done by determining the width of the tank wall *t* and the area of reinforcement $A_{st}$ required to resist the circumferential tension only,

- The varying value of circumferential tension per unit height T at depth z below the top and area of steel required to resist circumferential tension and thickness of wall are given by equations below:

$$T = \gamma r z$$

$$A_{st} = \frac{T}{\sigma_{st}}$$

Where:

$r$ is the internal radius of the tank ,

$\gamma$ is unit weight of the liquid,

$z$ is the depth below the top of tank,

$A_{st}$ is the area of circumferential tension steel,

$\sigma_{st}$ is the permissible tension strength of steel,

$T$ is circumferential tension (Hoops tension),

**Circular Tank with fixed joint at base:**

- If the wall of the Tank is supported at the base such that no radial movement occurs; then the wall will be subjected to radial shear, vertical bending and circumferential tension, the value of circumferential tension is always zero at the bottom of the wall,
- The assumption should be made that some portion of the wall at base acts as cantilever and thus some load at bottom are taken by the cantilever effect. Load in the top portion is taken by the hoop tension. The cantilever effect depends on the height of the wall,
- The bottom part of the tank wall about $\frac{1}{3}$ of the tank height H, or 1 meter from bottom whichever is greater is acted upon by a cantilever moment,
- For walls with free tops and a bottom that is either fixed or hinged values of circumferential tension, vertical moments and radial shear may be calculated from values of coefficients given in Tables D.2 and D.3,
- The design of the tank wall should be by determining the width of the tank wall $t$ and the area of reinforcement required to resist the circumferential tension, shear forces and bending moments determined in the design of slabs,

**Rectangular Tanks:**

- In the case of rectangular or polygonal walls, the sides act as two-way slabs, whereby the wall is continued or restrained in the horizontal direction, fixed or hinged at the bottom and hinged or free at the top. The walls thus act as thin plates subjected triangular loading and with boundary conditions varying between full restraint and free edge.
- Analysis for moments and shear forces should be done as that of two-ways slabs considering the walls as individual rectangular slab panels under action

## Table D.1: Design Requirements and Partial Safety Factors (BS 8007)

| | | Limit state | Design requirement | Means of compliance |
|---|---|---|---|---|
| Design requirements | Ultimate | Structural stability | Structure, whose resistance is based on the design strengths of materials, should be able to support without collapse, the design effects of specified combinations of design loads. | By calculation in accordance with BS 8110. |
| | Serviceability | Cracking | Design surface crack width due to applied loads, or thermal and shrinkage effects, not greater than:<br>Severe or very severe exposure 0.2 mm<br>Critical aesthetic appearance 0.1 mm | Allowable steel stresses or by calculation using equations in BS 8007. |
| | | Deflection | Final deflection not greater than *l*/250, where *l* is span of member, or length of cantilever (including walls). | Limiting span/effective depth ratios or by calculation. |
| | | Flotation | For structures subject to groundwater pressure, where the groundwater level can be reliably assessed, the deadweight of the empty structure with any anchoring devices should give a minimum factor of safety of 1.1 against uplift forces. | By calculation. |
| | Other considerations | Durability | Structure should perform satisfactorily in the anticipated environment for its intended lifetime (40–60 years), with all embedded metal adequately protected from corrosion. | Minimum concrete strength class and cover according to exposure classification. |
| | | | Exposure classification not less than severe (*Table 3.9*). For extended design life, consideration should be given to increasing the cement content or the cover, or using special reinforcement (galvanized, epoxy-coated, stainless steel). | For severe exposure:<br>concrete grade C35A<br>nominal cover 40 mm.<br>Otherwise, as BS 8110. |

**Partial safety factors**

Load combinations and values of $\gamma_f$ for the ultimate limit state (see Note 1)

| Load combination | Load type | | | | | |
|---|---|---|---|---|---|---|
| | Dead (see Note 2) | | Imposed | | Wind | Earth[a] and water[b] pressure |
| | Adverse | Beneficial | Adverse | Beneficial | | |
| 1. Dead and imposed (with earth and water pressure) | 1.4 | 1.0 | 1.6 | 0 | — | 1.2[a,b]<br>1.0[a] |
| 2. Dead and wind (with earth and water pressure) | 1.4 | 1.0 | — | — | 1.4 | 1.2[a,b]<br>1.0[a] |
| 3. Dead and wind and imposed (with earth and water pressure) | 1.2 | 1.2 | 1.2 | 1.2 | 1.2 | 1.2[a,b]<br>1.0[a] |

Note 1. Values of $\gamma_f$ are those given in BS 8110: Part 1: 1997. In BS 8007, which refers specifically to BS 8110: Part 1: 1985, the value for retained liquid loads is given as 1.4 for load combinations 1 and 2, and 1.2 for load combination 3. For containment structures, liquid levels should be taken to the tops of walls, assuming that liquid outlets are blocked.
Note 2. Earth covering on reservoir roofs may be taken as dead load, but due account should be taken of construction loads from plant and heaped earth, which may exceed the intended design load.

[a] The earth pressure is that obtained from BS 8002, including an appropriate mobilisation factor. The more onerous of the two factored conditions should be taken. The value of $\gamma_f$ should be taken as 1.2, unless an allowance for unplanned excavation in accordance with BS 8002, 3.2.2.2 is included in the calculations, when it may be taken as 1.0.
[b] Where the maximum credible water level can be clearly defined, $\gamma_f$ may be taken as 1.2. Otherwise, use $\gamma_f = 1.4$.

Values of $\gamma_m$ for the ultimate limit state

| Concrete | | | | Reinforcement |
|---|---|---|---|---|
| Compression | Shear | Bond | Bearing etc. | |
| 1.5 | 1.25 | 1.4 | ≥ 1.5 | 1.15 |

Note. For the serviceability limit states, the values of $\gamma_f$ and $\gamma_m$ should be taken as 1.0. For containment structures, the liquid level should be taken to the working top or overflow level, as appropriate to working conditions.

(Source: Reynolds et al., 2008)

of triangularly distributed loads; no need of modification to be applied to continuous walls provided there is no rotation about the vertical edges,

- In plane walls, the liquid pressure is resisted by both vertical and horizontal bending moments. An estimate should be made of the proportion of the pressure resisted by bending moments in the vertical and horizontal planes. The direct horizontal tension caused by the direct pull due to water pressure on the end walls, should be added to that resulting from horizontal bending moments.
- Magnitudes of moments and shear forces may be calculated using values of constants given in Tables 2.53 for square tanks; in rectangular tanks distribution of the unequal fixity moments at the wall junctions is needed, additional tables 2.78,and 2.79 may be used,
- When a tank is empty and acted upon by earth loading, consider fixed end condition at the edge of tank wall,
- The design of the tank wall should be by determining the width of the tank wall t and the area of reinforcement required to resist bending moments and shear forces as determined in the design of slabs
- At the vertical edges where the walls of a reservoir are rigidly joined, horizontal reinforcement and haunch bars should be provided to resist the horizontal bending moments even if the walls are designed to withstand the whole load as vertical beams or cantilever without lateral supports

## D.3 Procedures for Sizing of Concrete Members

**Step 1. Establish dimensions**

Establish the height and diameter/radius of the tank based on the volume:

Diameter, $D=\sqrt{\frac{4V}{3.14*(H-Fb)}}$

Where V=Capacity of Water Tank

H= Height of Water Tank

Fb= Free board of the water tank.

**Step 2. Analysis of forces**

**Sliding joint at base, sliding joint at top slab and resting on firm ground:**

There is only one force acting of the wall of the tank is the Hoop Tension, this should be calculated based from equation: $T = \gamma rz$

Where:

$r$ is the internal radius of the tank ,

$\gamma$ is unit weight of the liquid,

$z$ is the depth below the top of tank

**Rigid joint at base, sliding joint at top slab and resting on firm ground:**

There are three forces acting on the wall of the tank; Hoop tension, bending moment and shear force.

Assume tank section wall thickness t;

Calculate the variable $\frac{H^2}{2_{rt}}$, referring to table D.2 read corresponding values of coefficients $a_n$, $a_m$, $a_v$, for calculating hoop tension, bending moment and shear force on the tank wall.

Calculate these forces using equations below:

Hoop Tension, $T = a_n * \gamma * H * r$

Bending Moment, $M = a_m * \gamma * H^3$

Shear Force, $V = a_v * \gamma * H^2$

Step 3. Calculate area of reinforcement

Sliding joint at base, sliding joint at top slab and resting on firm ground:

Assume a tank section width 1000mm width and section thickness equal to, *t*:

Check the section compliance to Fire Resistance as per table 3.4 BS 8110;1;1997,

Check concrete cover and grade compliance to durability requirement as per table 3.3 BS 8110; 1; 1997.

Calculate the area of steel from the Hoop tension force only as given by equation; $A_{st} = \frac{T}{\sigma_{st}}$

Provide reinforcement horizontally spanning at spacing as provided by section 3.12.11 of BS 8110-1;

Check for section adequacy to resist shear stress; design shear stress , is given by equation

$$v = \frac{V}{b_v d}$$

Where:

V = Shear Force calculated from above calculation,

$v$ = design shear stress,

$b_v$ = breadth of section,

*d* effective depth.

In no case should $v$ exceed $0.87\sqrt{fcu}$ or $5N/mm^2$, where this condition is not satisfied consider change of width of section or provision of reinforcement.

**Rigid joint at base, sliding joint at top slab and resting on firm ground:**

Assume tank section with 1000mm width and section thickness equal to, *t*;

Check the section compliance to Fire Resistance as per table 3.4 BS 8110;1;1997,

Check concrete cover and grade compliance to durability requirement as per table 3.3 BS 8110; 1; 1997.

Assume diameter of main bars $\phi$, calculate effective depth d, by subtracting the from the width of section *t*, the concrete cover and half diameter of bars ($\frac{\phi}{2}$);

$d = t - cover - \frac{\phi}{2}$

Check the condition for section moment M, is not greater than ultimate moment of resistance $M_u$, $Mu = 0.156 fcubd^2 \geq M$ is satisfied, if this condition is not satisfied refer to section 3.4.4.4 of BS 8110-1; 1997 for provision of double reinforcement,

Calculate the area of steel from the following equations:

$$k = \frac{M}{fcu * b * d^2}$$

$$z = \text{d}[0.5 + \sqrt{0.25 - \frac{\text{K}}{0.9}}]$$

$$As = \frac{M}{0.87 * fy * z}$$

Provide reinforcement vertically spanning at spacing as provided by section 3.12.11 of BS 8110-1; 1997

Check for section adequacy to resist shear stress, design shear stress , is given by equation

$$v = \frac{V}{b_v d}$$

Where:

V= Shear Force calculated from above calculation,

$v$ = design shear stress,

$b_v$ = breadth of section,

$d$ = effective depth.

In no case should $v$ exceed $0.87\sqrt{fcu}$ or $5 N/mm^2$, where this condition is not satisfied consider change of width of section or provision of reinforcement,

Calculate area of steel from the Hoop tension force as given by equation;

$$A_{st} = \frac{T}{\sigma_{st}}$$

Provide reinforcement horizontally spanning at spacing as provided by section 3.12.11 of BS 8110-1; 1997,

Where a tank floor slab is resting on firm ground, the weight of water is carried by the ground. Provide floor slab of minimum thickness about 150mm. Minimum reinforcement should be provided as per table 3.25 in section 3.12.5.4 BS 8110-1; 1997.

$As = \frac{0.13*150*1000}{100}$ , for steel of strength 460N/mm$^2$

**Sliding joint at base, sliding joint at top slab and resting on firm ground:**

Assume tank section with 1000mm width and section thickness equal to, *t*;

Check the section compliance to Fire Resistance as per table 3.4 BS 8110;1;1997,

Check concrete cover and grade compliance to durability requirement as per table 3.3 BS 8110; 1; 1997.

Calculate area of steel from the Hoop tension force only as given by equation; $A_{st} = \frac{T}{\sigma_{st}}$

Provide reinforcement horizontally spanning at spacing as provided by section 3.12.11 of BS 8110-1;

Check for section adequacy resist shear stress design shear stress , is given by equation

$v = \frac{V}{b_v d}$ Where;

V= Shear Force calculated from above calculation,

$v$ = design shear stress,

$b_v$ = breadth of section,

$d$ = effective depth

In no case should $v$ exceed $0.87\sqrt{fcu}$ or 5N/mm$^2$, where this condition is not satisfied consider change of width of section or provision of reinforcement,

Where a tank floor slab is resting on firm ground, the weight of water is carried by the ground. Provide floor slab of minimum thickness about 150mm. Minimum reinforcement should be provided as per table 3.25 in section 3.12.5.4 BS 8110-1; 1997.

$As = \frac{0.13*150*1000}{100}$ , for steel of strength 460N/mm$^2$, for the case where bearing pressure of ground cannot support the weight of tank, design of foundation should be carried out as per BS8110.

**Joints**

To avoid the possibility of sympathetic cracking it is important to ensure that movement joints in the roof correspond with those in the walls, if roof and walls are monolithic.

However, provision should be made by means of a sliding joint for movement between the roof and the wall, Correspondence of joints is not so important. Moreover, in case of tanks intended for the storage of water for domestic purpose, the roof must be made water-tight. This may be achieved by limiting the stresses for the rest of the tank, or by the use of the covering by a waterproof membrane or by providing slopes to ensure adequate drainage.

## D.4 Units costs

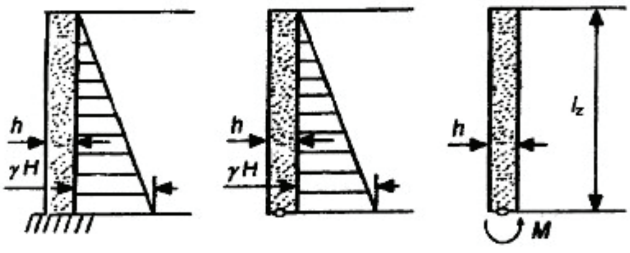
All standard drawings have been provided with their respective BoQ to enable accurate establishment of unit costs of the structures. These can be seen on each of the drawings provided that can be accessed on the MoW Website as Design Manual (DCOM) link, also can be accessed directly through the address: design.maji.go.tz

## Table D.2: Cylindrical Tanks: Elastic Analysis 1

Coefficients for circumferential tensions, vertical moments and radial shears in wall of constant thickness

| Load case | $\alpha$ | $z/l_z$ | Values of coefficient $\alpha$ for values of $l_z^2/2rh$: 2 | 3 | 4 | 5 | 6 | 8 | 10 | 12 | 16 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (1) Triangular load (fixed base) | $\alpha_{n1}$ | 0 | 0.234 | 0.134 | 0.067 | 0.025 | 0.018 | -0.011 | -0.011 | -0.005 | 0 |
| | | 0.5 | **0.274** | **0.362** | **0.429** | **0.477** | 0.504 | 0.534 | 0.542 | 0.543 | 0.531 |
| | | 0.6 | 0.232 | 0.330 | 0.409 | 0.469 | **0.514** | **0.575** | **0.608** | 0.628 | 0.641 |
| | | 0.7 | 0.172 | 0.262 | 0.334 | 0.398 | 0.447 | 0.530 | 0.589 | **0.633** | **0.687** |
| | | 0.8 | 0.104 | 0.157 | 0.210 | 0.259 | 0.301 | 0.381 | 0.440 | 0.494 | 0.582 |
| | | 0.9 | 0.031 | 0.052 | 0.073 | 0.092 | 0.112 | 0.151 | 0.179 | 0.211 | 0.265 |
| | $\alpha_{m1}$ | 0.6 | **0.0115** | **0.0097** | **0.0077** | **0.0059** | 0.0046 | 0.0028 | 0.0019 | 0.0013 | 0.0004 |
| | | 0.7 | 0.0075 | 0.0077 | 0.0069 | 0.0059 | **0.0051** | **0.0038** | **0.0029** | 0.0023 | 0.0013 |
| | | 0.8 | -0.0021 | 0.0012 | 0.0023 | 0.0028 | 0.0029 | 0.0029 | 0.0028 | **0.0026** | **0.0019** |
| | | 0.9 | -0.0185 | -0.0119 | -0.0080 | -0.0058 | -0.0041 | -0.0022 | -0.0012 | -0.0005 | 0.0001 |
| | | 1.0 | **-0.0436** | **-0.0333** | **-0.0268** | **-0.0222** | **-0.0187** | **-0.0146** | **-0.0122** | **-0.0104** | **-0.0079** |
| | $\alpha_{v1}$ | 1.0 | **0.299** | **0.262** | **0.236** | **0.213** | **0.197** | **0.174** | **0.158** | **0.145** | **0.127** |
| (2) Triangular load (hinged base) | $\alpha_{n2}$ | 0 | 0.205 | 0.074 | 0.017 | -0.008 | -0.011 | -0.015 | -0.008 | -0.002 | 0.002 |
| | | 0.5 | **0.434** | 0.506 | 0.545 | 0.562 | 0.566 | 0.564 | 0.552 | 0.541 | 0.521 |
| | | 0.6 | 0.419 | **0.519** | **0.579** | **0.617** | 0.639 | 0.661 | 0.666 | 0.664 | 0.650 |
| | | 0.7 | 0.369 | 0.479 | 0.553 | 0.606 | **0.643** | **0.697** | **0.730** | **0.750** | 0.764 |
| | | 0.8 | 0.280 | 0.375 | 0.447 | 0.503 | 0.547 | 0.621 | 0.678 | 0.720 | **0.776** |
| | | 0.9 | 0.151 | 0.210 | 0.256 | 0.294 | 0.327 | 0.386 | 0.433 | 0.477 | 0.536 |
| | $\alpha_{m2}$ | 0.6 | 0.0199 | 0.0127 | 0.0083 | 0.0057 | 0.0039 | 0.0020 | 0.0011 | 0.0005 | -0.0004 |
| | | 0.7 | **0.0219** | 0.0152 | 0.0109 | 0.0080 | 0.0062 | 0.0038 | 0.0025 | 0.0017 | 0.0008 |
| | | 0.8 | 0.0205 | **0.0153** | **0.0118** | **0.0094** | **0.0078** | **0.0057** | 0.0043 | 0.0032 | 0.0022 |
| | | 0.9 | 0.0145 | 0.0111 | 0.0092 | 0.0078 | 0.0068 | 0.0054 | **0.0045** | **0.0039** | **0.0029** |
| | $\alpha_{v2}$ | 1.0 | **0.189** | **0.158** | **0.137** | **0.121** | **0.110** | **0.096** | **0.087** | **0.079** | **0.068** |
| (3) Moment at hinged base | $\alpha_{n3}$ | 0 | -0.68 | -1.78 | -1.87 | -1.54 | -1.04 | -0.24 | 0.21 | 0.32 | 0.22 |
| | | 0.5 | 3.69 | 4.29 | 4.31 | 3.93 | 3.34 | 2.05 | 0.82 | -0.18 | -1.30 |
| | | 0.6 | 4.30 | 5.66 | 6.34 | 6.60 | 6.54 | 5.87 | 4.79 | 3.52 | 1.12 |
| | | 0.7 | **4.54** | **6.58** | 8.19 | 9.41 | 10.3 | 11.3 | 11.6 | 11.3 | 9.67 |
| | | 0.8 | 4.08 | 6.55 | **8.82** | **11.0** | **13.1** | **16.5** | 19.5 | 21.8 | 24.5 |
| | | 0.9 | 2.75 | 4.73 | 6.81 | 9.02 | 11.4 | 16.1 | **20.9** | **25.7** | **34.7** |
| | $\alpha_{m3}$ | 0.6 | 0.193 | 0.087 | 0.023 | -0.015 | -0.037 | -0.062 | -0.067 | -0.064 | -0.051 |
| | | 0.7 | 0.340 | 0.227 | 0.150 | 0.095 | 0.057 | 0.002 | -0.031 | -0.049 | -0.066 |
| | | 0.8 | 0.519 | 0.426 | 0.354 | 0.296 | 0.252 | 0.178 | 0.123 | 0.081 | 0.025 |
| | | 0.9 | 0.748 | 0.692 | 0.645 | 0.606 | 0.572 | 0.515 | 0.467 | 0.424 | 0.354 |
| | | 1.0 | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** |
| | $\alpha_{v3}$ | 1.0 | **-2.57** | **-3.18** | **-3.68** | **-4.10** | **-4.49** | **-5.18** | **-5.81** | **-6.38** | **-7.36** |

Load Cases (top of wall free)




(1) Triangular load (fixed base)  (2) Triangular load (hinged base)  (3) Moment at base (hinged base)

For $\alpha$ values shown above, positive signs indicate for:

($\alpha_n$) tension, ($\alpha_m$) tension in outside face, ($\alpha_v$) force acting inward

Circumferential tensions, vertical moments and radial shears, at depths denoted by $z/l_z$, are given by the following equations, where $l_z$ is height of wall, $r$ is radius to centre of wall, $z$ is depth from top of wall and $\gamma$ is unit weight of liquid.

Load cases (1) and (2):

Hoop tension: $n = \alpha_n \gamma l_z r$ (per unit height)

Vertical moment: $m = \alpha_m \gamma l_z^3$ (per unit length)

Radial shear: $v = \alpha_v \gamma l_z^2$ (per unit length)

Load case (3): $M$ = edge moment per unit length

Hoop tension: $n = \alpha_n M r / l_z^2$ (per unit height)

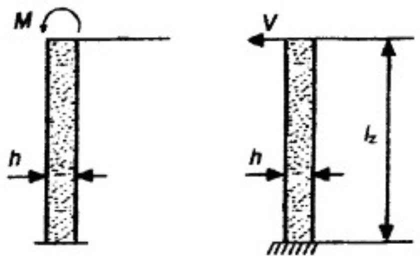
Vertical moment: $m = \alpha_m M$ (per unit length)

Radial shear: $v = \alpha_v M / l_z$ (per unit length)

## Table D.3: Cylindrical Tanks: Elastic Analysis 2

| Coefficients for circumferential tensions and vertical moments in wall of constant thickness | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Load case | $\alpha$ | $z/l_z$ | Values of coefficient $\alpha$ for values of $l_z^2/2rh$ | | | | | | | | |
| | | | 2 | 3 | 4 | 5 | 6 | 8 | 10 | 12 | 16 |
| (4) Moment at top | $\alpha_{n4}$ | 0 | **−13.63** | **−20.45** | **−27.26** | **−34.08** | **−40.89** | **−54.52** | **−68.15** | **−81.78** | **−109.0** |
| | | 0.1 | −7.43 | −9.43 | −10.77 | −11.61 | −12.03 | −11.90 | −10.77 | −8.87 | −3.46 |
| | | 0.2 | −2.98 | −2.22 | −0.87 | 0.85 | 2.78 | 6.95 | 11.18 | **15.85** | **22.45** |
| | | 0.3 | −0.02 | 1.92 | 4.03 | 6.11 | 8.05 | **11.33** | **13.76** | 15.38 | 16.66 |
| | | 0.4 | 1.74 | 3.81 | **5.62** | **7.04** | **8.09** | 9.21 | 9.33 | 8.78 | 6.65 |
| | | 0.5 | 2.60 | **4.25** | 5.31 | 5.86 | 6.00 | 5.48 | 4.40 | 3.15 | 0.90 |
| | $\alpha_{m4}$ | 0 | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** | **1.0** |
| | | 0.1 | 0.943 | 0.918 | 0.894 | 0.872 | 0.850 | 0.810 | 0.773 | 0.738 | 0.675 |
| | | 0.2 | 0.810 | 0.738 | 0.675 | 0.619 | 0.568 | 0.481 | 0.408 | 0.347 | 0.249 |
| | | 0.3 | 0.646 | 0.534 | 0.443 | 0.369 | 0.306 | 0.210 | 0.140 | 0.088 | 0.022 |
| | | 0.4 | 0.481 | 0.347 | 0.249 | 0.176 | 0.121 | 0.046 | 0.003 | −0.022 | −0.041 |
| | | 0.5 | 0.333 | 0.196 | 0.109 | 0.052 | 0.014 | −0.026 | −0.041 | −0.043 | −0.034 |
| (5) Shear at top (fixed base) | $\alpha_{n5}$ | 0 | **5.12** | **6.32** | **7.34** | **8.22** | **9.02** | **10.42** | **11.67** | **12.76** | **14.74** |
| | | 0.1 | 3.83 | 4.37 | 4.73 | 4.99 | 5.17 | 5.36 | 5.43 | 5.41 | 5.22 |
| | | 0.2 | 2.68 | 2.70 | 2.60 | 2.45 | 2.27 | 1.85 | 1.43 | 1.03 | 0.33 |
| | | 0.3 | 1.74 | 1.43 | 1.10 | 0.79 | 0.50 | 0.02 | −0.36 | −0.63 | −0.96 |
| | | 0.4 | 1.02 | 0.58 | 0.19 | −0.11 | −0.34 | −0.63 | −0.78 | −0.83 | −0.76 |
| | | 0.5 | 0.52 | 0.02 | −0.26 | −0.47 | −0.59 | −0.66 | −0.62 | −0.52 | −0.32 |
| | $\alpha_{m5}$ | 0.1 | −0.077 | −0.072 | −0.068 | −0.064 | −0.062 | −0.057 | **−0.053** | **−0.049** | **−0.044** |
| | | 0.2 | −0.115 | −0.100 | **−0.088** | **−0.078** | **−0.070** | **−0.058** | −0.049 | −0.042 | −0.031 |
| | | 0.3 | **−0.126** | **−0.100** | −0.081 | −0.067 | −0.056 | −0.041 | −0.029 | −0.022 | −0.012 |
| | | 0.4 | −0.119 | −0.086 | −0.063 | −0.047 | −0.036 | −0.021 | −0.012 | −0.007 | −0.001 |
| | | 0.5 | −0.103 | −0.066 | −0.043 | −0.028 | −0.018 | −0.007 | −0.002 | 0 | 0.002 |
| | | 1.0 | 0.019 | 0.024 | 0.019 | 0.011 | 0.006 | 0.001 | 0 | 0 | 0 |

Load Cases (top of wall free)




(4) Moment at top (any base)

(5) Shear at top (fixed base)

For $\alpha$ values shown above, positive signs indicate for: ($\alpha_n$) tension, ($\alpha_m$) tension in outside face

Circumferential tensions and vertical moments, at depths denoted by $z/l_z$, are given by the following equations, where $l_z$ is height of wall, $r$ is radius to centre of wall, $z$ is depth from top of wall and $\gamma$ is unit weight of liquid.

Load case (4): $M$ is edge moment per unit length

Hoop tension: $n = \alpha_n M r / l_z^2$ (per unit height)

Vertical moment: $m = \alpha_m M$ (per unit length)

Load case (5): $V$ is edge shear per unit length

Hoop tension: $n = \alpha_n V r / l_z$ (per unit height)

Vertical moment: $m = \alpha_m V l_z$ (per unit length)

Note. Coefficients for load case (4) apply to a semi-infinite cylinder. Since the effect of the moment dies out rapidly as $z/l_z$ increases, the same values may be used for all base conditions with errors that are reasonably small for $l_z^2/2rh > 2$, and negligible for $l_z^2/2rh > 8$.

| Coefficients for rotational stiffness of wall and fixed edge moment (FEM) for load cases (1) and (5) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $l_z^2/2rh$ | 2 | 3 | 4 | 5 | 6 | 8 | 10 | 12 | 16 | 20 |
| Stiffness | $\alpha_w$ | 0.445 | 0.548 | 0.635 | 0.713 | 0.783 | 0.903 | 1.010 | 1.108 | 1.281 | 1.430 |
| FEM | $\alpha_{w1}$ | −0.0436 | −0.0333 | −0.0268 | −0.0222 | −0.0187 | −0.0146 | −0.0122 | −0.0104 | −0.0079 | −0.0063 |
| | $\alpha_{w5}$ | −0.019 | −0.024 | −0.019 | −0.011 | −0.006 | −0.001 | 0 | 0 | 0 | 0 |

Rotational stiffness and fixed edge moments are given by the following equations:

Rotational stiffness of wall (hinged base and free top): $K_w = \alpha_w E_c h^3 / l_z$ where $E_c$ is modulus of elasticity of concrete

Fixed edge moment for load case (1): $M_w = \alpha_{w1} \gamma l_z^3$

Fixed edge moment for load case (5): $M_w = \alpha_{w5} V l_z$

# APPENDIX E: SUPPLY PUMPING SYSTEMS

It is important to understand the different types of pumps, design procedures, source of pumping power, motor starting, machine protections and economics of electric power systems.

## E.1 Rationale

The running and the economy of a water production line relies mainly on the success of the following project planning sub-components: intake and plant design, pumping system design, equipment type design, equipment selection, plant, pumping system and equipment protection, accuracy and comprehensiveness of erection, operation and maintenance instructions, economics of electrical power systems or other power systems, energy considerations, compliance with instructions, observation of the factory ordinance. The main goal of any water pumping plant and pumping system is to lift water from a lower to a higher level.

## E.2 Common Types of Pumps used in Water Supply

There are two main pump types used in the water supply projects which are different in design and application. Table E.1 shows the most commonly used pump types.

**Table E.1: Most Commonly Used Pump Types**

| Main Types | Sub-types | Specific types |
|---|---|---|
| **Rotordynamic** | Centrifugal | **Single-stage** |
| | | **Multi-stage shaft driven** |
| | | **Multi-stage submersible** |
| | Peripheral | **Axial flow** |
| | | **Mixed flow** |
| | | **Turbine** |
| | | **Submersible** |
| **Positive displacement** | Reciprocating | **Suction (shallow well)** |
| | | **Lift (deep well)** |
| | **Rotary** | **Helical Rotor** |

(Source: Modified from Uganda water design manual 2013)

### Rotordynamic Pumps

In the rotordynamic-type pump water while passing through the rotating element (impeller or a rotor) gains energy which is converted into pressure energy by an appropriate impeller casing and consists of three types: centrifugal, peripheral and special pumps.

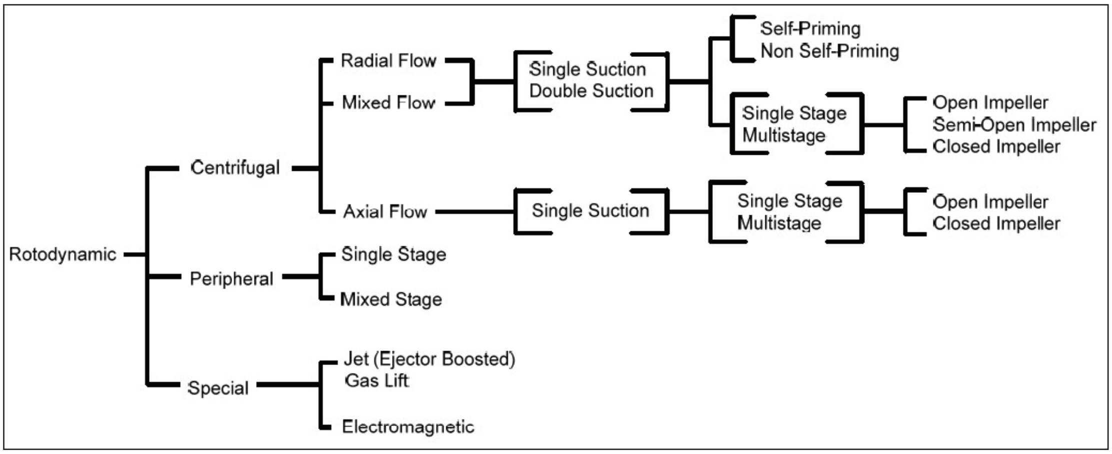



**Figure E.1: Classification of Rotary Pumps**

(Source: MoW Design Manual, 2009, 3rd Edition)

## E.3 Centrifugal Pumps

Centrifugal pumps are available in a wide variety of arrangements for mounting both above and below water. They are available as single or multi-staged units, and can be arranged for either horizontal or vertical mounting and for above water use come with fully enclosed or split casings. They can be direct coupled to a prime mover or in vertical mode driven via a shaft from a motor mounted above. The capacity of the centrifugal pump is greatly influenced by the pressure it works against, and also by the speed, form and diameter of its impeller. Low speed centrifugal pumps wear less and last longer than high speed pumps. Generally, speeds selected for raw water pumps should be limited to a maximum of 1500 rpm (Source: Water Supply Design Manual 2$^{nd}$ edition, Uganda.).

## E.4 Standard (dry mounted) Centrifugal Pump Sets

Standard, above ground centrifugal pump sets will either be horizontally or vertically mounted with the prime mover either at one end or immediately above the pump. For general waterworks purposes, the maximum pressure normally developed by a single stage pump will be 80 -100 m so that for heads greater than this, a multi-stage pump is usually required although increased head can also be achieved by increased speed and/or larger impellors. However, and as a general rule, the lower the speed, the longer the life of the pump.

With single stage horizontal pumps, end entry suction with side or top outlet is offered by some manufacturers whilst for vertically mounted units and for multistage pumps either side or top entry and exit ports are necessary. An end entry, single stage pump is illustrated in Figure E.2and a multi-stage pump in Figure E.3.

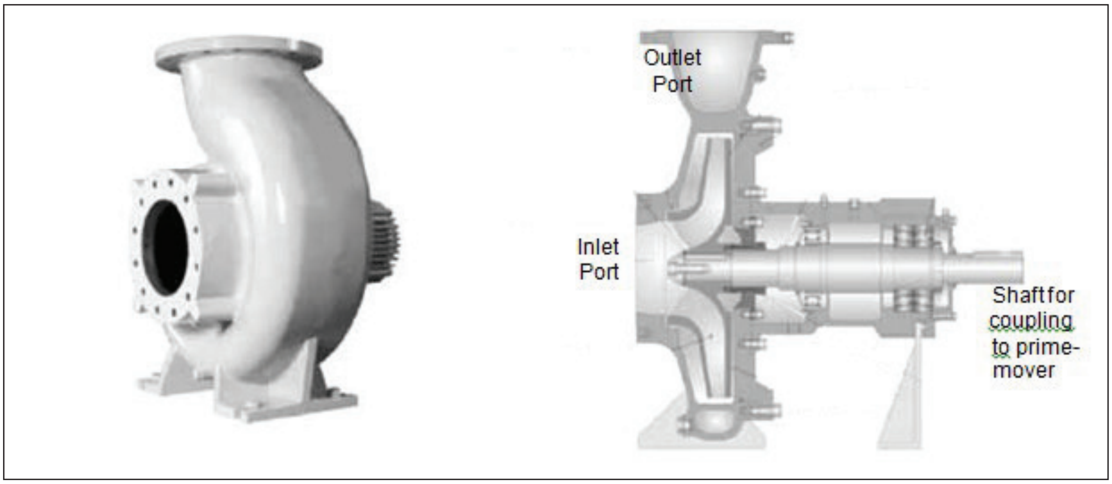



**Figure E.2: Section Through Single Stage End Suction Centrifugal Pump**

(Source: MoW 3rd Edition Design Water Manual, 2009)

Multistage unit casings can be either axially split (Fig. E.3) or radially split. Multistage pumps can be either horizontally or vertically disposed. A section through a typical multistage pump is illustrated in Figures E.3 and E.4.

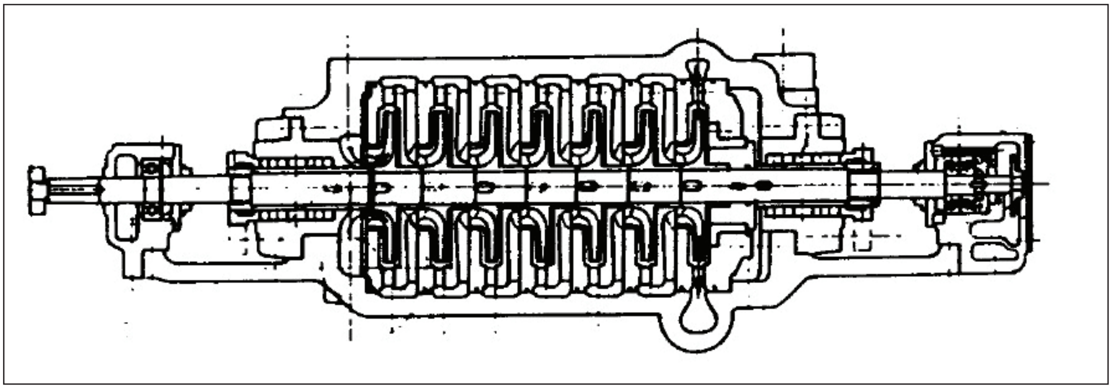


**Figure E.3: Axially Split Multistage Pump**

(Source: MoW 3rd Edition Design Manual, 2009)

The characteristics and applicability for the different types of centrifugal pumps include:

Single-stage: the usual depth range is 20 – 35 m. it requires skilled maintenance; not suitable for hand operation, powered by engine or electric motor;

Multi-stage shaft-driven: the depth range is 25 – 50 m. it requires skilled maintenance; the motor is accessible, above ground; alignment and lubrication of shaft critical; it has a capacity range of 25 – 10,000 l/min; and

Multi-stage submersible: the depth range is 30 – 120 m. its operation is smoother but maintenance is difficult; repair to motor or pump requires pulling the unit from the well; it has a wide range of capacities and heads; subject to rapid wear when sandy water is pumped.

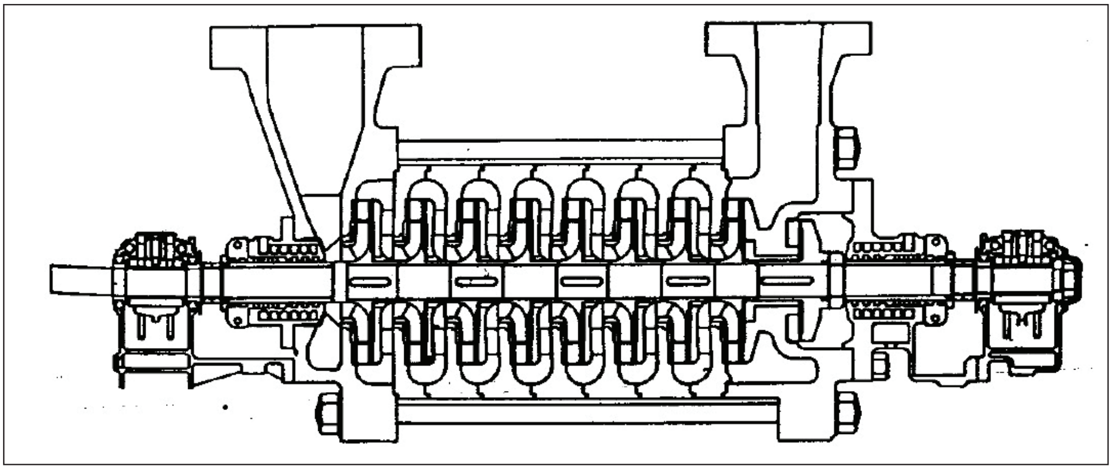


**Figure E.4: Radially Split Ring Section Multistage Pump**
(Source: Philippine's water design manual 2012).

## E.5 Axial- and Mixed-flow Centrifugal pumps

An Axial-flow propeller pump consists of a propeller which thrusts rather than throws the liquid upwards. Impeller vanes for mixed low centrifugal pumps are shaped to provide partial throw and partial push of the liquid outward and upwards. Axial and mixed-flow designs can handle large capacities but only with reduced discharge heads. They are constructed vertically. Axial flow pumps are used mostly for high- capacity and low-lifting pumping. They can pump water containing sand or salt. Axial flow pumps are the nominal choice for high-volume, low head raw water pumping. They are available in a wide range of capacities and sizes. They are usually installable to a depth range of 5 – 10 m.

## E.6 Specific Speeds

Specific speed ($N_s$) is the parameter which characterizes the rotordynamic pumps more explicitly and is given in Table E.2.

**Table E.2: Specific Speeds for Rotordynamic Pumps**

| Type | Specific Speed |
|---|---|
| Radial flow | 300-900 |
| Slow speed | 900-1,500 |
| Medium speed | 1,500-2,400 |
| High speed | |
| Mixed flow | 2,400-5,000 |
| Axial flow | 5,000-15,000 |

(Source: Water Supply Design Manual 2nd Edition, Uganda)

Centrifugal pumps can be either self – priming or non self – priming and can have open, semi – open or closed impellers depending on the specific requirements of the particular pump.

## E.7 Positive Displacement Pumps

Positive displacement pumps are essentially rotary or reciprocating machines in which energy is periodically added by application of force to movable boundaries of enclosed fluid containing volumes, resulting in a direct increase in pressure. The various types are illustrated on Figure E.5.

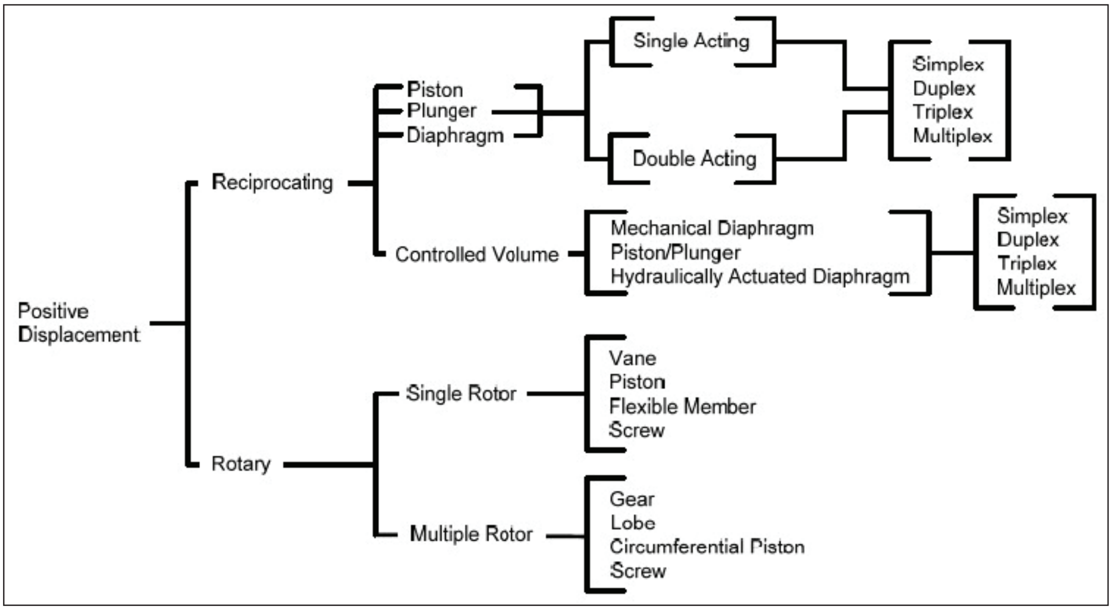



**Figure E.5: Types of Positive Displacement Pumps**

## E.8 Reciprocating Pumps

The reciprocating pump utilizes the energy transmitted by a moving element (piston) in a tightly fitting case (cylinder). Frequently in reciprocating pumps, a piston or plunger is used in a cylinder, which is driven forward and backward by a crankshaft connected to an outside drive. The reciprocating pumps can be divided into the two main categories: suction pumps and lift pumps.

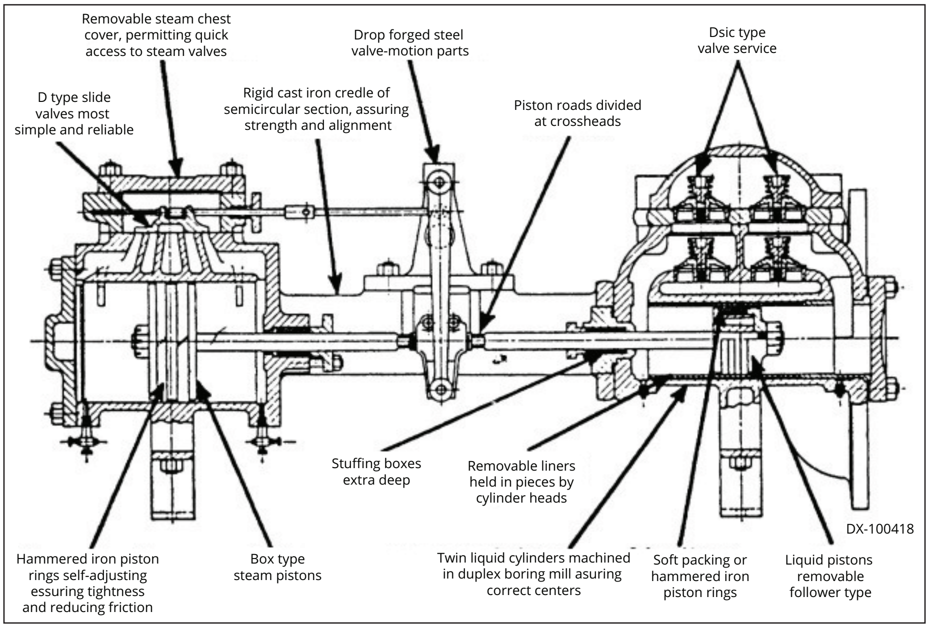



**Figure E.6: Duplex – Two Piston Pump[17]**

# E.8. Types of reciprocating pumps:

## E.8.1 Suction Pumps

Suction pumps are used in shallow wells. In a suction pump, the pump element (cylinder and plunger) is positioned above the water level, usually within the pump stand itself. A suction pump relies on atmospheric pressure for its operation. They lift water through a vacuum (sucking) action. All the moving parts are above the ground. Typically in the form of cast iron pumps, but also they come in different forms such as the plastic Rower pump and Diaphragm pumps. The suction pumps can be installed up to a depth of 7 m.

## E.8.2 Lift Pumps

Lift pumps are used in shallow wells. In a lift pump, the pump element (cylinder and plunger) is located below the water level in the well. Lift pumps create lift of the water, most commonly using a piston with leather, rubber or plastic washers (cup seals) located in a pump cylinder below the water level. The piston travels in an up and down motion at the pump head (direct action), a lever type handle, or a circular motion handle. Other mechanisms include spiral or helical stainless steel rotors encased in a rubber stator in the cylinder, and rubber diaphragms actuated hydraulically. The depth ranges[18] of the lift pumps are as follows:

[17] A. Kayode Coker, in Ludwig's Applied Process Design for Chemical and Petrochemical Plants (Fourth Edition), Volume 1, 2007.

[18] Water Supply Design Manual 2nd Edition, Uganda

**Table E.8: Pump Type and Depth Ranges**

| Pump Type | Depth Range |
|---|---|
| Low lift | up to 25 m |
| Intermediate lift | 25 to 50 m |
| Deep-set | 50 to 90 m |

Hand pumps are the most common used reciprocating pumps and, in most cases, the only economically feasible water lifting device for community needs (UNICEF, 1999). Yield depends on the depth and design, normally in the range of 600 to 1,500 litres per hour during constant use. The most important design criterion for a hand pump is its maintainability. Some typical maintenance programmes for Hand pumps include: periodic lubrication of above-ground components, replacement of washers and seals, replacement of plastic bearings, occasional replacement of individual rising mains. The maximum pumps (lifts) for comfortable operation of hand pumps are shown in the Table E.9.

**Table E.9: Maximum Heads (Lifts) for Hand Pumps**

| Cylinder Diameter (mm) | Head/lift (m) |
|---|---|
| 50 | Up to 25 |
| 65 | Up to 20 |
| 75 | Up to 15 |
| 100 | Up to 10 |

(Source: Water Supply Design Manual 2nd edition, Uganda)

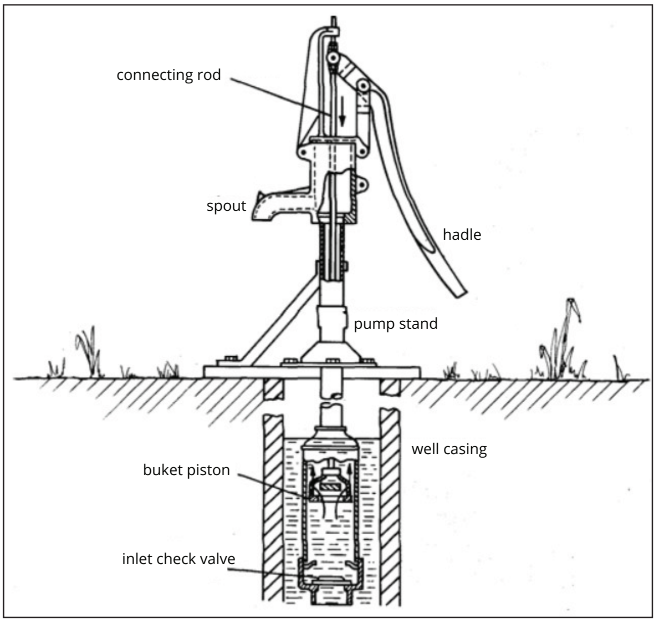



**Figure E.7: Hand Pump with Single-acting, Bucket Piston[19]**

[19] http://www.fao.org/3/ah810e/AH810E06.htm

## E.8.3 Rotary Pumps

Positive displacement rotary pumps work on the principle of rotation. Rotary pumps generally consist of gears, screws, vanes or similar elements enclosed within a casing. The rotation of the pump creates vacuum which draws in the liquid. The need to bleed the air from the lines manually is eliminated in rotary pumps because the air from the lines is naturally removed by the vacuum created. Positive displacement rotary pumps also have their weaknesses. Because of the nature of the pump, the clearance between the rotating pump and the outer edge must be very close, requiring that the pumps rotate at a slow, steady speed. If rotary pumps are operated at high speeds, the fluids will cause erosion, and thereby showing signs of enlarged clearances, which allow the liquid to slip through and detract from the efficiency of the pump. Rotary pumps are usually low in cost, require relatively small space, and are self priming[20].

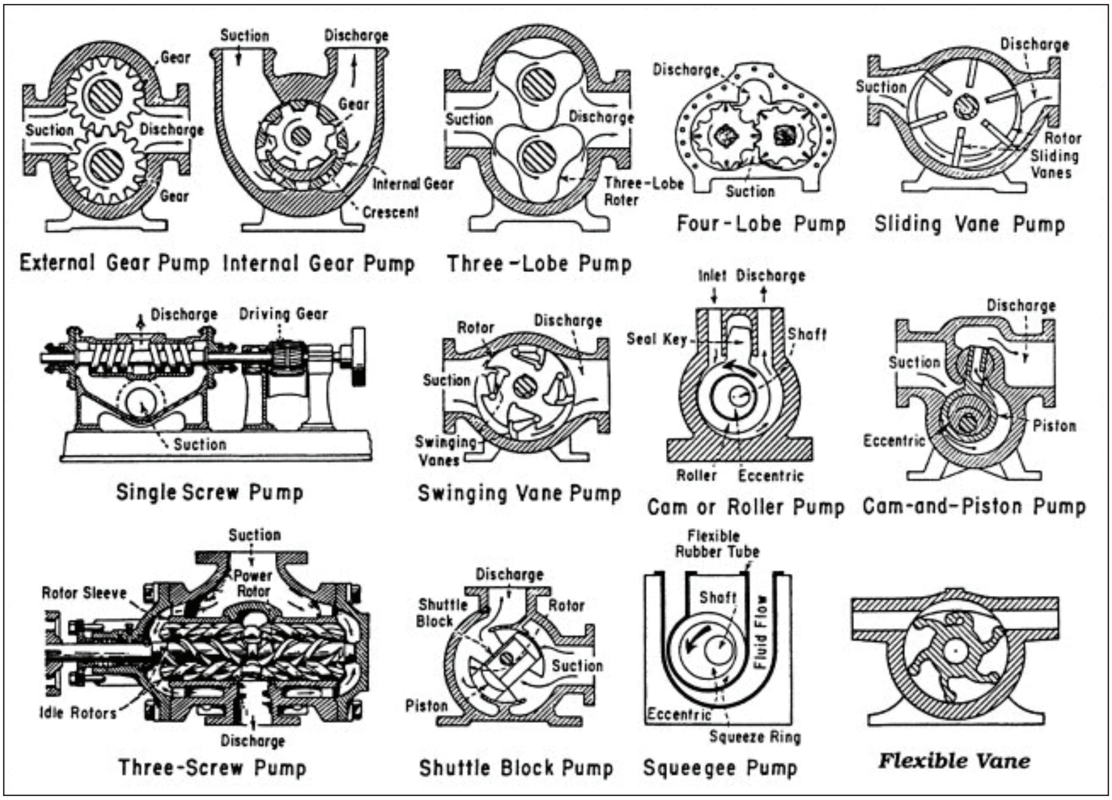



**Figure E.8: Types of Rotary Pumps**

(Source: Coker, 2014 #2381)

[20] William C. Lyons, Gary J. Plisga and Michael D. Lorenz.Standard Handbook of Petroleum and Natural Gas Engineering. 3rd Edition • 2016

Helical rotor pumps are the most commonly used type of rotary pump. A helical rotor pump consists of a single thread helical rotor which rotates inside a double thread helical sleeve, the stator. It is the meshing helical surfaces which force water up to create a uniform flow. Water delivery by rotary pumps is continuous and therefore smoother. However, internal losses in rotary pumps are normally higher through slip (internal leak-back). Slip increases with increasing pressure, making rotary pumps unsuitable for use in high pressure systems.

**Maximum Suction Lift Calculation**

In order to ensure continuous and smooth operation of any rotordynamic pump such as a centrifugal pump, there is a limit to the net positive suction head available (NPSHA) and hence the suction lift that can be achieved. The maximum suction lift shall be determined in accordance with the following formula:

$H_{s1}$ (Pepperberg) = B + $H_{suc}$ – Hfs........................................................................(3.29)

Where,

Pa = atmospheric pressure,

Vp = vapour pressure at the given temperature of the water source,

Hfs =friction losses in the suction line,

NPSHR =net positive suction head required,

Hfs = a value dependent upon the altitude and the water temperature and thus on the barometric pressure and vapour pressure of the water in meters head of water,

Hsuc = static height difference on the suction side of the pump in m (i.e. between the centre line of the pump element and the water level on the intake side of the pump).

## E.8.4 Turbine Pumps

The turbine pump motor is usually placed above the water level, but submersible types are available depending on the design requirements. Generally, turbine pumps have a constant head, and water flows uniformly at high pressure. The stages can be connected in series to increase the head capacity of the turbine pump. Two common types of turbine pump are submersible turbine pumps and deep well turbine pumps, which are also known as vertical turbine pumps.

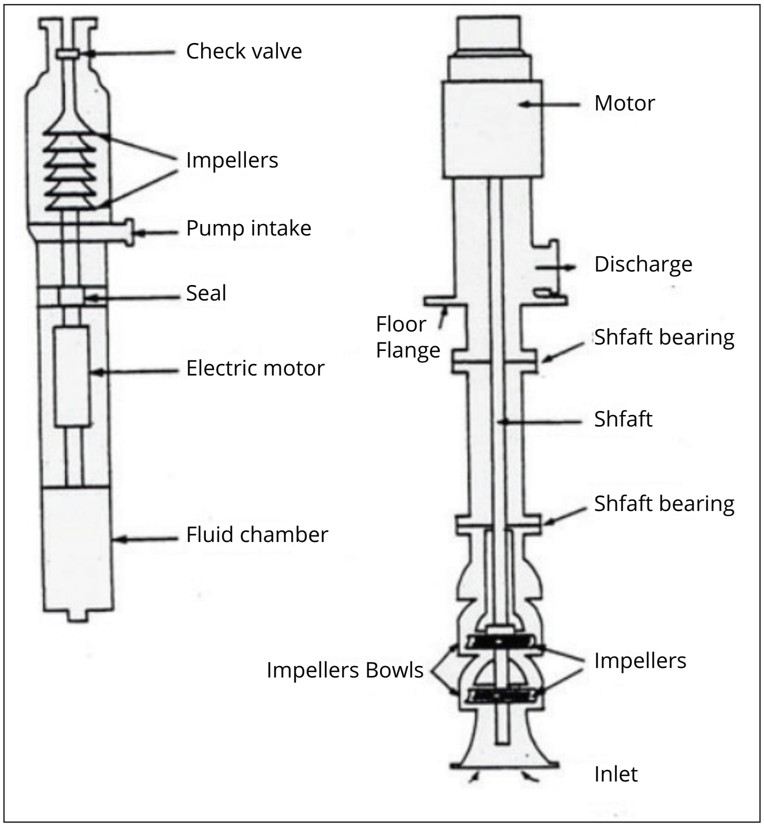



**Figure E.9: Turbine Pump**

## E.8.5 Submersible Pumps

The submersible pump, an illustration of which is shown in Figure E.10, is a pump which has a hermetically sealed motor close-coupled to the pump body. The whole assembly is submerged in the fluid to be pumped. The advantage of this type of pump is that it can provide a significant lifting force as it does not rely on external air pressure to lift the fluid. The pump is installed just above the motor, and both of these components are suspended in water. Submersible pumps use enclosed impellers and are easy to install and maintain. These pumps run only on electric power and can be used for pumping water from very deep and crooked wells. Moreover, they are unlikely to be struck by lightning and require constant flow of water across the motor.

The value is negative if the pump element is located above the water level on the intake side of the pump and positive when the pump is below the water level.

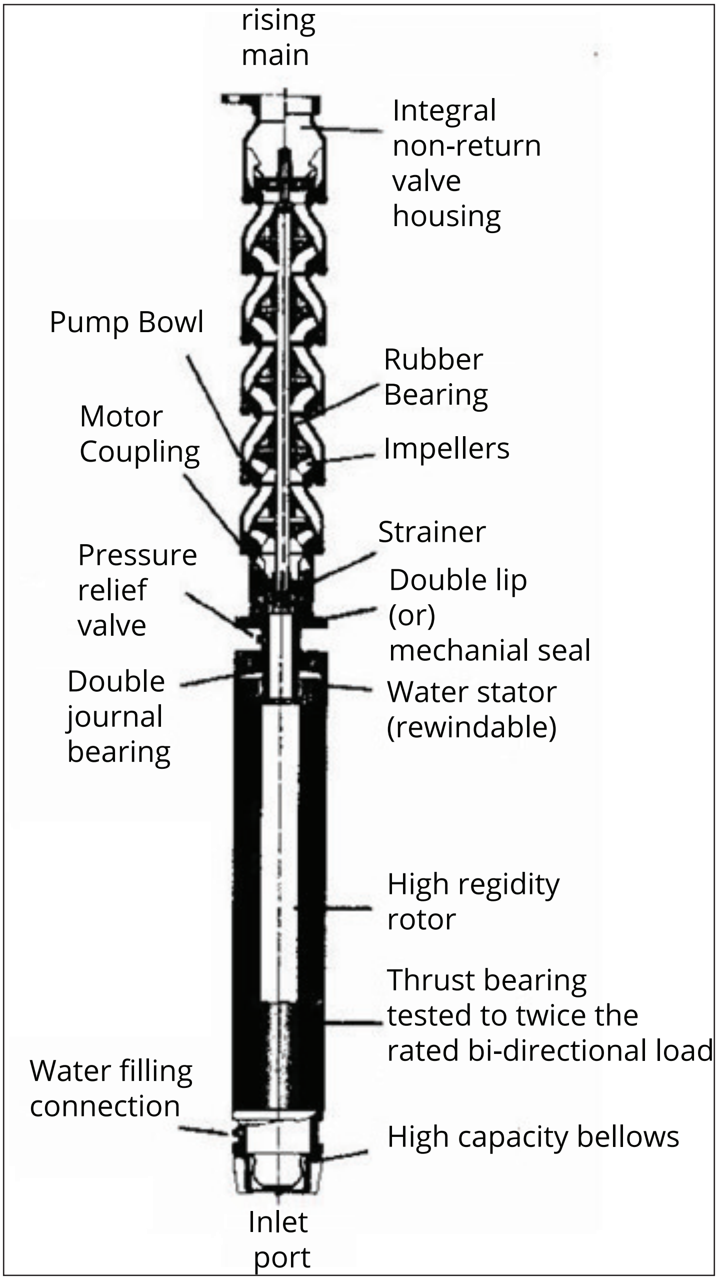



**Figure E.10: Submersible Pump**

(Source: Water Supply Design Manual, 2009)

**Notes:**

1. The NPSHR curve should be provided by the pump manufacturer or his agent. Otherwise the NPSHR value must be obtained from the manufacturer's catalogue or if even provided therein, it must be confirmed from the

relevant manufacturer who should give a written guarantee as to the value appropriate for the design head-flow point of the pump.

2. The term ($P_a$ - $V_p$ + $H_p$ + $H_{fs}$ + $H_{sl}$) = NPSHA, which for continuous operation must be at least 1 m.

**Example of Calculation of Head loss through the Strainer and the Foot valve:**

A pump is to be located 4 m above the minimum water level in an inlet sump and is to be used to pump 50 l/s. The suction pipe is 200 mm dia., 10 m long and has two 45° bends. There is a strainer with a foot valve at the inlet of the pipe at the altitude is 1,100 masl. Question: What should be the NPSH of the pump?

Solution:

$H_{suc}$ = - 4 m

$H_{fs}$ = 0.40 m (strainer & foot valve) + 0.07 m (bends) +

0.20 m (10 m of pipeline)

= 0.67 m

B (From Table 5.5, MoW 3rd Edition Design Manual; see also Appendix A) = 8.3 m

Hence, NPSHR = 8.3 – 4 – 0.67 = 3.63 m.

Therefore it is necessary to select a pump which has an NPSH of 3.63 metres or less for the capacity of 50 l/s.

**Pumping System Setup**

When setting up the pumping system, carefully calculate the driver HP required based on the data on the flow, pressure and efficiency of the pump. Check the pump RPM and drive RPM and select the proper size pulleys to achieve the desired flow. Review the maximum horsepower per belt to assure that the pump receives adequate power to deliver the desired flow. The correct belt length and centre distance must be established to achieve the proper HP. If in doubt, consult your pump and/or drive supplier for their recommendations.

# APPENDIX F: SOURCE OF PUMPING POWER

General

The different types of power sources commonly used for water supply pumps include:

- Grid power,
- Diesel/gasoline generators and engines,
- Natural gas/biogas generators,
- Solar energy,
- Wind energy.

The choice of water pumping energy depends on several factors namely:

- Availability of and proximity to grid power,
- Capital costs of the alternatives,
- Operational costs of the alternatives.

In Tanzania when deciding on the water pumping energy, grid power is considered as the basic source in the sense that when available it becomes the 1$^{st}$ choice. It is only when the grid source is too far from the pumping point that the other sources are considered. The three alternatives to grid power namely diesel/petrol/natural gas/biogas generators or engines have both positives and negatives. In the following subchapters each alternative shall be discussed.

**Grid power**

National grid connection enables a water pumping station to remain connected to the larger, wide-area electrical grid. The electrical grid is responsible for generating, distributing and balancing electricity across a wide area. Since most of the national electric power supply is mainly from hydropower plants (64% of the total generated electricity in 2016) (Aly, Moner-Girona et al. 2019),and natural gas the price is stable and the costs of power generation cheaper than the other sources except solar energy. Grid power is the most stable and trustable source of water pumping power (Although other renewable sources can also be made stable and trustable). It is also the cheapest in terms of capital cost since electric motors are cheaper than other forms of prime movers such as generators which also require engines in addition to electric motor for the pumps.

PV systems require higher investment costs due to the large area of PV required (It should however be noted that the costs of PV is getting lower). Similarly the running costs for electric motors is cheaper than that of engines except those utilizing own generated fuel such and gas or biogas fuel. In terms of running costs solar PV is the cheapest. Most of water supply sources requiring pumping are situated in rural areas where grid power may not be available. When deciding the alternatives the designers have to compare the costs of bringing grid power from the nearest point to the site against the capital costs of the other energy sources.

### Diesel Generators/Diesel Engines

There are two approaches applied when considering engines for water pumping. An engine may directly be coupled with the pump running on diesel or petrol. Most small irrigation pumps are of this type. The other way is a diesel run generator which provides electricity for running the pump. Here a normal electric driven pump is supplied with power from the generator instead of grid. Diesel run generators of different sizes (20 kW to over 3,000 kW) are available in the market to suit power requirements for various applications.


**Figure F.1: Modern Diesel Generator Set**

Designers of water supply systems would need to estimate the power requirement for pumping and distribution of water in order to be able to order the right size of a generator. Properly sized generator sets are easy to install. The running costs are high due to the requirement of fuel (Diesel/petrol) and the maintenance of the engine.

### Gas/biogas Generators

There are also generators which run on natural gas or biogas in case this type of fuel is readily available. Natural gas is locally available in some areas of Tanzania and already there are plans to introduce natural gas run automobiles. Biogas on the other hand can be produced locally in a bio-digester from industrial/ bio wastes containing high loading of organic matter. These two gases provide cheaper source of fuel for generating power. Note that when procuring gas engines one has to be specific whether the generator shall use biogas or natural gas. Natural gas and biogas are slightly different in terms of composition.

**Figure F.2: Biogas Generator[21]**

### Wind Driven Pump

The amount of water a wind-powered water pumping system can deliver depends on the speed and duration of the wind, the size and efficiency of the rotor, the efficiency of the pump being used, and how far the water has to be lifted. The power delivered by a windmill can be determined from the following equation:

$P = 0.0109D^2V^3e$ ........................................................................................ (3.30)

Where: P is power in watts, D is the rotor diameter in metres, V is the wind speed in kilometres per hour, and e is the efficiency of the wind turbine. As can be seen from this expression, relatively large increases in power result from comparatively small increases in the size of the rotor and the available wind speed; doubling the size of the rotor will result in a four-fold increase in power, while doubling the wind speed will result in an eight-fold increase in power.

However, the efficiency of wind turbines decreases significantly in both low and high winds, so the result is that most commercially-available windmills operate best in a range of wind-speeds between about 15 km/hr and 50 km/hr.

For adequate usage of wind power, the wind speed should be higher than 2.5 to 3 m/s for at least 60% of the time. The windmill should be placed above surrounding obstructions such as trees or buildings within 125 m; preferably, the windmill should be set out on a tower of 4.5 to 6 m high. Consider protection

[21] http://www.waltpower.com/methane-gas-generator-sets/biogas-generators

by provision of an automatic lubrication system or covering of the windmill driving mechanism. In order to directly pump water by a windmill, there is the need to match the characteristics of the local wind regime, the windmill and the pump. This therefore means that, the manufacturers should always be consulted regarding the selection of the equipment. The discharge, Q that can be pumped by a windmill can be estimated by the formula below:

$$Q = \left( \frac{2.8 D^2 V^3 e}{H} \right) \qquad (3.31)$$

Where,

Q = discharge in litres per minute (l/min),

D = wind rotor diameter in meters,

V = wind velocity in meters per second,

H = pumping head in meter,

e = wind to water mechanical efficiency, value 0-1.

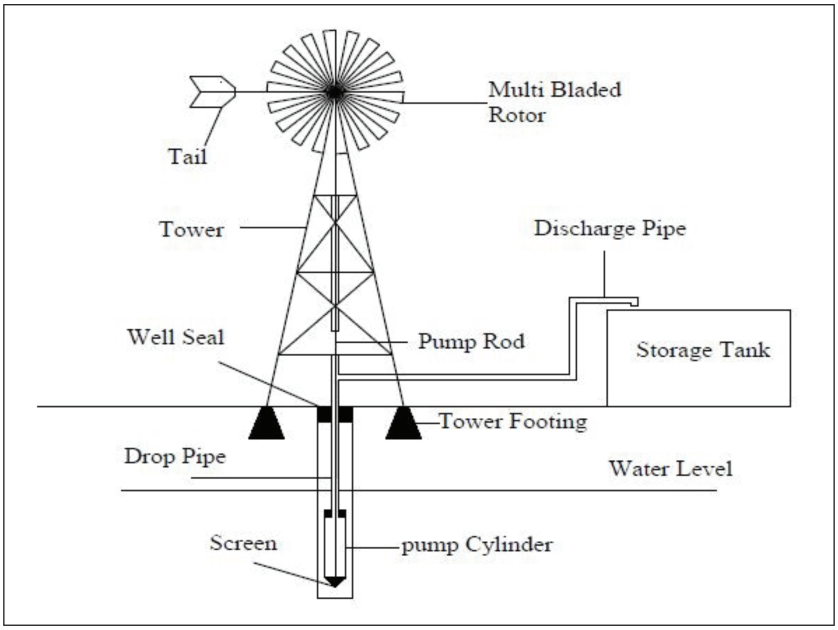



**Figure F.3: Windmill Driven Water Pumping**[22]

Windmills with rotor diameters between approximately 2 m to 6 m are usually available. The efficiency, e will rarely exceed 30%.About 5 years of data would give the designer reasonably representative averages following the fact that the monthly wind speed varies greatly between 10 % to 20 % from one year to the

[22] https://www.ecvv.com/product/4529384.html

next. For successful operation of a wind pump, at least wind speed of 2 to 3 m/s is required. Effort should be made to acquire a wind map of the area to guide with the wind speed throughout the year. This information is available from the Meteorological Agency but it may require interpretation and organization to ensure that it is applicable to the area in question.

### Solar Power

Solar-powered water pumps or photovoltaic pumps (PVP) are an effective alternative to conventional gas or electric pumps. Modern pumps are powered by solar energy effectively and used in different parts of the world. PVP systems offer numerous advantages over water supply system utilizing conventional power (adapted from Water Supply Manual, Uganda 2013):

- PVP systems may be the only practical water supply solution in many regions where the logistics make it too expensive or even impossible to supply diesel generators with the required fuel;
- PVP systems are ideal for meeting water requirements for villages between 500 and 2,000 inhabitants and small-scale irrigation purposes (up to 3 hectares);
- PVP systems run automatically, require little maintenance and few repairs;
- In areas where PVPs have entered into competition with diesel-driven pumps, their comparatively high initial cost is offset by the achieved savings on fuel and reduced maintenance expenditures;
- The use of solar energy eliminates emissions and fuels pills there by making photo voltaic pumps an environmentally sound and resource-conserving technology;
- There is no need for complicated wiring for the electricity and outside fuel is not needed;
- They can be designed not to require storage batteries, which are expensive and need a lot of maintenance; and
- The maintenance of a PVP system is restricted to regular cleaning of the solar modules.

The three components of a solar-powered water pumping system are:

Photovoltaic (PV) array (solar cells); Electric motor; and water pump.

The PV array generates direct current (DC) electricity when exposed to sunlight. This electricity is fed into the electric motor which in turn drives the water pump. Optional components of the system include the following:

Controllers (for regulating current and/or voltage);

Inverters (for converting DC power from the PV array to alternating current (AC) power for certain types of motors);

Electronic maximum power tracking devices, MPPT (to obtain a more efficient operation of the array and the motor); and

Batteries (used for both voltage regulation and energy storage and also for generating the required starting currents needed to overcome high electric motor starting torques).

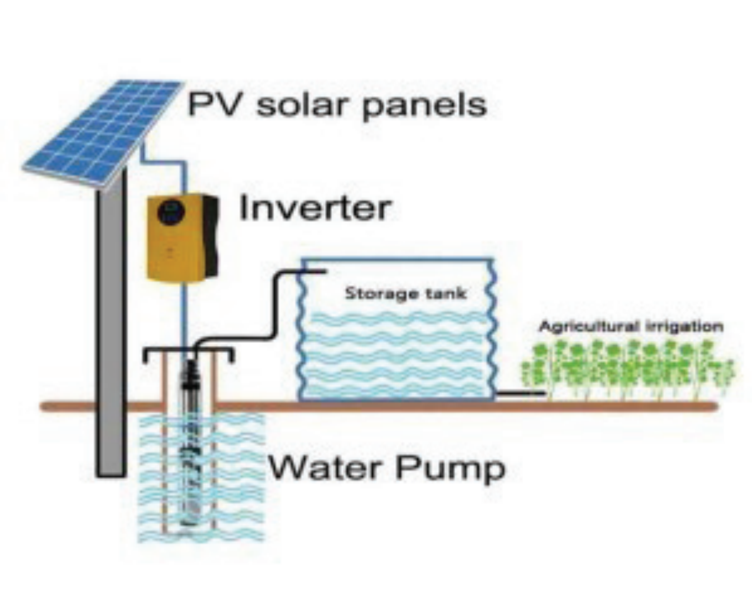



**Figure F.4: Arrangement for Solar Driven Pumping of Water[23]**

The selection of PV arrays and associated equipment should always be made in consultation with the relevant manufacturers. A solar generator provides electricity for driving a submersible electric pump, which in turn pumps water into an elevated water tank that bridges night-time periods and cloudy days. On a clear, sunny day, a medium-size PVP system with an installed power of 2 kW will pump approximately 35 $m^3$ of water per day to a head of 30 meters. That amount of water is sufficient for communities with populations up to 1,400. Today's generation of PVP systems is highly reliable. For the costliest part, the PV generator, the manufacturers give a 20 year guarantee on the power output. A crucial prerequisite for the reliability and economic efficiency is that the system be sized appropriate to the local situation.

The power output of a PV array is directly proportional to the solar irradiation falling on to the array. The power flow through a typical solar-powered water pumping system is in Figure F.4. Accordingly, the best efficiencies that are expected are as follows:

PV array -11% of the total solar power received by the PV array; and

Motor-pump unit - 4.5% of the total power received by the PV array

[23] https://www.commodoreaustralia.com.au/product-category/solar

The formula below can be used to estimate the daily solar power requirement of a water pumping system, in kWh.

$$P = \frac{\rho g Q H}{3600e} \quad \text{.......... (3.32)}$$

Where,

P = Power required in watt-hours/day,

ρ = density of water = 1,000 kg/m$^3$,

g = Gravitational acceleration (9.8 m/s$^2$),

Q = Daily water requirement in cu.m per day,

H = Total pumping head in m,

e = Overall mechanical efficiency of the system.

In practice, the overall mechanical efficiency of the system is about 30%. A typical performance curve, showing the power generated by a PV panel during the day is given in Figure F.5. Solar panels are specified by the Peak Power Rating, the power output of the PV panel in watts when the panel is receiving radiation of 1,000 watts per square metre, at ambient temperature of 25 °C. The available power in watt-hours per day is equal to the hatched area of Figure F.5. This is area is equal to the Peak Power Rating of the panel multiplied by 5 hours. Thus, if a panel is rated at 50 Watts Peak Power, then the useful Daily Output is 250 Watt-hours.

Thus the number of PV panels required can be found using the formula:

$$\mathbf{Numberofpanelsrequired} = \frac{\mathbf{DailyPowerRequired}}{\mathbf{PeakPowerRating}} \quad \text{.......... (3.33)}$$

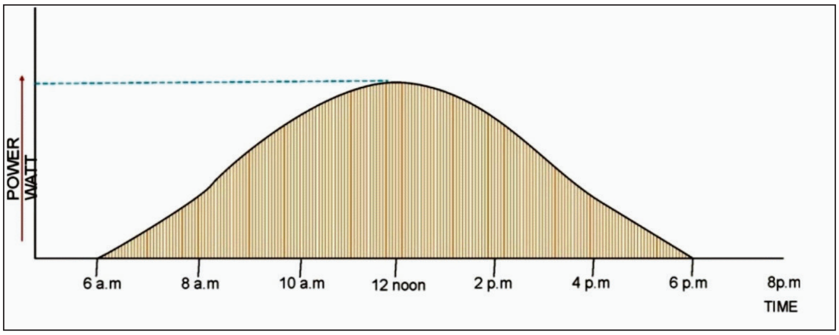



**Figure F.5: Diurnal Power Generation from a Solar PV Panel**

(Source: Uganda Water Design Manual)

When designing solar-powered water pumping systems, keep in mind that:

PV arrays panels are prone to vandalism and thus they should be protected by fencing; and

The shading of PV arrays should be avoided by siting obstructions on the south side of the arrays and by keeping the surrounding vegetation always cut.

### Sizing a PVP-System

APVP system is sized on the basis of the findings from a local data survey. While a non-site survey of meteorological and climatic data would be worthwhile it is usually hindered by lack of time and money. Many systems are based on the known data of a nearby reference location for which relevant measured values are available.

If it is possible to visit the intended location, the following field data should be gathered:

Demand for water in the supply area;

Pumping head with allowance for friction losses and well dynamics; and

Geographical peculiarities. e.g., valley locus.

It is also important to include sociological factors in the planning process. The future users should be involved in the data-gathering process at the intended PVP site in order to make early allowance for their customs and traditions in relation to water. Women in particular must be intensively involved in the planning, because they are the ones who are usually responsible for maintaining hygiene and fetching water. Thus, the planning base for each different location should cover both technical and sociological aspects.

The technical planner can choose from a number of design methods of various qualities. The most commonly employed approaches are outlined below.

### Estimation of PV Generator Output

To arrive at a first estimate of how much the planned PVP system will cost for a just selected site, it is a good idea to first estimate the requisite size of the PV generator. This, however, presumes knowledge of the essential sizing data, namely the daily water requirement within the area of supply (Vd), the pumping head to be overcome by the pump (H), and the mean daily total of global irradiation (Gd) for the design month. A simple arithmetic formula allowing for the individual system component efficiencies can be used to calculate the required solar generating power, PSG. The equation reads:

$$P_{SG} = 11.6 \frac{H V_d}{G_d} \quad \text{.......} (3.34)$$

According to this equation, it takes a 3.5-kWh PV generator to deliver water at the rate of 30 m³/d at a head of 50 m for a daily total global irradiation of 5 kWh/(m²*d). This gives the planner an instrument for estimating the size of the PV generator and, hence, the cost of the planned system at the time of site selection. Installation of the system must be done in accordance with the supplier's manuals and conditions and the relevant regulations. Relevant qualified personnel must be consulted at all times.

### Recommended use or non-use of solar battery

The solar driven pumping systems can be designed with storage batteries to make power available when there is solar radiation and when there is no solar radiation. The capital cost of a solar pump with power storage batteries is high while also its maintenance costs are also high. It is advisable that for water pumping installation of a large enough overhead reservoirs can allow a solar pump to store the required daily water so that when solar radiation is low the storage tank can continue to supply the needed water. However, in areas of poor security it may be worth to include a small battery to run the security lights during dark hours of the day.

Examples on pump calculations is provided below.

**Table F.1: Altitude and Temperature Values (Factor B)**

| | | | | | |
|---|---|---|---|---|---|
| **MASL** | 0 | 500 | 1000 | 1500 | 2000 |
| **B** | 9.4 | 8.9 | 8.4 | 7.9 | 7.4 |

### Example F.1: Sizing the pump from the desired flow rate

Same Mwanga Korogwe (SMK) projects aims to provide a total of 456,931 residents in the first phase, where, according to design manual, 250 residents are enough to be served by one water tap. Water requirement is 75 liters/p/day in urban areas. Estimate the flowrate that must be supplied by pumping

The total demand for water for residents is 34,269,825 liters per day.

Allow for 20% losses

Thus the total demand is 34,269,825 + 20% x 34269825 = 41,123,790 liters per day

Assume a peak factor of 1.2 The total daily requirement becomes 41,123.79 * 1.2 = 49,348,548 liters per day

Hourly requirement in $m^3/h$:

$$Q = \frac{49{,}348{,}548 \text{ liters per day}}{1000 \text{ liter per m3 x } 24 \text{ hrs per day}} = 2056.2 \text{ m3 per hour}$$

So the pumping plant (pump) should have at least 2,056.2 $m^3/h$.

With the intention of increasing efficiency we can put two pumps with a capacity of 1,028.09475 $m^3$/ h for each pump and a plant (pump) one standby.

**(The required pumps should be at least 1,028.1 $m^3$/hr)**

### Total head loss

This is pressure required to be overcome, sum of all opposition to the flow of water from the source to the destination tank or customer tap. Includes height required to be overcame pumped water, frictions inside pipes due to bends and actual water loss in joints. Therefore it is found from hydraulic calculations.

### Horsepower –kW

Source games (intake)

Head from the source until the plant is 50 m

Power pump (Power) = Energy (Time) Energy (Time)

Energy = mgH, m = density ($\rho$) x Volume (V), g = gravitational constant

Power = mgH / time Power = density x Volume x gH / time

Power = $\rho$ x V x gH / time V / time = power rate ($\varphi$) Power = $\rho$ x $\varphi$ x gH,

H = 50 m, $\varphi$ = 1,028.09475 $m^3$ / h,

= 1,000 $m^3$ / kg, g = 9.8 m / s2 Power = 1,000 x 995.328 x 9.8 x 50/3,600

Power = 139,935.119 Watts

Power = 139.935119 kWatts = 139.935119 + 20% (clearance)

= 167.922142 kW

**Pumps required should be at least 200 kWatts**

### Example F.2: Solar System Design

If a PV pump for water supply is to be installed in a village of 625 populations, then water is to be pumped from the height of 15 m. The water then has to be pumped to the storage tank that is at a height of 15 m. Dynamic head of the system is approximately 7 m (frictional losses of lifting water to the storage tank are included).

**Total water head** = Static head + dynamic head + height of storage tank

= 15+7+15 m

= 37 m.

**Daily water demand** = Population x Person daily water demand

In Tanzania a minimum of 40 litres per day per person should be made available.

= 625 x 40 (Litres per day)

= 25 $m^3$/day

**System size** for drinking water supply would also include the number of days the water has to be stored, two to three days of water storage is to be accounted for the designing the system. However the first step is to determine total energy required to lift the water. Although there is no charge controller.

Note: There is no charge controller or battery involved in the system, certain conversion losses at the time of water pumping occur.

Average solar insolation is 5.5 $kWh/m^3/day$

Energy required = 25 x 9.8 x 37 x 0.28

= 2 538 Wh/day (1 kJ = 0.28 Wh)

Array load = Energy required / System efficiency

= 2,538 / 0.85

Array load works out to be 2986 Wh

Array size = (Array load x Reserve days)/(Insolation x Mismatch factor)

= (2,986 x 2)/(5.5 x 0.85)

= 1,277.4 Wp (say 1.278 kWp)

**Design outputs:**

Array size = 1,278 Wp

Assume available solar panel = 74 Wp

1,278 Wp/74wp = 17.27 (say 18)

# APPENDIX G: ADDITIONAL DETAILS OF VARIOUS UNIT OPERATIONS

## 1. Horizontal-Flow Roughing Filters (HRF)

### Features of HRF

The filter box comprises of 2 to 4 compartments filled with gravel media of different sizes, coarsest in the first compartment where the bulk of solids will be retained, to finest in the last compartment which acts as polisher, removing the last traces of solids;

The raw water enters via an inlet channel across the filter width and falls over a weir into an inlet chamber which allows for settlement of coarse solids and separation of floating material.

Water passes into the first and subsequent filter compartments through openings in the separation walls. Treated water is collected in an outlet chamber and discharged over a weir- (for flow measurement) to an outlet channel and then to the SSF

The length of the filter box depends on raw water quality, hydraulic loading and media size.

Height should be limited to about 1.5m to allow for easy removal of media for manual cleaning.

The width depends on the required filter capacity.

A minimum of 2 HRF units are provided, to allow for one being out of service for cleaning, and an adjacent area for gravel cleaning should be provided.

### Design Considerations for HRF

(i) Characteristics of the raw water quality in both dry and rainy seasons i.e.;
Turbidity;
Suspended solids or filterability; and
E-coli
(ii) Raw water intake site location, elevation, high/low flows and water levels;
(iii) Treatment plant site, elevation, foundation conditions;
(iv) Hours of operation of intake, HRF, SSF and distribution;
(v) Availability of raw materials, gravel, sand, rock; and
(vi) Requirements for pre-sedimentation with or without combined raw water balancing tank.

**Table G.1 Design Guidelines for Horizontal Roughing Filters**

| Maximum suspended solids concentration in pre-settled water (mg/L) | > 300 high | 300 - 100 medium | 100 low |
|---|---|---|---|
| Filtration rate m/h | 0.5 | 0.75 - 1 | 1 - 1.5 |
| Filter length (m) for medium size (mm) | | | |
| 20 mm | 3 - 5 | 3* | 3* |
| 15 mm | 2 - 5 | 2 - 4 | 2 - 3 |
| 10 mm | 2 - 4 | 2 - 3 | 2 |
| 5 mm | 1 - 2 | 1 - 2 | 1 |
| Maximum suspended solids concentration in HRF effluent (mg/L) | 5 | 2 - 3 | 2 |

* This gravel size can possibly be omitted.

(Source: *http://www.nzdl.org/gsdlmod?*)

### Design Criteria of HRF

The criteria for design of HRF are as follows (also illustrated in Table G.1)

Filtration rate (m/s) ranges between 0.3 and 1.5 m/h. It is defined here as hydraulic load ($m^3/h$) per unit of vertical cross section area ($m^2$) of the filter;

Filter Box: - length 9-12 m

- Height 1-1.5 m

- Width 2-5 m per unit

Media size: - coarsest 35- 40 mm

finest 4 mm

Required effluent quality for SSF:

10 turbidity units 2-5 mg/L

2-5 mg/L SS

Openings in compartment walls; 10-20% of cross- sectional area, evenly distributed

Other criteria, such as expected efficiency of Suspended Solids (SS) removal, length of filter runs, required maximum head loss, etc., may be estimated by means of a monograph if necessary.

The outlet weir level should be 30-40 cm below the top level of filter material in order to allow for head loss across the filter, which builds up as the length of filter run increases and the filter loads up with silt.

Filter material ranges between 20 and 4 mm in size, and is usually distributed as coarse, medium and fine fraction in three subsequent filter compartments.

Filter length is dependent on raw water turbidity and usually lies within 5 to 7 m. Due to the comparatively long filter length.

Horizontal-flow roughing filters can handle short turbidity peaks of 500 to 1,000 NTU.

The filter has a series of differently graded filter material separated by perforated walls, to the filter outlet where the raw water runs in horizontal direction from the inlet compartment. These are: gravel; broken stones/rocks; burnt bricks; burnt charcoal; (prior testing at pilot and coconut fibre (plant level is necessary).

## 2. Vertical-flow roughing filters

### Design guidelines

The size of the three distinct filter material fractions is generally between 20 and 4 mm, and graded, for example, into fractions of 12-18 mm, 8-12 mm and 4-8 mm.

The filter layers are covered with a 0.1 m-deep layer of boulders, to avoid exposing the outflow directly to sunlight; this helps to prevent algal growth.

The filtration rate is approximately 0.6 m/h

The water to be treated flows in sequence through the three filter compartments filled with coarse, medium and fine filter material.

Water flows in through an under - drain system on the bottom, usually a perforated PVC pipe, which also permits rapid abstraction during cleaning when the flow direction is reversed (backwashing).

Vertical-flow roughing filters usually consist of three filter units arranged in series and operate either as down flow or up flow filter as shown in the Appendix B.2 (b).

An upflow filter has a round or rectangular shape, with vertical or partially inclined walls, and it is usually about 1.5 m deep. The filter box can be made of bricks, concrete or ferrocement. The under-drains are covered with a layer of coarse gravel, on top of which lie several layers of finer gravel and coarse sand.

Downflow filters have a better performance than upflow filters as the solid particles are more likely to settle on top of the gravel surface in the direction of flow than under counter current conditions.

In down flow roughing filters, the bulk of accumulated solids has to be flushed with a relatively small wash water volume from the soiled filter top through the lower and cleaner filter part to the filter bottom. The opposite is true for upflow roughing filters.

## 3. Dynamic Roughing Filters

### Design guidelines

Maximum available head loss is limited and ranges between 20 and 40 cm in spite of finer filter material and greater filter velocity.

The water quality between filter inlet and outlet hardly changes during periods of low raw water turbidity.

Dynamic filters act like turbidity meters connected to an open-close valve; i.e., they rapidly get clogged when raw water of high turbidity passes through the filter.

Dynamic filters are similar in layout to intake filters, but differ in filter material size and filtration rate. Especially the gravel size of the top filter layer is smaller; i.e., less than 6 millimetres in diameter, while filtration rate is usually more than 5 m/h.

Finally, the horizontal flow velocity over the filter bed surface should be small or non-existent; i.e., less than 0.05 m/s or nil, to prevent removal of accumulated silt during turbidity peaks. A typical layout of the Dynamic Roughing filter is illustrated in Appendix G.3.

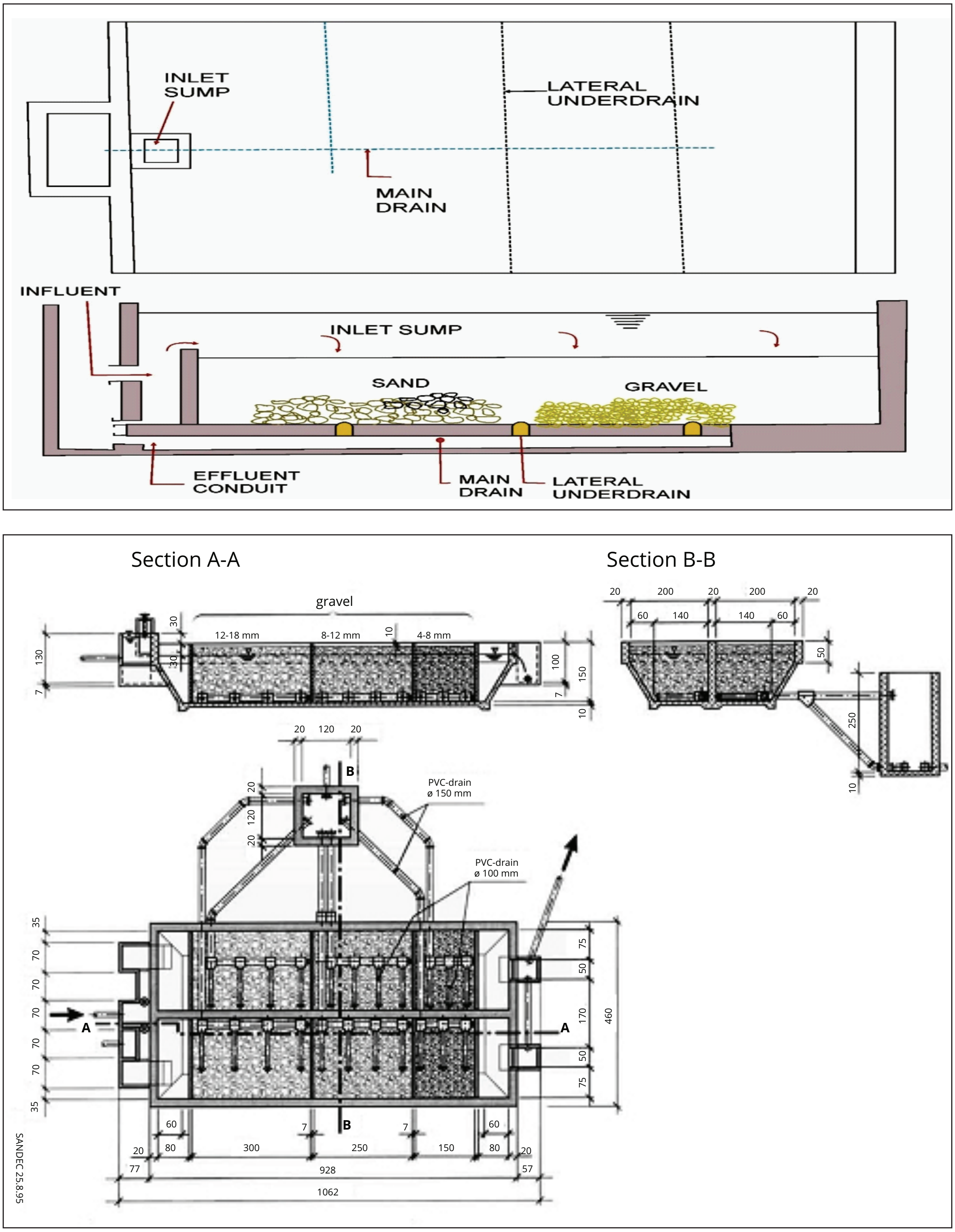



**Figure G.1: Layout of Horizontal-Flow Roughing Filter**

(Source SANDTEC – SKAT, 1996 as cited in the Water Supply Design Manual-Second Edition, Uganda 2013)

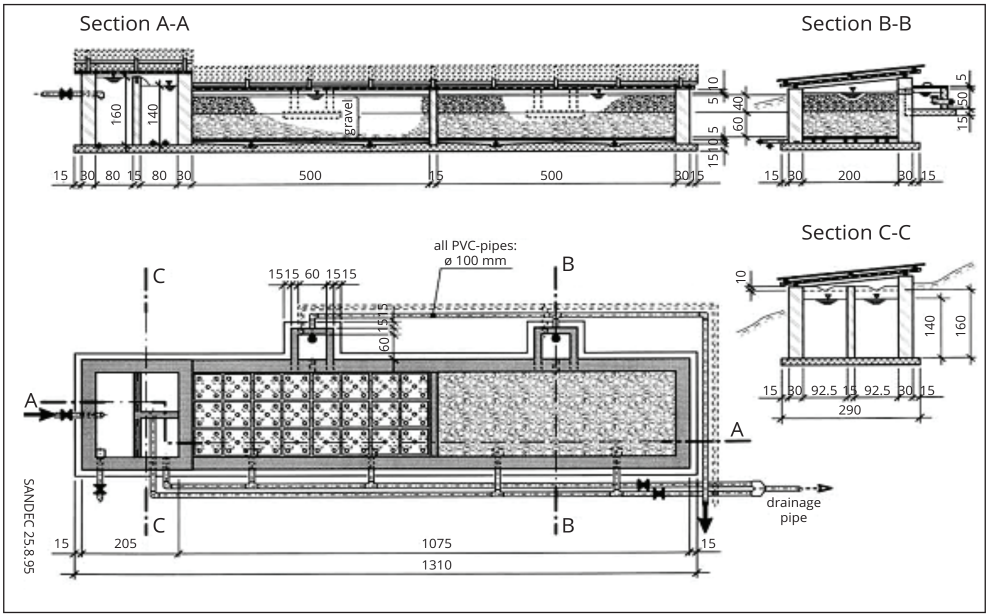
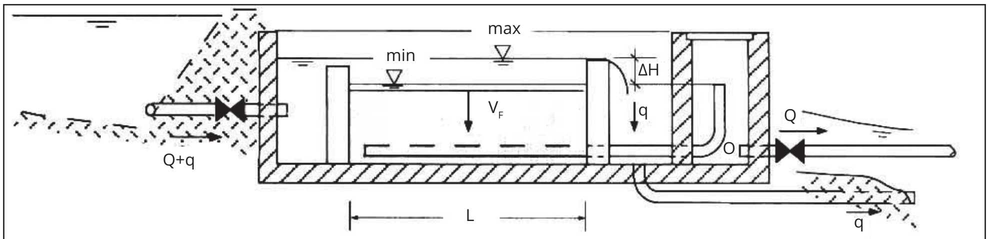



**Figure G.2: Layout of Upflow Vertical-Flow Roughing Filters**

(Source: SANDEC (1996): as cited in the link: Water Supply Design Manual, Uganda 2013)




| list of symbols | design guidelines filteration rate | intake filter | dynamic filter |
|---|---|---|---|
| $d_g$ (mm) gravel size | $V_F = \frac{Q}{L.W} = \frac{Q}{A}$ | 0.3 - 2 m/h | > 5 m/h |
| L (m) filter length<br>W (m) filter width | max. headloss (operation) ΔH | ~ 20 -40 cm | ~ 20 - 10 cm |
| A (m²) filter area | gravel size | gravel layer height | |
| ΔH (cm) headloss | $d_g$ = 2 - 4 mm | — | 20 - 30 cm |
| Q (m³/h) flow rate | $d_g$ = 4 - 8 mm | 20 - 40 cm | 10 cm |
| $V_F$ (m/h) filtration rate | $d_g$ = 8 - 12 mm | (10 - 20 cm) | 10 cm |
| q (m³/h) surplus flow rate | | | |

SANDEC 25.8.95

**Figure G.3: Layout of the Dynamic Intake and Dynamic Roughing Filter**

Source: as cited in the link: http://www.nzdl.org/gsdlmod?e=d-00000-00---off-0cdl--00-0----0-10-0---0---0direct-10---4-------0-0l--11-en-50---20-about---00-0-1-00-0--4----0-0-11-10-0utfZz-8-00&cl=CL2.19&d=HASH01165bbf8a8dc3251af16cd9.6.4.3&gt=1)

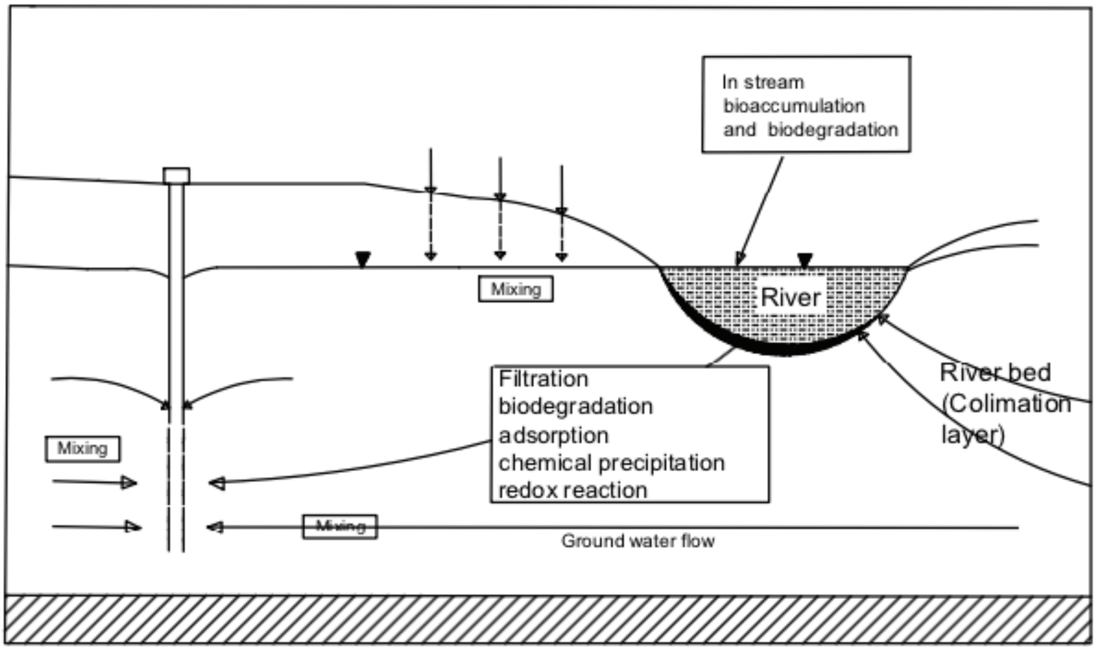



**Figure G.4: Schematic Diagram of Processes Affecting Water Quality During the Bank Filtration Process**

(Source: HISCOCK & GRISCHEK (2002 as cited in link: http://archive.sswm.info/category/implementation-tools/water-sources/hardware/surface-water-sources/bank-filtration

# APPENDIX H: METHODS FOR DISINFECTING WATER

## Physical Disinfection

The two principal physical disinfection methods are boiling of water, and radiation with ultraviolet rays.

### Boiling

Boiling is a safe process for it destroys pathogenic micro-organisms and is effective as a household treatment, but is not a feasible method for community water supplies.

### Ultraviolet radiation

Light radiation is an effective disinfection method for clear water but its effectiveness is significantly reduced when the water is turbid and contains constituents such as nitrate, sulphate, and ferrous iron. In addition, this disinfection method does not produce any residual that would protect the water against any new contamination that could serve for control and monitoring purpose. Hence it is not recommended.

## Chemical Disinfection

For chemical disinfection, the following points need to be followed:

- Good mixing between water and disinfection agent,
- Sufficient dosage compared to water quality and types of micro-organisms that are to be removed,
- Sufficient contact time between the water and the disinfectant,
- Suitable water quality with regard to turbidity and organic matter.

A good chemical disinfectant should possess a number of important characteristics, including:

- Quick and effective in killing pathogenic micro-organisms present in water,
- Readily soluble in water and in concentrations required for the disinfection, and capable of providing a residual,
- Does not impart taste, odour or colour to water at the concentrations used,
- Be easy to detect and measure in water,
- Be readily available at moderate cost.

The chemicals that have been successfully used for disinfection are: chlorine, and chlorine compounds and to a lesser extent iodine dosed in suitable form, ozone and other oxidants like potassium permanganate and hydrogen peroxide. Each one of these has its advantages and limitations.

### (a) Chlorine and Chlorine Compounds

These have the ability to destroy pathogens fairly and their widespread availability makes them well suited for water disinfection. Their cost is moderate and are for this reasons widely used as disinfectants throughout the world. Environmentally, the production of chlorine has negative health impacts. Some of their by-products are of possible health concern and include trihalomethanes.

**(b) Iodine**

In spite of its attractive properties as a disinfectant, iodine 'has serious limitations. High doses (10-15 mg/l) are required to achieve satisfactory disinfection. It is not, therefore effective when the water to be disinfected is coloured or turbid. Hence, it is not recommended.

**(c) Potassium Permanganate**

This is a powerful oxidizing agent and has been found to be effective against cholera vibrio but not for other pathogens. It leaves stains in the container and hence it is not a very satisfactory disinfectant for community water supplies. It is destructive to aquaculture. Hence it is not recommended.

**(d) Ozone**

Ozone is increasingly being used for disinfection of drinking water supplies in industrialized countries as it is effective in eliminating compounds that give objectionable taste or colour to water. Like ultraviolet light, ozone normally leaves no measurable residual which could serve good for monitoring the process. The absence of a residual also means that there is no protection against new contamination of the water after its disinfection. The high installation and operational costs and the need for continuous power make the use of ozone relatively expensive. Hence it is not recommended.

**Trihalomethanes (THMs)**

Trihalomethanes can be formed when raw waters containing naturally occurring organic compounds such as humic and fulvic acids are chlorinated. They are also formed by the reaction of chlorine with some algal derivatives. Control is best achieved by avoiding pre-chlorination and only using post chlorination with the removal of as much of the organic precursors as possible before the chlorine is introduced into the water.

There is evidence that trihalomethanes pose a cancer risk and for this reason the WHO has set guideline values and is recommended that total trihalomethanes (TTHM) in public water supplies be limited to 0.2 mg/l (200 μg/l), with any single trihalomethane limited to half of this level.

However, the WHO guidelines emphasize that the disinfection process must not be compromised, and that ‘inadequate disinfection in order not to elevate the THM level is not acceptable’.

## Choice of Chemical Disinfectant

As a result and despite a number of environmental health drawbacks in its production and sometimes in its use, disinfection is still overwhelmingly done using chlorination, and one of the following agents maybe used:

For large schemes:

- Gaseous chlorine

For medium sized schemes

- Sodium hypochlorite (NaOCL), especially for on-site production electrolytically using near-pure salt and electricity
- Calcium hypochlorite (HTH) with 65 – 70 % available chlorine

For small schemes:

- Chlorinated lime or bleaching powder $CaO_2.Ca(OCl)_2$ with up to 39 % available chlorine

Great care must always be taken when using chlorine as in gaseous form it is extremely poisonous. Only qualified and authorised personnel should be involved in mixing and dosing and under no circumstances should members of the public be allowed unaccompanied into the mixing and dosing facilities.

- Chlorine solutions should, whenever possible be fed into the water by means of gravity or displacement dosers, and dosing pumps should preferably be confined to medium and large plants.
- A contact period of at least 30 minutes in the clear water tank and the transmission main should be allowed before the water reaches the first consumer.
- Normally the necessary disinfectant added should be in the-range of 0.5 - 2.0 mg/l (free chlorine)

If chlorine gas is used the cylinders or drums must be stored in a cool, well-ventilated place. Valves must never be left open after use as any residual gas then combines with moisture to form hydrochloric acid. If bleaching powder is used separate mixing and dosing tanks must be provided so that solid deposits do not clog the dosing process sand the dose at the point of application must be visible.

## Chlorine

As noted above and although a number of disinfectants have been tried and used in the past in a very few instances they are still used. Notwithstanding its disadvantages, Chlorine is still the most widely used water supply disinfectant, either in the form of a gas or one of its several compounds such as chlorine of lime or sodium hypochlorite. In all cases, the active disinfectant is chlorine. Because of cost, dependability efficiency and relative ease of handling provided this is done with care, chlorine or chlorine compounds are almost always used.

As a matter of fact the term "chlorination" is generally used synonymously with disinfection in water works practice.

Chlorine may be applied either as a gas or as a solution, either alone or in conjunction with other chemicals. Regardless of the form of application, the quantity or dosage of chlorine is controlled by special apparatus called chlorinators or hypochlorinators.

The selection of the equipment depends on a particular installation.

## Chlorine Water Reactions

Chlorine reacts with water to form hypochlorous acid (HOCL) and hydrochloric acid (HCl) according to the equation.

$Cl_2 + H_2O = H^+ + C^{l-}$ ........................................................................................ (1)

This hydrolysis reaction is reversible. The hypochlorous acid disassociates into hydrogen ions ($H^+$) and hypochlorite ions ($OC^{l-}$) according to the equation

$HOCl = H^+OC^{l-}$ ........................................................................................................ (2)

HOCl is about 100 times more powerful for disinfection than OCl- ion. When the pH value of the chlorinated water is above 3, the hydrolysis reaction is almost complete and the chlorine exists entirely in the form of HOCl.

From a consideration of the second equation, it is evident that as the pH increases more and more HOCl disassociates to form OCl ions. At pH values of 5.5 and below, it is practically 100% unionised HOCl while above a pH 9.5, it is all OCl ions. Between pH 6.0 to 8.5, there occurs a very sharp change from undissociated to completely disassociated hypochlorous acid with 96% to 100% of HOCl, with equal amounts of HOCl and OCl being present at pH 7.5. The addition of chlorine does not produce any significant change in the pH of the natural waters because of their buffering capacity. Free available chlorine may be defined as the chlorine existing in water as hypochlorous acid and hypochlorite ions.

## Chlorinated Lime (Bleaching Powder)

Before the advent of liquid chlorine, chlorination was mostly accomplished through the use of lime and chlorine gas, with the approximate composition

$CaCl_2Ca(OH)_2.\ H_2O + CaCl_22Ca(HO)_2$ ................................................................. (3)

Fresh, chlorinated lime has a chlorine content of 33 to 37 percent. However, chlorinated lime is unstable and exposure to air, light and moisture causes the chlorine content to fall rapidly. The compound should therefore, be stored in a dark, cool and dry place and in closed, corrosion resistant containers.

It also produces waste sludge which has to be disposed of.

### High Test Hypochlorite (HTH)

These are not only twice as strong as chlorinated lime (60 to 70 percent available chlorine content) but retain their original strength for more than a year under normal storage conditions. HTH may be obtained in packages of 2-3 kg, and in cans of up to 50 kg, and are also available in granular or tablet form. It should therefore be used instead of chlorinated lime.

### Sodium Hypochlorite

Sodium Hypochlorite is a solution, (NaOCl) produced electrolytically and usually contains 10 to 15 percent available chlorine in the commercial form. Household bleach solutions of sodium hypochlorite usually contain only 3 to 5 percent available chlorine. It has a very short shelf life and should be produced only on site as required.

Where waterworks require electricity and nearly pure salt is available locally it is a preferred method.

### Chlorination Practice

Chlorination practises may be grouped into two categories depending upon the desired level of residual chlorine and the point of application. When it is required to provide a residual and the time of contact is limited, it is common practice to provide for free available residual chlorine. If combined available residual chlorination is used, the chlorine is applied to water to produce, with natural or added ammonia, a combined residual, effect. Pre-chlorination is the application of chlorine prior to any other treatment. This has been used for the purpose of controlling algae, taste and odour but should be avoided whenever possible because of trihalomethanes (THMs). Post chlorination refers to the application of chlorine after other treatment processes particularly after filtration and should be preferred.

### Chlorine Demand

This is the difference between the amount of chlorine added to water and the amount of free or combined available chlorine remaining at the end of a specified contact period.

### Residual Chlorine

Several methods are available to measure residual chlorine in water. Consult standard water treatment textbooks to select the most appropriate method for a particular situation.

### Gaseous Chlorine Storage

Gaseous chlorine is heavier than air and is lethal. Should a leak be detected, people nearby should be alerted and told to move upwind. Usually cylinders are used for chlorine storage. Their choice in terms of capacity depends on the chlorine requirement. Cylinders up to 67 kg capacity should be stored vertically so that a leaking container if found can be removed with the least possible handling of others. It is preferable to provide space for separate storage of full and empty cylinders. Care should be taken to prevent them from falling over or from being hit by moving objects. Any dropping of containers is very dangerous. Containers should be stored in a cool ventilated area protected against external sources of heat like steam, electric heaters and away from inflammable materials.

# APPENDIX I: MEASUREMENTS OF WATER HARDNESS

## Hardness and its Measurement

Hardness is expressed in terms of mg/l by weight and in terms of calcium carbonate. Water with hardness not exceeding 70 mg/l is termed ‘soft’ and above that 'hard'. In public water supplies, it used to be customary to reduce carbonate hardness to 35 - 40 mg/l and total hardness to between 50 and 100 mg/l. However as indicated above this is no longer recommended unless hardness exceeds about 130 mg/l, but should still be practised for strictly industrial supplies of water.

**Hardness = ∑ divalent cations = $Ca^{2+}$ + $Mg^{2+}$ + $Fe^{2+}$ + $Mn^{2+}$ + $Sr^{2+}$.....**

The principle cations causing hardness in water and major anions associated with them are as follows:

**Table I.1: Principle Cations and Anions Associated with Water Hardness**

| Cations | Anions |
|---|---|
| $Ca^{2+}$ | $HCO^{3-}$ or $CO_3^{2-}$ |
| $Mg^{2+}$ | $SO_4^{2-}$ |
| $Sr^{2+}$ | $Cl^{-}$ |
| $Fe^{2+}$ | $NO^{3-}$ |
| $Mn^{2+}$ | $SiO^{3-}$ |

If hardness is too high it results into precipitation of soap, scaling on pipes, boilers, cooling towers, heat exchangers. If hardness is too low, the water becomes corrosive. Hard water interferes with almost every cleaning task from laundering and dishwashing to bathing and personal grooming (IANR, Water Quality 1996). Hard water ranges between 120-250 mg/L as $CaCO_3$ or beyond 250 mg/L as $CaCO_3$ for very hard waters. The acceptable water hardness range is between 60-120 mg/L as $CaCO_3$ (Dipa Dey, Amanda Herzog and Vidya Srinivasan, 2007). The scale of hardness is shown in Table I.2:

**Table I.2: Scale of Hardness (Classification of Hard Waters)**

| Hardness in mg/l | Scale Description |
|---|---|
| 15 | Extremely soft |
| 30 | Very soft |
| 45 | Soft |
| 90-110 | Moderately soft |
| 130 | Hard |
| 170 | Very hard |
| 230 | Excessively hard |
| 250 | Too hard for use |

(Source: 3rd Edition Design manual, 2009)

### Temporary and Permanent Hardness

Hardness can be described as temporary or permanent as shown in the Table: I.3 Carbonate hardness is called temporary because it precipitates readily at high temperatures since it is sensitive to heat and precipitates readily at high temperatures. Therefore, hardness can be removed by boiling the water. Non-carbonate hardness is called permanent because it does not precipitate readily at high temperatures.

**Table I.3: Compounds Producing Temporary and Permanent Hardness**

| Causing Temporary Hardness (Carbonate Hardness) | Causing Permanent Hardness (Non-Carbonate Hardness) |
|---|---|
| **Calcium bicarbonate [$Ca(HCO_3)_2$]** | Calcium sulphate [$CaSO_4$] |
| **Magnesium bicarbonate [$Mg(HCO_3)_2$]** | Magnesium sulphate [$MgSO_4$] |
| | Calcium chloride [$CaCl_2$] |
| | Magnesium chloride [$MgCl_2$] |

(Source: 3rd Edition Design Manual, 2009)

# APPENDIX J: BASIC STATISTICS USED IN ESTIMATION OF DESIGN OF FLOOD EVENTS

Generally, the basic statistics such as mean, standard deviation, variation coefficient and skewness coefficient give basic information on data.

### Distributions

The daily maximum rainfall for a given return period is obtained through methods such as two-parameter log-normal, three-parameter log-normal, Gumbel, type III log-person distributions as expressed here below.

***Two parameter log-normal distribution***

$$\ln \quad X_T = Y_T = \mu_Y + K_T \cdot \sigma_Y \qquad \text{(1)}$$

Where ln $X_T$ : Probable Rainfall

$Y_T$ : ln $X_T$ ............................................................ (2)

$\mu_Y$ : Average of log-value

$\sigma_Y$ : Standard Devitation of log-values

$K_T$ : Coefficient of frequency

$$K_T = \frac{\exp\left[\left(\ln(1+z^2)\right)^{1/2} \cdot t - \frac{1}{2}\left(\ln(1+z^2)\right)\right] - 1}{z} \qquad \text{(3)}$$

Where z: coefficient of variation

t: Standard normal deviate

***Three parameter log-normal distribution***

$$\ln \quad X_T = Y_T = \mu_Y + K_T \cdot \sigma_Y \qquad \text{(4)}$$

Where $X_T$ : Probable Rainfall

$\mu_Y$ : Average of log-value

$\sigma_Y$ : Standard Deviation of log-values

$K_T$ : Coefficient of frequency

$$K_T = \frac{\exp\left[\left(\ln(1+z_2^2)\right)^{1/2} \cdot t - \left[\frac{\ln(1+z_2^2)}{2}\right] - 1.0\right]}{z_2} \qquad \text{(5)}$$

$$\text{Where } z_2 = \frac{1 - w^{\frac{2}{3}}}{w^{\frac{1}{3}}} \qquad \text{(6)}$$

$$w = \frac{-\gamma_1 + (\gamma_1^2 + 4)^{1/2}}{2} \qquad \text{(7)}$$

***Type I extremely distribution (Gumbel)***

$$X_T = \mu + K_T \cdot \sigma \quad \text{.......} (8)$$

Where $X_T$ : Probable Rainfall

$K_T$ : Coefficient of frequency

$\mu$ : Average

$\sigma$ : Standard Deviation

where $$Y_m = Y_T = -\ln[-\frac{\ln(T-1)}{T}] \quad \text{.......} (9)$$

$$\mu_y = (\sum_{i=1}^{n} y_i)/n \quad \text{.......} (10)$$

$$\sigma_y^2 = \{\sum_{i=1}^{n}(y_i - \mu_y)^2\}/m$$

$$y = \alpha(X - \beta) \quad \text{.......} (11)$$

$$\alpha = 1.2825/\sigma$$

$$\beta = \mu - 0.4500\,\sigma$$

***Log Pearson type III distribution***

$$\ln \quad X_T = Y_T = \mu_Y + K_T \cdot \sigma_Y \quad \text{.......} (12)$$

Where $X_T$ : Probable Rainfall

$K_T$ : Coefficient of frequency

$\mu$ : Average

$\sigma$ : Standard Deviation

$$K_T = t + (t^2 - 1)\frac{Y_1}{6} + \frac{1}{3}(t^3 - 6t)(\frac{Y_1}{6})^2 - (t^2 - 1)(\frac{Y_1}{6})^3 + t(\frac{Y_1}{6})^4 + \frac{1}{3}(\frac{Y_1}{6})^5 \quad (13)$$

Where t : Probable Rainfall

$Y_1$ : Coefficient of skew

**Testing the goodness of fit**

Three goodness-of-fit tests are described in the manual. Note that X denotes the random variable and; n, the sample size.

The tests are as follows:

***Kolmogorov-Smirnov (K-S) test***

This test is used to decide if a sample comes from a hypothesized continuous PDF. It is based on the largest vertical difference between the theoretical and empirical CDF. For a random variable X and sample (x1, x2, .......xn) the empirical CDF of X (Fx(x)) is given by:

$$F(x) = \frac{1}{n}\sum_{i=1}^{n} I(X_i \leq X) \quad (14)$$

Where, I : condition = 1 if true and 0 otherwise

Given two cumulative probability functions Fx and Fy, the Kolmogorov-Smirnov test statistics (D+ and D-) are given by:

$$D_+ = max_x(F_x(x) - F_y(x)) \quad (15)$$

$$D_- = max_x(F_x(x) - F_y(x)) \quad (16)$$

***Cramer-von-Mises (CVM) test***

The criterion is a form of minimum distance estimation applied in judging the goodness-of-fit of a probability distribution compared to a given empirical distribution function in statistic.

The definition of CVM is given as follows:

$$\omega^2 = \frac{1}{n}\left\{\frac{1}{12n} + \sum_{i=1}^{n}\left[\frac{2i-1}{2n} - F(x_i)\right]^2\right\} \quad (17)$$

Where, $x_i$ : Increasing ordered data and n is the number of sample size

***Chi-squared (C-S) test***

This test simply compare show well the theoretical distribution fits the empirical distribution PDF. The C-S test statistic is given by:

$$X^2 = \sum_{i=1}^{k}\frac{(O_i - E_i)^2}{E_i} \quad (18)$$

Where, $O_i$ : The observed frequency for bin i

$E_i$ : The expected frequency for bin i

K : The number of classes

$E_i$ is given by:

$$E_1 = F(x_2) - F(x_1) \quad (19)$$

and $X_1$ and $X_2$ are the lower and upper limits for bin i.

*NB statistical method are well elaborated in different references.*

The estimation of design flood (peak discharge) is by using Rational method.

The rational method is one of the useful methods deployed in discharge scarce rivers. The major limitation of the method is that it is deployed in the small catchments due to complicity of estimating the runoff coefficient for large area with different land uses. The governing question for rational method is expressed here below:

$$Q_T = C.I_{Ifect}.A \quad \text{..........} (20)$$

Where,

QT is discharge at a certain return period,

C is run off coefficient,

$I_{Infect}$ is effective rainfall,

A is catchment area.

**Procedure**

The rainfall intensity at different return period is calculated by using combined statistical and probabilistic formula. Estimation of the intensity at different return period is determined by the following procedure:

The set of daily rainfall data collected from a station which is located is within the catchment or close to the catchment having similar rainfall patterns is used.

From the daily rainfall data, 2, 5 and 10 days rainfall were derived.

Frequency analyses of the rainfall extreme event are performed based on classified rainfall events at constant class interval. $P=n/n_t$ and T=1/P whereby; n=number of occurred extreme event, nt = total number of chances (total number of years) T=Return period and P=Probability

The classified rainfall in mm against return period should be plotted on the semi logarithmic scale and resulted on a straight line which make easier extrapolation of higher return periods (see Figure J.1).

The extrapolated rainfall is converted to different rainfall duration (i.e. 1, 2, 5 and 10 days rainfalls) so that to be plotted with their respective return periods.

The return periods of 1 to 10,000 are used during the preparation of the IDF Curve. Then, after the corresponding intensities at different return period are estimated as illustrated on Figure J.2.

**Rainfall Runoff Coefficient (C)**

Rainfall coefficient is the ratio of runoff and total precipitation over the catchment. The coefficient covers all loses due to infiltration, evaporation, land cover and other related. The coefficient should be estimated based on the available hydrological reference. After estimating runoff coefficient, runoff of the catchment area may be computed at the different return periods.

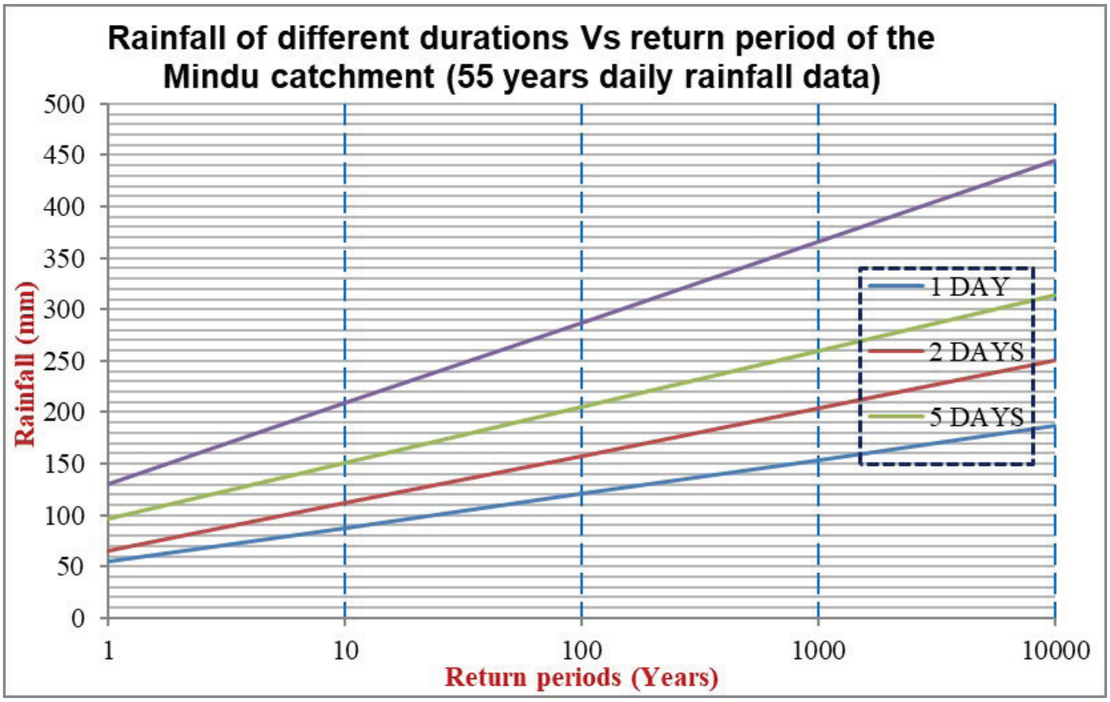



Figure J.1: Typical Example Linearized Rainfall at Different Return Periods

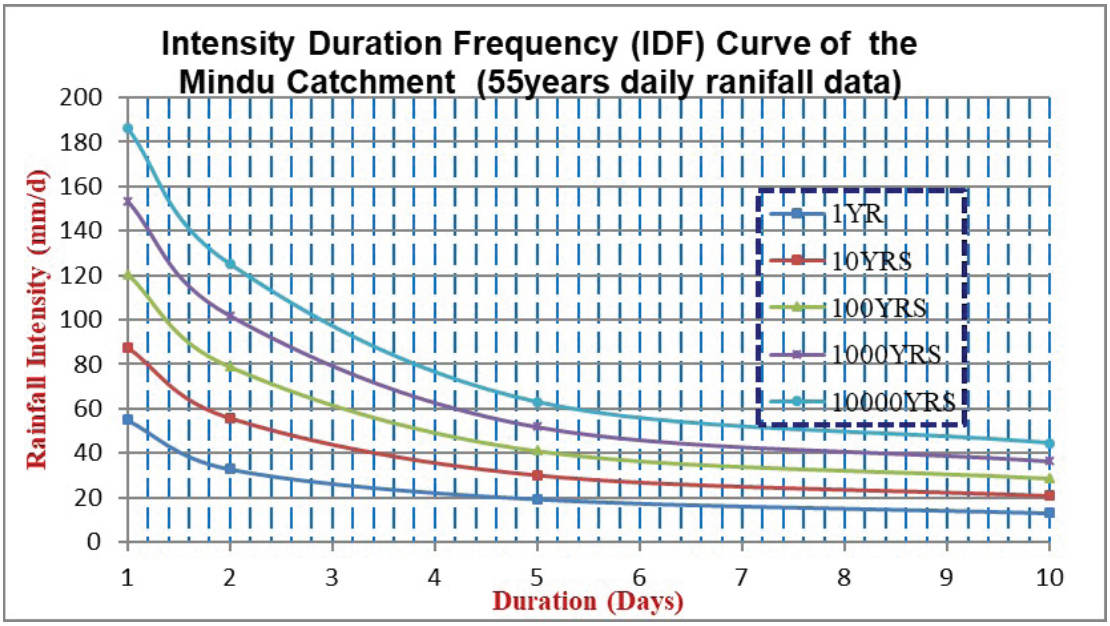



Figure J.2: Typical Example IDF Curve

# APPENDIX K: DESIGN OF DAMS FOR WATER SOURCES

## K.1 Introduction

In this Appendix, condensed information has been provided in order to assist design engineers to design dams that can provide water for domestic use, to support livestock as well as for commercial and some industrial use. It should be borne in mind that the major water demand will be for domestic and livestock use. Furthermore, in Tanzania, majority of the existing dams for water sources are earthfill dams and only a few are rockfill dams. Furthermore, where a water source comes from a concrete dam, the primary purpose of that dam is for hydropower generation and hence, the dam design will be undertaken by the respective power generation agency. The Appendix has been broken into 20 sections that include:

- Types of Dams,
- Dam and Reservoir Pre-Surveys and Detailed Studies,
- Reconnaissance of Potential Dam Sites,
- Empirical Methods for Capacity Evaluation of Dam Construction Sites,
- Preliminary Data Collection (Pre-design),
- Designs of Earthfill Dams,
- Height of the Dam,
- Design of the Embankment Filter,
- Check of the Rip-rap Adequacy for the Safety of the Structure,
- Seepage and Stability Analysis of Earthfill Dams,
- Sediment Yield in the Catchment,
- Estimation of Evaporation from the Reservoir,
- Different Types of Spillways,
- Free board (Fb) Estimation,
- Estimation of Freeboard due to wave action ($f_{wave}$),
- Embankment Crest Width (W),
- Embankment Slopes,
- Design Requirements for Tailings Storage Facility (TSF),
- Dam Break/Failure Analysis,
- Design Drawings.

## K.2 Types of Dams

The general choice of a dam will depend to a large extent on the site selected for it. There are mainly three general types of dams that include:

- Earthfill dams which also includes rockfill dams;
- Concrete and Masonry dams;
- Sub-surface dams.

In the case of earthfill dams, the pressure exerted by their weight spreads over a much greater area by virtue of having flatter embankment slopes. A rockfill dam consists of an embankment made of stones of irregular sizes and shape faced either with concrete or masonry earth and clay. Concrete dams require foundations of much greater bearing pressure (strength) than earthfill dams which can be more easily constructed. A slight settlement in the foundation of a concrete or masonry dam may give rise to fractures whereas the same can often be well accommodated in the earth-fill dams. Technically, concrete dams are rigid structures while earthfill and rockfill dams are deformable structures. Because concrete dams are not common in water supply projects in Tanzania, no further discussion of these type of dams is presented in the manual.

Sub-surface dams are usually constructed by excavating a trench within the sand-bed of a river during the dry season which is then filled with impervious material that may or may not be zoned. Further design specifications of dams for water sources focus on earthfill dams. Rock-fill and earth-fill dams are usually built in relatively broad valley sites because large-sized machines can be operated efficiently in such sites. The selection of the dam type should be carefully made, firstly basing this on materials procurable at the dam site or in the neighbourhood, and secondly in case of fill type and where this can be found. Rain and temperature greatly affects the work period available.

A flood spillway is an indispensable part of any dam structure and therefore its size, type and the natural conditions involved in its layout often proves to be a governing factor in the selection of dam type. Because of the cost of spillways and the high sediment load in main watercourses, designers should always look at the possibility of off-channel storage sites on tributaries as these can often prove to be the more cost effective alternative, even though they often involve pumped storage.

## K.3 Dam and Reservoir Pre-Surveys and Detailed Studies

Pre-surveys should be undertaken for each of the factors listed below. Subsequent to the pre-surveys, detailed studies have to be conducted on each of the following aspects:

(a) Precipitation and reservoir evaporation,
(b) Inflow of sediments from main and any of the tributary rivers,
(c) Planned highest water levels,
(d) Watershed surroundings,
(e) Water quality,
(f) Topographical data,
(g) Geology of the area,
(h) Foundation characteristics,
(i) Preliminary location and hydraulic analysis of the spillways,
(j) Water demand and use conditions,
(k) Choice of the most suitable dam site.

## K.4 Reconnaissance of Potential Dam Sites

A suitable site for a good reservoir shall depend upon the following factors:

(a) Geological condition of the catchment area. This should be such that percolation losses are minimum and favours only maximum runoff,

(b) The reservoir site should be such that the quantity of leakage through it should be minimum. Reservoir sites having highly permeable rocks reduce water tightness of the reservoir. Good types of rocks that may not allow passage of much water includes shales, slates, schists, gneisses, and crystalline igneous rocks such as granite,

(c) The dam should be founded on sound, water tight rock base, and percolation below the dam should be minimum. The cost of the dam is often a controlling factor in selection of the dam site,

(d) The reservoir basin should have a narrow opening in the valley so that the length of the dam is less,

(e) The cost of real estate for reservoir including road, rail road, dwelling relocation, etc., must be as little as possible,

(f) The topography of the reservoir site should be such that it has adequate capacity without submerging excessive land and other properties,

(g) The site should be such that a deep reservoir is formed. A deep reservoir is preferable to a shallow one because of:
- Lower cost of land submerged per unit of capacity,
- Less evaporation losses because of reduction in the water spread area,
- Less likely hood of weed growth.

(h) The reservoir site should be such that it avoids or excludes water from those tributaries which carry a high percentage of silt in water,

(i) The reservoir site should be such that the water stored in it is suitable for the purpose for which the project is undertaken. The soil and rock mass at the reservoir site must not contain any objectionable soluble minerals and salts (Punmia et al., 2010).

## K.5 Empirical Methods for Capacity Evaluation of Dam Construction Sites

The cross sections of the proposed dam sites are usually plotted after acquisition of data from the field by using horizontal interval and the elevations taken by GPS during reconnaissance. The plotted cross sections will show that, there is a potential of building a dam of a certain type and at different heights and embankment lengths. The proposed dam sites may face different challenges including a small catchment area and hence poor water yield that can fill the dam, catchment degradation due to different human activities and hence sediment transport challenges along the river valleys, etc. All these challenges have to be addressed during the design. Usually the proposed dam sites have to be refined during detailed design phase as they may be shifted on either side downstream or upstream for appropriate dam axis setting with respect to appropriate dam heights and their capacities.

Through site verifications, a number of potential dam sites can be identified and verified with their initial capacities being empirically computed. Three empirical methods may be used to make rough estimation of the reservoir capacities and these are illustrated as methods 1, 2 and 3 as described in the next paragraphs. The appropriate dam capacities will further be appropriately determined by using contours during the detailed survey stage of undertaking the project. Materials for construction (sand, gravel and stones) of such water storage facility are to be investigated and if possible to be cheaply and locally available within the project sites with clearly noted distance from the respective sites in-line with the specifications as presented in Chapter Twelve of the Volume I of the DCOM Manual.

### K.5.1 Method No.1: Queensland Water Resources Commission

The capacity for each dam site is estimated in accordance with an appropriate empirical formula to get initial guess of the expected potential storage volume for each dam. The reservoir capacity is worked out based on empirical formula estimation as per the Queensland Water Resources Commission method (cited by Nkuba, 2009). The Commission developed a method which involves determining the shape of the gully/valley slice across section and then selecting a corresponding shape from a list each of which has its own co-efficient between 0.5 and 1.6 as illustrated in Figure K.1.

$$V_{storage} = 0.22(K) \times W \times D \times L \quad \text{.........} \quad (1)$$

Where:

$V_{storage}$ = Volume of water stored ($m^3$),

L = Longest length of water surface/fetch distance (m),

W = Width of the water across the dam wall (m),

D = Water depth at the base of the embankment (m),

K = Appropriate coefficient for gully/valley shape ranging from 0.5 to 1.6 as indicated in Figure K.1, whereas according to the shape from the plot in Figure K.1, 0.5 factor can correlate to the site conditions.

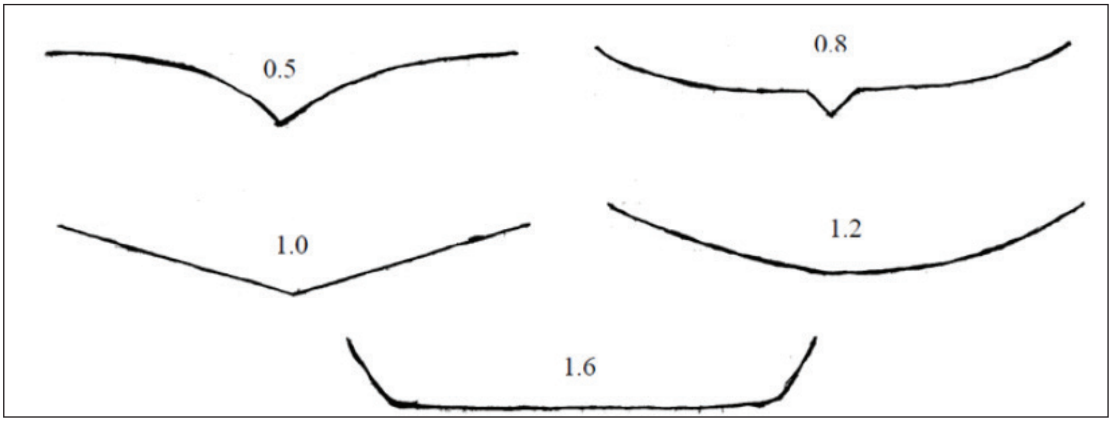



**Figure K.1: Gully Profile with Shape Factor K Varying from 0.5 to 1.6**
(Source: Nkuba, 2009)

### K.5.2 Method No.2: Best Practice from FAO Experience

Reservoir storage capacity can alternatively be estimated basing on field experience of FAO as follows (FAO, 2010):

$$Q = \frac{LTH}{6} \quad \text{.......} \quad (2)$$

Where: Q = The storage capacity in m$^3$ and should not exceed the catchment Yield (Y).

L = The length of the dam wall at full supply level (FSL) in m.

T = The throwback, in m and approximately in a straight line from the wall.

H = The maximum height of the dam, in m, at FSL.

6 is a factor (generally conservative) that can be adjusted (to 5 or 4) with experience and local knowledge. All the parameters in method 2 can be determined by the use of a level or theodolite (or any accurate GPS equipment) at the site. The capacity estimated in this way is accurate to within about 20 percent, but it must be revised by a more detailed survey when the site has been approved for possible construction. The formula considers the water volume to be an inverted pyramid with a triangular surface area (LT/2) and H/3 for the height/depth, and is a simplification of reality. With experience, one is able to judge fairly accurately how an individual valley will compare with such an idealized picture and therefore, to adjust the resulting conclusions (FAO, 2010).

### K.5.3 Method No.3: Nelson Equation

Nelson (1991) equation as cited by Nkuba (2009) is one of the recommended equations for quick dam capacity estimations.

Capacity (C) = 0.264*D*W*TB ........................................................................ (3)

Where:

C = Dam capacity ($m^3$),

D = The maximum water depth, i.e. the difference in elevation between lowest point in the reservoir bed at the dam and spillway crest level (m),

W = The width of water surface at the dam at the spillway crest level (m),

TB = The "throwback" at the spillway crest level (m).

## K.6 Preliminary Data Collection (Pre-design)

K.6.1 Geotechnical Investigations

In all major Civil Engineering projects, geotechnical investigations are important in guiding designers on the most essential items to be considered. This important item may only take 0.25 - 1% of the whole cost of the project. Such investigation is required to give detailed information on the following aspects (Punmia et al., 2010):

(a) Water tightness of the reservoir Basin,
(b) Suitability of the foundation for the Dam,
(c) Geotechnical and structural features including; folds, faults, fissures, lineaments etc., of the underlying rock at the site,
(d) Type and depth of overburden (superficial deposit),
(e) Location of permeable and soluble rocks, if any,
(f) Ground water conditions in the region,
(g) Location of quarry sites and other sources for materials to be used for dam construction and the estimated quantities of the intended materials,
(h) Study of the geology for the catchment area as it has effects on the runoff and percolations,
(i) The geology of the dam site is important from the point of view that it secures the foundation for the dam,
(j) There should be special requirements for the geology of the reservoir site such that there is no danger of serious leakage when the ground is under pressure from the full head of water in the reservoir,
(k) The nature of the sub surface geology should be explored by trial bores or various means of geological explorations.

### K.6.2 Catchment Yield

The total quantity of surface water that can be expected in a given period from a stream at the outlet of its catchment is known as the "yield" of the catchment in that period. In determining the catchment potential in yield to the dam reservoir, some basic factors are usually considered. Such factors include catchment area, catchment characteristics and the designed maximum flood expected to pass through the spillway during extreme events.

### K.6.3 Topographical Surveys

This stage is executed when the site is suitable for dam construction and already being confirmed against its potential to store enough water. Topographical surveys will include contours generation after taking the ground levels as per directed intervals by the Engineer. It is important to take the levels at the reservoir area and locations of the spillway, other related infrastructure with the dam, appropriate contours, and spatial arrangement of facilities with readable scale. Propose survey control points for horizontal and vertical movements of the embankment, baseline information, propose spacing and bench marks usually away from the Tailings Storage Facility (TSF) embankment. Some of the relevant and popular equations which may be used for volume computation include using areas covered by the contours and those include the following:

a) End Area Method: $V = h\frac{(A_1 + A_n)}{2}$ ........................................................ (4)

b) Cone Formula: $V = \sum \frac{h}{2}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$ ............................................... (5)

c) Prismoidal formula:

$$V = \sum \frac{h}{3}(A_1 + 4A_2 + A_3) = \frac{h}{3}\{(A_1 + A_n) + 4(A_2 + A_4 + \ldots) + 2(A_3 + A_5 + A_7 + \ldots)\}$$

.......................................................................................................... (6)

d) Trapezoidal formula $V = \sum \frac{h}{2}(A_1 + A_2) = h\left[\frac{A_1 + A_2}{2}\right] + A_2 + A_3 + \ldots A_{n-1}$ ... (7)

Where: *V* = the reservoir Capacity, or Volume of storage for the designed reservoir, $A_n$ = the area of the contour corresponding to the water surface elevation in the proposed reservoir and *h* = Contour interval (Punmia etal., 2010).

### K.6.4 Catchment Area

Catchment area estimation is nowadays done by digitizing the map of the project area, using GIS computer based tool (Arcview, ArcGIS, etc.). The total catchment area for each dam site catchment is normally computed in sq. km-unit.

### K.6.5 Flood Estimation for Dam Design

Some of the existing popular methods used for flood estimation include the following:

(a) Envelope curves,

(b) Empirical formulae,
(c) Physical indication of past floods,
(d) Rational method,
(e) Probable Maximum Precipitation (PMP) chart,
(f) Rating curves or rating equations,
(g) Unit hydrograph method,
(h) Flood frequency analysis,
(i) Frequency distribution functions.

Out of all these methods, the five (5) commonly used ones include empirical formula, rational method, unit hydrograph method, Flood frequency analysis and Frequency Distribution Functions. However flood frequency analysis and frequency distribution functions methods based on probability and statistics are the most favourable and these give quite reliable flood values compared to the rest of the methods. Frequency Distribution Functions for extreme flood prediction values include the following:

- Gumbel's extreme value distribution,
- Log Peason Type III distribution,
- Log normal distribution.

These have been further discussed in Chapter Three of Volume I of the DCOM Manual.

## K.6.6 Estimation of Discharge for Charco and Small Dams

**(a) Mean Annual Runoff**

The Mean Annual Runoff (MAR) can be estimated from a dam catchment conventionally by being expressed as an equivalency of runoff depth, in mm. The simplest method of estimating mean annual runoff is to apply a runoff coefficient to the mean annual runoff as illustrated below.

MAR=ROC*MAP ........................................................................................ (8)

Where:

MAR = Mean annual runoff

MAP = Mean annual precipitation (mm)

ROC = Runoff coefficient

This approach is used in many of the simpler small dam design manuals, where for semi-arid areas the runoff coefficient is often set at 0.1. Runoff depends on many parameters in addition to rainfall and this is recognized in some procedures, which adjust the runoff coefficient to account for land forms, soil type and drainage, and cover etc. This approach requires a considerable quantity of catchment specific data that will be not readily available to most small dam designers (Nkuba, 2009).

### (b) Mean Annual Flood Peak Discharge

A spillway must be designed to pass the design flood without damage, and the peak flood without the dam overtopping, i.e. the peak flood must be contained within the dry freeboard. The design has to be checked by ensuring that the 250 year return flood can be passed without the dam overtopping i.e. without the additional head needed exceeding the dry freeboard. Using regional frequency method, the mean annual peak flood (MAF) is estimated as follows (Nkuba, 2009):

$MAF = 0.114*(CA^{0.52})*MAP^{0.537}$ ........................................................................ (9)

Where:

MAF= Mean annual flood peak discharge ($m^3/s$),

CA = Catchment area ($km^2$),

MAP = Mean annual precipitation (mm).

Thus, the 1 in 100 year and 1 in 250 year return period floods are the calculated as multiples of MAF as follows:

$Q_{100}$=6.51*MAF and

$Q_{250}$=9.86*MAF

### (c) Specific Yield of the Catchment

The majority of earth dam failures (around 90%) in the world are due to under-designed spillway or under estimated floods which cause overtopping of the embankment followed by erosion of the downstream slope and finally the breaching of the embankment. Consequently, obtaining accurate figures for the flood expectations is the most important part of the design from the safety point of view. Figures in Table K.1 are for $Q_{100}$ or 100 year return period from flood studies in Kenya as cited by Nkuba (2009). These should be used with great personal judgment based on catchment characteristics and knowledge of rainfall patterns of the catchment.

**Table K.1: Catchment Area and Corresponding Flood Magnitudes from Flood Studies in Kenya (cited by Nkuba, 2009)**

| S/No. | Catchment Area ($Km^2$) | Q(flood) in $m^3/s/km^2$ of the catchment |
|---|---|---|
| 1. | <1 | 15 |
| 2. | 1-5 | 12 - 10 |
| 3. | 5-25 | 8 - 6 |
| 4. | 25-100 | 3 - 2 |
| 5. | 100 -1000 | 1 - 0.4 |
| 6. | >1000 | <0.3 |

The required design flood = Specific Yield of the Catchment [$m^3/s/km^2$] x Catchment Area [$Km^2$].

## K.7 Design of Earthfill Dams

### K.7.1 The Design of Earthfill dams includes the following components:

(a) Embankment design (filter),
(b) Dam height,
(c) Foundation (Core trench),
(d) Rip-rap adequacy,
(e) Seepage and stability analysis,
(f) Catchment sediment yield,
(g) Estimation of Evapotranspiration or Evaporation from a reservoir,
(h) Different Types of Spillways,
(i) Freeboard estimation including wave action,
(j) Embankment crest width,
(k) Embankment slopes,
(l) Design for TSF,
(m) Dam break/failure analysis,
(n) Design drawings.

### K.7.2 Important Data

(a) The dam is intended for use mainly for domestic and livestock water demand (allows for commercial and institutional demands),
(b) Population of the area (people and livestock),
(c) Design period of the proposed dam structure,
(d) Catchment area in sq. km,
(e) Ground levels along the proposed dam axis and within the reservoir,
(f) Levels of the proposed dam at different heights.

### K.7.3 Design Assumptions

(a) The dam shall be mainly for Livestock and Domestic demand as qualified above,
(b) The design period shall be for 50 years,
(c) The water losses shall be taken as 20% of the total volume for evaporation and seepage,
(d) Embankment shall be safe against overtopping by provision of adequate spillway capacity and sufficient free board as well as being stable against seismic aspects,
(e) There should be no seepage to allow free flow of water,
(f) Water should not infiltrate through bed/cracks of embankment and hence proper compaction is necessary,
(g) The upstream and downstream slopes of the embankment should be stable under all conditions. This will be taken care of through the provision of stone pitching/ riprap to the upstream face to protect the embankment from wave

action and grass plantation on the downstream face to protect possibility of soil erosion.

## K.8 Height of the Dam

Dam height refers to the embankment height taken at the lowest ground level along the valley of the proposed dam axis to the embankment top level. Note that the embankment top level includes also the freeboard height of the dam.

The height of a storage dam is governed by two factors:

- The amount of storage that can be developed economically, and
- The storage volume afforded by the topography of the reservoir site.

The height of the dam above the bed of the stream is determined by the level to which it is desired to raise the inflowing stream. This may however be restricted by the limitation of funds, by the cost of land which will be under water, by interference with highways railways water power development schemes, by the backwater curve effect upstream of the reservoir, by the need to relocate inhabitants in the reservoir area, or any other interests having prior rights on the land, etc.

**Environmental conditions and their protection:** Detailed ESIA should be conducted to come up with positive and negative impacts and proposed enhancement and mitigation measures, respectively. Some further literature reviews should be done to confirm the validity of previous studies. Finally, field collected information should also be confirmed through further web searches.

## K.9 Design of the Embankment Filter

In order to lower the phreatic surface within the embankment section, filter materials are designed so as to allow safe evacuation of seepage in the dam embankment section without carrying away fine construction materials. The movement of seepage with fines is what is called piping. This phenomenon is mitigated by designing filter materials that allow a reasonable quantity of seepage to pass through the embankment wall without harming the structure. During the execution of the task, the sieve analysis test results of the construction materials plotted on the particle size distribution curve are used for the establishment of a sound relationship between the construction materials and the filter materials designed to be placed at the downstream toe of the embankment. Using the same principles, the dam supervisor should establish the grading curve envelope for the filter materials by using the following empirical equations.

$$\frac{D_{50}\,(Filter)}{D_{85}\,(Soil)} \leq 5 \qquad \text{.......................................................................} (10)$$

$$\frac{D_{15}\,(Filter)}{D_{15}\,(Soil)} \geq 5 \qquad \text{.......................................................................} (11)$$

$$\frac{D_{50}(Filter)}{D_{85}\ (Soil)} \leq 25 \quad \text{.......} (12)$$

Whereby $D_{50}$ etc. refers to the passing size etc.as determined from particle size distribution analysis of the materials. Equations (10) and (11) set out piping and permeability criteria, respectively; Equation (12) further defines the permeability ratio. During construction of the facility the supervisor will have to find suitable materials to fit the criteria established empirically in equation (10) to (12). Therefore, the materials used for filter design should follow the empirically established relationship. Hence, the supervisor should establish a grading curve for filter materials plotted on sieve analysis result of the construction materials.

## K.10 Check of the Rip-rap Adequacy for the Safety of the Structure

The riprap size and layer depends on the side slope of the embankment and the wave height calculated or estimated. During computation of the riprap sizes the following guidelines are used.

According to the table of related values developed by Sentürk, the riprap weight ($D_{50}$) and the size of riprap can be extracted. During the evaluation process, values of riprap size were obtained depending on the mentioned empirically developed tables, whereby wave heights and embankment slope were the input parameters (Sentürk, 1994, pp. 459-464).

According to Varshney, riprap sizes were computed by using the equation (13) (Varshney at el., 1982).

$$D_m = 2.23.C.\frac{h_w \gamma_w}{(\gamma - \gamma_w)}.\frac{\sqrt{1+s^2}}{s\sqrt{2+s}} \quad \text{.......} (13)$$

Whereby:

$D_m$ is the diameter of stone brought to form a ball in m,

$\gamma$ is the unit weight of the stone,

$\gamma_w$ is the unit weight of the water,

$h_w$ is the height of wave in m,

$s$ is the slope of the embankment,

$C$ is the factor depending on the type of the protection. For hand placed riprap C is 0.54; For dumped riprap C is 0.85.

## K.11 Seepage and Stability Analysis of Earthfill Dams

### K.11.1 Seepage analysis

Seepage analysis is computed by using the finite element equation which operates by construction of triangles whose nodes are used as collection points of seepage from upstream to downstream. The following equation represents the finite element equation which operates under combination of the continuity equation and Darcy’s law. The combination of the continuity equation and the Darcy's law gives the two dimensions Laplace finite element equation as given below.

$$\frac{\partial}{\partial x}\left(-k_x\frac{\partial \varphi}{\partial x}\right)+\frac{\partial}{\partial y}\left(-ky\frac{\partial \varphi}{\partial x}\right)=Q \quad \text{.......} (14)$$

Whereby:

kx and ky are the permeability in x and y direction respectively,

Q is the discharge along the boundaries or the discharge of internal sources,

Seepage analysis is computed by Geostudio software package such as Seep/W, Deltares software such as Mseep and other related software. The computed phreatic line from this software is exported to the stability analysis software for stability analysis.

### K.11.2 Stability Analysis

During stability analysis of the embankment dams, the most important factor to be taken into account is the input data analyzed during geotechnical investigations and laboratory soil test and assumptions made. There are several methods used to compute the stability of the embankment.

There are also many methods used for stability analysis of embankment slopes. Some of the commonly used methods are the Bishop Method slice method (automatic generated slip plane), Spencer method (manual constructed slip plane), Fellenius (automatic generated slip plane), uplifting Van method, uplifting Spencer method, Finite element method (time and data demanding method) and horizontal balance method (Deltares, 2011).

Based on the existing problem and the available computation methods, Bishop is a convenient model to be deployed due to the fact that the method incorporates both forces and moment balance of the slices during stability computation. Furthermore, the method is faster and accurate (Deltares, 2011); (USACE, 2003). This method is applied with the condition of the slip plane failure method; the main assumptions of the method are:

The forces between the slices are neglected in such a way that the forces are assumed to be acting in the horizontal direction. The shear strength along the slip plane is assumed to be constant for the entire slip circle. The equilibrium of the vertical forces moments are taken into account while the horizontal forces moments are neglected.

***Limitations***

The method does not fulfil the horizontal forces equilibrium since it excludes the computation of the horizontal forces' moments. If there is a great dispersion of the construction materials on the structure, the method is not recommendable due to its assumption of equal shear strength along the slip circle.

In spite of the limitations of the method, it is still considered to be among the best in the world due to the accurate results obtained out of it. Based on the professional recommendations, the method is accurate and simple to use even by hand, the computations can be executed. In this manual the emphasis is given to the use of the Bishop method whose equation in expressed below.

$$F = \frac{\dfrac{\sum\left[c'\Delta x + (W + P\cos\beta - u\Delta x\sec\alpha)\tan\varphi'\right]}{m_\alpha}}{\sum W\sin\alpha - \dfrac{\sum M_p}{R}} \qquad \text{.......... (15)}$$

$$m_\alpha = \cos\alpha - \frac{\sin\alpha\tan\varphi'}{F} \qquad \text{.......... (16)}$$

Whereby:

$\Delta x$=width of the slice, $\varphi'$=angle of internal friction for Mohr-Coulomb diagram plotted in terms of effective normal stress, $\sigma'$ (degrees), $\beta$=inclination from horizontal of the top of the slice (degrees), R=Radius of the circle, $\alpha$=inclination from the horizontal of the bottom of the slice, W=Weight of the slice, c'=effective cohesion of the material, ma=term used in simplified bishop method, Mp=moment produced by the force P about the centre of the slice and u= pore water pressure.

**Scenarios used during stability analysis of earthfill dam:**

During computation of the stability of the embankment, the following are the possible scenarios which are to be assumed and computed using the Geostudio software package such as Slope/W and Deltares software such as Mstab:

- Earthquake at normal reservoir operation level,
- Earthquake under flood reservoir operation level,
- Safety at rapid drawdown condition of the reservoir.

## K.12 Sediment Yield in the Catchment

The estimation of sediment yield from the catchments is important for soil and water conservation practices in catchments during planning, design and operation of reservoir. Some few available procedures for such an exercise are (Punmia et al., 2010):

- Flow duration curve and sediment rating curve procedure,
- Reservoir sedimentation surveys,
- Estimation of water catchment erosion and sediment delivery ratio.

Sediment yield refers to the amount of sediment exported by a basin over a period of time, which is also the amount that will enter a reservoir or pond located at the downstream limit of the basin (Morris and Fan, 1998). The estimates of long-term sediment yield have been used for many decades to size the sediment storage pool and estimate reservoir life. Sediment yield from the dam catchment is one of the parameters for controlling sedimentation of small dams. The correlation of sediment yields to erosion is complicated by the problem of determining the sediment delivery ratio, which makes it difficult to estimate the sediment load entering a reservoir/pond on the basis of erosion rate within the catchment. This has to be estimated if future sedimentation rates in a dam are to be predicted. In literature (Ndomba, 2011) developed sediment yield prediction equations by regression analysis approach for small catchments in Tanzania in terms of climatic zones of dry, moderate and wet zones. He generated equations according to the specific zones in terms of the area of catchment that can be used to generate estimated amounts of sediment load. The map below illustrates the sediment zones as developed (Ndomba, 2011).

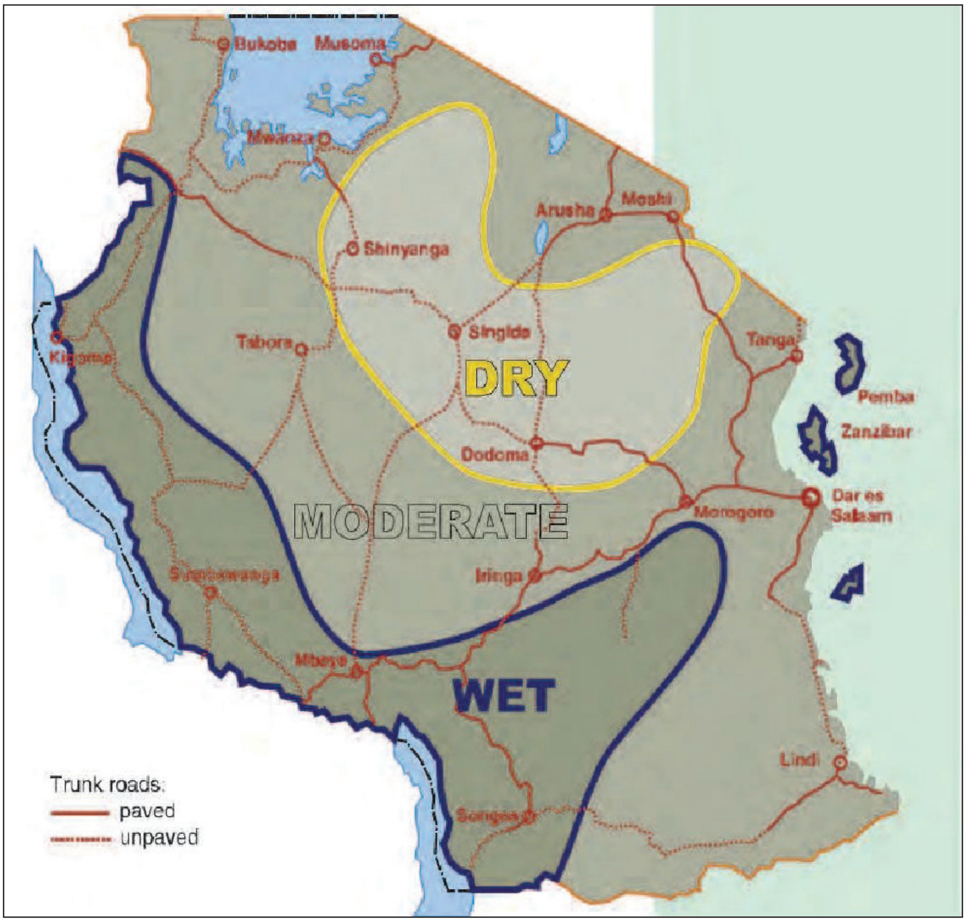



**Figure K.2: Rainfall Zones of Tanzania**

(Source: URT, 1999 as cited by Ndomba, 2011)

According to Ndomba (2011), three major regions were established as dry, moderate and wet zones in the whole of Tanzania and equations were fitted with correlations at an appropriate confidence interval. For dry and moderate climatic zones the fitted equation is:

$$SF = 556.2\ A^{0.4313} \qquad (17)$$

Where: SF= Sediment fill ($m^3$/yr); A = Catchment area ($km^2$, the equation for sediment fill-yield was fitted in catchment areas for both dry and moderate climatic zone (all regions being analyzed) satisfactorily to small catchment at 95 % confidence interval. Table K.2 shows the detail for more fitted equations for specific regions according to their climatic zones (Ndomba, 2011).

The sediment yield in a particular catchment will be used to estimate the dead storage portion of the reservoir depending on the sediment transport rate per annum (Sediment loading to the reservoir).

### K.12.1 Sediment measurement and computation for sediments in water streams and rivers

The total sediment load that is transported out of the catchment by a stream is classified into (Subramanya, 2008):

- **Wash load:** This is composed of fine grained soils of very small fall velocity, originated from land surface of the watershed and is transported into streams by means of splash, sheet, rill and gully erosion. Such portion is taken care of during water treatment process (coagulation and flocculation process).
- **Suspended load:** Refers to relatively finer bed materials that are kept in suspension in the flow through turbulence eddies and transported in suspension mode by flowing water. The suspended load particles move considerably long distances before settling on the bed and sides. This portion of sediment forms a small part of total sediment load (usually<25%). This is the commonly measured part of sediment in gauged rivers by sampling the river flow.
- **Bed load:** Refers to relatively coarse bed load material that is moved at the bed surface through sliding, rolling and siltation. Usually for planning and design purposes, the bed load of a stream is estimated by either the use of available bed load equations such as Einstein equation bed load equation or is taken as a certain percent of the measured suspended load.

For planning and design purposes, the bed load of a stream is estimated by using either the available bed load equations such as Einstein equation or being taken as a certain percentage of measured suspended load. Usually the sediments from the collected sample of sediment by filtering and its dry weight is measured. It is expressed in parts per million (ppm). Suspended sediment concentration ($C_s$) is given by:

## Table K.2: Fitted Sediment Yield Prediction Equations for Specific Climatic Zones

| Climatic zone | Region (s) | No. of data points, n | Range of α=Coefficient ± Standard Error | Average values of α | Range of β =Coefficient ± Standard Error | Average values of β | Sediment Fill/Yield prediction Equation (SF or SY=αA^β) | $R^2$ |
|---|---|---|---|---|---|---|---|---|
| Dry climatic zones | Dodoma, Shinyanga, & Singida | 18 | 508.42 – 663.8 | 580 | 0.3712 – 0.4914 | 0.4313 | $SF = 580A^{0.4313}$ | 0.4586 |
| | Dodoma | 13 | 471 - 815 | 619.6 | 0.0636 – 0.3288 | 0.1962 | $SF = 619.6A^{0.1962}$ | 0.5708 |
| Moderate climatic zone | Tabora | 17* | 505 - 802.8 | 637.2 | 0.2989- 0.4745 | 0.3867 | $SF = 637.2A^{0.3867}$ | 0.7591 |
| | Arusha | 11 | 333.43 – 31974.22 | 3264.37 | 0.4609 – 0.5613 | 0.511 | $SF = 3264.4A^{0.511}$ | 0.7689 |
| Dry and Moderate zone | All regions analyzed** | 41 | 439.2 – 632.27 | 556.16 | 0.3824 – 0.4802 | 0.4313 | $SF = 556.2A^{0.4313}$ | 0.7318 |

Note:
* Only 70% of data points for Tabora region were used to fit the regression relationship. Thirty percent (30%), i.e. 7 data points were used for validation purposes.

** The data for Arusha region was not included in fitting the regression for all regions (i.e. dry and moderate climatic zone) as it presents itself with unique soil/erodibility characteristics

$$C_s = \left( \frac{Weight\,of\,sediment\,in\,the\,sample}{Weight\,of\,(Sediment + Water)\,of\,the\,sample} \right) \times 10^6 \quad \text{......................(18)}$$

Thus sediment transport rate in a stream of discharge Q [m/s] is given by (Subramanya, 2008):

$Q_s = (Q \text{ X } C_s \text{ X } 60 \text{ X } 60 \text{ X } 24)/10^6 = 0.086QC_s[tones/day]$....................................(19)

The estimations for bed load is given in Table K.3.

**Table K.3: Estimations of Bed Load from Suspended Loads (Subramanya, 2008)**

| S/N | Concentration of suspended load (ppm) | Type of material forming the stream channel | Texture of suspended Material | Percentage of measured suspended load that could be taken as Bed Load |
|---|---|---|---|---|
| 1 | Less than 1,000 | Sand | Similar to bed material | 25 to 150 |
| 2 | Less than 1,000 | Gravel, rock or consolidated Clay | small amount of sand | 5 to 12 |
| 3 | 1,000 t0 7,500 | Sand | Similar to bed material | 10 to 35 |
| 4 | 1,000 t0 7,500 | Gravel, rock or consolidated Clay | 25% sand or less | 5 to 12 |
| 5 | Over 7,500 | Sand | Similar to bed material | 5 to 15 |
| 6 | Over 7,500 | Gravel, rock or consolidated Clay | 25% sand or less | 2 to 8 |

## K.12.2 Methods for Estimation of Bed Load

The popular methods for this case include the following estimation formulae:

**(i) Einstein Bed Load Function**

This is the widely used approach of which defines the intensity of bed load transport and given by:

$$\phi = \frac{G_i}{w}\sqrt{\left(\frac{\rho}{\rho - \rho_s}\right)\left(\frac{1}{gd^3}\right)} \quad \text{......................(20)}$$

and flow intensity

$$\psi = \left(\frac{\rho_s - \rho}{\rho}\right) \times \frac{d}{gR} \quad \text{......................(21)}$$

Whereby:

*Φ* = Intensity of Bed Load transport, *Ψ* =Flow intensity, *R* = Hydraulic radius, $G_1$ = Bed load transport per unit width of the river, *w* =Specific weight of water, *d* = diameter of grains and *g* =acceleration due to gravity.

**(ii) Meyer-Peter, Muler Equation**

$$G = 5.1844B \times \left(\frac{10.844QB}{Q}\right)\left(\frac{D_{90}}{n_p}\right) \times ds - 0.637Dm \quad \text{......................................(22)}$$

G = Bed Load [t/day], B = Width of rivers[m], $Q_B$ = Water just flowing over the bed load [$m^3/s$], $D_{90}$ = Particle size of stream of bed such that 90% of the particle is finer than this size, $n_p$ = Maning's n for the river bed, $D_m$ = Effective Bed material size (weight mean diameter) in mm, *d* = mean diameter of water and *s* = Hydraulic gradient

**(iii) Du Boys Formula**

Bed load transport has been based on classical solution of Du Boys and given as:

$$G_i = \gamma \frac{\lambda_0}{w}(\lambda_0 - \lambda_c) \quad \text{......................................(23)}$$

Whereby: $G_1$ =Bed load transport per unit width of river, $\gamma$ =empirical coefficient depending on the size and shape of sediment particles, *w* =Specific weight of water, $\lambda_0$ = Shear stress at stream bed, $\lambda_0$ = Critical or magnitude of shear stress at which transport starts.

**(iv) Laurence and Garde and Albert formula**

$$q_s = C_4 \frac{V^4 n^3}{d_p^{1.5} d} \quad \text{......................................(24)}$$

Whereby:

$q_1$ = sediment transport rate per unit width of the river, $C_4$ = constant of river, *V* = water velocity, *n* = Maning's roughness of the river bed, $d_p$ = sediment diameter, *d* = depth of water (Punmia et al., 2010).

**Empirical estimation of Reservoir Basing on Catchment Area**

(i) **Khosla's equation:** The annual sediment yield on volume basis is related to the catchment area as below. The annual sediment yield rate (on volume basis) is given by the equation:

$$q_{sv} = \frac{0.00323}{A^{0.28}} \quad Mm^3 / km^2 / year \quad \text{......................................(25)}$$

or Volume of sediment yield per year from a catchments:

$$Q_{sv} = 0.00597A^{0.72} \quad Mm^3 / year \text{ ......} (26)$$

where, $A$ = Area of catchment[km²]

(ii) **Joglekar's equation:** Based on data from reservoirs from India and abroad, Joglekar expressed the annual sediment yield rate as:

$$q_{sv} = \frac{0.00597}{A^{0.24}} \quad Mm^3 / km^2 / year \text{ ......} (27)$$

or Volume of sediment yield per year from a catchments:

$$Q_{sv} = 0.00597A^{0.76} \quad Mm^3 / year \text{ ......} (28)$$

where,  = Area of Catchment[km²]

(ii) **Dhruv Narayan et al. 's Equation:** Annual Sediment rate is related to annual runoff as (Subramanya, 2008):

$$Q_s = 5.5 + 11.1Q \text{ ......} (29)$$

Whereby:

$Q_s$ = Annual sediment yield rate [tones/year] from upstream catchment of reservoir,

$Q$ =Annual runoff volume [M.ha.m]

### K.12.3 Reservoir Sediment Control

Sediments in reservoirs can be reduced if the following methods in catchment and reservoir site can be taken with extra care. The common methods for sedimentation control include:

(a) Proper selection of a reservoir site with less sediment inflow to the river,
(b) Control of sediment by constructing a series of low dams upstream of the reservoir,
(c) Management of the catchment by vegetal cover to arrest the sediment to flow with runoff,
(d) Soil conservation and watershed management like strip cropping, control farming, crop rotation, terracing, benching, on steep hill slopes, control and management of grazing field, small embankment to prevent erosion and arrest sediment,
(e) Removal of sediment from reservoir by excavation, draining, and flushing by sluices after disturbing the sediment by mechanical method,
(f) Opening dam sluices to remove sediment coming in during flood time,
(g) Constructing the dam in stages,
(h) Afforestation and control of deforestation in the reservoir catchment.

**Note: All the above methods can be summarized into three major types as mentioned below.**

(i) ***Reduction in Sediment yield from catchments (effective catchment conservation).*** This include:
- Environmental friendly tree planting, terracing, strip cropping, etc.,
- Check dams to reduce sediment inflow to the stream,
- Vegetal cover, grassed waterways and afforestation to reduce run off rates and hence reducing erosion.

(ii) ***Reduction in rate of accumulation of sediment in reservoirs.*** This will include the following aspects:
- Provision of scouring sluice at lower elevations in the dam in order to flush out high concentration of sediments and density currents,
- Appropriate operation of gates overflow outlets and other sluices in the dam.

(ii) ***Physical removal of already deposited sediments (Draging) just after reservoir sedimentation survey*** (Bathmetric Survey)

Sedimentation against reservoir functions Subramanya, (2008) and Mohan Das and Saikia (2009):

a) Sedimentation reduces the useful life of reservoir if not properly managed,
b) Outlet scouring sluices may be blocked by sediments,
c) Aggredations upstream of dam due to sediment deposition,
d) Degradation below reservoir by outflow from scouring sluices.

## K.13 Estimation of Evaporation or Evapotranspiration from a Reservoir

Estimation of evaporation (E) or evapotranspiration (ET) is one of the essential parameters in designs of reservoirs, Irrigation canals, water balance and any project relating to water projects. Sometimes, only evaporation as a process of water loss in vapour form due to sun radiations is enough instead of ET especially when inside the reservoir there is negligible vegetation. The key factors for evaporation includes solar radiation, vapour pressure, temperature, wind velocity, atmospheric pressure, area of water surface, quality of water surface, nature of evaporating surface, salinity of water, depth of water in the water body and humidity (Mohan Das and Saikia, 2009).

### K.13.1 Methods for Measurement or Estimation of Evaporation

The cheapest method for reservoir evaporation estimation in Water Resources Panning is setting reasonable percentage of live capacity of reservoir as evaporation losses through long term experience or by using different hydrological studies within the project area. A value of 15% of live storage capacity is adopted for major water projects and 25% of live storage capacity for minor water projects. Generally there are different evaporation methods which include the following (Punmia et al., 2010):

(a) Empirical formulae,
(b) Water budget method or storage equation,
(c) Energy budget equation,
(d) Mass transfer method,
(e) Combined energy and mass transfer approach,
(f) Pan Measurement method,

Mohan Das and Saikia (2009).

## K.14 Different Types of Spillways

### K.14.1 Introduction

Spillways are provided for storage and detention dams to release surplus or flood water which cannot be contained in the allocated storage space, and at dams to bypass flows exceeding those which are turned into the diversion system. Ample spillway capacity is of paramount importance for earth fill and rock fill dams, which are likely to be destroyed if overtopped, whereas concreted Spillway requirements are:

- Spillway must have capacity to spill safely the excess water from the reservoir,
- A spillway must be hydraulically and structurally adequate,
- It must be located or provided with an energy dissipating outlet structure, such that spillway discharges will not cause downstream erosion or undermine the toe of the dam (downstream (d/s) of the dam).

**Design considerations:**

Spillway components include:

- A control structure to regulate flow into the channel,
- A discharge channel to convey the flow released through the control device to a stream below the dam,
- A terminal structure which enables the spillway discharge to return into the stream without scour or erosion of the toe or downstream side of the dam.

**Type of spillways:**

(a) Free over-fall (straight drop),
(b) Ogee (over flow),
(c) Side channel,
(d) Open channel (through a chute),
(e) Conduit,
(f) Tunnel,
(g) Baffled apron drop,
(h) Culvert and apron,
(i) Morning glory,
(j) Drop inlet (shelf or morning glory) [This type should not be considered except on very small dams or on off-river tributaries with very small catchments].

## K.14.2 Spillway Types

A spillway is a hydraulic structure built at a dam site for diverting the surplus water from a reservoir after it has been filled to its maximum capacity. Spillways are classified into different types on the basis of the arrangement of the control structure, a conveyance channel and a terminal structure. Different types of spillways include the following (Source: The constructor, 2020):

(a) Straight drop spillway
(b) Ogee spillway
(c) Shaft spillway
(d) Chute spillway
(e) Side channel spillway
(f) Siphon spillway
(g) Labyrinth spillway

The selection of a certain type of spillway will depend on the following factors:

(a) Topography of the proposed spillway site (Steep slopes call for chute spillway)
(b) Geological formation: the more stable geological formation the less erosive power of the spillway, hence low operation and maintenance of the spillway
(c) Hydraulic characteristics of the flow (subcritical flow is more preferred in side channel spillways)
(d) Availability of land for location of spillway in the constricted land, morning glory and overflow spillways may be suitably applied
(e) Hydrological condition of the catchment (the width and height of the spillway depends on the magnitude of the estimated flow to be evacuated)

### (i) Straight Drop or free over fall Spillway

A Straight drop spillway consists of low height weir wall having its downstream face roughly or perfectly vertical. When the water level in the reservoir rises above the normal pool level, the surplus water falls freely from the crest of the weir.

To prevent the scouring of downstream bed from falling water jet, an artificial pool with a concrete apron and low secondary dam is constructed on the downstream side. Proper ventilation should be provided on the underside portion of a falling jet to prevent pulsating and fluctuating effects. Sometimes, an overhanging projection is provided on the crest of the weir to prevent the entrance of small discharges onto the face of the weir wall. Straight drop spillways are most suitable for thin arch dams, earth-fill dams or bunds.

### (ii) Ogee Spillway

Ogee spillway, as the name says, represents the shape of the downstream face of the weir. It is an improved form of a straight drop spillway. In this case, the downstream face of the weir is constructed corresponding to the shape of lower nappe of freely falling water jet which is in ogee shape. The ogee shape of the

**Figure K.3: Typical example of Straight Drop Spillway**
(Source: The Constructor, 2020)

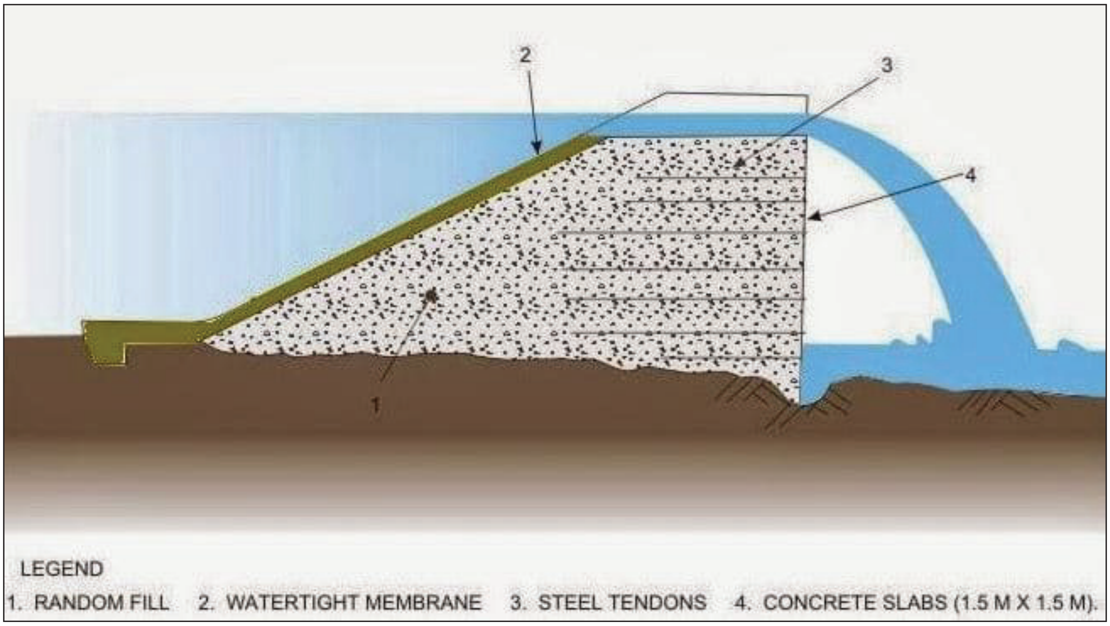



**Figure K.4: Typical Example Straight Drop Spillway Components**
(Source: The Constructor, 2020)

downstream face is designed on the basis of the principle of a projectile. In general, the shape of lower nappe of the water jet is not constant for all water heads hence, the shape obtained for the maximum head is taken into account while designing ogee spillway.

**Figure K.5: Typical Example of Ogee Spillway of Walayar Dam, India**
(Source: The Constructor, 2020)

Whenever there is surplus water, it will be freely disposed of through ogee spillway along its ogee shaped crest hence it can also be called as an overflow spillway. Ogee spillways are most commonly used in cases of gravity dams, arch dams, buttress dams, composite dams, etc. For gravity dams, it is generally located within the dam body. According to K Subramanya in 2012 it is suggested that, Ogee spillway with S-shaped over flow profile is the most extensively used to safely pass the flood flow out of the reservoir.

**(iii) Shaft Spillway**

A Shaft spillway is a type of spillway which consists of a vertical shaft followed by a horizontal conduit. The surplus water enters into the vertical shaft and then to the horizontal conduit and finally reaches the downstream of the channel.

**Figure K.6: Typical Example of Shaft Spillway**
(Source: The Constructor, 2020)

The shaft constructed is either artificial or natural. Excavation for the natural shaft is possible only when the hard rocky layer is present on the upstream side. The horizontal conduit either passes through the dam body or through the foundation of the dam. In the case of large projects, the inlet hole of the vertical shaft is specially shaped which is called morning glory or glory hole of the spillway. Hence, shaft spillway is also called **Morning Glory Spillway or Bell Mouth Spillway**. Shaft spillway is recommended when there is no space to provide for other types of spillways such as ogee spillway, straight drop spillway, etc.

**Figure K.7: Typical Example of Morning Glory Spillway, Monticello Dam, USA**
(Source: gfycat.com)

### (iv) Chute Spillway

Chute spillway is a type of spillway in which surplus water from upstream is disposed to the downstream through a steeply sloped open channel. It is generally constructed at one end of the dam or separately away from the dam in a natural saddle in a bank of the river. Chute spillway is suitable for gravity dams, earthfill dams, rockfill dams, etc. But it is preferred when the width of the river valley is very narrow. The water flows along the steeply sloped chute or trough or open channel and reaches the downstream of the river. Chute spillway is also called as trough spillway or open channel spillway. The slope of chute spillway is designed in such a way that the flow should be always in supercritical condition. To dissipate energy from the falling water, energy dissipaters can be provided on the bed of chute spillway.

**Figure K.8: Typical Example of Chute Spillway, Tehri Dam, India**
(Source: The Constructor, 2020).

### (v) Side Channel Spillway

Side channel spillway is similar to chute spillway but the only difference is the crest of side channel spillway is located on one of its sides whereas crest of chute spillway is located between the side walls. In other words, the water spilling from the crest is turned to 90 degrees and flows parallel to the crest of side channel spillway unlike in chute spillway.

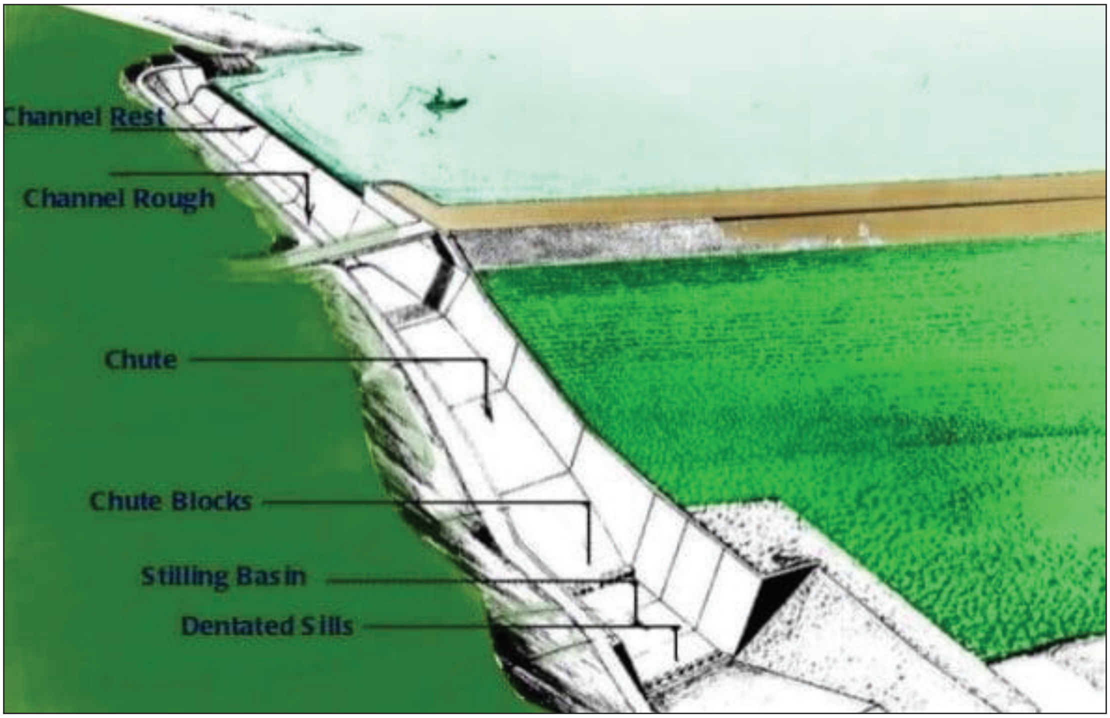



**Figure K.9: Typical Example of Side Channel Spillway**

(Source: The Constructor, 2020)

Side channel spillways are preferred over chute spillways when flanks of sufficient width are not available, usually to avoid heavy cutting. The angle of turn of water flow after passing weir crest can also be kept between 00 and 90.

**Figure K.10: Typical Example of Side Channel Spillway of Hoover Dam, USA**

(Source: The Constructor, 2020)

### (vi) Siphon Spillway

A siphon spillway is a type of spillway in which surplus water is disposed to downstream through an inverted U shaped conduit. It is generally arranged inside the body or over the crest of the dam. In both types of siphon spillways, air vents are provided at the bent portion of the upper passageway to prevent the entrance of water when the water level is below the normal poll level. Whenever the level rises above normal pool level, water enters into the conduit and is discharged to the downstream of the channel by siphonic action.

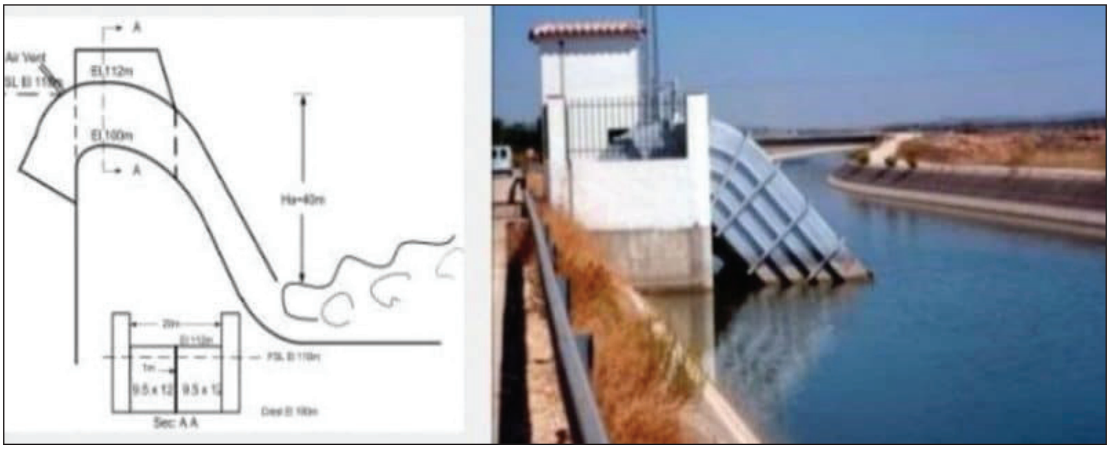



**Figure K.11: Typical Example of Siphon Spillway**

(Source: The Constructor, 2020)

### (vii) Labyrinth Spillway

A labyrinth spillway is a type of spillway in which the weir wall is constructed in a zigzag manner in order to increase the effective length of the weir crest with respect to the channel width. This increase in effective length raises the discharge capacity of the weir and hence higher water flow at small heads can be conveyed to the downstream easily.

**Figure K.12: Typical Example of Labyrinth spillway of Ute Dam, Mexico**
(Source: The Constructor, 2020)

## K.14.3 Spillway Dimensioning

The spillway is dimensioned after estimating the flood design which is a function of peak flood or probable maximum flood estimated from the catchment that contributes yield to the dam reservoir. The spillway is provided to accommodate any excess water above full supply level. The governing formula depends on the type of spillway decided by an Engineer as there are several types of spillway and morphology.

The commonest type of spillways and simple to handle is **over fall spillway** (ogee/weir type) the formula was suggested by Creager, Justin, Hinds 1945 and US Bureau of Reclamation (*Novack et al., 2001*). This formula has to be simplified to quicken the design and it is given by:

$$Q_d = \frac{2}{3} \times C_d \times b_{weir} \times \sqrt{2g}\left[\left(h_d + \frac{V_{apr}}{2g}\right)^{3/2} - \left(\frac{\alpha V_{apr}}{2g}\right)^{3/2}\right] \qquad (30)$$

Whereby $Q_d$ = Design discharge ($m^3/s$)

$C_d$ = Coefficient of discharge

$b_{weir}$ = Width of the spillway (m)

g = Acceleration due to gravity ($m/s^2$)

$V_{apr}$ = Approach velocity (m/s)

α = Curve factor

$h_d$ = Height of water above the crest weir (m)

From above the following equation the following are always assumed:

- Approach velocity ($V_{apr}$) is very small ≈ 0; the velocity that water passes over the crest weir after heating the crest structure.
- Pressure on the spillway is greater than atmospheric
- Coefficient of discharge of discharge is in the range $0.578 < C_d < 0.745$; Cd is assumed to be 0.62

The formula is reduced and simplified to:

$$Q_d = \frac{2}{3} \times C_d \times b_{weir} \times \sqrt{2g} \times h_d^{3/2} \quad \text{.........} (31)$$

By using trial and error or excel program the value of $b_{weir}$ can be computed by trying different values of $h_d$. Different values of $h_d$ at different trials give different values of $b_{weir}$ and hence a need for an accurate judgment for an appropriate value to be used with respect to site conditions. The shape of the profile of the crest downstream of the apex can be expressed by using the equation:

$$\frac{y}{H_d} = K\left(\frac{x}{H_d}\right)^n \quad \text{.........} (32)$$

where K and n are constants and their values depend upon the inclination of the upstream face and depend also on the velocity of approach. Typical values for K and n are given in Table K.4

**Table K.4: Typical Values for K and n of the Crest Downstream of the Apex**

(Source: Novack et al., 2001)

| S/N. | Upstream face | K | n |
|---|---|---|---|
| 1 | Vertical | 0.500 | 1.850 |
| 2 | 1Horizontal: 1/3 vertical | 0.517 | 1.836 |
| 3 | 1Horizontal: 1vertical | 0.534 | 1.776 |

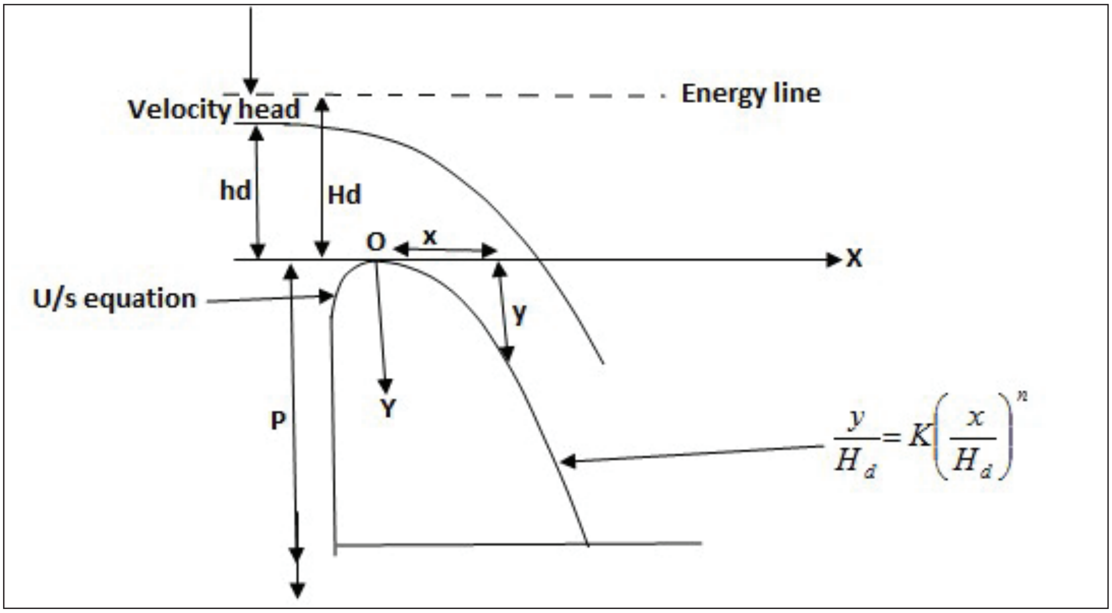



**Figure K.13: Typical Elements of Spillway Crest**

Where: X = Coordinate, Y = Coordinate, O = Original of coordinates and apex of crest, Hd = Design energy head, P = Pressure zone

The crest profile upstream of the apex is usually given by a series of compound curves. Cassidy reported that, the equation for the upstream portion of a vertical faced spillway as here below

$$\frac{y}{H_d}=0.724\left(\frac{x}{H_d}+0.270\right)^{1.85}-0.432\left(\frac{x}{H_d}+0.270\right)^{0.625}+0.126 \qquad (33)$$

This equation is valid for the following region $0\geq\frac{x}{H_d}\geq-0.270$ and $0\leq\frac{y}{H_d}\leq0.126$ and the coordinate system is the same as the downstream profile equation (Novack et al., 2001).

### Shaft Spillway

$$Q_d=\frac{2}{3}\times C_d\pi D_c\times\sqrt{2g}\times H^{3/2} \qquad (34)$$

Where by $Q_d$ = Design discharge ($m^3/s$)

$C_d$ = Coefficient of discharge

$D_c$ = Cone diameter of the spillway (m)

g = Acceleration due to gravity ($m/s^2$)

H = Height of water above the cone shape (m)

(Novack et al., 2001).

**Siphon Spillway**

This kind of spillway at very low flows operates as a weir; as the flow increases the upstream water level rises, the velocity in the siphon increases, and the flow in the lower leg begins to exhaust air from the top of the siphon until this primes and begins to flow full as a pipe with a discharge given by:

$$Q_d = C_d A(2gH)^{1/2} \quad \text{.......... (35)}$$

Whereby $Q_d$ = Design discharge ($m^3/s$)

$C_d$ = Coefficient of discharge

A = throat cross-section of the siphon

g = Acceleration due to gravity ($m/s^2$)

H = Difference between the upstream water level and siphon outlet or downstream water level if the downstream is submerged and

$$C_d = \frac{1}{(k_1 + k_2 + k_3 + k_4)^{1/2}} \quad \text{.......... (36)}$$

where $k_1$, $k_2$, $k_3$, and $k_4$ are head loss coefficients for the entry, bend, exit and friction losses in the siphon (Novack et al., 2001).

## K.15 Freeboard (Fb) Estimation

This is a very important parameter in dam designing Engineering. It is the vertical distance between the top of the dam and the full supply level in the reservoir. The top of the dam is the highest water tight level of the structure and could thus be the top of the water tight parapet. The free board has the following components:

(a) Rise in reservoir level due to flood routing(flood surcharge),
(b) Seiche effects,
(c) Wind set up of the water surface, and
(d) Wave action and run-up of waves on the dam.

Components ii, iii and iv are considered to be the key aspects in free board estimation and they are all sometimes termed as wave free board. Another component may be considered to account for effects of shore instabilities (rock fall, landslides etc). In embankment dams, the total free board must also include embankment and foundation (core trench) settlements as every material has its allowable settlement.

In estimating of free board the following are important:

- General engineering judgments depending on the prevailing situation of the site,
- Hydrological analysis(statistical analysis),
- Damage that would result from the overtopping of the dam.

Estimation of each of the components are clearly illustrated below:

(i) The Seiche parameter is usually ignored especially in medium sized reservoirs and their effects are included in a safety margin added to other components. The effects of Seiche goes as high as 0.5 m in some of the very large reservoirs.

(ii) Wind set up (wind tide) depends on reservoir depth(d), wind fetch (F) of which is the maximum free distance which wind can travel over the reservoir, usually in km units, the angle of the wind to the fetch(θ) and the wind speed U in kmh$^{-1}$. The Zuider Zee equation can be used to combine all these aspects as follows:

$$S=\frac{U^2FCos\theta}{kd} \qquad (37)$$

where S = wave set up in m and k = 62000 when using m units. This equation was modified further into the following equation:

$$S=k_1\frac{U^2}{gd}FCos\theta \qquad (38)$$

Where, $k_1 = 2x10^{-6}$

(iii) Allowance for wave height (H) and the run up ($f_w$) of wind generated waves are the most significant components of free board estimation for a reservoir. Estimation of these two parameters are given (measured in m units) below:

$$H=0.34F^{1/2}+0.76-0.26F^{1/4} \qquad (39)$$

where F = Fetch distance (m)

For large values of fetch (F>20km) the last two terms in the above formula may neglected and becomes:

$$H=0.34F^{1/2} \qquad (40)$$

The above first equation was also further modified to:

$$H_{hw}=0.034(UF)^{1/2}+0.76-0.24(F)^{1/4} \qquad (41)$$

it includes wind speed

In case of medium to large reservoirs wave free board (Hw) is given by:

$$H_{hw}=0.75H+\frac{c^2}{2g} \qquad (42)$$

where H = Wave Height that does not consider wind speed,
g = 9.81m/s$^2$, c =wave propagation velocity (m/s)
But c = 1.5+ 2H, hence

$$f_w=0.75H+\frac{(1.5+2H)^2}{2g} \text{ (m units)} \qquad (43)$$

(Novack et al., 2001).

"Normal" freeboard should be based on a wind velocity of 160 km/hr; and "Minimum" freeboard on 80 km/hr. Based on these assumptions and other considerations of the purpose of freeboard as previously discussed, the following tabulation lists the "LEAST" amount recommended (USBR) on riprapped earthfill dams.

**Table K.5: Recommended "Least" Normal and Minimum Freeboard Values**

| Fetch (km) | Normal Freeboard (m) | Minimum Freeboard (m) |
|---|---|---|
| < 1.6 | 6.4 | 4.8 |
| 1.6 | 8.0 | 6.4 |
| 4.0 | 9.6 | 8.0 |
| 8.0 | 12.8 | 9.6 |
| 16.0 | 16.0 | 11.2 |

(Source: Kalyo, 2018)

It is recommended that the amount of freeboard shown in this table should be increased by 50% if a smooth pavement is to be provided on the upstream slope. The minimum height of the freeboard for wave action is generally to be equal to "1.5 ":

Where:

For $F < 32\ km;$ $\quad h_{wv} = 0.032\sqrt{v.F} + 0.763 - 0.271F^{1/4}$ ............... (44)

For $F > 32\ km;$ ....... $h_{wv} = 0.032\sqrt{v.F}$ ............................................................ (45)

$h_{wv}$ = Wave height i.e. Height of water from top of crest to bottom of trough (in *m*);

*v* = Wind velocity (in *km/hr*); and

*F* = Fetch or straight length of water expense (in *km*)

(Source: Kalyo , 2018)

**Table K.6: Recommended "Least" Normal and Minimum Freeboard Values**

(Source: Kalyo,2018)

| Nature of Spillway | Height of Dam H (in *m*) | Freeboard f over Maximum Water Level (in *m*) |
|---|---|---|
| Uncontrolled (i.e. free) Spillway | Any height, *H* | $2.0 \leq fbd \leq 3.0$ |
| Controlled Spillway | *H* < 60 | 2.5 Above top of Gates |
| | *H* < 60 | 3.0 Above Top of Gates |

## K.16 Estimation of Freeboard Due to Wave Action ($f_{wave}$)

In order to estimate the freeboard due to wind action, different empirical formulae are applied and averaging is highly advisable. The following terminologies are commonly applied in the estimation of the freeboard due to wave action.

Fetch length: is the horizontal distance over which the wind blows across the reservoir surface.

***Effective fetch length:*** is the averaged distance which is computed to cover the malt directional of the wind when it blows into the reservoir

***Wave height:*** is the height attained by the wave when it hits on the dam wall.

***Wave set up:*** is the increase in mean water level due to presence of waves

***Wave run up:*** is the height which can be attained by the wave when it hits on the sloping surface of the embankment. Due to the availability of a sloping surface the wave will tend to raise an extra height, this is what is termed wave run up.

### *(i) Wave height computation ($Z_d$)*

The following formulae were used to compute the wave height for both dams.

Molitor's formula ($Z_d$)

$Z_d = 0.032\sqrt{VF} + 0.75 - 0.27\sqrt[4]{VF}$ For F< 32 Km................................................(46)

$Z_d = 0.032\sqrt{VF}$ For F>32 Km.......................................................................(47)

For shallow waves

$Z_{da} = Z_{dd}\left(1 + \frac{V}{420+V}\right)$ ..............................................................................(48)

Ocean Formulas

$Z_d = 0.034\, V^{1.06}\, F^{0.47}$ ..............................................................................(49)

Thomas Stefenson's formula

$Z_d = 0.034\sqrt{VF} + 0.75 - 0.26\sqrt[4]{VF}$ For metric units...........................................(50)

U.S. Army Corps of Engineers Formula

The US Corps suggests using the empirical curves whereby, the wave height is interpolated in the curves depending on the wind velocity of the area and effective fetch length of the reservoir (Sentürk, 1994). Figure K.14 represents the curve which is used to define the wave heights for the embankment dams.

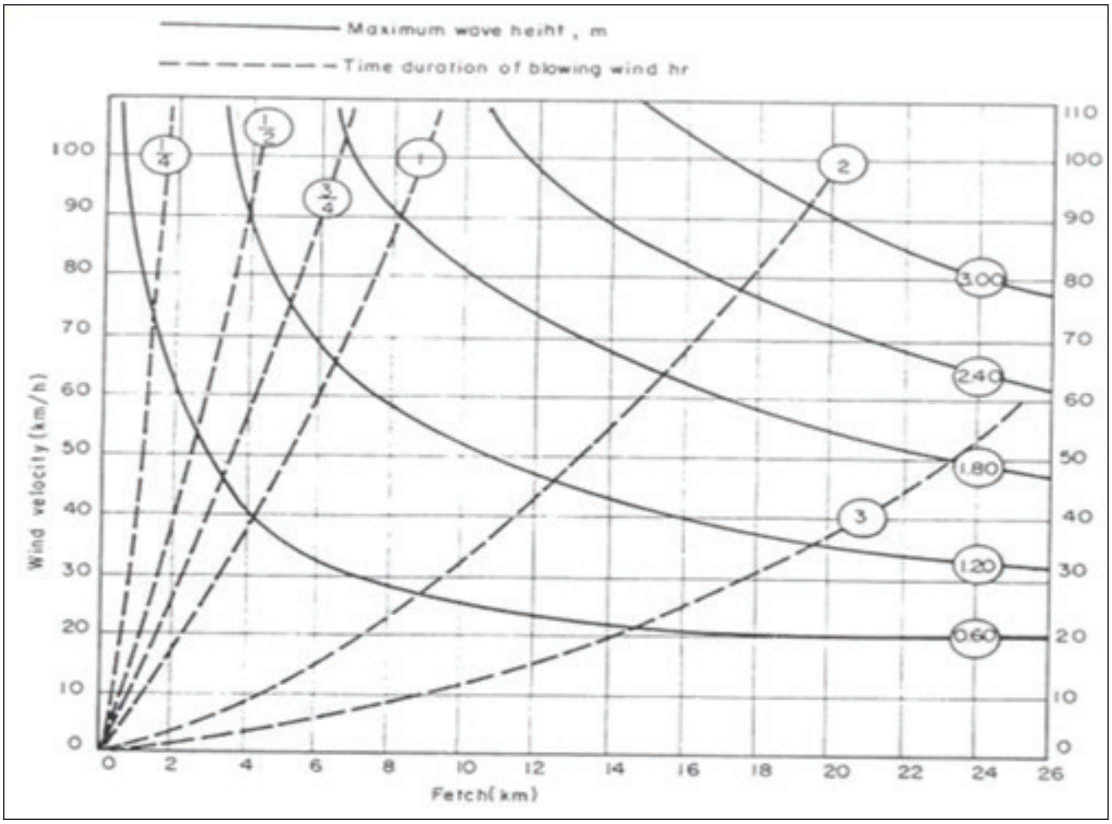



**Figure K.14: USACE Curve for Interpolation of Wave Height**
(Source: Senturk, 1994)

### *(ii) Wave run up ($Z_r$)*

The value of wave run up is obtained first by calculating the value of $t_r$, then after the value of λ should be estimated. Since the value of wave height is known from the computations above, the ratio of $Z_d/\lambda$ was calculated hence $Z_r$ is obtained from the graphical established relation by McClendon and Cochran (Sentürk, 1994, page 422).

$$t_r = 0.46V^{0.44}F^{0.28} \quad \text{.......... (51)}$$

$\lambda = 5.15t_r^2$ Then the value of λ is computed.

$Z_r/Z_d = k$ Where by the value of κ is obtained in the curve, and then the value of $Z_r$ is to be computed.

### *(iii) Wave set up ($Z_s$)*

The wave set up was calculated by using Zuider-Zee formula as illustrated below:

$$z_s = \frac{V^2}{63000d}Cos\theta \quad \text{.......... (52)}$$

Therefore the total freeboard due to wave action obtained by; $f_{wave} \geq Z_d + Z_r + Z_s$

Whereby:

$V$ is the wind velocity in km/hr,

$Z_d$ is the wave height in m,

$\theta$ is the angle between the effective fetch and the actual blowing wind,

$\lambda$ is the wave length in feet,

$t_r$ is the wave period,

$d$ is the mean reservoir depth in the direction of the effective fetch in m.

The above equations adopted are summarized below:

Estimating the run-up, denoted R, subsequently determined from the height of the waves, their angle of incidence and the shape of the dam upstream face;

Estimating the local elevation of the water, denoted S thereafter, corresponding to a lower elevation of the reservoir due to the accumulation of water against the dam upstream face. Summing the two parameters, denoted by E (E = R + S), to deduce the maximum rise of the water level against the dam; adding a freeboard **f** meeting the design criteria.

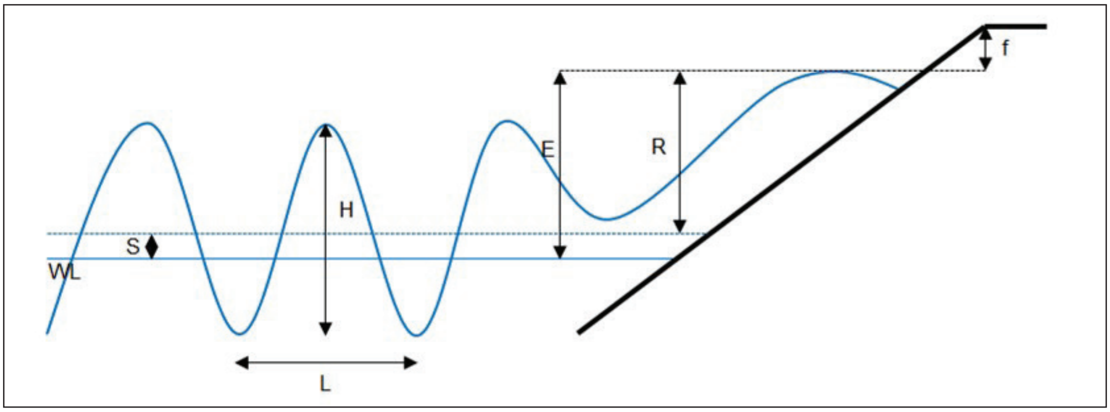



**Figure K.15: Components of a Typical Freeboard Estimates**

With :

WL Reservoir water level for the considered scenario

S Local raising of the water level

L Length of the wave

H Height of the wave

R Run-up

E Maximum rising of the water level of the dam crest

f Freeboard

## K.17 Estimation of Embankments Crest Widths

The width of the embankment should be optimized to allow the passage of heavy equipment during construction and operation and maintenance. The following are the formulae used for estimation of the embankment crest width.

Sentürk suggests that, the minimum crest width of the embankment should be 10m in seismic free areas, while in seismic areas 20% of the minimum suggested width should be added (Sentürk, 1994). The commonly recommended empirical formulas to be used for estimation of the crest embankment width are indicated below.

USBR's Formula (Sentürk, 1994); $b = \frac{h}{3} + 1.0$ ................................................ (53)

Preece (1938) in (Sentürk, 1994); $b = 1.1\sqrt{h} + 1.0$ (54)

Whereby:

*b* is the crest width in m

*h* is the dam height in m measure from the deepest point

### K.17.1 Embankment Crest Width (W)

The crest (or top) of width of large earthfill dams should be sufficient to keep the seepage line well within the dam body when the reservoir is full. It should also be sufficient to withstand earthquake shocks and wave action. For a small dam, the top width is generally governed by "minimum roadway" width requirements. Generally, the crest width depends on the following considerations:

(a) Nature of the embankment materials, and minimum allowable percolation distance through the embankment at normal reservoir water level. Minimum crest width should be the one which will provide a safe percolation gradient through the embankment at the level of the water surface,
(b) Height and importance of the structure,
(c) Possible roadway requirements, and
(d) Practicability of construction.

There are different criteria for estimating the top width of an embankment (embankment crest) and the following formulas are suggested for the determination of crest width of small earthfill dams:

For very low dams $W = \frac{H}{5} + 3$ ................................................................. (55)

where w = crest width and H = dam Height

For dams < 30 m height, $W = 0.55\sqrt{H} + 0.2H$.................................................... (56)

For dams > 30 m height, $W = 1.65(H + 1.5)$..................................................... (57)

(Novack et al., 2001)

Furthermore, width of an embankment Crest is designed as a function of the Dam height according to Horton and Jobling (Novack et al., 2001) and the formula below is also commonly used during design to dimension the best top width of the embankment.

$$W = \sqrt{H} + 1 \quad (58)$$

Where H = Height of Embankment/Dam Height.

## K.18 Embankment Slopes

The slopes of the embankment depends on the angle of repose of the fill material when it is wet and when it is dry and according to the soil results the embankment can be stable for the following slopes without causing any failure due to sloughing.

**Table K.7: Recommended Slopes for Small Homogeneous Earthfill Dams on Stable Foundations**

| Case | Type | Purpose | Soil Classification | U/S slope | D/S slope |
|---|---|---|---|---|---|
| **A** | Homogeneous or Modified-Homogeneous | Detention or Storage | GW, GP, SW, SP | Pervious, **NOT** Suitable | |
| | | | GC, GM, SC, SM, | 2½ : 1 | 2 : 1 |
| | | | CL, ML, | 3: 1 | 2½ : 1 |
| | | | CH, MH, | 3½ : 1 | 2½ : 1 |
| **B** | Modified Homogeneous | Storage | GW, GP, SW, SP | Pervious **NOT** Suitable | |
| | | | GC, GM, SC, SM, | 3 : 1 | 2 : 1 |
| | | | CL, ML, | 3½: 1 | 2½ : 1 |
| | | | CH, MH, | 4 : 1 | 2½ : 1 |

(Source: Kalyo, 2018)

**Table K.8: Recommended Slopes for Small Zoned Earthfill Dams on Stable Foundations**

| Type | Purpose | Subject to Rapid Drawdown | Shell Material Classification | Core Material Classification | U/S Slope | D/S Slope |
|---|---|---|---|---|---|---|
| Zoned With Minimum Core A | Any | Not Critical* | Rockfill, GW, GP, SW (gravelly), or SP (gravelly) | GC, GM, SC, SM, CL, ML, CH or MH | 2 : 1 | 2 : 1 |
| Zoned With Maximum Core | Detention or Storage | NO | Rockfill, GW, GP, SW (gravelly), or SP (gravelly | GC, GM, ........................ | 2 : 1 | 2 : 1 |
| | | | | SC, SM, ........................ | 2¼ : 1 | 2¼ : 1 |
| | | | | CL, ML, ........................ | 2½ : 1 | 2½ : 1 |
| | | | | CH, MH ........................ | 3 : 1 | 3 : 1 |
| | Storage | YES | Rockfill, GW, GP, SW (gravelly), or SP (gravelly | GC, GM, ........................ | 2½ : 1 | 2 : 1 |
| | | | | SC, SM, ........................ | 2½ : 1 | 2¼ : 1 |
| | | | | CL, ML, ........................ | 3 : 1 | 2½ : 1 |
| | | | | CH, MH ........................ | 3½ : 1 | 3 : 1 |

(Source: Kalyo, 2018)

## K.19 Design Requirements for Tailings Storage Facility (TSF)

(a) Tailings Storage Facility (TSF) may include a tailings dam (impoundment or pond), decant structures and spillways. A TSF can also be an open pit, dry staking or an underground storage,

(b) Tailings dam encompasses embankments, dam walls or other impounding structures, designed to enable the tailings to settle and be retained and the process water is decanted for further treatment. The impounding structure must be constructed in a controlled manner,

(c) Tailings impoundment is the storage/volume created by tailings dam or dams where tailings are deposited and stored. Tailings Storage Facility (TSF) has to be water tight without spilling as it is the general case in normal water dams,

(d) Tailings management facility shall be located, designed and be built to satisfy established criteria for stability of the downstream slope, seepage, overtopping, and earthquake and landslides resistance,

(e) The starter dam shall be designed and shall specify the internal and external geometry of the starter dam shall be specified and specifications for drainage, seepage control and in some cases liner systems required to maintain embankment stability and control release to the environment should be included. (MoW, Dam safety Regulations, 2013).

## K.19.1 Topographical Survey

Including levels of the TSF footprint area and locations of the facility, appropriate contours, spatial arrangement of facilities with readable scale. Propose survey control points for horizontal and vertical movements of the embankment, baseline information, propose spacing and bench marks usually away from the TSF embankment.

## K.19.2 Hydrological Analysis

(a) Rainfall and temperature trends including rainfall contribution to TSF supernatant water notably during extreme events,
(b) Intensity (at different return periods) duration frequency including design period for extreme events consideration,
(c) Yield from the upstream catchment and storm water management plans,
(d) Appropriate ditches to separate and covey storm water from TSF system and propose a well designed storage reservoir for the storm water,
(e) Address Sediment transport and propose the appropriate remedial measures, and
(f) General water balance within the mining proximity.

## K.19.3 Hydrogeology and Geotechnical Aspects

(a) Surface geology of the area,
(b) Seismic issues within the project area,
(c) Rock type and rock characteristics,
(d) Ground water flow direction including matrix and fracture contributions if any,
(e) Designed a number of monitoring Bore holes and design appropriate spacing, and
(f) Materials for construction.

## K.19.4 Hydraulic Issues

(a) Slope of the catchment,
(b) velocity of flow within the catchment, and
(c) Catchment cover properties.

## K.19.5 Laboratory Tests

(a) Permeability of the material,
(b) Sieve analysis,
(c) Specific gravity,
(d) Shear tests,

(e) Atterberg limit and Linear shrinkage of the material,
(f) Shrinkage limit,
(g) Slurry density, and
(h) Bulk and dry density of tailings material.

### K.19.6 Detailed Design of TSF Storage Capacity

(a) Expected Operation time or mining life of the mining depending on the mining license or mining survey (Years) of the mineral reserve,
(b) Expected rate of mining per year (tones),
(c) Density of the tailings to be achieved(tones/$m^3$),
(d) Capacity of the tailings storage facility(TSF) per year($m^3$),
(e) Conveyance of the tailings slimes from processing plant,
(f) How much water is recycled per day,
(g) Estimation of Tailings Volume to be stored in the TSF,
(h) How this can lead into resizing of the of the TSF starter dam including cross section and side slopes,
(i) Rate of embankment raising (construction),
(j) Freeboard estimation,
(k) Stability analysis including, construction method (upstream, centre line downstream etc.) appropriate beach de-positioning (single or multiple pipes), solid water ration, and consolidation aspects, and
(l) Associate the environmental and social concerns addressed in the Environmental Impact Statement.

### K.19.7 Seepage Control

(a) Low permeability of soil liner,
(b) HDPE Plastic material Liner,
(c) Decant collection system,
(d) Embankment toe drain, and
(e) Polishing ponds depends on the design assumptions and decisions of the designer.

### K.19.8 Other Designs

(a) Design of decant pond,
(b) Design of spillway in case of storm water ponds,
(c) Design of emergence water ponds if any, and
(d) Construction methods.

### K.19.9 Monitoring Aspects

(a) Solid tonnage of the TSF,
(b) Water volume of the facility,
(c) Rainfall add up and evaporation loss,

(d) Water return from TSF to processing plant (recycled water),
(e) Flow through the Decant pump, and
(f) Proper location of the tailings pipe.

## K.19.10 Monitoring Instrumentation

(a) Automatic cut outs in case of any pipe failure,
(b) Warning system for alerting whenever there is any risk associated phenomenon,
(c) Leakage detection system,
(d) Piezometers at core trench and at TSF embankment portion,
(e) Appropriate decision as to where each instrument has to be put with appropriate intervals whenever deemed necessary,
(f) Boreholes for groundwater Quality assessment including baseline information, daily operations and security aspects, and
(g) Set the daily operation schedule at site for monitoring purposes.

## K.19.11 Future Planning Issues

(a) Prepare future operation plans,
(b) Prepare rehabilitation plans including embankment raising of the TSF,
(c) Prepare Risks and emergency action plans,
(d) Plans for water reduction in the TSF in case of access water; including mechanical /forced evaporation systems, recycling and treatment of tailings water if any,
(e) Propose a closure and decommissioning plans,
(f) Make sure the TSF design is water tight, saves the operational life of the mining and not for water storage.

**(a) Sizing of TSF**

The formula for sizing the minimum area for TSF is given by (PHB Billion, 2005):

$$A_{\min} = \left( \frac{Q_{Ta}}{(q_{STa})(h_{TaR})} \right) \quad \text{.......... (59)}$$

$A_{min}$ =Minimum area of tailings storage (m$^2$), $QT_a$ = Quantity of tailings produced (tones/year), $q_{STa}$ = Dry density of stored tailings (tones/m$^3$), $h_{TaR}$ = Height of compacted tailings rise (m/year)

$$A_{\min} = \left( \frac{Q_{Ta}}{(q_{STa})(h_{TaR})} \right) = [L \times W] \quad \text{.......... (60)}$$

whereby for rectangular shape; L=2W and L= Length and W= Width

$A_{min} = 2W \times W = 2W^2$ .......... (61)

It means $A_{min} = 2W^2$ .......... (62)

and hence:

$$W = \sqrt{\frac{A_{\min}}{2}} \quad \text{......(63)}$$

The minimum area for TSF does not include the following:

- Area covered by the embankment of TSF,
- Area for polishing ponds or return water ponds associated with the TSF,
- Space to allocate monitoring Bore Holes.

If the mining life is  year, and expected production per day is  [m³/day]

$$V_{Ta} = \frac{p}{day} \times \frac{365days}{yr} \times T[yr] \quad \text{......(64)}$$

where, $V_{Ta}$ = Volume of tailings produced.

### (b) Solid concentration in tailings

If the tailings slurry has solid concentration ($f_{sc}$) say; Therefore Volume of solid tailings will be:

$$V_{Tas} = V_{Ta} \times f_{sc} \quad \text{......(65)}$$

### (c) Water content in Tailings

The volume of water from Tailings is given by considering the water content ($f_{sw}$):

$$V_{Taw} = V_{Ta} \times f_{sw} \quad \text{......(66)}$$

If there is an evaporation rate of R % per year say; Water lost through evaporation will be:

$$V_{Taw}e = V_{Taw} - V_{Taw} \times R\% \times T \quad \text{......(67)}$$

Such extra volume has to be added to the reservoir volume before the losses occur

If content for  water content of Tailings has to be recycled in a day, then some assumptions ha to be set, say 60 days to cuter for unforeseen events. Assume 60 days to cutter for T years:

$$V_{Taw} = \frac{V_{Taw} \times \tau R}{\tau_D} \quad \text{......(68)}$$

Whereby $V_{Taw}$ = Fraction of total volume of Tailings effluent for recycling [m³], $\tau_R$ = retention time for fraction of Tailings effluent for recycling [days], $\tau_D$ =Design time[days].

## (d) Free Board Estimation for TSF

The purpose of free board is to provide minimum safety margin, over and above the estimated inflows of fluids (tailing slimes) from extreme natural events and operational situations, so that the risk of overtopping leading to TSF structure erosion and ultimate failure is minimized. The probable maximum discharge and probable maximum precipitation have to be considered.

Total freeboard = Beach freeboard (Horizontal) +Operational (Vertical) freeboard

Beach freeboard is defined as the vertical height between the waterline and the beached tailings against the embankment facility. Vertical Free board has to accommodate maximum rainfall and wave effects during extreme events. Minimum considerations for free board need to be confirmed by the hydrology of the project area:

(i) For TSF with water ponds normally located away from any perimeter retaining structure

Total free board = Operational freeboard + beach free board

= 500 mm with a sub minimum of 300 mm operational free board

(ii) For TSF with water ponds normally located against a perimeter retaining structure but with no upstream catchment apart from the storage it self

Total free board = Operational freeboard = 500 mm

(iii) For TSF with water ponds normally located against a perimeter retaining structure but with upstream catchment in addition of the storage it self

Total free board = Operational freeboard = 1,000 mm

Total free board is the vertical height between the lowest point on the crest of the perimeter of retaining structure of the TSF and the normal operating pond level plus allowance for an inflow corresponding to the 1-in -100-years for 72 hours rainfall event falling in the catchment of the pond assuming that no decant delivery is taking place for the duration of rainfall event (U.S. Environmental Protection Agency, 1994).

## (e) Consideration of Stability of TSF

(i) Appropriate slope provision on both upstream and downstream according to the materials laboratory results that gives the actual angle of repose. Otherwise TSF may fail due to sloughing,

(ii) Appropriate compaction at an optimum moisture content to avoid failure due to improper compaction during construction as Laboratory tests indicate the appropriate compaction value to be attained,

(iii) Appropriate positioning of the Phreatic Line in the TSF embankment and hence avoid failure due to liquefaction in case of any shaking like seismic, abnormal blasting, heavy construction,

(iv) Storm water management to avoid overtopping due to access storm water and hence to be separated from the TSF Reservoir. Separation ditches have to be appropriately designed,
(v) Modelling for stability analysis is important to ascertain the actual field situation,
(vi) Piezometers for monitoring TSF foundation, failure due to uplift as well as over saturation of the TSF embankment due to poor operation or lining,
(vii) Monitoring Bore Holes (BHs) for groundwater looking for stability and quality aspects on downstream part of the TSF embankment. May be both shallow and deep BHs whenever there exist shallow and deep aquifers. Quality aspects and water levels are usually monitored. Proper location will be the actual flow ground water movement directions (A hydrological study report will depict the actual situation),
(viii) Monitoring of TSF movements (vertical and horizontal), Bench marks, Survey pins on top of embankment have to be established,
(ix) Operation rules shall include a monitoring Plan for all the Safety aspects (U.S. Environmental Protection Agency, 1994).

## K.20 Dam Break/Failure Analysis

Dam break analysis is very important aspect to be conducted at detailed design stage of the project. The main objective of this analysis is to predict failure of the embankment ant anticipate damage related to its failure. Through the study the following will be envisaged:

- Area to be flooded after the dam break or zone of influence,
- Anticipated property to be lost,
- Suitable area for evacuation of people and property,
- Time taken for the flood to inundate human settlement and their property,
- The magnitude of flood and anticipated water depth.

After conducting the dam break analysis, the next stage will be to prepare Emergency Preparedness plan (EPP). The EPP should comprise early warning system to the community, communication strategy in case of an emergency and evacuation procedure. Therefore, this should commence at Design stage of the project and be refined after completion of the project implementation.

During the execution of this task, inundation maps have to be well prepared and warning signs should be placed in the flood prone area to inform and warn the community. The communication strategy should be well defined to the community to be affected by the project hazard.

## K.21 Design Drawings

- Design drawings should be well prepared so as to enable the contractor of the project to execute the project smoothly,

- Some of the details should be drawn in three Dimension to enable easy understanding by technical and non-technical staff,
- The scale used for preparation of details should be checked for clear visibility and pleasant appearance,
- All drawings of the project should be drawn to scale,
- The preferable size of drawing paper is $A^2$ which is easy for filling and handling,
- The layout scale for topographical map should range from 1:2000 to 1:25000 depending on the size of the project. The paper size of more than $A^2$ may be used on some special circumstances (Senturk, 1994, page 421).

## SUGGESTED REFERENCES FOR FURTHER READING


FAO (2010). Irrigation and Drainage Paper 64: Manual on Small Dams Guide to Siting, Design and Construction.

Government of Western Australia (2015), Department of Mines and Petroleum, "Guide to the preparation of design report for tailings storage Facilities (TSFs)" East Perth WA, Australia

Kalyo, J. (2018). Hydraulic structures Lecture Notes at Mbeya University of Science and Technology.

Madan Mohan Das, Mimi Das Saikia (2009)," Hydrology", Eastern Economy edition. PHI Learning Private Limited Publishers, New Delhi

Ministry of Water (2012). Dam safety Guidelines.

Ndomba, P.M. (2011)," Developing Sediment Yield Prediction Equations for Small Catchments in Tanzania, Advances in Data, Methods, Models and Their Applications in Geoscience, Dr. Dongmei Chen (Ed.), ISBN: 978-953-307-737-6, InTech, Available from: https://cdn.intechweb.org/pdfs/25388.pdf

Nkuba I. I. (2009), "Ministry of Water Class D Dams"

Novack et al., (2001),"Hydraulic Structures", 3rd ed. Taylor& Francis group, London and New York

PHB Billion (2005), "Tailings Storage Facility Design Report". https://www.bhp.com/-/media/bhp/regulatory-information-media/copper/olympic-dam/0000/draft-eis-appendices/odxeisappendixf1tailingsstoragefacilitydesignreport.pdf

Punmia, B.C., Jain, A.K. and Jain, A.K. (2010)," Environmental Engineering-1; Water Supply Engineering", Laxim Publication (P), 113, Golden House, Daryaganj, New Delhi, India.

Raghunath, H.M. (2006),"Hydrology-Principals, Analysis and Design," 2nd ed. New Age International (P) Ltd Publishers, New Delhi, India.

Subramanya, K. (2008),"Engineering Hydrology", 3rd ed. Tata McGraw Hill Education Private Limited Publishers, 7West Patel Nagar, New Delhi, India.

Subramanya, K. (2015),"Flow in Open Channels", 4rd ed. Tata McGraw Hill Education (India) Private Limited Publishers, P-24, Green Park Extension, New Delhi, India.

The Constructor (2020). 7 Different Types of Spillways. Available at: https://theconstructor.org/water-resources/hydraulic-structures/different-types-spillways/32484/, visited on 01 March 2020.

U.S. Environmental Protection Agency (1994), "Design and Evaluation of Tailings Dam" Office of Solid Waste Special Waste Branch 401 M Street, SW Washington, DC 20460.

# APPENDIX L: STANDARD DRAWINGS

This is a list of standard drawings of common structures associated with drinking water supply structure. It enables engineers and project managers to save time during design, and preparation of design report and most importantly during implementation by using drawing and bill of quantity from the MoW server with the URL of **www.maji.go.tz**

These standard drawings available can be accessed through official link **Design Manual (DCOM)**

| S/N | DRAWING NAME | DRAWING NUMBER |
|---|---|---|
| 1 | 45 $m^3$ CIRCULAR TANK WITH REINFORECED COVER | S/AR/370 |
| 2 | 45 $m^3$ R.C CIRCULAR TANK ON 6 m HIGH CONCRETE BLOCKWORK RISER | S/MO/826 |
| 3 | 90 $m^3$ R.C CIRCULAR TANK ON 6 m HIGH CONCRETE BLOCKWORK RISER | S/MO/827 |
| 4 | 90 $m^3$ CIRCULAR TANK WITH REINFORECED COVER | S/AR/370 |
| 5 | 135 $m^3$ CIRCULAR TANK WITH REINFORECED COVER | TY/TA/14 |
| 6 | 135 $m^3$ R.C CIRCULAR TANK ON 6 m HIGH CONCRETE BLOCKWORK RISER | TY/TA/41/A |
| 7 | 225 $m^3$ R.C CIRCULAR TANK ON GROUND | S/MO/257 |
| 8 | Intake of clear stream-to be found on rock | TY/ST/40 |
| 9 | Intake of clear stream-to be found soft ground | TY/ST/40 |
| 10 | Typical sand trap intake (small stream) | 22109 |
| 11 | Spring intake structure (spring box) | MAR/SK/150 |
| 12 | Double tap D.P (type IA) | TY/WS/23 |
| 13 | Single tap D.P (type IB) | TY/WS/23 |
| 14 | Substitute to domestic point type I (Box Kiosk design type A) | S/MO/1051 |
| 15 | Type borehole pump house (with Lister 6 H.P engine & Climax 28/3 well head) | TY/ST/12/A |
| 16 | Type pump house for centrifugal pump | TY/ST/49 |
| 17 | Break pressure tank combined with Domestic point and washing slab | S/MO/852 |
| 18 | Short cattle trough | TY/TA/5 |
| 19 | Short Goat and sheep trough | S/MO/880 |
| 20 | Spring weir | MAR/SK/146 |
| 21 | Slow sand filter capacity 5,000 gallons/day | TY/TA/54 |

# APPENDIX M: SELECTED INTAKE DESIGNS

Typical types of intake structure are provided to enable appropriate choice of as per the water source condition

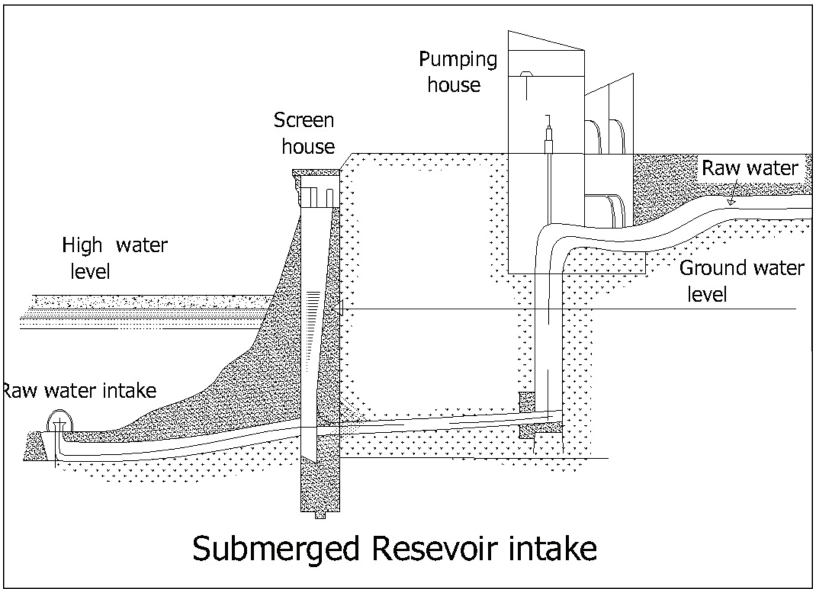



Figure M.1: Submerged Reservoir Intake

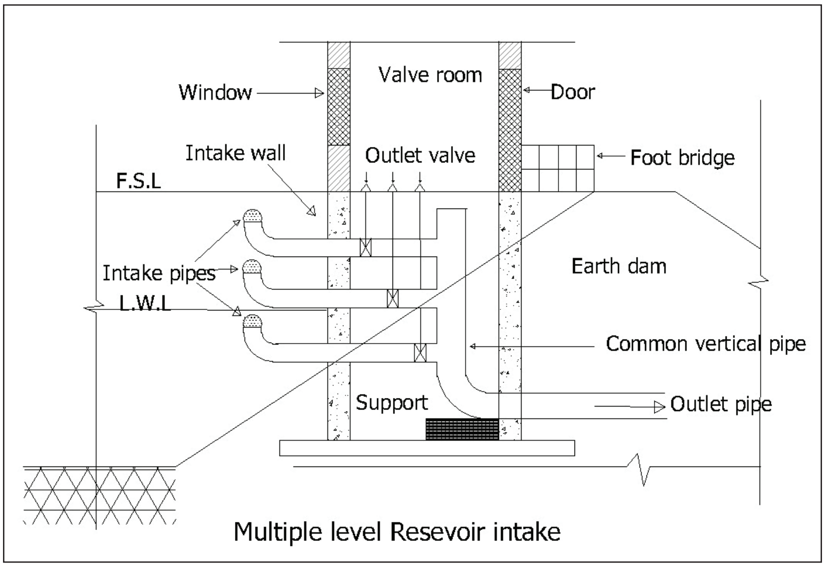



Figure M.2: Multiple Level Reservoir Intake

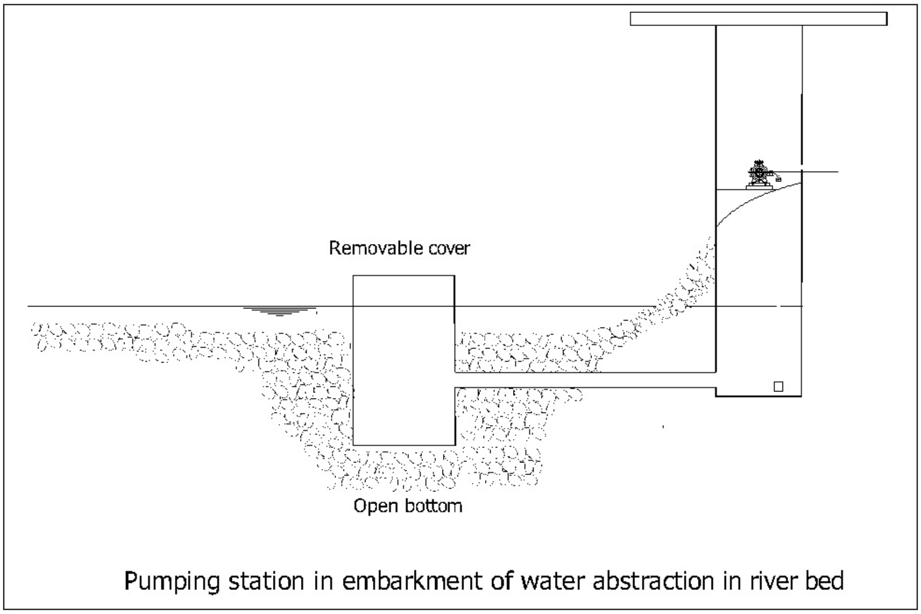



**Figure M.3: Riverbed Bottom Intake**

# SUBJECT INDEX