THE UNITED REPUBLIC OF TANZANIA

MINISTRY OF WATER

DESIGN, CONSTRUCTION SUPERVISION, OPERATION AND MAINTENANCE (DCOM) MANUAL

VOLUME II

DESIGN OF SANITATION PROJECTS

FOURTH EDITION

PROJECT PREPARATION,

COORDINATION AND DELIVERY UNIT (PCDU)

March 2020

PREFACE

The Government of the United Republic of Tanzania through the Ministry of Water is overseeing the implementation of the Water Supply and Sanitation Management projects in the country. The Ministry of Water published several editions of the Design Manuals. The First edition was the Water Supply and Waste Water Disposal Manual of1985/86. The Second edition was titled Design Manual for Water Supply and Waste Water Disposal of 1997. The Third edition was titled Design Manual for Water Supply and Waste Water Disposal of 2009. These manuals contributed in guiding the Ministry and the general public in the planning and design of water supply projects in the country. As it is now well over ten years since the Third edition of the Design manual was adopted and in the meantime, many scientific and technological changes have taken place including the conclusion of MDGs and adoption of the SDGs in 2015 as well as learning some useful lessons out of implementation of the WSDP I and WSDP II (which is still on-going); it is felt it is high time to revise the 2009 design manual. Notably, the 3rd edition Design Manual has limited coverage on the impact of climate change, application software and sanitation management issues among other things.

The Ministry is now at various stages of instituting policy and legal reforms that are deemed necessary for improving the design, construction supervision, operation and maintenance of water supply and sanitation management projects in Tanzania. Therefore, the 4th edition Design, Construction Supervision, Operation and Maintenance (DCOM) Manual will make invaluable contribution in this regard. It is important to recall that the Government has in 2019 established the Rural Water Supply and Sanitation Agency (RUWASA), which is responsible for supervision, execution and management of rural water supply and sanitation projects. RUWASA is expected to improve the responsibility and accountability in the management of the water and sanitation services in rural areas. The 4th edition DCOM Manual will support the National WSSAs, UWSSA, RUWASA, CSO funded, DP funded projects and will provide valuable information related to implementation of water supply and sanitation projects at various stages, from pre-feasibility and feasibility studies, planning, design, construction supervision and during operation and maintenance.

It is expected that the 4th edition DCOM Manual will position the Ministry to systematically and comprehensively implement the design, construction supervision, operation and maintenance of water supply and sanitation projects in order to ensure sustainability of water supply and sanitation projects in Tanzania. 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 with regard to the "Africa that we want" and the Sustainable Development Goals (SDGs) on water and sanitation (SDG No. 6).

Preparation of this Water Supply and Sanitation Management DCOM Manual, required contribution in form of both human and financial resources. The Ministry of Water would therefore like to take this opportunity to thank the members of the Special Committee for Reviewing and Updating the 3rd edition Design Manual for Water Supply and Wastewater Disposal of 2009 for their immense efforts in preparation of this comprehensive 4th edition DCOM Manual as well as the World Bank for financing the major part of the activities, and 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 address changes in policy and societal needs, emerging technologies, and sustainability concerns in implementation of water supply and sanitation projects in the country.

Prof. Makame Mbarawa (MP)
Minister
Ministry of Water **14th March 2020**

ACKNOWLEDGEMENTS

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

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

i. Review of the 3rd edition including benchmarking with design manuals from other countries,
ii. Website reviews and review of other manuals prepared by consultants who have worked in Tanzania,
iii. Review of Literature data collection and design methods review,
iv. Data collection from stakeholders: Primary stakeholders-MoW technical and management staff; Private companies that deal with implementation of the water supply and sanitation projects; Beneficiaries of water supply and sanitation projects,
v. Collection of existing standard drawings and digitisation after conversion to metric units for some drawings,
vi. Review of the 4th edition drafts by various stakeholders: MoW staff and other stakeholders outside MoW,
vii. Revision of the 4th Edition by incorporating comments and views from all the stakeholders,
viii. Preparation and submission of the 4th edition of the DCOM Manual.

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

Among the new features in the 4th edition DCOM Manual include mainstreaming of climate change impacts and use of various types of software in design of water supply and sanitation projects. These facilitate faster and more accurate analysis. The DCOM manual has also encouraged use of Supervisory Control and Data Acquisition Systems (SCADA) for large urban or national projects where local capacity building can be guaranteed by the providers. It should be borne in mind that software can allow a wide variety of scenarios to be considered. However, it should be noted that, despite the

critical role of software/models in guiding decision-making, its limits should be realised so as to avoid that it becomes a substitute for critical practical evaluation.
I wish to thank the different stakeholders for their active participation in contributing various inputs during the course of preparation of this DCOM Manual from within and outside the Ministry of Water including Development Partners, NGOs, Consultants, Suppliers and Contractors as well as other Ministries for their support. 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 **14th March 2020**

**MEMBERS OF THE SPECIAL COMMITTEE
ON REVIEW AND UPDATING
THE 3RD EDITION, 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	NMAIST	....
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 **14th March, 2020**

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ABR	Anaerobic Baffled Reactor
AF	Anaerobic Filter
ATAWAS	Association of Tanzania Water Suppliers
BICO	Bureau for Industrial Cooperation
BOD	Biochemical Oxygen Demand
CBWSO	Community Based Water Supply Organisations
CDD	Consortium for DEWATS Dissemination
CMIP5	Coupled Model Inter-comparison Project Phase 5
COD	Chemical Oxygen Demand
COWSOs	Community Owned Water Supply Organizations
CRD	Contractors Registration Board
DAWASA	Dar es Salaam Water Supply and Sanitation Authority
DCOM	Design, Construction, Supervision, Operation and Maintenance
DEWAT	Decentralized Wastewater Treatment Systems
EAC	East Africa Community
EIA	Environmental Impact Assessment
EIS	Environmental Impact Statement
EMA	Environmental Management Act
ENSO	El Nino-Southern Oscillation
ERB	Engineers Registration Board
ERB	Engineers Registration Board
EWURA	Energy and Water Utilities Regulating Authority
FS	Faecal Sludge
FSM	Faecal Sludge Management
FSTP	Faecal Sludge Treatment Plant
FYDP	Five Years Development Plan
GCMs	Global Climate Models
GHGs	Greenhouse gases
GWPTZ	Tanzania Global Water Partnership
IHP	International Hydrological Programme
IOD	Indian Ocean Dipole
IPCC	Intergovernmental Panel on Climate Change
ISO	International Standardisation Organization
ITCZ	Inter-tropical Convergence Zone
IWA	International Water Association
MAPET	Manual Pit Emptying Technology
MOEST	Ministry of Education, Science and Technology
MoW	Ministry of Water
NAWAPO	National Water Policy
NEMC	National Environmental Management Council
NMAIST	Nelson Mandela African Institution of Science and Technology
NSSS	Non-sewered Sanitation System

NWSDS	National Water Sector Development Strategy
OSS	On-site Sanitation System
PGF	Planted Gravel Filter
PP	Polishing Pond
PPRA	Public Procurement Regulatory Authority
Q&Q	Quantity and Quality
RCPs	Representative Concentration Pathways
RUWASA	Rural Water Supply and Sanitation Agency
SACADA	Supervisory Control and Data Acquisition System
SADC	Southern Africa Development Community
SDGs	Sustainable Development Goals
SST	Sea Surface Temperature
TAWASANET	Tanzania Water Supply and Sanitation Network
TBS	Tanzania Bureau of Standards
TDV	Tanzania Development Vision
UDSM	University of Dar es Salaam
UHT	Underground Holding Tank
UNECE	United Nations Economic Commission for Europe
UNESCO	United Nations Education and Science Organisation
UNFCC	United Nations Framework Convention on Climate Change
URT	United Republic of Tanzania
UWSSA	Urban Water Supply and Sanitation Authority
WMO	World Meteorological Organization
WSDP	Water Sector Development Programme
WSPs	Waste Stabilization Ponds

CHAPTER ONE
INTRODUCTION

The preparation of this DCOM manual has been preceded by an overview of five important global considerations of Water Supply and Sanitation prior to reviewing the water and sanitation sector in Tanzania. It is followed by explanation of the rationale for 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 came together at the United Nations in New York and adopted the 2030 Agenda for Sustainable Development. Governments responded to the common development challenges they faced and the changing world around them by uniting behind a truly forward-looking, yet urgent plan to end poverty and create shared prosperity in a healthy and peaceful planet. The Agenda 2030 central principle is leaving no one behind in achieving the 17 Sustainable Development Goals (SDGs) through 169 targets. The 2030 Agenda for Sustainable Development adopted at the UN Summit includes the 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 (https://sustainabledevelopment.un.org/). It should also be noted that, water and sanitation are at the heart of the Paris Agreement on climate change 2015 (https://unfccc.int/process-and-meetings/the-paris-agreement/the-paris-agreement).

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 now to transform the way it manages its water resources and the way it delivers water and sanitation services for billions of people (https://sustainabledevelopment.un.org/sdg6).

The designers and engineers therefore have the responsibility to support the Government of Tanzania to achieve 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 and intense extreme weather events have resulted in a higher incidence of floods and droughts around the planet. The ensuing adverse impacts of climate change on water and sanitation services constitute a clear and present danger 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.

Flood occurrences 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 basic water and sanitation infrastructure. Such damage can take years to repair.
- On a smaller scale, drinking-water infrastructure can be flooded and be 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 therefore 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 (or adapt to) the climate change within the time frame of their project design (i.e. design life).

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

A key challenge faced by water authorities in developing countries is how to manage their service delivery obligations to rural communities. Even in decentralized sectors, the 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 the 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 is how to regulate and monitor performance under these contracts.

Globally, activities undertaken in 2005 suggested that private participation in the water sector is entering a new phase. New private firm involvement is focusing on smaller projects and bulk facilities. Contractual arrangements involving utilities are combining private operation with public financing and new players are entering the market. Water is so crucial to food security and irrigation and is much affected by climate change.

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 apart from giving low priority to sanitation. In this context, Public Private Partnerships (PPPs) can be a mechanism (among others) to help Governments fund the much needed investment and bring technology and efficiency that can improve the performance and financial sustainability of the water and sanitation sector.

Governments are currently using the private firms in the water and sanitation sector increasingly to finance and operate bulk water supply and wastewater treatment. New technologies and innovation are currently being introduced, where traditional water sources are being scarce, such as in desalination and wastewater re-use. Utilities are drawing on specific expertise, such as Non Revenue Water (NRW) reduction and pressure management, to bring efficiencies and service improvements. Private investors and providers are increasingly local and regional, increasing competition and bringing down prices.

Most utilities are increasingly turning to the private sector for turnkey solutions to design, build and operate water and wastewater treatment plants, and in some cases 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 more 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 World Bank 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 (https://ppp.worldbank.org/public-private-partnership/sector/water-sanitation).

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 types of 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 the non-navigational uses of international watercourses (https://www.siwi.org/icwc-course-international-water-law/).

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* is the 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 doctrines 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 Sustainable Development Goals (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 decision making.

Management of water resources that entails extraction of shared international water resources in form of rivers, lakes, seas and oceans as sources are guided by the 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 include the Tanzania Development Vision (TDV) 2025 (https://www.mof.go.tz/mofdocs/overarch/vision2025.htm). The realization of TVD 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 (https://mof.go.tz/mofdocs/msemaji/Five%202016_17_2020_21.pdf).
The Government adopted the TDV in the mid-1986s for socio-economic reforms and continue 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 is shaping the future in which water and sanitation services will be delivered to enhance the health of normal citizens who are very important 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, namely industrialization, human development, and implementation effectiveness, and is aligned well to its SDGs. Importantly, industrialization will place a huge demand on utility supplies e.g. energy and water, so subscribing on addressing the SDG Goals 6: on water and sanitation.

Chapter 4 of FYDP II, sub-chapter 4.3.4 on Water Supply and Sanitation Services sets key targets as follows; 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: 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 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 in terms of water coverage for rural areas and urban areas are 85% and 95% by 2025, respectively which are also articulated by 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 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 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. As regards 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 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 explained to be put in place. The policy aimed at ensuring that 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 by assurance of improved water supply and sanitation services to the water users and to identify and preserve the water sources.

1.2.2 Legal and Institutional Framework for Water Supply and Sanitation Services

Basically, the water and sanitation sector is governed through 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: 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 will be 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 thoroughly in most of the implemented projects through establishments of COWSOs in every completed projects that were given all the mandate of making sure the projects are sustainable. Amongst the lessons learnt from 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 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 (community). 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 to be 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, RUWASA also has 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

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 that was further articulated by the National Water Sector Development Strategy of 2006 - 2015 (URT, 2008) and the WSDP of 2006-2025. There are current 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 (www.maji.go.tz) 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 wa Usafi wa Mazingira*" (Guidelines for Construction of Toilets and Sanitation), (MoHCDGEC, Oct. 2014).

1.2.5 Major Stakeholders in Water Supply and Sanitation Projects

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

(i) Regulatory Authorities

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

i. Public Procurement Regulatory Authority (PPRA) (https://www.ppra.go.tz/),
ii. Tanzania Bureau of Standards (TBS) (http://www.tbs.go.tz/),
iii. Engineers Registration Board (ERB) (https://www.erb.go.tz/),
iv. Contractors Registration Board (CRB) (http://www.crb.go.tz/),
v. Energy and Water Utilities Regulating Authority (EWURA) (https://www.ewura.go.tz/).
vi. The National Environmental Management Council (NEMC) (http:www.nemc.go.tz)

(ii) 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

(iii) National Water Supply and Sanitation NGOs and networks

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

- Association of Tanzania Water Suppliers (ATAWAS)(http://atawas.or.tz/),
- Tanzania Water Supply and Sanitation Network (TAWASANET)(http://www.tawasanet.or.tz/),
- Tanzania Global Water Partnership (GWPTZ) (https://www.gwptz.org/about/).

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 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 the basket funding.

In support of the Government 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, MoW has managed to create the necessary environment for supporting the private sector such that, a sizeble 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 is the inclusion of the private sector in water supply and sanitation projects implementation. Notwithstanding the good experiences, 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 closely.

On the other hand, the Ministry of Water organized a forum on enhancing public private partnership in the water sector, which 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 implementation effectiveness, which earmarked water supply and sanitation as among the key interventions for its 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 for executing water supply projects,
h) Database issues especially on water resources information, which may ends up with over- or under- designing water supply facilities.
i) Design specifications based on use of obsolete technologies was also concluded to be 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 are recommended:

a) MoW through the established in-house Design Unit to 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) Awareness on other operational modes in PPP as per water policy be enhanced;
c) Where applicable, private operators to 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;

d) Private operators to 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 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) stakeholder's meeting hosted by the MoW in 2018. During that meeting, the issue of providing designs/specifications that use old technologies in procurement was mentioned as well as the need to adopt the latest appropriate technology was also stressed. 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 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 undertake their supervisory and coordination roles well and the consultants to undertake designs using the guidelines recommended in the MoW manual only. 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 the water supply and sanitation projects, the manuals will provide guidance on how they have to 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 facilitate one 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 it will eliminate the recurring problem of consultants designing water supply and sanitation management projects that are below minimum standards.

1.4 About the Fourth Edition of the DCOM Manual

The 4th edition of the DCOM Manual has been prepared in the year 2020, following review and updating of the Third Edition of the Water Supply and Wastewater Disposal

Design Manual of 2009. The former, was prepared in three separate volumes. These volumes included eight chapters on water supply, three chapters on waste water disposal and one chapter on water pipelines standards and specifications. It should be however be remembered that the 2nd Edition of the Design Manual that was titled *Design Manual for Water Supply and Waste Water 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 the year 1985/86, a few years after conclusion of the International Water and Sanitation Decade that ended in 1981. Thus, the current edition of DCOM Manual is adequately informed by previous edition evaluations which incorporate the topical and existing DCOM Manual challenges and issues.

Preparation of the four volumes was undertaken by a Special Committee of twelve members from The Ministry of Water, RUWASA, University of Dar es Salaam (UDSM), Private sector consultant and The Nelson Mandela African Institution of Science and Technology (NMAIST). The process of preparation of 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 ,Material suppliers and Development Partners. It also involved undertaking of an extensive search of literature from libraries, conference proceedings, journal publications, websites of various entities and design manuals from various global entities, 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* that is organized into thirteen chapters,
- **Volume II** that dwells into *Design of Sanitation Projects* is divided into six chapters,
- **Volume III** which is 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, which can be offered as separate packages for training of different groups of users:

  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 Manual

The purpose of preparation of Volume II is to guide planners and engineers responsible for design of either a complete sanitation system or component of the same to plan, select options and design units in the sanitation chain. The Volume has also provided link or hyperlink to many other websites that may be relevant and also to use the index provided at the end of the volume to make instant search for a topic.

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CHAPTER TWO
PLANNING FOR SANITATION PROJECTS

2.1 Introduction

Safe sanitation and good hygiene practices are fundamental to human development and well-being, including the achievement of adequate nutrition, gender equality, education and the eradication of poverty (Supply, 2015). It is estimated that for every $1 invested in sanitation, there is a return of $5.50 in lower health costs, more productivity and fewer premature deaths (National Bureau of Statistics, 2014). In Tanzania, the increase of unplanned settlements has intensified the challenge of increasing access to sanitation and hygiene services to urban dwellers. It is estimated that only 34.2 percent of the urban population has access to improved sanitation and the remaining using basic sanitation facilities or practicing open defaecation (National Bureau of Statistics, 2014).

In Tanzania, planning for the delivery of sanitation services is done during urban and town master planning process, through the Ministry of Lands, Housing and Human Settlements Development (MLHHSD). The process as a whole is highly centralized and often resorts to recommending, the centralized sanitation service chain. They are not context specific and in most cases the master plans are not informed by research or understanding of the local sanitation status. Demand for sanitation is not assessed and there is little, or no communication between the project planners and future users. Consequently, social, gender, cultural and religious aspects are not sufficiently considered when designing sanitation projects. In other cases, environmental factors are not considered in the design, which sometimes leads to the unsafe situations. For example, in low-income urban areas where pit emptying is often a necessity, such sanitation services are often absent or too expensive. Also, hygiene education to improve the sanitation behaviour of the community was rarely included in sanitation projects, because community education and sanitation projects had different implementation time-scales.

In the light of the above background, this design manual for sanitation projects is providing the planning framework that will assist in selection of sanitation system that are appropriate and deemed to be sustainable for various contents.

2.2 Planning Stages and Processes

In this section, planning process stages that will lead to selection of the most appropriate and effective sanitation systems are presented in the next paragraphs.

2.2.1 Community mobilization and sensitization campaigns

Again, the first step is for the community to request improved sanitation services. It will be conducted through community mobilization and sensitization campaigns to create demand for sanitation services. Community mobilization is presumed to be undertaken by the CBWSO leadership in collaboration with the Local Government and village leadership. At times a trained community moderator can be sought from MoW or RUWASA. The manner of mobilization is through formal village or area meetings.

Once a demand for improved sanitation facilities has been expressed, technology selection should be preceded by or based upon, a participatory needs assessment. Hygiene awareness and promotion campaigns can increase demand for improved sanitation facilities. Site visits by the community representatives led by CBWSO can assist to conclude the technology choice.

2.2.2 Analysis of the Existing Sanitation Situation

This is the stage of planning where the sanitation situation at hand will be analyzed and assessed. Situation analysis is undertaken through a participatory assessment involving the target community. A participatory assessment should be carried out to determine if there are problems related to:

(i) the existing human excreta-disposal system;
(ii) hygiene and defaecation behaviour (among men, women and children);
(iii) the overall hygienic environment and
(iv) prevalence of human excreta-related diseases.

Also necessary are:
(i) a participatory assessment of the cultural, social and religious factors that may influence the choice of the sanitation technology;
(ii) a participatory assessment of the local conditions,
(iii) capacities and resources (material, human and financial);
(iv) community ability and willingness to pay,
(v) the identification of local preferences for sanitation facilities, and
(vi) possible variations.

2.2.3 Synthesis

Data should be collected on all the factors that affect sanitation system performance. Several criteria can help in the analysis of the data and in choosing the design of the sanitation system:

(i) Match user preferences according to local capacities and environmental conditions, such as whether there is a risk of contaminating water sources. The preferences of all users should be considered, including men, women and children.
(ii) Match investment requirements to the costs of the technology and to the community's ability/willingness to pay.
(iii) Match community needs to the availability of materials.
(iv) Match the proposed design options to the availability of craftsmanship.
(v) Match O&M requirements to the prevailing sanitation behaviour and to local capacities.
(vi) Identify promotional campaigns, micro-credit mechanisms and hygiene education programmes that could accompany the technology selection and installation process.

2.2.4 Hold discussions with the target community

Discussions should be held with the community about sanitation options, and include discussions about the technical, environmental, financial and hygiene implications of each option. Discussion processes should consider religious/cultural sensitivities, gender and minorities.

2.2.5 Select the most appropriate technology

There are a wide range of technologies available for managing domestic wastewater and excreta. In addition, designing a sanitation chain means using a series of complementary components, the organization and combination of which will vary according to the physical context, user demand and the level of treatment required, etc.

For a designer, selecting an appropriate sanitation solution that is adapted to the context of the local environment may be a bit complex. Wastewater and excreta management is linked to many different domains (technical, sociological, political, land use, financial, etc.) and depends on numerous criteria (topography, geology, urban population density, user demand, water consumption, temperature etc.).

Different contexts can exist side by side; each with its own particularities and requiring its own type of sanitation chain. It is necessary to consider this concept of complementary systems when defining the overall strategy at municipality level. This guide (and Steps 1 and 2 of the planning process, in particular) will enable one to identify the chain(s) best suited to the particular locality.

2.3 Planning for Various Sanitation Chains

Urban environment in developing countries is not uniform. Urban and peri-urban areas differ greatly in terms of sanitation facilities used. Similarly within the urban area different sub areas differ in terms of sanitation facilities and connectivity to central systems. A good example is the City of Dar es Salaam where there are areas which are planned and others which are not planned. At the same time some areas can be connected to the central sewer but others cannot. The different types of sanitation technologies, which can evolve over time, use either improved on-site systems, or small-piped or conventional sewerage systems. The management option for urban environment is shown in Fig. 2.1. The sanitation system is selected by considering the demand from the population, the requirements imposed by the natural environment, the local context, the population density and local practices. Given these considerations, the different sanitation chains are defined in Table 2.1.

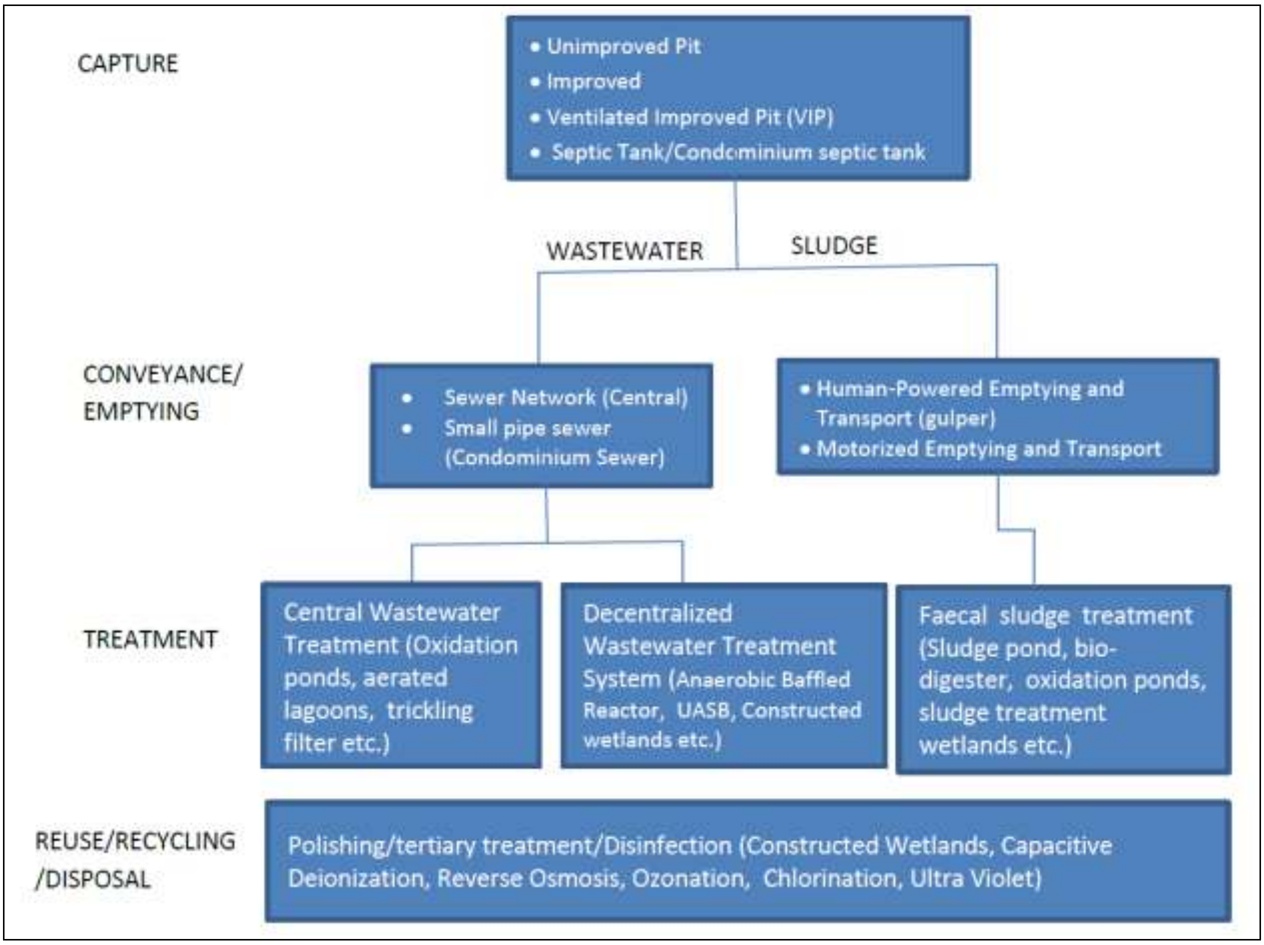

Figure 2.1: Sanitation management options for urban environment
Source: (Modified from Tilley, 2014)

At local authority level, it is important to consider these different systems (on-site sanitation, small-piped and conventional sewerage systems) as being complementary to each other: several sanitation chains can co-exist within the same area.

Table 2.1: The different sanitation chains

CHAIN	DESCRIPTION	PROS	CONS
On-site sanitation	These are technologies that enable wastewater storage within a plot (e.g. simple latrines). Storage can be combined with pre-treatment (such as a septic tank). These installations often require periodic emptying and transportation of the resulting sludge to suitable disposal and treatment plants.	- Low investment costs; - Can be constructed and required using locally available materials; - Techniques can be mastered locally (they don't require great technical expertise); - Not necessary to have a constant water source.	- Costs of emptying; health risks linked to sludge if this is not sanitized; - Risk of underground pollution.
Small-piped sewerage system	These are technologies, such as simplified sewerage systems used by multiple plots that collect wastewater and excreta produced at neighbourhood level or from several houses. The wastewater thus collected can either be treated on-site or be directly transported to a treatment plant.	- Medium-level operation and maintenance costs; - Very convenient; - Extension possible should the population evolve; - Permanent evacuation of pollution far from the population's place of residence.	- Design and construction requires expert intervention; - Qualified labour required for care and maintenance.
Conventional sewerage system	These are sewerage systems to which households are directly connected. These systems transport wastewater and excreta to treatment plants which reduce the pollution content of effluent* prior to this being discharged into the environment.	- Highly convenient; - Long lifespan of the system; - Permanent evacuation of pollution far from the population's place of residence; - Adopted for areas of high population density and where large volumes of wastewater are produced.	- Very high investment costs; - Design and construction requires high-level expert intervention; - Qualified labour required for care and maintenance.

Source: Monvois et al, 2010

In practice, this is very common and is even to be encouraged. The urban development of a commune (local authority) is never uniform. Different contexts can exist side by side; each with its own particularities and requiring its own type of sanitation chain. It is necessary to consider this concept of complementary systems when defining the overall strategy at municipality level. This design manual (and Steps 1 and 2 of the planning process, in particular) will enable one to identify the chain(s) best suited to the particular town or part of a town or locality.

2.3.1 The five successive segments of a sanitation chain

Regardless of the sanitation chain under consideration, the management of wastewater and excreta can generally be divided into five segments, as shown in Table 2.2. Breaking down sanitation into successive segments in this way enables one to better understand this complex field. Indeed, each segment has different, yet complementary, objectives and sets out a specific approach for meeting the requirements. It is vital that, there is coherence between these five successive segments (and so between the different technologies used); to ensure this coherence is in place for a given area and for each of the segments, it is necessary to choose technologies from the same sanitation chain (on-site, small-piped or conventional sewerage system). Within each segment, there are specific technologies available that enable the required objectives to be met. It is these technologies that are the focus of this guide. Upon completion of Step 3 of the planning process, one will be in a position to select the appropriate technologies to be put in place.

Specific technical solutions for each chain and for each segment Sanitation technologies are very diverse and vary according to both the sanitation chain used and to the segment within the chain. This ‘chain’/‘segment’ double entry is summarized in a non-exhaustive manner in Table 2.3.

Table 2.2: Five successive segments of a sanitation chain

SEGMENTS	SEGMENT-RELATED OBJECTIVES AND METHODS	TECHNICAL SOLUTIONS TARGETED BY THIS SEGMENT
Segment 1 **……………..** **Containment**	**Objective:** To improve hygiene of people by minimizing contact with human excreta **Methods**: contain excreta in a safe holding facility such as septic tank, pit latrine, etc.	This segment group together all technologies that can be used to safely hold human excreta/faecal sludge before it is taken out for further processes
Segment 2 **……………..** **Access / Collection**	**Objective**: To improve the sanitary conditions in people's homes. **Methods**: removal of wastewater and excreta from households' dwellings.	This segment together those technologies with which the user has direct contact. These technologies enable wastewater and excreta to be collected, temporarily stored and, if appropriate, to be partially treated: latrines, septic tanks, soakaways, etc.
Segment 3 **……………..** **Evacuation / Transport**	**Objective**: To ensure the health and hygiene of the neighbourhood. **Methods**: evacuation of wastewater and excreta from the neighbourhood.	This segment includes all those technologies that transport wastewater and excreta away from the user's home to discharge and final treatment sites: vacuum trucks, sewerage systems, etc.
Segment 4 **……………..** **Treatment**	**Objective**: To reduce pollution. **Methods**: physico-chemical and / or biological treatment of effluent (followed by utilization, if appropriate).	This segment brings together those technologies used to dispose of wastewater, excreta and sludge, used for treatment to reduce the pollution load and, if appropriate, utilization of the end-product.
Segment 5	**Objective**: To recover and use	This segment brings together all

……………. Reuse, Recycling & Disposal	safe component of treatment and ensure safe disposal not cause pollution of environment **Methods**: recovery of water, nutrients, biogas for utilization in agriculture, energy, etc.	technologies that can be used to recover by-products of water and sludge treatment such as anaerobic systems for recovery of biogas, irrigation systems.

Source: Modified from Monvois *et al*, 2010

Table 2.3: Examples of technologies in relation to the chain used.

	CHAIN		
	ON-SITE SANITATION	**SMALL-PIPED SEWERAGE SYSTEM**	**CONVENTIONAL SEWERAGE SYSTEM**
Containment (Segment 1)	Simple toilet [1], VIP latrine [2], soakaway, septic tank, infiltration trenches, flush toilet	Cistern flush or pour flush toilet, septic tank, grease trap	Cistern flush toilet
Access/collection (Segment 2)	Pit of a latrine, soakaway, septic tank, infiltration trenches, flush toilet	Cistern flush or pour flush toilet, septic tank, grease trap	Cistern flush toilet
Evacuation, transport (Segment 3)	Manual pit emptying, vacuum truck	Small-piped system (simplified or settled sewerage system)	Conventional evacuation system
Treatment (Segment 4)	Sludge treatment plant	Intensive or extensive, decentralized treatment plant	Intensive or extensive, centralized treatment plant
Reuse/Recycling/ Disposal/ (Segment 5)	Drying of sludge and application on land	Polishing of treated wastewater in tertiary treatment, disinfection	Polishing of treated wastewater in tertiary treatment, disinfection

[1] These are simple non-ventilated pit toilets. This term 'simple toilet' is also regularly used throughout this guide as a straightforward means of describing this technology.
[2] These are Ventilated Improved Pit (VIP) toilets.

Source: Modified from Monvois *et al*, 2010

2.3.2 A three-step planning process

To determine the relevant sanitation technologies, the design engineer is advised to use a three step process (Monvois, 2010), composed of (1) Characterization of the Town with regard to sanitation (2) Determination of a sanitation chain for each area identified (3) Selection of appropriate technical solution for each sanitation chain

The steps are explained herein after

Step 1. Characterizing the town with regard to sanitation

- a first 'sub-step' (characterize the town in its entirety) provides an understanding of the sanitation situation at overall town level and enables one to anticipate any urban development that may take place over the next 10 to 20 years;
- this is then followed by a more refined analysis (characterize the neighbourhoods to identify homogeneous areas) to identify areas that are homogeneous in terms of physical, urban and socio-economic context. An appropriate sanitation technology that is adapted to the context will then be implemented in each area.

Step 2. Determining a sanitation chain for each area identified

For each of the areas identified, it is possible to select a sanitation chain based on an initial simplified approach, as presented in Figure 2.1.This simplified approach makes use of a limited number of the criteria for which information was collected during Step 1 and which needs to be satisfied to validate the selection of a given chain. One therefore proceeds by elimination. For instance, if there is low water consumption in a given area, a conventional sewerage system sanitation chain will not be possible. In the same way, if there is dense housing and so no space to build a pit for a household latrine, then on-site sanitation will not be appropriate.

It may however be the case that, based on the simplified approach shown in Table 2.1, several sanitation chains are possible for the same area. This type of situation is not unusual. To deal with such a situation, a second qualitative approach is proposed in Table 2.4. This table describes the pros and cons of each sanitation chain based on indicators previously identified at Step 1. From this table, one can make a choice which, at this stage in the planning process, does not have to be definitive: if, at Step 3, it transpires that this choice is not the most appropriate, it is always possible to go back and explore a different sanitation chain for this area.

Figure 2.2 shows an example of the distribution of sanitation facilities in a small town of Babati. As it can be seen the wet sanitation facilities are concentrated on the Central Business District only while dry sanitation (various forms of pit latrines) are dominant. Choice of sanitation chain for the town has to take this into consideration during planning. Figure 2.3 shows a simplified diagram for selection of sanitation chains

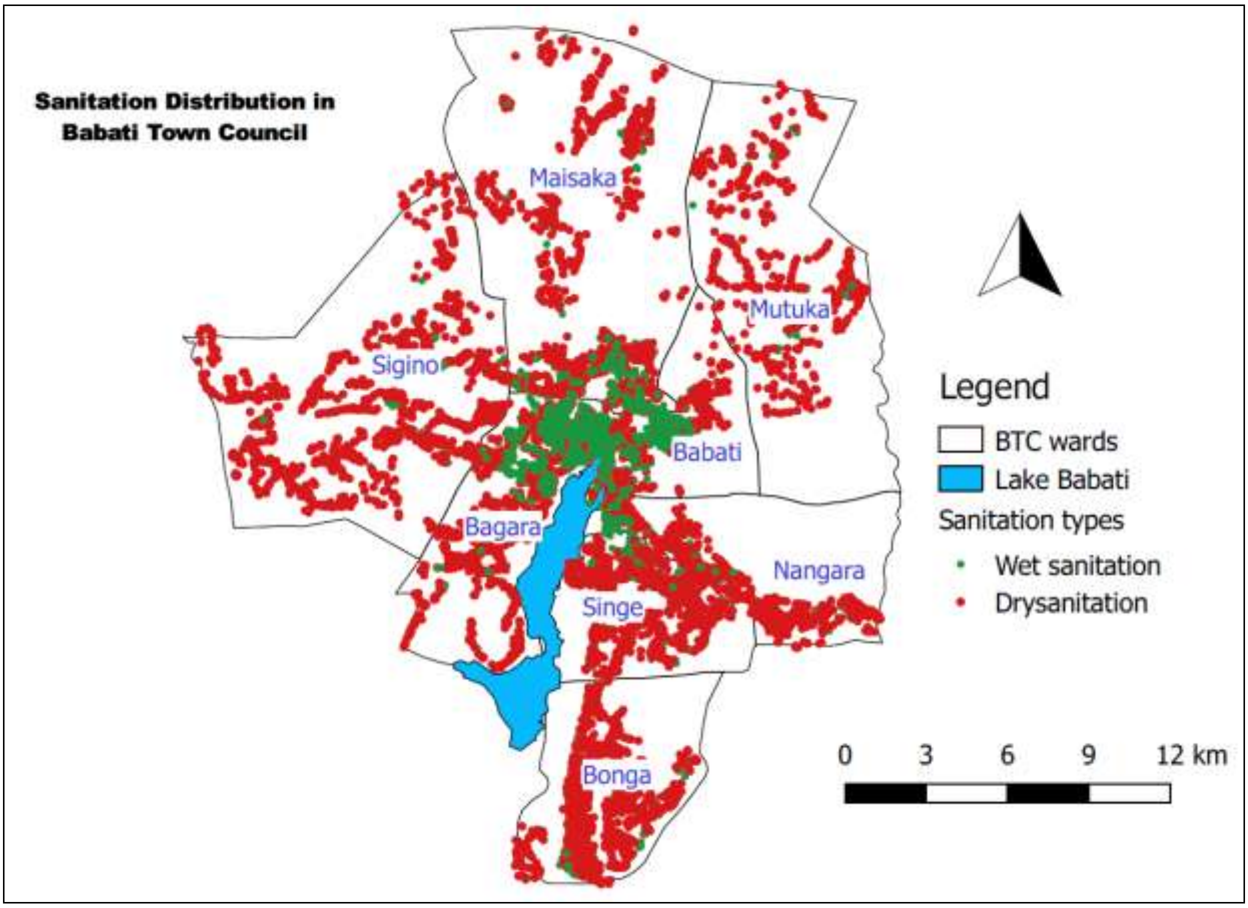

Figure 2.2: Distribution of sanitation facilities in Babati Town Council (2019)

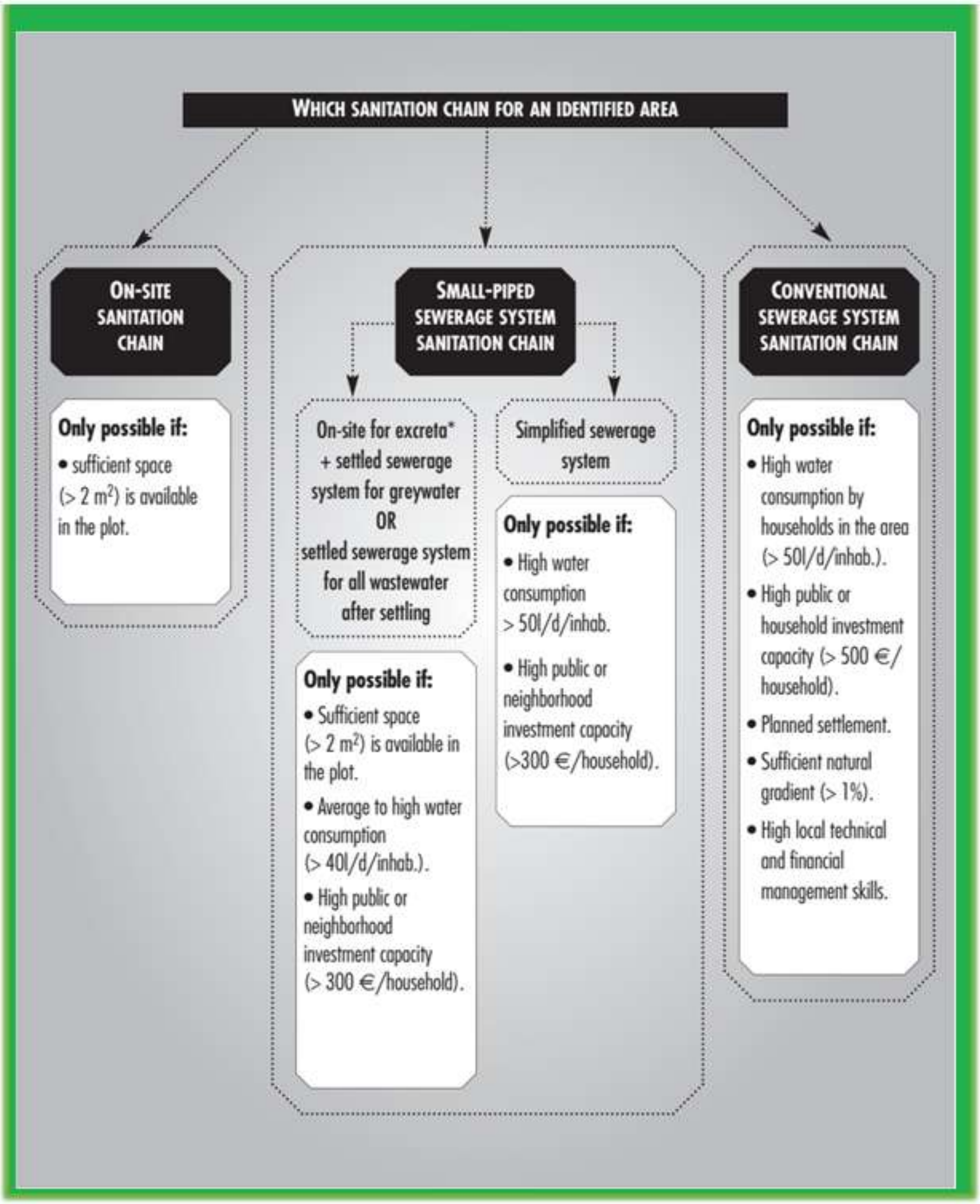

Figure 2.3: Simplified diagram for selection of sanitation chains
Source: (Monvois *et al*, 2010)

Table 2.4: Precise selection of sanitation chains

	CRITERIA	QUESTIONS	RESPONSES	PROS AND CONS ON-SITE SANITATION CHAIN
PHYSICAL	Soil type	Does the soil enable the absorption of wastewater and excreta in the area of intervention?	☐ YES ☐ NO	Certain technologies used in this chain (simple latrines, VIP), which are also the least costly, require permeable soil as they work through the partial infiltration of blackwater into the soil. Where the soil is impermeable, other 'on-site' technologies can be put in place (septic tanks, urine diverting dry toilets).
		Is the soil rocky?	☐ YES ☐ NO t	All technologies used with this sanitation chain require digging work. If the soil is rocky, then this will increase the cost of construction. In this case it will be necessary to raise the pit, ensuring that its volume is as small as possible (micro-septic tank) to reduce costs. This constraint means using technical solutions that require little water (urine diverting dry toilets, etc.) to ensure that the emptying frequency remains acceptable.
	Groundwater table	Is there a ground -water table near the surface? At what depth?	☐ YES ☐ NO Depth: .... meters	For technical solutions used in this chain that require infiltration, there is an increased risk of contamination if the groundwater table is high, particularly if it is less than 3 metres from the base of the pit. Where there is a recognized risk of contamination due to proximity to the groundwater table, it will be necessary to use watertight pits or to study the possibility of using the small-piped or conventional sewerage system sanitation chains.
	Topography	Is the gradient sufficient to enable the gravitational flow	☐ YES: > 1% (1m/100m) ☐ NO	A very steep gradient can pose problems for vacuum trucks. Where this is the case, preference should be given to an on-site

	of effluent?	< 1%	sanitation chain using simple toilets or to small-piped or conventional sewerage system sanitation chains.

PROS AND CONS		
SMALL-PIPED SEWERAGE SYSTEM SANITATION CHAIN	**CONVENTIONAL SEWERAGE SYSTEM SANITATION CHAIN**	
Conventional and small-piped sewerage sanitation chains are to be implemented in areas of high population density. Using these two chains in sparsely populated areas involves very high investment costs (in total and per user) that are difficult to withstand and also means users need to discharge large volumes of wastewater (to guarantee effective sludge removal from the system and to prevent clogging), which rarely happens in sparsely populated areas.		**URBAN**
Small-piped sewerage systems do not require a lot of space in the home	The conventional sewerage system sanitation chain does not take up any significant surface area in the home.	**URBAN**
This chain can be developed in unplanned settlements and where residents do not possess title deeds. However, should the area be subsequently developed, some households risk expulsion and so will lose their sanitation facilities at the same time.	Given the collective dimension and investment required to develop this chain, it needs to be located in planned settlements where the land status is clearly defined	**URBAN**
• **For a small-piped greywater and blackwater (simplified) sewerage system, average to high consumption is required to prevent the risk of clogging.** • **For a small-piped (settled) sewerage system carrying greywater only, low consumption will suffice.**	High household water consumption is crucial for ensuring the sewerage system functions correctly.	**SOCIO-ECONOMIC**
Medium to high investment is required for the small-piped sewerage system sanitation chain, depending on the technical options selected.	High levels of investment are required for the conventional sewerage system sanitation chain.	**SOCIO-ECONOMIC**
High level skills are usually required for small-piped	High level skills are required for the technologies used within this chain.	**SOCIO-ECONOMIC**

sewerage systems. **The skills required for the on-site facilities within this chain (latrines, septic tanks, etc), are the same as those in the on-site sanitation chain.**		

Source: Monvois *et al*, 2010

Step 3. Selecting appropriate technological solutions

The selection criteria

Each technical solution has its own characteristics, as well as its own pros and cons. For any given area, the selection process consists of assessing the extent to which the characteristics of a technical solution fits the context and constraints of the area under consideration. Lastly, it is necessary to establish whether or not a technical solution is feasible for a given area. A technological solution is feasible if:

- it meets local demand;
- the financial resources are available for its construction; and
- the technical and management skills exist to ensure its sustained operation.

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CHAPTER THREE
ON-SITE SANITATION SYSTEMS

3.1 Introduction

Sanitation refers to preventing people from coming into contact with wastes generated in homes, workplaces and public buildings by providing facilities and services for the safe management of human excreta from the toilet to containment and storage and treatment on-site or conveyance, treatment and eventual safe end use or disposal (https://www.who.int/topics/sanitation/en/). On the basis of the transport and treatment mode, there are two main types of sanitation systems, namely; on-site sanitation and off- site sanitation systems. Under the off-site system, there are two sub-categories of centralized and decentralized wastewater treatment (DEWAT). The manual provides guidance to design both types of systems. This chapter focuses on on-site faecal management systems.

3.2 Faecal Sludge Management

In Tanzania, it is estimated that about 90% of the population is served by On-site Sanitation Systems (OSS) and technologies which includes pit latrines, septic tank and soak-away (Brande et al, 2015). These systems end up generating huge volumes of Faecal Sludge (FS). FS is what accumulates in on-site sanitation technologies.

'Faecal Matter (*FM) is defined as raw or partially digested, a slurry or semisolid, and results from the collection, storage or treatment of combinations of excreta and black water, with or without grey water. Examples of on-site technologies include pit latrines, un-sewered public ablution blocks, septic tanks, aqua privies, and dry toilets. Faecal sludge management includes the storage, collection, transport, treatment and safe end-use or disposal of faecal sludge. Faecal sludge is highly variable in consistency, quantity, and concentration*.

Recently, FS has presented itself to be one of the major sanitation management challenges in urban areas of Tanzania. Thus, it calls for adequate and appropriate faecal sludge management (FSM).

To better address FSM challenges, it is important to consider FSM as sanitation service delivery chain. Faecal sludge management (FSM) in developing country settings refers to organized programmes that provide safe and hygienic septic tank and pit emptying services, along with the proper treatment of liquids and re-use of bio-solids where possible. The adequate and proper FSM follows a 'service chain' approach. The service chain includes the collection, storage, transport, treatment and safe end-use or disposal of FS as depicted on **Figure 3.1**.

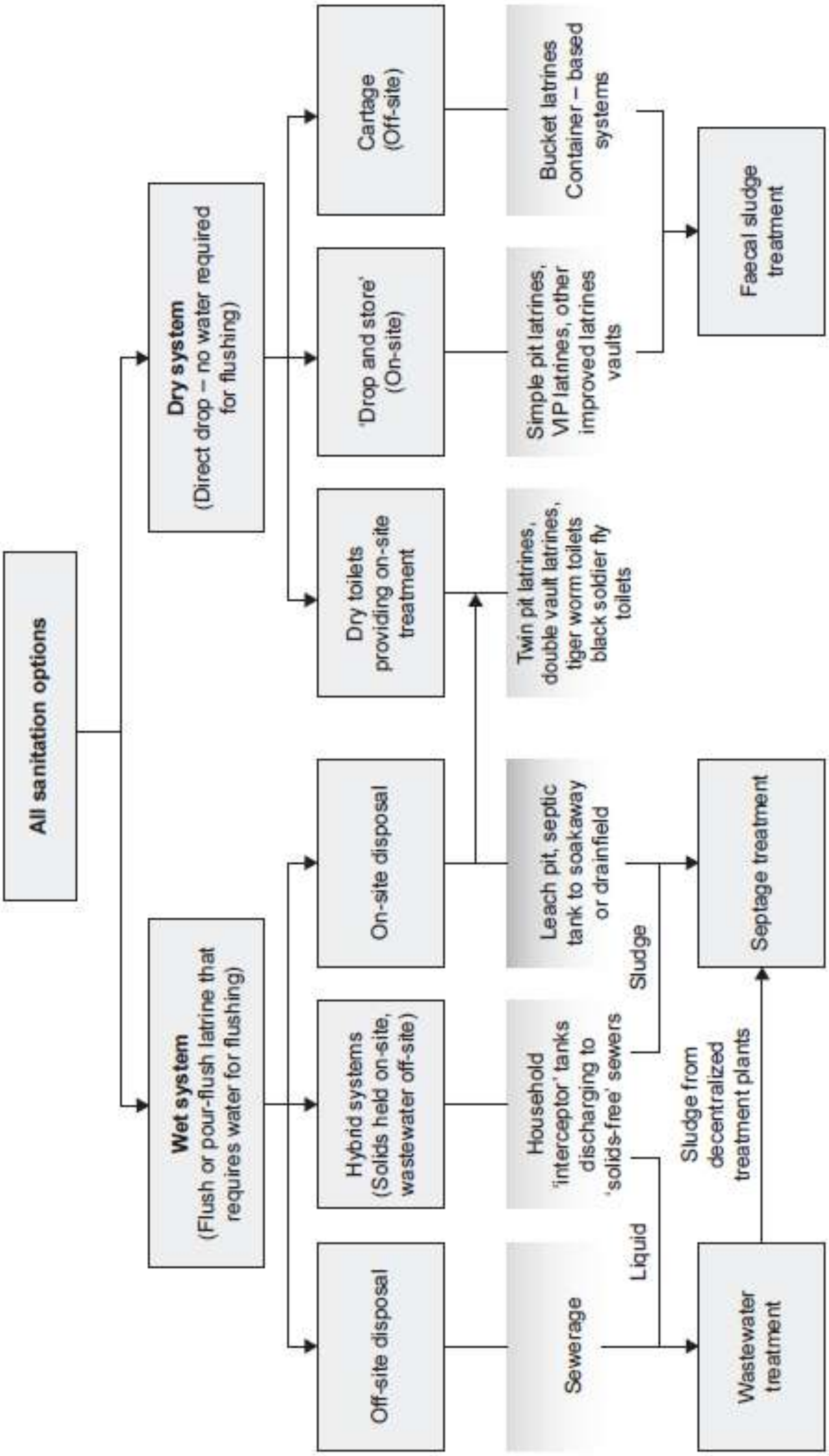


Figure 3.1: Alternative sanitation options

Source: (Kevin Tayler, 2018)

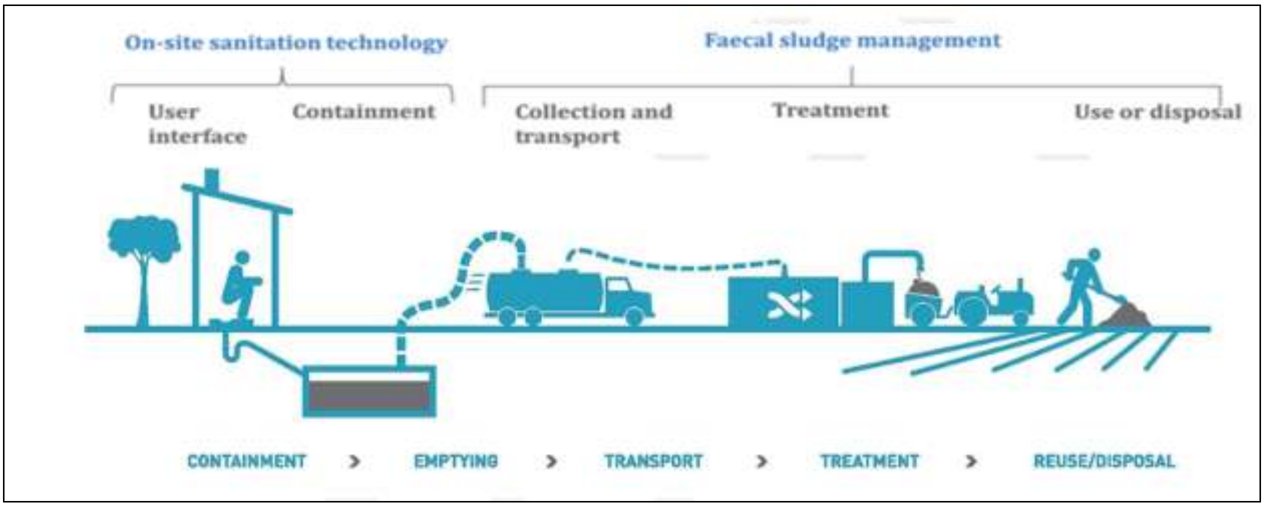


Figure 3.2: A typical faecal sludge management system
Source: (Modified from Wikipedia.org ,2020)

The complete sanitation service chain is shown in Figure 3.2, the FSM component is specifically the emptying, collection, transport, treatment and end-use or disposal of FS. This design manual provides a framework for design of sanitation in all its components of sanitation service delivery chain

3.2.1 Quantification of Faecal Sludge

Deriving accurate estimates for the volume of FS produced is essential for the proper sizing of infrastructure required for collection and transport networks, discharge sites, treatment plants, and end-use or disposal options. The first step in designing faecal sludge (FS) treatment technologies that will meet defined treatment objectives is to quantify and characterise the FS to be treated. Ideally, this should be carried out as part of the feasibility study

Accurate estimate of faecal sludge volume is crucial for the appropriate sizing of collection and transport systems, treatment facilities, discharging sites and disposal options. While methods for quantification of faecal sludge are still being evaluated globally, the following key points have been evaluated during quantification of faecal sludge:

(i) The amount of excreta going into the toilet depending on how much people are eating and excreting,
(ii) How much faecal sludge is produced adding toilet paper and flush water to the previous number,
(iii) The amount of faecal sludge accumulated by balancing accumulation and degradation (currently it is done empirically),
(iv) The amount emptied from the containment,
(v) The amount that is illegally discharged,
(vi) The actual amount delivered to the treatment plant

Additionally, the Figure 3.1 should be used together with the shit flow diagram (see Figure 3.3) to estimate the quantity of FS.

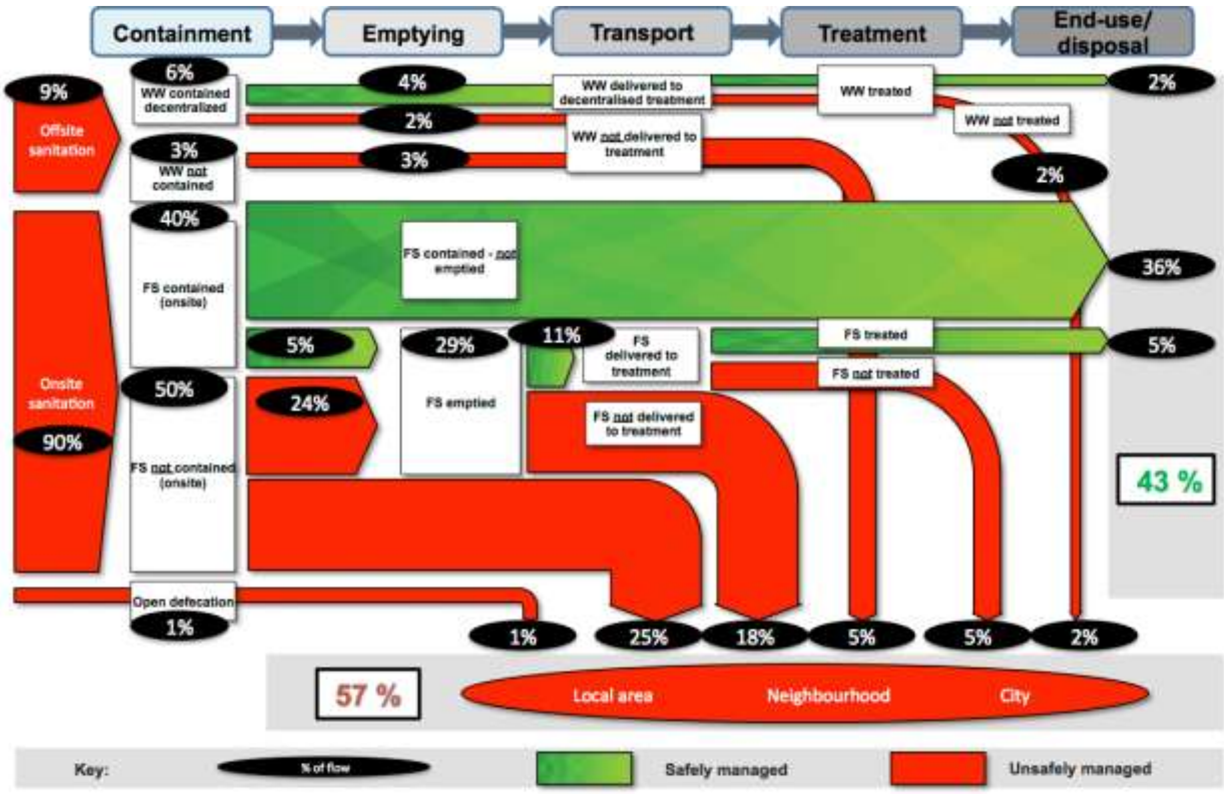


Figure 3.3: Shit flow diagram for Dar es Salaam
Source: (Brande *et al.,* 2015)

Due to the variability of FS volumes generated, it is important to make estimates based on the requirements specifically for each location and not to estimate values based on literature. For example, from FS quantification diagram, bullet six will be obtained from the number of trucks and their capacity. Then from the shit flow diagram, this amount is equal to 11%. So, the total quantity of FS generated is (quantity at point 6/0.11).

However, no proven methods exist for quantifying the production of FS in urban areas, and the data collection required in order to accurately quantify FS volumes would be too labour intensive, especially in areas where there is no existing information.

3.2.2 Characterisation of Faecal Sludge

Parameters that should be considered for the characterisation of FS include Solids - Total Solids (TS), Total Volatile Solids (TVS) and Total Suspended Solids (TSS) concentration, Chemical Oxygen Demand (COD), Biochemical Oxygen Demand (BOD), Nutrients, Pathogens and metals. These parameters are the same as those considered for domestic wastewater analysis. However, it needs to be emphasised that the characteristics of domestic wastewater and FS are very different. In the absence of

Standard methods for Faecal Sludge analysis, Standard methods for analysis of water and wastewater as provided for by APHA, 2012 should be used.

Table 3.1 present examples from literature illustrating the high variability of FS characteristics and provides a comparison with sludge from a wastewater treatment plant. The organic matter, total solids, ammonium, and helminth eggs concentrations in FS are typically higher by a factor of ten or a hundred compared to wastewater sludge (Montangero and Strauss, 2002).There is currently a lack of detailed information on the characteristics of FS. However, research is actively being conducted in this field. Research results, together with empirical observations, will continue to increase the knowledge of FS characteristics, and allow more accurate predictions of FS characteristics using less labour intensive methods.

3.2.3 Design Considerations for Containment

3.2.3.1 Pit latrines, Septic tank-Soakaway pit systems

Containment for On-site systems can be Pit Latrines of different kinds (simple or traditional pit latrine, improved pit latrines, pour flush or ecosan. Pit latrines can be stand alone or combined with septic tank-soakaway pit. For hygiene reasons Tanzania is encouraging use of improved pit latrines but also recognising that many people in rural and peri-urban areas are using traditional pit latrines. The most common type of on-site sanitation system in urban and peri-urban areas is flush toilet-septic tank-soakaway pit combination. The Ministry responsible for Health has prepared guidelines for planning and implementing improved pit latrines, septic tank and pits systems for individual households, community level, institutional and for disaster situations. This document titled "MWONGOZO WA UJENZI WA VYOO BORA NA USAFI WA MAZINGIRA" should be referred to for guidance whenever on-site sanitation composed of any of the aforementioned systems is concerned. The Ministry of Health Guidelines for Improved Latrines and Environmental Sanitation should be widely disseminated and used by LGAs to ensure that these Guidelines are used. The link to the document can be found at http://www.moh.go.tz/en/enviromental-health.

3.2.3.2 On-site systems for areas of high water table and soils of low permeability (e.g. Clay Soils)

Pit latrines and septic tank-soakaway pits described in 3.2.3.1 above can be implemented in areas of low water table and soils of moderate to high permeability in order to allow the necessary treatment to occur. In areas of high water table latrines and soakaway pits tend to overflow during rainy seasons and may require frequent emptying which can be very expensive. There is also concern over public health regarding sewage running freely in the environment exposed to people. It is recommended that for such areas septic tanks or latrines be combined with technologies such as constructed wetlands or subsurface drains. For areas of low permeability it is recommended to use lateral subsurface drain connected to the soakaway pit.

Septic Tank-Constructed Wetland combination

Figure 3.4 and Figure 3.5 show household and institutional systems of septic tank-constructed wetland combination applied to an area of clay soil and high water table, respectively. Design of constructed wetland is explained in section 4.2.4.4.1 of this manual.

Figure 3.4: Constructed wetland treating household septic tank wastewater at Kimara Temboni in Dar es Salaam.

(Courtesy: Prof. K.N. Njau)

Figure 3.5: Constructed wetland treating septic tank effluent from dormitory A of the NM-AIST.
(Courtesy Prof. K.N. Njau)

Septic tank-Soakaway pit-Subsurface (French) field drain combination

When infiltration from the soakaway pit is a problem due to low permeability of the soils, increasing the area of infiltration by introducing lateral subsurface drains also commonly known as French drains to the soakaway pit. Subsurface drains involve excavating a trench and placing a perforated pipe and coarse material such as gravel in the trench. The lateral drains should follow the sloping of the land for easy draining.

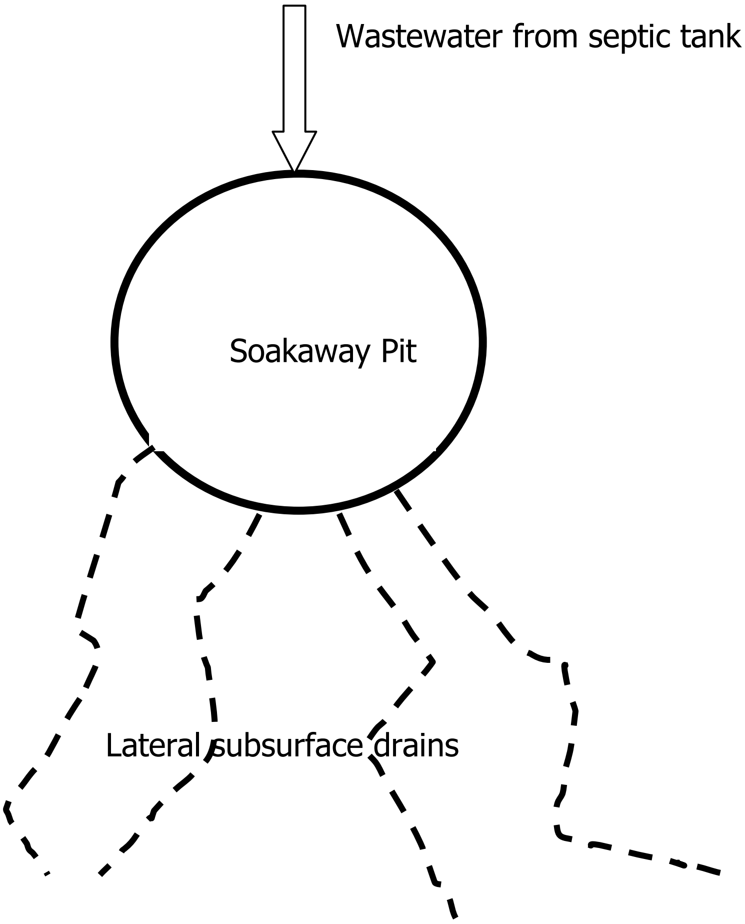


Figure 3.6: Arrangement of lateral drains to a soakaway pit.

Figure 3.7: Subsurface drain perforated pipe with layout of gravels (https://i.ytimg.com/vi/U6r4ZSbTQ8U/maxresdefault.jpg, https://www.ncdrainage.com/)

3.2.4 Design Considerations for Collection

Collection technologies can, in general, be categorized into either manually operated or mechanized mechanical collection technologies. Emptying services in low and middle income countries are frequently a mix between mechanical tools and a manual workforce.

3.2.4.1 Manually operated mechanical collection

Recent innovations in human powered mechanical devices are assisting service providers in servicing septic tanks and pit latrines more quickly, safely and efficiently. There are four (4) common types of mechanical pumping equipment that have been developed and tried; namely, the Sludge Gulper, the diaphragm pump, the Nibbler, and the Manual Pit Emptying Technology (MAPET). A designer of manually operated mechanical collection is advised to refer to the Table 3.2 taking into consideration of the characteristics of of faecal sludge presented in table 3.1.

Table 3.1: Characteristics of faecal sludge from on-site sanitation facilities and sludge

Parameter	FS source		WWTP	Reference
	Public toilet	Septic tank	Sludge	
pH	1.5-12.6			USEPA (1994)
	6.55-9.34			Kengne *et al.* (2011)
Total Solids, TS(mg/L)	52,500	12,000-35,000	-	Koné and Strauss (2004)
	30,000	22,000	-	NWSC (2008)
		34,106		USEPA (1994)
	≥3.5%	<3%	<1%	Heinss *et al.* (1998)
Total Volatile Solids, TVS (as % of TS)	68	50-73	-	Koné and Strauss (2004)
	65	45	-	NWSC (2008)
COD (mg/L)	49,000	1,200-7,800	-	Koné and Strauss (2004)
	30,000	10,000	7-608	NWSC (2008)
	20,000-50,000	<10,000	500-2,500	Heinss *et al.* (1998)
BOD (mg/L)	7,600	840-2,600	-	Koné and Strauss (2004)
	-	-	20-229	NWSC (2008)
Total Nitrogen, TN (mg/L)	-	190-300	-	Koné and Strauss (2004)
			32-250	NWSC (2008)
Total Kjeldahl Nitrogen, TKN (mg/L)	3,400	1,000	-	Katukiza *et al.* (2012)

Parameter	FS source		WWTP	Reference
	Public toilet	Septic tank	Sludge	
NH_4-N (mg/L)	3,300	150-1,200	-	Koné and Strauss (2004)
	2,000	400	2-168	NWSC (2008)
	2,000-5,000	<1,000	30-70	Heinss *et al.* (1998)
Nitrates, NO_3 (mgN/L)	-	0.2-2.1	-	Koottatep *et al.* (2005)
Total Phosphorus, TP (mgP/L)	450	150	9-63	NWSC (2008)
Faecal coliforms (cfu/100mL)	$1x10^5$	$1x10^5$	$6.3x10^4$-$6.6x10^5$	NWSC (2008)
Helminth eggs (Numbers/L)	2,500	4,000-5,700	-	Heinss *et al.* (1994)
	20,000-60,000	4,000	300-2,000	Heinss *et al.* (1998)
		600-6,000		Ingallinella *et al.* (2002)
		16,000		Yen-Phi *et al.* (2010)

Source: Strande et al., 2014

Table 3.2: Summary comparison Table of manually operated mechanical equipment

Equipment Type	Performance	Purchase/Operating cost (USD)	Challenges
Gulper	• Suitable for pumping low viscosity sludges • Average flow rates of 30 L/min • Maximum pumping head is dependent on design	• Capacity Cost: 40 – 1,400 (depending on design)/ • Operating Cost: Unknown	• Difficulty in accessing toilets with a small superstructure • Clogging at high non-biodegradable material content • PVC riser pipe prone to cracking • Splashing of sludge between the spout of the pump and the receiving container
Manual diaphragm pump	• Suitable for pumping low viscosity sludges • Maximum flow rate of 100L/min • Maximum	• 300 – 850 (depending on manufacturer and model) • Operating Cost: Unknown	• Clogging at high non-biodegradable content • Difficult to seal fittings at the pump inlet resulting in

Equipment Type	Performance	Purchase/Operating cost (USD)	Challenges
	pumping head of 3.5m – 4.5m		entrainment of air • Pumps and spare parts currently not locally available
Nibbler	• May be suitable for pumping higher viscosity sludges	• Capital Cost: Unknown • Operating Cost: Unknown	• May be unsuitable for dry sludge with high non-biodegradable material content
MAPET	• Maximum flow rates of between 10 and 40L/ min depending on the viscosity of the sludge and the pumping head • Maximum pumping head of 3.0m	• Capital Cost: 3,000 (1992) • Operating Cost: 175 per annum (maintenance costs only) (1992)	• Requires strong institutional support for MAPET service providers • A reliance on the importation of a key space part • MAPET service providers unable to recover maintenance and transport costs from emptying fees

Source: Strande et al., 2014

3.2.4.2 Fully Mechanized Collection

Fully mechanized technologies are powered by electricity, fuel or pneumatic systems. They can be mounted on a frame or trolley for increased mobility, or mounted on vehicles for emptying and transporting large quantities of sludge over longer distances. A designer is guided to choose any the presented methods in the Table 3.3.

Table 3.3: Summary of mechanised mechanical sludge emptying equipment

Equipment Type	Performance	Cost (USD)		Challenges
		Capital	**Operating**	
Motorized diaphragm pump	• Can handle liquid sludge and solid particles 40 to 60mm in size • Maximum flow rate of 300 to 330L/min • Maximum pumping head of 15m (can easily empty from variable depths)	2,000	Unknown	• Blocking due to non-biodegradable waste in the sludge • Spare parts not available locally
Trash pump	• Can handle very liquid sludge and solid particles 20 to 30 mm in size • Maximum flow rate of approximately 1,200 L/min. Maximum pumping head of 25 to 30m (can easily empty from variable depths)	500 – 2,000	Unknown	• Difficult to find spare parts • Requires containment system • Potential for clogging
Pit screw auger	• Can handle liquid sludge and a small amount of non-biodegradable waste • Flow rates of over 50 L/min. Pumping head of at least 3m (difficulty emptying from variable depths)	700	Unknown	• The fixed length of the auger and riser pipe • Unsuitable for use with dry sludge and large quantities of non-biodegradable waste • Difficult to clean after use • Difficult to manoeuver due to weight and size
Gobbler	• Blocks easily due to sludge build up in the	1,200	Unknown	• Complex fabrication process and a

Equipment Type	Performance	Cost (USD)		Challenges
	working parts • Pumping head of at least 3m • Difficult emptying from variable depths			high number of parts • Weight of the pump • Length not adjustable
Vacutug	• Can handle low-viscosity sludge well and some non-biodegradable waste • Ideal for areas with limited access • Pumping head varies depending on model used	10,000 – 20,000	25 USD/load[1]	• Can be slow to transport • Difficulty emptying high viscosity sludge • Small volume (500 to 1,900 litres) • Not financially viable for long-haul transport
Conventional vacuum tanker	• Can easily handle low-viscosity sludge well and some non-biodegradable waste • Ideal for transporting large quantities of sludge over long distances • Pumping head varies depending on pump model used	10,000 – 100,000[2]	Highly variable	• Difficulty accessing high-density areas • Difficult to maintain in low-income contexts due to specialized parts • Prohibitively expensive for some service providers

[1] Assuming two loads emptied per day from an average distance of 10 kilometres from the disposal point and an average travel speed of 10 km/h (Mikhael and Parkinson, 2011).
[2] The price range of conventional vacuum tankers varies significantly depending on whether the vehicle is brand new or used, capacity, extra capabilities 9e.g. jetting), and shipping costs.

Source: Strande et al., 2014

3.2.5 Faecal Sludge Transport Technologies

There are two types of FS transport mechanisms, namely manual and mechanical transport. The type of transport technology to be opted may be guided by a number of aspects including:

(i) the type of vehicle to be used including its road worthiness, maintenance, licenses and permits, and where it is kept when it is not in service;
(ii) the type of sludge removal equipment, including hoses, pumps, augers, and other tools of the trade;
(iii) the spill management equipment to be used including shovels, disinfectants, sorbents, and collection bags;
(iv) the skills of the operator including the training and certifications that might be required to perform the work;
(v) procedures that need to be followed including rules of the road and activities at the treatment plant and
(vi) other aspects such as the use of transfer stations, worker health and safety, and emerging technologies.

3.2.5.1 Manual transport

Figure 3.8: Example of manual transport of FM
Source: https://www.worldbank.org/en/topic/sanitation/brief/fecal-sludge-management-tools

Today, both standard carts used for general transport of materials, as well as customized carts designed specifically for transport of FS, can be found in many low-income countries, an example of which is shown in Figure 3.4.

3.2.5.2 Motorized transport

Motorized transport equipment (Figure 3.9) offers the potential for larger load capacities and increased speed, leading to reduced travel times and a greater range compared to manual transport. The operation and maintenance of motorized transport is generally more complex than that of manual transport, however many variations are widely used in low-income countries. Before selecting the type of transport system, it is important to verify that the knowledge and skills to carry out repairs are locally available.

Figure 3.9: Examples of motorized transport of Faecal Sludge
Source: (ENVICON Report on Sludge Management, 2019)

3.2.6 Transfer Stations

It has also been observed that distances from the emptied on-site sanitation systems to a regulated disposal facility of greater than 5km often result in illegal dumping of sewage in creeks and rivers (Kone & Peter 2014). In order for operators to get enough trips done in a day, while keeping the service affordable, has resulted in this illegal practice, which has obvious health and environmental concerns through the contamination of water and attracting vermin and flies. One response to this problem is, to install faecal sludge transfer stations at close proximity to densely populated areas. This approach creates a two-step process for handling the waste matter (SSWM, 2014). Faecal sludge can be safely offloaded at the transfer station (Figure 3.10) by local operators primary transport) and temporarily stored.

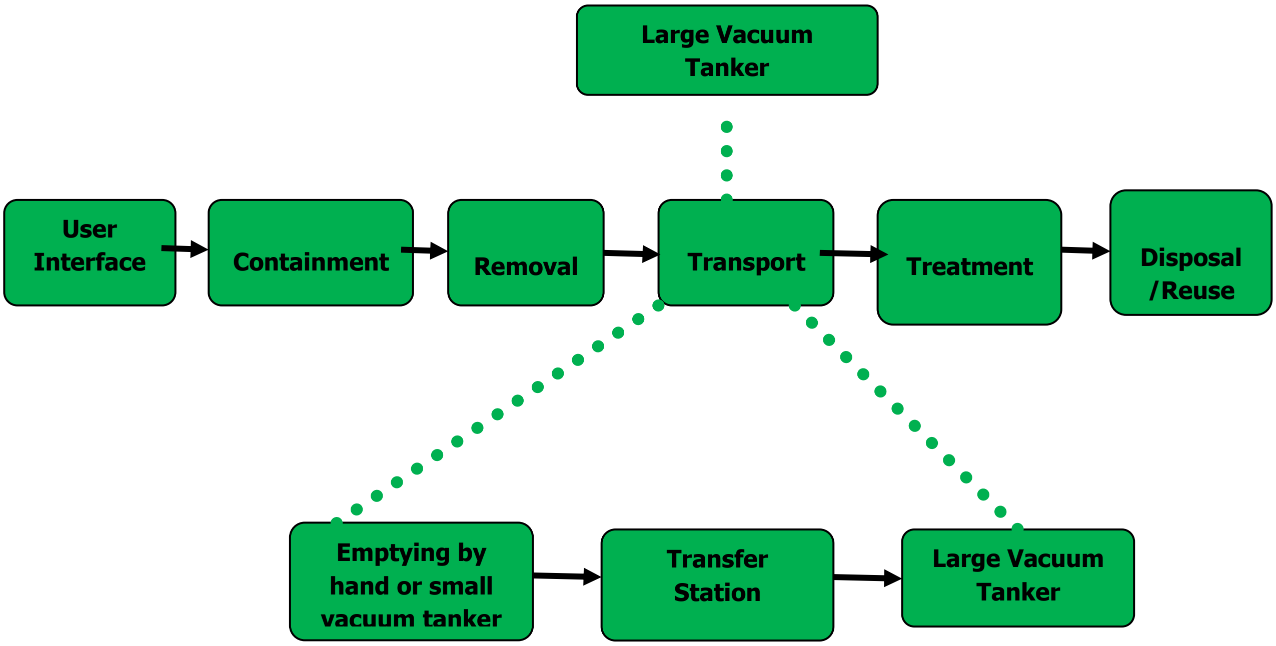


Figure 3.10: Sanitation service chain indicating a transfer station
(Source: BICO, 2017)

Figure 3.7 presents the spatial puzzle when locating a transfer stations and Table 3.4 provides the advantages and disadvantages of the FS transfer stations to assist the designers to make informed choices sooner.

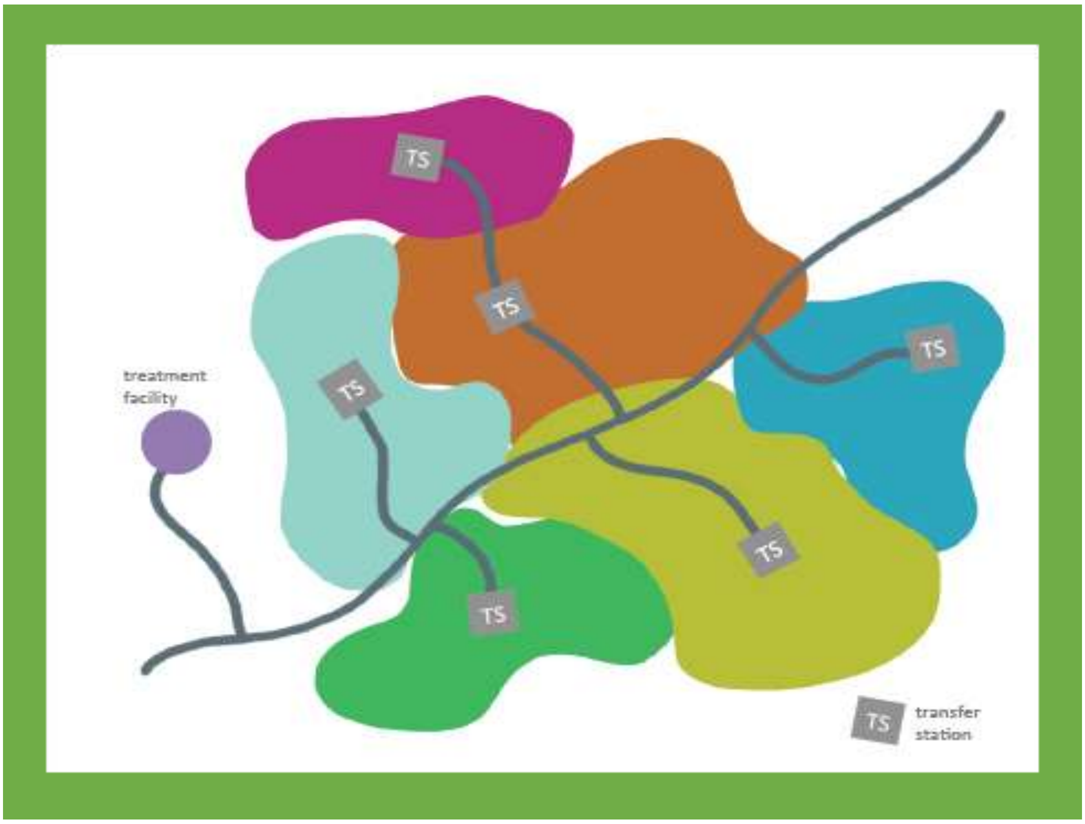


Figure 3.11: The spatial puzzle when locating a transfer station
(Adapted from researchgate.net)

Table 3.4: Advantages and Disadvantages of FS Transfer Station

Advantages of FS transfer station	Disadvantages of FS transfer station
i. Reduces transport distance and makes sludge transport to the treatment plant more efficient, especially where small-scale service providers with slow vehicles are involved	i. Fixed stations require expert design, location and construction supervision
ii. May reduce the illegal dumping of faecal sludge	ii. May cause blockages and disrupt sewer flow in the case of sewer discharge stations
iii. May reduce accidents and spillage	iii. The sludge still requires secondary treatment and/or appropriate disposal
iv. Moderate capital and operation costs	iv. Requires an institutional and regulatory framework for taking care of access fees, connection to sewers or regular emptying and maintenance
v. Latrine deslugers can receive a payment from the utility/operator per load delivered to the transfer station, thereby ensuring safe disposal of the septage	v. Can lead to bad odours and vermin if not properly maintained
vi. May encourage more community-level emptying solutions	vi. May inconvenience a few for the benefit of the whole community
vii. High potential for local job creation and replication of income generation	

3.2.6.1 Fixed transfer stations

Fixed transfer stations can be divided into four main categories, the first of which are 'permanent storage tanks'. Constructed as vault-like concrete structures, these tanks are designed to provide storage capacity for FS over a short period of time without capacity for treatment. An example of such tanks are the underground holding tank (UHT) reported by Boot (2007) in Accra, Ghana. With capacities of approximately 23m^3, the UHTs were designed to provide access to pan latrine collectors (primary transport) and vacuum trucks (secondary transport). Care must be taken that to ensure that FS is NOT stored over long periods as it may cause operational challenges due to settling of sludge.

The fixed transfer station essentially serves the role of a secure, safe, storage facility and can be designed according to the type of containers used. For example, in one project in Ghana a concrete-lined pit within a fenced compound was used to store the containers in order to avoid tampering, flooding or spillage. Once full the IBCs are emptied with a vacuum truck. Figure 3.12 presents an example of a fixed transfer station showing the top of an underground holding tank, a chain to lock the lid, a vent pipe and a wall to prevent overflowing of excreta traveling onto the road.

Figure 3.12: Fixed transfer station
Source: (Boot, 2008)

3.2.6.2 Mobile transfer stations

Mobile transfer stations consist of easily transportable containers providing temporary storage capacity at any point near the structure being emptied - essentially a tank fitted on a wheeled chassis. Examples of such transfer stations include motorised collection vehicles, or tanker trailers pulled via a truck or tractor. Figure 3.13 presents a schematic presentation of a typical mobile transfer station.

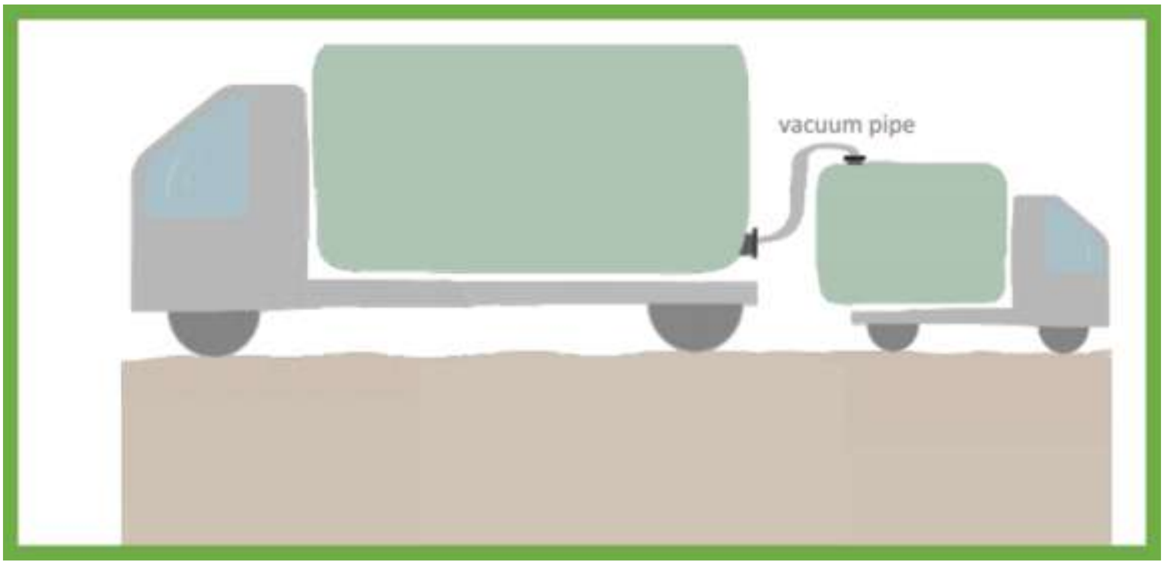


Figure 3.13: A schematic presentation of a typical mobile temporary transfer station.

3.2.7 Design Procedures for Faecal Sludge Treatment

The engineering design approach provides a systematic way of setting appropriate treatment objectives in the absence of a clear regulatory framework. The three steps of the engineering design approach for faecal sludge treatment plants that are briefly introduced in this chapter are:

- Setting treatment objectives based on the final end-use or disposal option,
- Estimating quantities and qualities (Q&Q) of incoming faecal sludge,
- Evaluating selection of management solutions and treatment technologies.

The engineering design approach is an iterative process that needs to take into account the dynamics of the full faecal sludge management chain and an enabling environment.

Step 1

First, options for the final end-use or disposal of treated faecal sludge should be evaluated. Based on the end-use, treatment objectives and corresponding performance goals have to be defined that can be used to work backwards, to inform design variables for the treatment technology that can achieve these goals. For example, pathogen inactivation for a certain treatment product, and an acceptable level of pathogen reduction that can be measured to ensure the treatment objective. It is important to keep in mind that over-designing treatment systems wastes money and resources, while under-designing does not provide adequate protection of human and environmental health.

Step 2

Second is developing an understanding of the faecal sludge quantity and quality (Q&Q) that will be arriving at the treatment facility. Based on these estimates, technologies can be selected and sized appropriately. For more information on methods to determine Q&Q of faecal sludge at community to city-wide scales relevant for the design of treatment and management solution the designer is referred to latest research findings from the area or similar neighbourhood.

Step 3

Based on the defined treatment objectives and the estimated incoming Q&Q of faecal sludge, feasible technology and management options can be selected. To select the most appropriate options, the following factors will have to be considered;

i. existing infrastructure and services,
ii. available skills and capacities,
iii. legal requirements and regulations,
iv. social acceptance and norms,
v. operation and maintenance and

vi. financial viability.

Various trade-offs will exist among all these factors, and it is important to find an acceptable balance together from all stakeholders.

When selecting an appropriate technology for faecal sludge treatment, it is very important to consider whether technologies are established, being transferred from other sectors, or are still at the innovation phase of development. Technologies are classified based on

i. the level of adaption,
ii. research,
iii. innovation and
iv. expert knowledge that is required for successful implementation.

Technologies are considered established if their design and their operational and maintenance guidelines can be readily recommended. Transferring technologies are still in the process of being adapted for faecal sludge management from other sectors such as wastewater treatment or solid waste management. Innovative technologies are promising and potentially ready to be scaled up, but are currently still at the pilot scale of development.

Table 3.5 summarizes faecal sludge treatment technologies and their level of development. The implementation of transferring and innovative technologies has an increased risk due to there being no or less operating experience. Hence, technology development needs to be managed during operation. This could be done through, for example, public or private partnerships or research collaboration with research entities or universities. The first step in the engineering design approach is defining the treatment objectives based on the effluent and treated sludge standards, resource recovery and/or disposal options. Selecting technologies is also based on factors such as cost, operational and maintenance requirements, and faecal sludge quantities and qualities (Q&Q) that need to be treated.

Table 3.5: The three levels of treatment technology development

Established	Transferring	Innovative
Settling-thickening tanks	Anaerobic digestion	Ammonia treatment
Unplanted drying beds	Incineration	Black soldier fly (BSF)
Planted drying beds	Lime treatment	Thermal drying
Co-composting	Mechanical dewatering	Vermicomposting
Deep row entrenchment	Pelletizing	
	Solar drying	

Source: Englund and Strande, 2019

Figure 3.14 presents an example of a treatment plant process flowsheet, and the treatment objectives achieved with each technology.

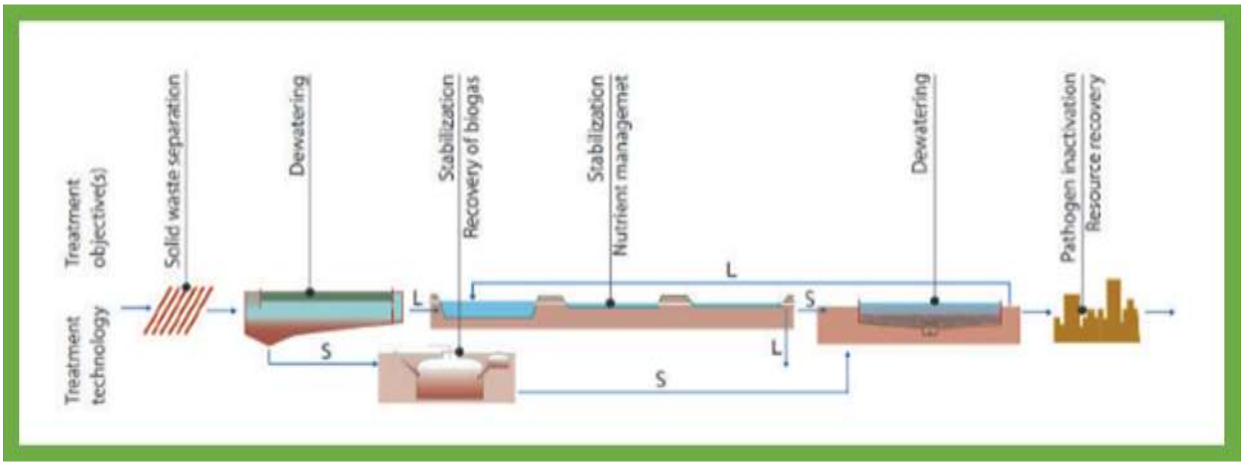


Figure 3.14: Example of a process flowsheet of a faecal sludge treatment plant.
(Adapted from Englund and Strande, 2019)

The treatment process includes screening to remove solid waste, Anaerobic Stabilization Reactor (ASR) for stabilization, unplanted sludge drying beds for dewatering. The dewatered solids from the unplanted drying beds are either co-composted or thermal dried in a solar heated greenhouse to inactivate pathogens prior end-use. The liquid stream from dewatering unit is treated by Anaerobic Baffled Reactor (ABR), Anaerobic Filter (AF), Planted Gravel Filter (PGF), Pressure Sand and Activated Carbon Filter (PS &ACF) and finally disinfected in polishing ponds (PP). See Figure 3.15 for the process flowsheet Faecal Sludge of the Treatment Plant (FSTP) as proposed by CDD society.

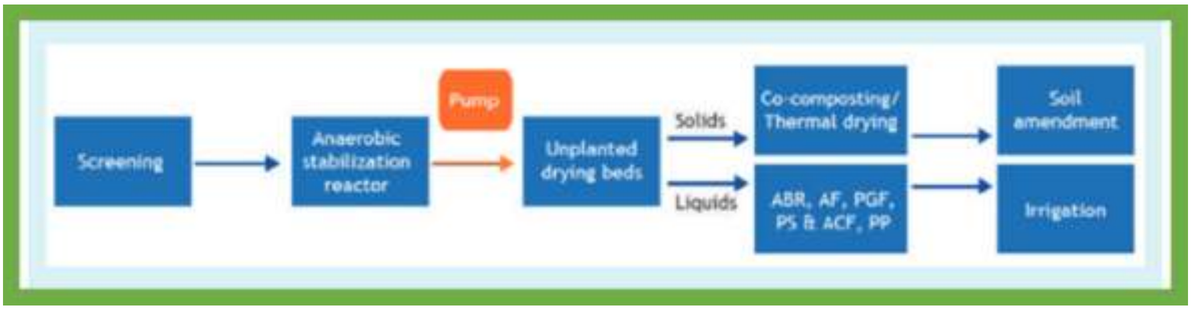


Figure 3.15: Process flow of the concept FSTP proposed by CDD Society
(Source: Englund and Strande, 2019)

3.2.7.1 Settling-thickening tanks

The main treatment objective of settling-thickening tanks is solid-liquid separation; pathogen inactivation does not occur, and both liquid effluent and settled sludge require further treatment. Thickened sludge is normally removed after 5 to 30 days by a combination of pumps, front-loaders, and/or manually with shovels. The loading period should be adjusted with the TSS in the supernatant layer. If the outlet's TSS concentration is too high, adjustments are required. There are normally two parallel tanks to allow for sludge removal and maintenance.

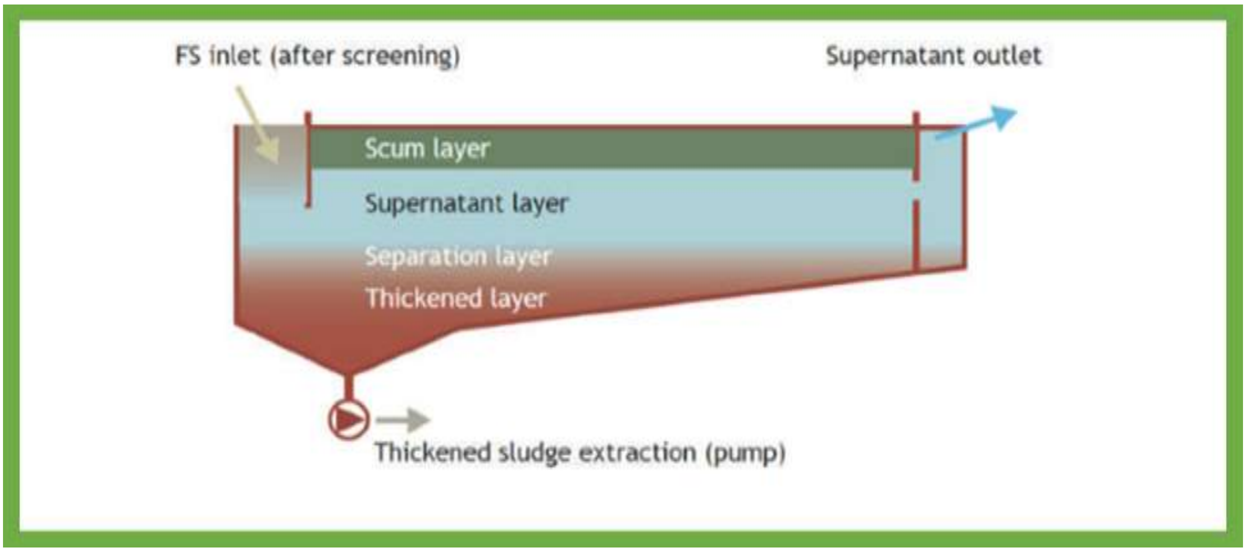


Figure 3.16: Schematic sketch of the settling-thickening tank configuration
(Source: Englund and Strande, 2019)

Settling-thickening tanks have an inlet with a baffle to ensure quiescent flow, and an outlet with a baffle to allow only the supernatant to pass through the outlet. Figure 3.16 presents a schematic sketch of a typical settling-thickening tank. The floor is commonly either sloped so sludge can be pumped out, or else flat with access for removal by a front loader. Total suspended solids settle out into the thickened layer, and fats, oils and grease float in the scum layer.

Prior to designing a settling-thickening tank, tests should be conducted with Imhoff cones to determine the settleability of the specific sludge. Settleability (recommended to be <100 mL/g TSS for settling) is determined by the Sludge Volume Index (SVI) and is estimated by settling of 1 litre of faecal sludge in Imhoff cones or columns for 30 to 60 minutes. The Sludge Volume Index is then calculated from the volume that settles at the bottom of these Imhoff cones divided by the initial total suspended solids concentration.

$$SVI = \frac{Volume\ settled\ FS}{C_{TSS}} \left(\frac{mL}{g}\right) \quad (3.1)$$

In general, it has been observed that sludge that is more stable settles better than sludge that has been stored in the containment for shorter periods of time. Treatment performance can be measured by TSS concentrations in the effluent/supernatant.

Sizing settling-thickening tanks

The approach taken for the sizing of settling-thickening tanks is summarized in Table 3.6.

Table 3.6: Step by step approach for the design of settling-thickening tanks

Step	Activity
Step 1	Establish design criteria
Step 2	Calculate surface area
Step 3	Calculate thickened sludge volume
Step 4	Determine tank dimensions
Step 5	Configure inlet and outlet
Step 6	Operation and maintenance

Source: Englund and Strande, 2019

Step 1: Establish design criteria

The design of settling-thickening tanks is based on the TSS concentration of the incoming sludge and its settleability. Faecal sludge can either be discharged directly to the settling thickening tanks, in which case following the Q&Q method to determine quantities and qualities and quantities would be most appropriate, or can be loaded using other technologies. The following parameters are important:

- *SVI:* Sludge Volume index (mL/g)
- Q_S: Daily inflow of faecal sludge (m^3/day)
- $C_{TSS,in}$: TSS concentration of influent sludge (kg TSS/m^3)
- f_{op}: Delivery/operating days per year (d)
- *e*: Settling efficiency (-) (should be designed as 80%; in reality a reduction will probably occur, so a safety factor is recommended

Step 2: Calculate surface area

The surface area is calculated based on the peak flow to ensure enough time for the particles to settle.

$$SA = \frac{Q_p}{V_u}\,[m^2] \quad \text{......(3.2)}$$

$$Q_p = \frac{Q_s \times C_p}{h}\left(m^3/_h\right) \quad \text{......(3.3)}$$

SA: Surface are of the tank (m^2)
Q_p: Influent peak flow (m^3/h)
V_u*:* Settling velocity (m/h)
C_p: Peak flow coefficient (-)
Q_S: Daily inflow of faecal sludge to the settling-thickening tank (m^3/d)
h: Number of operating hours of the treatment plant per day (h/d)

Based on existing experience, the settling velocity can be estimated as 0.5 m/h in rectangular settling-thickening tanks to treat faecal sludge with a SVI of less than 100 mL/g TSS.

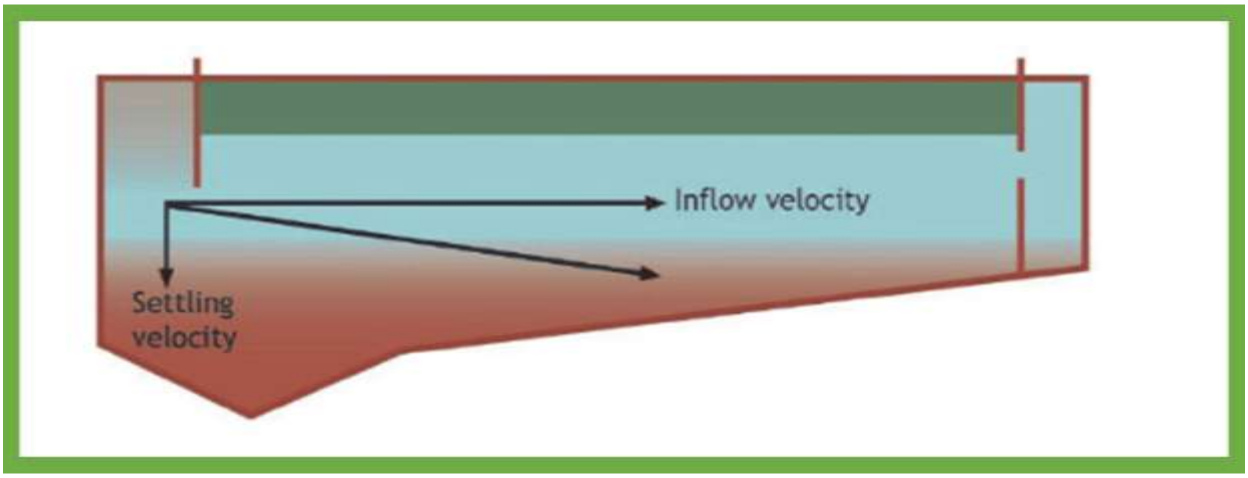


Figure 3.17: Schematic presentation of the relation between settling velocity and inflow velocity for a particle to settle in the tank.
(Source: Englund and Strande, 2019)

The tank is designed based on the desired removal of particles from the supernatant to the solids layer. In order for the particles to settle in the tank, a settling velocity greater than the inflow velocity is required, as shown in Figure 3.17. The recommended ratio between width and length is between 1:10 and 1:5. Lower settling velocities require longer tanks for particles to settle and remain in the tank. Depending on the planned operation of the tank, the design will vary. Manual emptying, either by truck or pump, of solids will result in different layouts and loading and discharge cycles. The calculated area should be doubled so that two parallel-operated settling-thickening tanks can receive discharged faecal sludge

Step 3: Sizing of thickened sludge layer

The thickened sludge layer depth will vary with TSS concentration and will govern the design of the settling-thickening tank.

$$V_t = \frac{Q_s \times C_{TSS,in} \times e \times N}{C_{TSS,out}} \, [m^3] \quad \text{(3.4)}$$

$$d_t = \frac{V_t}{SA} \, [m] \quad \text{(3.5)}$$

V_t: Volume of thickened sludge storage zone (m^3)
Q_s: Daily inflow of faecal sludge to the settling-thickening tank (m^3/d)
$C_{TSS,in}$: TSS concentration of incoming faecal sludge (kg TSS/m^3)
$C_{TSS,out}$: TSS concentration of thickened faecal sludge (kg TSS/m^3)
e: Settling efficiency

N: Faecal sludge loading time (d)
d_t: Thickened faecal sludge depth (m)

To find out how much sludge has accumulated in the thickened sludge layer, the average daily inflow is used. However, in reality the faecal sludge loading period (N) is also determined by a trade off or consideration of the treatment technologies that then follow, although it should not exceed four weeks. As a principle, the more time the thickened faecal sludge has to compact, the more challenging the removal with pumps will be. Expected TSS concentrations in the solids and supernatant layers can be estimated by using Imhoff cones or column testing prior to designing the system.

Step 4: Determine tank depth
The total depth is established by assumptions of the scum, supernatant and separation layer depth. Nevertheless, excavation costs for building tanks are commonly high and are a limiting factor in the maximum depth.

Table 3.7: Preliminary guidelines based on studies in Accra

Sludge layer	Depth of respective layer (m)
Scum	0.4-0.8
Supernatant	0.5
Separation	0.5
Thickened sludge	To be calculated, from step 3

Source: Heinss *et al.,* 1998

$D_t = V_t/SA + \sum$ respective layer depths (m)..(3.6)
Where:
D_t: Total tank depth (m)
V_t: Volume of thickened sludge storage zone (m^3)
SA: Surface area of the tank (m^2)

Step 5: Configure inlet and outlet
Without an adequate inlet and outlet design, turbulent flow will disrupt the solid-liquid separation and the risk of short circuiting increases. The outlet needs to be placed horizontally lower than the inlet to avoid backflow. The baffles are important to stop scum leaving the tank with the outlet and slowing down the influent streams. Also, an inlet chamber is useful to slow down and decrease the inflow forces of the discharged faecal sludge.

3.2.7.2 Unplanted drying beds
The main treatment objective of unplanted drying beds is the dewatering and drying of faecal sludge; they are not intended for pathogen inactivation. The leachate requires further treatment, since the effluent nutrient and organic content from unplanted drying

beds can be higher than for typical wastewater treatment influent. The dewatered and dried solids might need further treatment depending on the end-use.

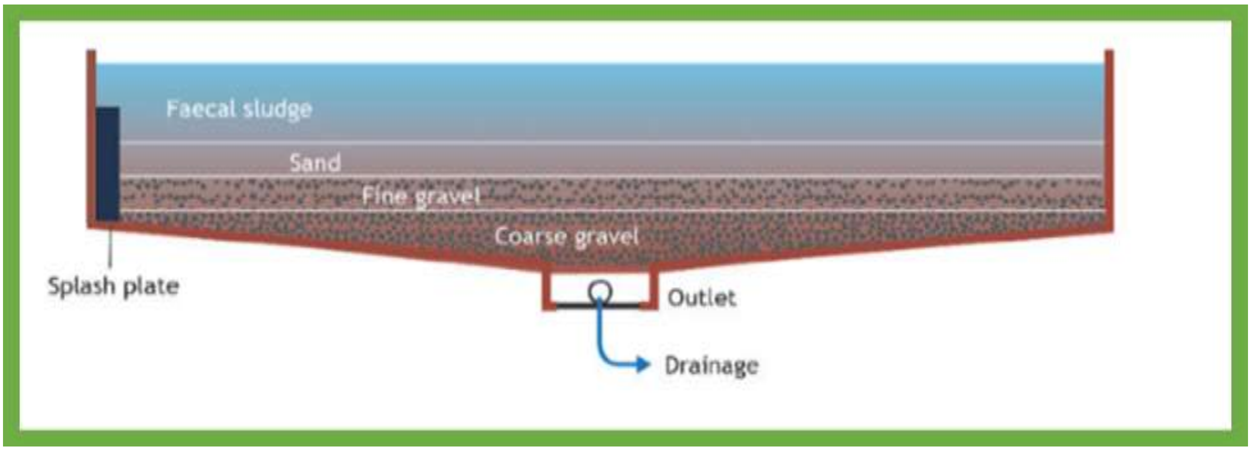


Figure 3.18: Cross section of an unplanted drying bed and its filter media.
(Source: Englund and Strande, 2019)

When using wheelbarrow and shovel to remove the dried sludge cakes, a ramp is recommended to include in the design.

Drying beds are typically rectangular and constructed out of concrete, bricks or stone masonry. The sides need to be high enough to account for hydraulic loadings. The design includes a splash plate to disrupt flow during loading, and a ramp for solids removal. The filter media usually consists of layers of sand and gravel, increasing in diameter with depth, see Figure 3.18 and Table 3.8. Treatment capacity is controlled by solids and hydraulic loading rates, and the treatment performance is based on the filter layer separating solids by exclusion.

Table 3.8: Recommended layer depth and particle sizes for sand and gravel in unplanted drying bed

	Layer depth [cm]	Particle size [mm]
Faecal sludge	20 – 30	-
Sand	10 – 20	0.1 – 0.5
Fine Gravel	10	5 – 15
Coarse gravel	15 – 20	20 – 40

Source: Englund and Strande, 2019

Prior to designing an unplanted drying beds, Capillary Suction Time (CST) test standards and dewaterability standards (TS following centrifugation) should be carried out to determine the dewaterability of the specific faecal sludge. It has been observed that stabilized faecal sludge dewaters better than faecal sludge with short storage time in containment or with high fats, oils and grease content. However, further research is needed to provide additional understanding. To address this, faecal sludge with different levels of stabilization could be mixed. The required level of dewatering will depend on the intended end-use. For example, 50% dryness is recommended prior to pelletizing with the Bioburn pelletizer, 90% for use as a dry combustion fuel and 40-60% for co-composting. However, for faecal sludge to be removed from the drying beds, it needs to be spadable (easily removed with a shovel). Usually it takes 10-30 days for faecal sludge to reach 20-50 % TS, depending on the faecal sludge characteristics, loading rates, filter media and climate.

Sizing unplanted drying beds

The approach outlined below for the sizing of unplanted drying beds is summarized in Table 3.9

Table 3.9: Step-by-step approach for design of unplanted drying beds

Step 1	**Establish design criteria**
Step 2	Determine total drying cycle
Step 3	Calculate surface area
Step 4	Validate surface area
Step 5	Determine number of drying beds
Step 6	Specific concerns with operation and maintenance

Source: Englund and Strande, 2019

Step 1: Establish design criteria
The design of unplanted drying beds is based on the discharged TS concentration and volume. Faecal sludge can be loaded directly onto the drying beds or can be loaded from prior technologies such as settling-thickening tanks. Capillary Suction Time (CST) and dewaterability (TS following centrifugation) should be conducted using standard methods to evaluate the times required for dewatering.

The following parameters are required to calculate the surface area and the required numbers of drying beds:

- Q_s: Discharged faecal sludge (m^3/d)
- C_{TS}: Incoming TS concentration kg TS/m^3
- t_{tot}: Total drying cycle (d)
- f_{op}: Delivery/operating days per year (d/yr)
- *HLR:* Hydraulic loading rate (m/loading)
- *SLR:* Solids loading rate (kgTS/m^2.yr

Step 2: Determine total drying cycle
The total time required for each drying cycle (t_{tot}), is the sum of the time for loading the faecal sludge onto beds (t_l), drying time (t_d) and the time for dried solids removal (t_{sr}). t_{tot} is affected by climate (e.g. humidity, wind, temperature and rainfall). In areas with heavy rain, roofs can be constructed over drying beds.

$$t_{tot} = t_l + t_d + t_{sr} \quad \text{...(3.7)}$$

t_{tot}: Total drying time (d)
t_l: Loading time (d)
t_d: Drying time
t_{sr}: Faecal sludge removal time (d)

Number of drying cycles per year
After determining the time required for one drying cycle, the number of drying cycles in one year can be determined. Drying cycles per year can be calculated by dividing days per year with drying cycle time.

$$n = \frac{days\ per\ year}{Total\ drying\ time} \quad \text{...(3.8)}$$

n: Drying cycles per year (cycles/yr)

Unplanted drying beds will be loaded and unloaded depending on the different treatment technologies that are used before and after the drying beds. For example, settling-thickening tanks that are manually emptied cannot be discharged on a daily basis, rather on a monthly basis.

Step 3: Calculate the surface area

a) Select the loading rate

Area requirements are estimated by either the hydraulic loading rate or solids loading rate. Recommended hydraulic loading rates are 0.2-0.3 m per cycle. Solids loading rates are 50-300 kg TS/m^2.yr in general; however, 100-200 kg TS/m^2.yr is usually recommended in tropical countries.

b) Surface area calculations based on solids loading rate

Solid loading: $\dot{m}_s = Q_s \times C_{TS} \times f_{op} \left[\frac{kgTS}{yr}\right]$ ....(3.9)

Surface Area: $SA_{SLR} = \sum \frac{\dot{m}_s}{SLR} \quad [m^2]$

$\dot{m}_s$: Solids loading kg TS/yr

Q_s: Discharged faecal sludge (m^3/d)

C_{TS}: TS concentration of incoming faecal sludge (kg TS/m^3)

f_{op}: Delivery/operating days per year (d/yr)

SA_{SLR}: Surface area based on SLR (m^2)

SLR: Solids loading rate (kg TS/m^2,yr)

Step 4: Validate surface area

It is important to validate the calculation, since the largest calculated area will govern the design.

$SA = max(SA_{SLR,} SA_{HLR})[m^2]$...(3.10)

SA_{SLR}: Surface area based on SLR (m^2)

SA_{HLR}: Surface area based on HLR (m^2)

$SA_{HLR} = Q_s \times t_{tot} \div HLR[m^2]$...(3.11)

SA_{HLR}: Surface area based on HLR (m^2)

Qs: Daily inflow faecal sludge to the drying bed (m^3/d)

t_{tot}: Total drying time (d)

HLR: Hydraulic loading rate (m/loading)

The relationship between hydraulic loading rate and solids loading rate can be used to calculate the actual operating hydraulic loading or solids loading rate based on the governing surface area.

$$SLR = HLR \times C_{TS} \times n \left[\frac{kgTS}{m^2.yr}\right] \qquad (3.12)$$

SLR: Solids loading rate kg TS/m^2.yr
HLR: Hydraulic loading rate (m)
C_{TS}: TS concentration of incoming faecal sludge (kgTS/m^3)
n: Drying cycles per year (l/yr)

Step 5: Determine number of the drying beds
To determine the number of drying beds, the total drying time, total incoming faecal sludge quantity and its loading frequency on the drying beds need to be considered. However, the actual number of beds also includes a safety factor, for example, the amount of time or space covering one additional day of operation, and enough beds not to exceed 300 m^2 per bed.

For example, if faecal sludge need to be loaded daily, a simplified assumption of that the number of operating days until next loading can be used as the minimum amount of required beds. With a total drying time of 14 days and if the treatment plant operates six full days per week (Monday through Saturday). Then there are 12 operating days between each loading and 12 drying beds are required plus a safety factor of for example one bed, see Figure 3.19 which will allow for desludging or fluctuating quantities and qualities of the incoming faecal sludge.

Another example, if one has 180 m^3 to load and a hydraulic loading of 30 cm, on a hydraulic loading basis, one need 180 m^3/0.3m = 600 m^2 to accommodate that. Since 300 m^2 per bed should not be exceeded, two beds are required.

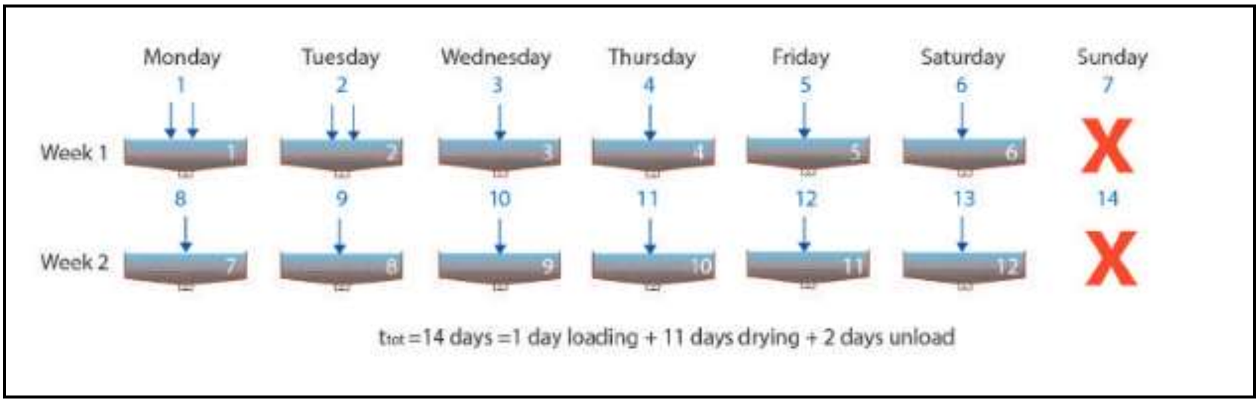


Figure 3.19: Illustration of operation of drying beds
(Englund and Strande, 2019)
Figure 3.19 shows that 12 drying beds are required for a total drying cycle of 14 days, if faecal sludge are loaded every day, with 6 days operation at the faecal sludge treatment plant. The same principal can be applied to other drying requirements and operation days at the treatment plant

3.2.7.3 Planted drying beds

Planted drying beds are used to dewater and stabilize faecal sludge. They are similar to unplanted drying beds in that they both consist of a gravel and sand filter bed, and are designed based on hydraulic and solids loading rates. Faecal sludge is loaded onto the top and the leachate percolates through the bed and is drained away in an under-drain. The difference between planted and unplanted drying beds is that in planted drying beds, the filter bed is used for growing plants and is fed continuously with faecal sludge, whereas unplanted drying beds are batch operated. Planted drying beds are loaded 1 – 3 times a week, with a hydraulic loading rate of 7.5 – 20 cm of sludge per loading depending on the context. For practical reasons, thinner layers are more difficult to evenly distribute faecal sludge.

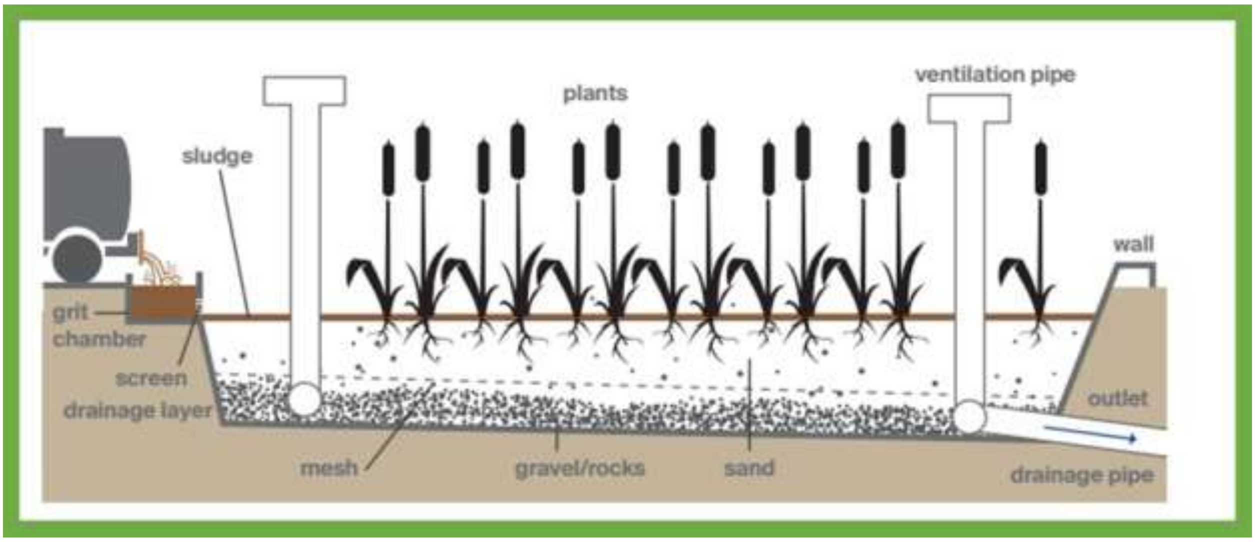


Figure 3.20: Schematics sketch of a suggested layout of a planted drying bed
Source: Englund and Strande, 2019)

As shown in Figure 3.20, the planted drying bed is preceded by a grit chamber so that the incoming sludge does not disrupt the filter media and evenly discharges faecal sludge; a screen separates solid waste. Ventilation pipes ensure a constant air flow through the media, though the plant roots and stems from micro-channels which also aid in ventilation. The sidewalls should be high enough to contain the total volumes of loaded sludge, generally recommended to be 1.5 – 2 metres, and the drain should slope towards the outlet; the beds can be loaded for 5 – 10 years with a few months resting prior to desludging. The size of the beds will depend on the topography of the site.

Table 3.10: Filter layer thickness and particle size

Filter layer	Thickness [cm]	Particle size [mm]
Sand	10 – 15	10 – 40
Fine grave	20 – 25	5 – 10
Coarse gravel	20 – 45	0.2 – 0.6

Source: Strande *et al.*, 2014

The plants help to facilitate dewatering and the stabilization of organic matter and nutrients of faecal sludge. They keep the bed from clogging and provide a more complex environment for the growth of bacteria within the bed. For the treatment process to be successful, the plants need to survive which means they must be tolerant of fluctuating water levels and salinity, be fast-growing, have high transpiration rates, have deep-growing rhizomes and roots, and be non-invasive. It is crucial that the acclimatization period is long enough to establish plant growth and allow adaption to hostile faecal sludge conditions.

Plant species should be selected based on the local conditions and the resource recovery objectives. For example, from research in Senegal, *E. crus-galli* had optimal growth at loading rates of 200 and 300 kg TS/m^2.yr, whereas *P. geminatum* and *P. vaginatum* had growth at 100 and 200 kg TS/m^2.yr, but both species of plants could be used as animal fodder (Mbeguere *etal.,* 2016). For places where *Phragmites australis* or *Phragmites mauritianus* grow well they are preferred. A planted drying bed also known as a sludge wetland being established in Zanzibar shall be planted with *P. mauritianus*

Sizing planted drying beds

The approach outlined for the sizing of planted drying beds is summarized in Table 3.11

Table 3.11: The approach for the sizing of planted drying beds

Step 1	**Establish design criteria**
Step 2	Calculate surface area
Step 3	Determine number of beds
Step 4	Validate surface area calculations
Step 5	Establish acclimatization conditions
Step 6	Operation and maintenance

Source: Englund and Strande, 2019

Step 1: Establish design criteria

The design of planted drying beds depends on solids loading, feeding frequency, resting periods, plant density, plant acclimatization and plant harvesting. Faecal sludge can be loaded directly onto drying beds or can be loaded using other technologies such as settling-thickening tanks. To determine influent values, the Quantity and Quality (Q&Q) methodology should be used. See the list below for input parameters to the design of the sludge planted drying beds.

- Q_S: Daily inflow of faecal sludge (m^3/d)
- C_{TS}: Average TS concentration of influent sludge (kg TS/m^3)
- f_{op}: Delivery/operating days per year (d/yr)
- SLR: Solids Loading Rate (kg TS/m^2.yr), typically 100-300kg TS/m^2.yr
- HLR: Hydraulic loading (m/loading), typically 0.075-0.2 per loading

Step 2: Calculate surface area

Calculate the surface area based on the SLR.

$$\dot{m}_s = Q_s \times C_{TS} \times f_{op} \quad [kgTS] \ldots\ldots(3.13)$$

$$SA_{SLR} = \Sigma\{\dot{m}_s \div SLR\} \quad [m^2] \ldots\ldots(3.14)$$

$\dot{m}_s$: Solids loading kg TS/yr

Q_S: Discharged faecal sludge (m^3/d)

C_{TS}: TS concentration of incoming faecal sludge (kg TS/m^3)

f_{op}: Delivery/operating days per year (d/yr)

SA_{SLR}: Surface area based on SLR (m^2)

SLR: Solids loading Rate (kg TS/m^2,yr)

Step 3: Determine number of beds

The number of beds will depend on the loading frequency of each bed, e.g. 1-3 times a week depending on climate (Sonko *et al.,* 2014) and the frequency and volumes of sludge delivery. There is no standard size for planted drying beds, but for operational purposes, they should not exceed 300 m^2. Multiple beds in parallel are recommended to enable sequential loading and allow a resting phase.

$$n_{db} = \frac{f_{op}}{f_{load}} [bed] \ldots\ldots(3.15)$$

n_{db}: Number of drying beds (bed)

f_{op}: Operating days (d/w)

f_{load}: Loading frequency (d/w.bed)

Step 4: Validate surface area calculations

The design was based on SLR, but each time the beds are loaded the hydraulic loading should be within the range of 7.5 – 20 cm. It now needs to be verified that the SLR and HLR can both be met. Otherwise, the largest surface area will govern the design.

$$HLR = \frac{Q_S}{SA_{SLR}} [m/loading] \ldots\ldots(3.16)$$

$$A_{db} = \frac{SA_{SLR/HLR}}{n_{db}} \qquad (3.17)$$

HLR: Hydraulic loading (m/loading)
Q_s*:* Daily inflow of FS to the planted drying bed (m^3/d)
A_{db}*:* Area for one drying bed (m^2)
$SA_{SLR/HLR}$*:* Surface area based on either SLR or HLR (m^2)

Step 5: Establish acclimatization conditions

The acclimatization phase takes, on average, six months, and in arid climates it should start during the rainy season for best growth. One way to acclimatize the bed is to gradually increase the solid loadings from 50 kg TS/m^2.yr to 200 kg TS/m^2.yr, with a feeding frequency of at least twice a week. Another way would be starting out with dilute wastewater or settling thickening tank effluent, and gradually moving to full strength faecal sludge. Visual indicators of plant stress such as a yellowish colour or slow growth rates should be carefully observed during this period.

3.2.7.4 Co-composting

Composting is the controlled decomposition of organic material into biologically stable humic substances, carried out by micro-organisms and invertebrates in the presence of oxygen. The compost that is finally produced is stable, does not degrade further and is an excellent soil amendment that can improve soil structure and provide nutrients. The need for co-composting faecal sludge with another organic substrate is due to both the low carbon to nitrogen (C:N) ratio in faecal sludge and its high liquid content. If the co-composting process is operated appropriately, treatment objectives such as pathogen reduction, nutrient management, and stabilization can all be achieved. The treatment objectives are fulfilled by regulating the moisture content (~65%), C:N ratio, aeration and temperature. Furthermore, co-composting can reduce around 50% of the volume entering the heap.

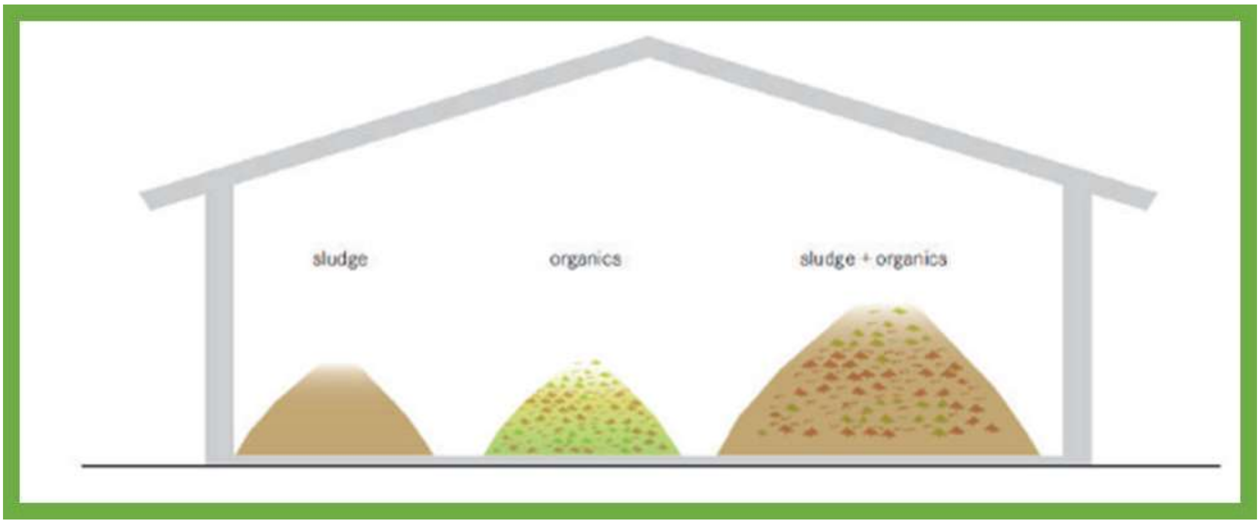


Figure 3.21: Overview of co-compost as a treatment technology
(adapted from Englund and Strande, 2019)

The input material for composting needs to have a C:N ratio between 20:1 and 35:1, so it is necessary to mix faecal sludge with carbon-rich organic matter, and it also needs to have a moisture content of 50-60 %; typically dewatered faecal sludge reaches 30-50 % and a C:N ratio of 18.22±11.12. Therefore, it is recommended to mix faecal sludge in a 1:2-10 ratio with other organic waste. For example, in Ghana, the faecal sludge that International Water Management Institute (IWMI) is working with is 93-99% water, and following dewatering with drying beds, the dried faecal sludge has a C:N ratio of 11±3 (Cofie *et al.* 2016). Figure 3.21 presents the schematic components of the co-compositing treatment technology.

Pathogen inactivation during co-composting is achieved by maintenance of thermophilic conditions over a period of time. Monitoring and maintaining the operating parameters as designed is important to achieve this. If the composting process is not properly maintained, pathogen inactivation could also occur due to storage time in the heap, but this is not reliable and should not be designed for. The key point is that, in reality, the mixing, moisture content and storage are all arbitrary, and the outcome is difficult to control. Local operators usually gain their own knowledge about how the pile should be operated in order not to overheat. Three composting technologies are: 1) windrow; 2) aerated static; and 3) in-vessel. Solid waste such as plastics will not decompose and should be removed prior to co-composting.

Sizing co-composting treatment

The approach taken here for the sizing of co-composting is summarized in Table 3.12.

Table 3.12: Step-by-step approach for the design of co-composting with faecal sludge

Step 1	**Establish design criteria**
Step 2	Determine required mass of bulk material
Step 3	Determine C:N ratio and moisture content
Step 4	Calculate area requirements
Step 5	Calculate pathogen inactivation
Step 6	Operation and maintenance

Source: Englund and Strande, 2019

Step 1: Establish design criteria

Presented in Table 3.13 are the operation and design recommendations for co-composting. It is important not to be overly influenced by these numbers but to understand how the parameters function and how they relate to each other. In reality there is a lot of trial and error needed in order to reach the correct operational parameters for each local context.

Table 3.13: Collection of recommended ranges for various parameters

Parameter	Range in literature
Temp [°C]	>50
Moisture content [weight %]	50 – 60
Turning frequency [-]	3 – 6 turnings per 3 months
Pile size (Width : Height : Length) [m]	2 : 1.6 : Length
Faecal sludge : organic solid waste	1 : 2 -3
Particle size [cm]	1 – 2.5 cm forced aeration systems
	5 – 10 cm passive aeration
	<5 cm static piles
C : N ratio in pile	20 – 35 : 1
Co-compost cycle [weeks]	6 – 12

Source: (Cofie *et al.,* 2016; Strande *et al.,* 2014; Tilley *et al.*, 2014)

To start the design of co-composting, establish the C:N ratio and moisture content of the feedstock. In reality, it will be adjusted and fine-tuned during the operation, but it is important to consider both the availability of materials and the inter-relation of the operating parameters. Listed in Table 3.14 are types of organic wastes that you could use for co-composting, depending on their availability, cost and qualities (e.g. moisture, C:N, and particle size). These qualities will also affect the composting times required to achieve stabilization.

Table 3.14: Types of organic waste that can be co-composed with faecal sludge

Source of material	Type of waste
Residences and gardens	Garden trimmings, leaves, grass cuttings
Restaurants and canteens	Raw peelings and stems, rotten fruit, vegetables and leftover food
Market	Organic waste of vegetable and fruit markets
Agro-industries	Food waste, bagasse, organic residues
Parks	Grass clippings, twigs and branches, leaves
Municipal areas	Residential solid waste, human and animal excreta
Dumping sites	Decomposed waste
Animal excreta	Cattle, poultry, pig dung from urban and peri-urban farms
Slaughterhouses	Contents of the digestive system

Source: Cofie *et al.,* 2016

Examples of ranges of C:N ratios are provided in Table 3.15; for more input material refer to databases such as Phyllis2.

Table 3.15: Typical characteristics of co-composting input material

Input material	MC [wt %]	% c_n	% n_n	C:N
Dewatered faecal sludge(1)	42.3±0.42	11.39±7.7	1.05±1.02	18.22±11.12
Household waste(1)	50.65±0.92	30.2±14.9	1.43±0.33	31.44±6.93
Municipal organic waste(1)	68.05±1.34	32.81±19.08	1.25±0.93	28.49±6.00
Sawdust(2)	8	46.8	0.11	425.45
Newspaper(3)				120
Sugarcane waste(3)				50

Source: Cofie et al., 2009; Phyllis, 2018; Brady and Weil, 2002.

Step 2: Determine required mass of bulk material

The mass of each input material will affect both the moisture content and C:N ratio; the process to find the optimum is iterative. To adjust the moisture content in the co-compost heap to a range between 40-60 %, the faecal sludge needs to be mixed with a bulk material with compatible moisture content. The following equation can be used to calculate the required mass of bulk material

$$m_1 = \frac{m_{FS}(MC_{FS} - MC_{mix})}{MC_{mix} - MC_1} [kg] \quad (3.18)$$

MC_n: Moisture content of n (weight %)
m_n: Mass of organic material n (kg)

The same equation can be used to extract moisture contents if the mass of both materials is known. The required volume of each input material is calculated using the density of the bulk agent and calculated mass.

Step 3: Determine C:N ratio and moisture content

The moisture content and C:N ratio are design variables that are interrelated with the input material mass. To determine the C:N ratio of the mixed pile, Table 3.15 or databases such as Phyllis2 (https://phyllis.nl) can be used as a template to collect information similar to the specific context. This can then be solved for example with any solver data software such as Excel to iteratively find a C:N ratio between 20:1 and 35:1 and moisture content of 40-65 % for your pile and, as Step 2 mentions, what mass is required.

$$C:N_{mix} = \frac{m_1 \times C_1(100 - MC_1) + m_{FS} \times C_{FS}(100 - MC_{FS}) + .. + m_n \times c_n(100 - MC_n)}{m_1 \times n_1(100 - MC_1) + m_{FS} \times n_{FS}(100 - MC_{FS}) + .. + m_n \times n_n(100 - MC_n)} \text{...............(3.19)}$$

$C:N_{mix}$: C:N ratio of co-compost mix (-)
c_n: Carbon content (%) of material n (%)
n_n: Nitrogen content (%) of material n (%)
m_n: Mass of organic n (kg)
MC_n: Moisture content of n (weight %)

Step 4: Calculate area requirements

Depending on whether mechanical or manual turning is used, differently sized piles are practical. Piles higher than 1.6m and wider than 2m should be avoided. With mechanical turning, other sizes are possible. The total surface area required includes space for the heaps and storage of input material and a finished product.

SA: Surface area [m²]	$SA = SA_{Co-compost} + SA_{Storage}$ [m²]

................(3.20)

As seen in Figure 3.21, the co-compost can be roofed to protect it from rain and other unfavourable weather conditions. Extreme humid conditions might require more than roofing in order to keep it to a maximum of 60 %, such as walls, additional coverage of the pile, fans and improved drainage. Additionally, the flooring should be lined and the drainage in place, if needed.

Step 5: Calculate pathogen inactivation

Pathogen inactivation can occur as a result of the heat that is generated during the active phase of the composting process; the full process takes up to 90 days. Based on the review of field data compiled by Feachem *et al.* (1983), Vinnerås *et al.* (2003) derived equations to predict the relationship between composting temperature and time required for total removal of viable Ascaris. The equation for Ascaris is:

$t = 177 \times 10^{-0.1922(T-45)}$ [d] ....(3.21)

Where
t: Total removal time [d]
T: Co-compost temperature [^oC]

When depending on pathogen inactivation, temperatures need to be adequately measured to ensure thermophilic temperatures prevailing during the full period of time.

3.2.7.5 Co-treatment of faecal sludge with wastewater

If an area is served by both centralized, sewer-based sanitation and on-site sanitation technologies, it can be efficient to manage and treat them together. However, the risk of failure is high, and the consequences of failure are significant, so the options must be carefully considered and managed. Faecal sludge can have higher concentrations of TS, organic matter and nutrients than wastewater, is more variable (by 1-2 orders of magnitude), and has varying levels of stabilization. Hence, faecal sludge cannot necessarily be treated in the same ways as domestic centralized wastewater.

The focus of this sub-section is how to modify existing centralized wastewater treatment plants for co-treatment with faecal sludge; note also, that the same considerations are relevant for designs of new co-treatment plants. Discharging faecal sludge into sewers is not recommended, although treating with a supernatant stream following dewatering can be a possibility. Sewers are designed for the gravity flow of wastewater, and thicker faecal sludge does not have the same flow properties. The thicker faecal sludge can result in blockages, preventing wastewater flows and causing overflows (e.g. through manholes and pumping stations). Loadings at the subsequent treatment plant must also be considered.

To avoid shock loadings, the preferred method for co-treatment of faecal sludge would be a continuous flow at a rate that is proportional to the wastewater influent, not only volumetrically, but also considering its characteristics. If there is adequate space, influent variability could be reduced with homogenization tanks. It is also NOT recommended to directly discharge faecal sludge into the headwork of a wastewater treatment plant. This can cause aeration technologies to become over-stressed, resulting in aerobic processes to turn anaerobic, incomplete oxidation, filamentous bacteria, and overloading of settling tanks and clarifiers.

If an existing wastewater treatment plant is currently under capacity, there are two ways that co-treatment with faecal sludge can be considered, see Table 3.16.

Table 3.16: Two options for process flow integration

Option 1.	Option 2.
Dewater the faecal sludge and retreat the supernatant with liquid wastewater streams, and the solids with wastewater solids (biosolids). Example of dewatering include settling-thickening tanks, geo-textile bags, drying beds, and mechanical dewatering.	Treat the faecal sludge together with the wastewater solids stream (biosolids). Examples include co-composting, and co-digestion in an anaerobic digester.

Source: Englund and Strande, 2019

Evaluate co-treatment with existing treatment plants

The approach taken here is summarized in Table 3.17. This is a simplified outline for planning purposes only; in-depth calculations of acceptable loadings and technical adoptions will be required by qualified professionals.

Table 3.17: Step-by-step approach for evaluation of co-treatment with existing treatment plants

Step 1	Form a competent team
Step 2	Assess quantities and qualities of faecal sludge
Step 3	Characterize existing wastewater treatment chain
Step 4	Integrate process flow
Step 5	Model and verify
Step 6	Monitoring and operation

Source: Englund and Strande, 2019

Step 1: Form a competent team

Identify and contract a competent engineering team, with sanitary engineers that have expertise in both faecal sludge and wastewater treatment to understand the complexities of co-treatment. Engineers should be qualified to design and model the proposed modifications, and oversee in-field testing for verification. There should also be adequate expertise at the treatment plant for implementation and operation.

Step 2: Assess quantities and qualities of faecal sludge

Conduct an evaluation of faecal sludge quantities and qualities to determine potential treatment plant loadings (refer to the Sandec news article *Method to Estimate Quantities and Qualities of Faecal Sludge*), including estimating the total future demand. For details refer section 3.2.1

Step 3: Characterize existing wastewater treatment chain

Evaluate the existing process flow, treatment performance, and treatment capacity for each step in the treatment chain. If the plant is at capacity, not operating as designed or not meeting effluent guidelines, then co-treatment should not be considered.

Step 4: Integrate process flow

Based on the results of steps 2 and 3, possibilities for co-treatment can be evaluated. The two possibilities for co-treatment are summarized in

Table 3.16. It is of utmost importance to investigate options for pre-treatment of the faecal sludge prior to co-treatment, and the potential to incorporate them in to existing and future infrastructure (e.g. a settling-thickening tank). No matter how the faecal sludge is integrated into the process flow, it is important to consider not only the consequences of the increased loadings on all the treatment steps but also the final quality of the treated effluent and solids.

Step 5: Model and verify

Model the effects of additional loadings to the existing wastewater treatment plant based on the monitoring data and quantity and quality results. Conduct any additional laboratory tests necessary for validation of the characteristics. Implement co-treatment with a slow start-up period to verify the accuracy of the assumptions and incorporate any necessary modifications.

3.2.8 Faecal Sludge Effluent treatment technologies and their objectives

In general, liquid streams that come from faecal sludge treatment technologies, and are discharged to the environment, are referred to as effluent. Leachate refers specifically to liquid that is drained (or percolates, or is leached) from drying beds, which still requires further treatment prior to discharge. Concentrations of constituents in the effluent are dependent on factors such as the influent characteristics, the treatment technology chain, and treatment performance. Compared to conventional domestic wastewater influent, the liquid stream from faecal sludge treatment technologies such as drying beds, typically has higher concentrations of organics, nutrients and salts, as shown in the example in Table 3.18.

Table 3.18: Leachate characteristics compared to domestic wastewater influent, and discharge standards.

Parameter	Unit	Leachate unplanted drying beds	Leachate planted drying beds	Influent domestic wastewater	Effluent discharge standards
TSS	Mg/L	290-720	49-730	200-450	10-100
COD	Mg O_2/L	3,600-6,500	92-2,200	450-800	50-200
Ammonium nitrogen	Mg/L	150-520	5-200	20-35	5-30

Source: (Koottatep et al.,2004; Heinss et al.,1998, Von Sperling et al., 2005; Sonko et al., 2014; Manga et al., 2016; NEMA, 1999; Kone et al., 2007; Cofie et al., 2006; Strande et al., 2014)

The engineering design approach sets treatment objectives based on local effluent discharge standards or on the desired end-use. Effluents can have salt concentrations that are too high for irrigation, ammonia concentrations that can be harmful to plants, ammonia and organic concentrations that have negative impacts on aquatic environments, and high levels of pathogens. Therefore, it is important to consider the appropriate levels of treatment for the end-use.

There are important differences between effluents from faecal sludge treatment compared to influent domestic wastewater to consider when designing treatment solutions. Effluent from faecal sludge treatment tends to have:

- Higher concentrations of total organic matter (as COD), however, the biodegradable COD fraction can be lower due to the longer on-site retention time. This is important to consider in biological processes for the stabilization of organic matter.
- Higher ammonia concentrations. This is important to consider, as it can inhibit biological processes.
- More variability in the quantities and qualities arriving at the treatment plant.

As a result, it is important to consider actual concentrations, rather than using percent removals, as well as minimums and maximums, rather than averages. Equalization tanks are one example of potential measures that can be put in place to buffer quantities and qualities.

There are several established treatment technologies that work well for effluent depending on the available space. For example, infiltration beds, planted drying beds, constructed wetlands, anaerobic baffled reactors (ABR), waste stabilization ponds and anaerobic filters. These technologies have proved their reliability and capacity to treat high organic loading from faecal sludge effluent. They do not require a constant energy supply and only need light operation and maintenance.

Other technologies that could potentially be transferred from wastewater treatment include activated sludge processes, moving bed bioreactors, trickling filters or sequence batch reactors. These might reach better effluent standards and require less land. However, they need a constant energy supply, are more expensive, require skilled operators and their ability to handle hydraulic and organic shock loads inherent to faecal sludge effluent has not been confirmed. While designing the effluent treatment system, all these parameters have to be carefully evaluated with regards to the local context. For more information on designing technologies, please refer to *Domestic wastewater treatment in developing countries,* Mara (2013).

Step-by-step approach for selection of effluent treatment

The approach taken here to determine the appropriate effluent treatment technology is summarized in Table 3.19. This is a simplified outline for planning purposes. In-depth calculations of acceptable loadings and technical adaptions made by qualified professionals are also required.

Table 3.19: Step-by-step approach for selection of effluent treatment

Step 1	**Define treatment objectives**
Step 2	Assess faecal sludge pre-treatment
Step 3	Assess faecal sludge effluent quantity and quality
Step 4	Select appropriate effluent treatment technology
Step 5	Monitor and operate

Source: Englund and Strande, 2019

Step 1: Define treatment objectives

Identify what the treatment needs to achieve. This includes defining the end-use/disposal of the treated effluent and the corresponding local effluent application/discharge standards that have to be reached. If local standards are not available, use international guidelines and a risk based approach depending on the intended end-use or disposal.

Step 2: Assess faecal sludge pre-treatment

The faecal sludge treatment chain will influence the quantity and quality of the effluent and whether it needs further treatment prior to discharge. For example, settling-thickening tanks, unplanted or planted drying beds will result in different hydraulic loads and nutrient levels in the effluent. Therefore, it is important to understand which treatment steps the faecal sludge is undergoing to be able to evaluate the quantities and qualities that will be handled.

Step 3: Assess faecal sludge effluent quantity and quality
Based on the treatment chain identified in step 2, calculate the quantity (e.g. hydraulic load) and quality (e.g. COD, nutrient and pathogen concentration) of the faecal sludge effluent that needs to be treated prior to discharge. It is crucial to understand not only the dynamics of the quantities and qualities from the respective treatment technology, but also the full chain in order to choose and properly dimension the treatment technologies.

Step 4: Select appropriate effluent treatment technology
Knowing the quantity and quality of faecal sludge effluent as well as the effluent standards for the end-use will define the amount of pollutant that has to be removed from the effluent. With this required level of treatment in mind together with other local specific parameters (e.g. availability of land, electricity, operation and maintenance capacity and financial resources), select the most appropriate treatment technology. The treatment design also needs to take the variability of the quantities and qualities of faecal sludge into account.

3.2.9 Identification of FS treatment construction sites

The identification of existing sites, former sites and potential sites is carried out through discussions with the key stakeholders who should be given a number of alternative scenarios (Table 3.20)

Table 3.20: Criteria for site evaluation

Criteria	*Prerequisite*
1. Average transport distance for mechanical service providers	Acceptability and affordability for service providers, as defined during interviews
2. Accessibility	Ease of access
3. Surface area	Surface area > 0.3 ha
4. Land ownership and price	Guarantee to be able to buy, at a reasonable price
5. Neighbourhood/potential for urbanization	Risk of future access due to urbanization
6. Topography	No risk of flooding
7. Soil type	Free soil (unconsolidated)
8. Groundwater table	> 2m deep
9. Opportunities for disposal of treated effluent and sludge	Must have disposal and endues possibilities

3.2.10 Non-Sewered Sanitation

3.2.10.1 Overview

Non-sewered sanitation System (NSSS) defines a new era of safe non-sewered sanitation. It is a reinvented toilet system which focuses on treating human waste within the toilet itself, operating completely off the grid and recovering valuable resources onsite. NSSS is next generation of off-grid, innovative and novel technological options for sanitation which take into account available water and energy resources, user preferences, variable user population, and are able to treat human wastes at source, eliminating pathogens, and generating products of beneficial value

Innovation of re-invented toilet is a fierce determination to unleash the sanitation revolution, moving to scale and application. NSSS is a prefabricated integrated treatment unit, comprising frontend (toilet facility) and backend (treatment facility) components that:

- collects, conveys, and fully treats the specific input within the system, to allow for safe reuse or disposal of the generated solid, liquid, and gaseous output, and
- is not connected to a networked sewer or networked drainage systems.

NSS provides a cohort of the new disruptive non-sewered sanitation and off-grid sanitation solutions. They are the next-generation toilet technologies that are flourishing – shifting away from the current 'flush-and-dispose' and 'drop-and-store' models to systems that apply circular sanitation thinking and design.

3.2.10.2 Components of a non-sewered sanitation system

In NSSS, the frontend includes user interfaces such as a urinal, squatting pan, or sitting pan, which may apply evacuation mechanisms ranging from conventional flush, pour flush, and dry toilets to novel evacuation mechanisms such as those employing mechanical forces requiring little to no water. Figure 3.22 shows different components of NSSS.

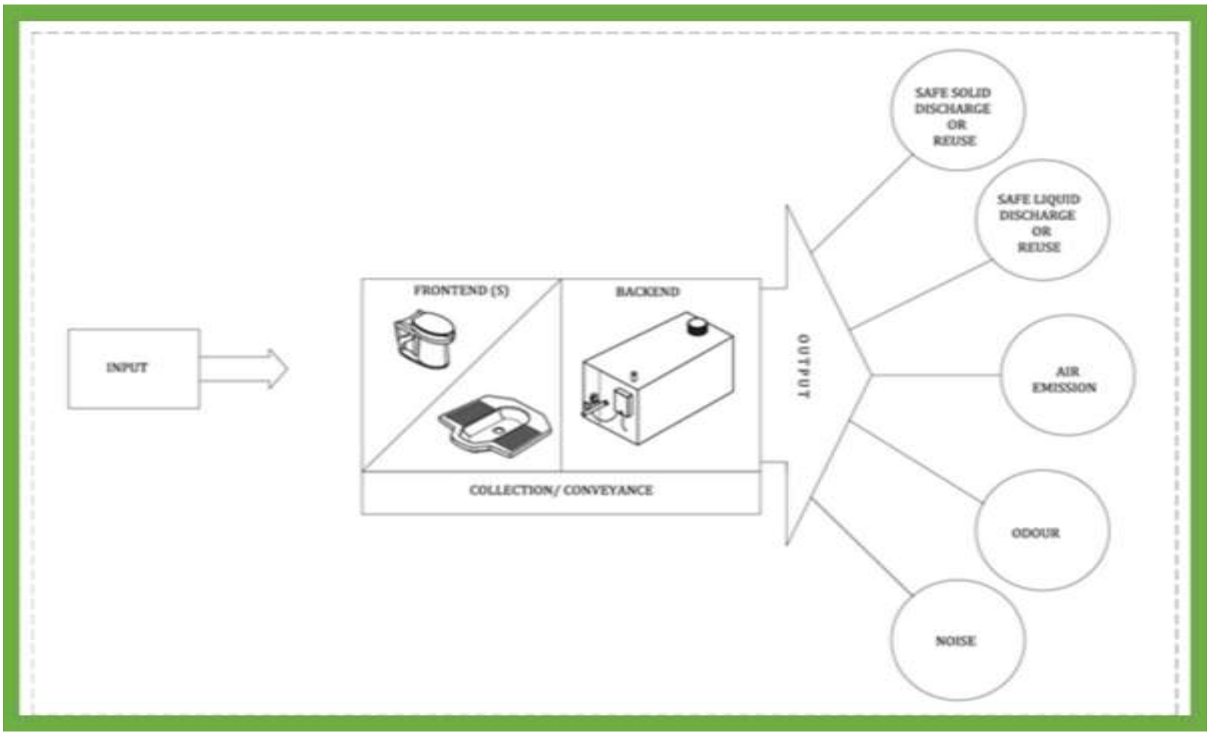


Figure 3.22: Components of the NSSS
Source: https://www.iso.org/obp/ui/#iso:std:iso:30500:ed-1:v1:en

Conventional and novel evacuation mechanisms may be combined with urine diversion applications (e.g. urine diversion flush toilet, urine diversion dry toilet). Backend treatment technologies and processes of NSSS range from biological or chemical to physical unit processes (e.g. anaerobic and aerobic digestion, combustion, electrochemical disinfection, membranes). Some systems use only one of these technologies or processes while others apply various unit processes in combination through several treatment units.

(c) Types of Non-Sewered Sanitation Systems
Two versions of NSSS exist, namely single and multi-unit. Figure 3.23 presents the two versions of NSSS.

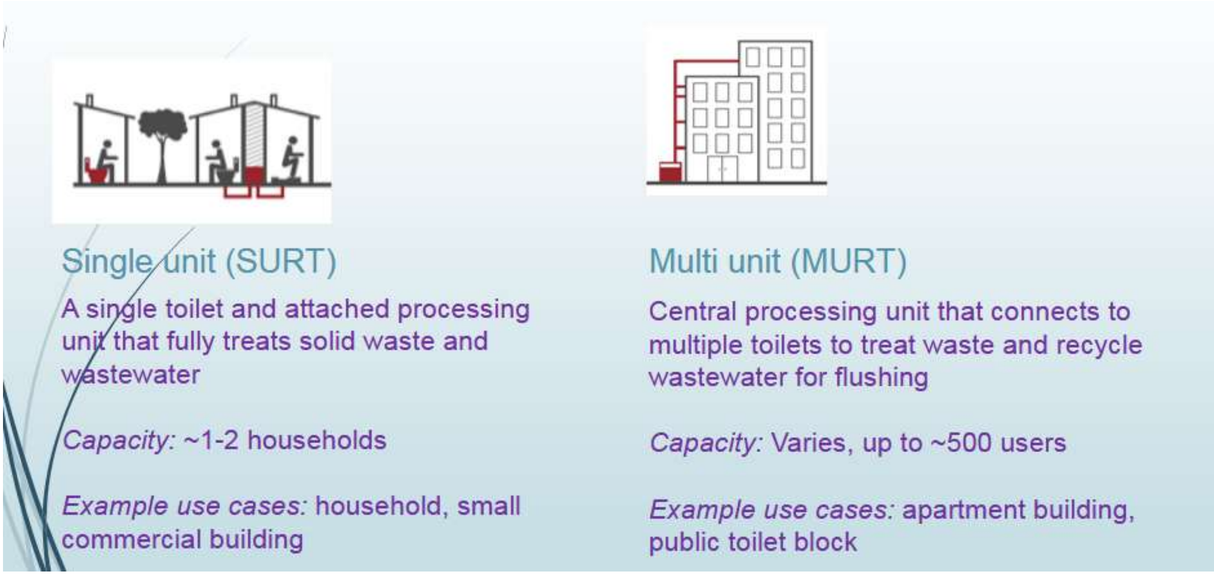


Figure 3.23: Two versions of Reinvented Toilets for different scales: single and multi unit

(d) Design considerations and procedures for NSSS

ISO standards (ISO 30500:2018) provide for design considerations and procedures of NSSS. Title of this standard is ISO 30500, Non-sewered sanitation systems – Prefabricated integrated treatment units – General safety and performance requirements for design and testing. ISO 30500 seeks to provide general safety and performance requirements for the product design and performance testing of non-sewered sanitation systems for prefabricated integrated treatment units.

The user of this manual is thus advised to consult ISO 30500:2018 for more detailed account for design, selection and specifications of NSSS.

References

For Chapter 3 references please refer to the end of chapter four.

CHAPTER FOUR
OFF-SITE SANITATION SYSTEMS

Off-site sanitation refers to a sanitation system in which wastewater and excreta are collected and conveyed away from the plot where they are generated. An off-site sanitation system relies on a sewer technology (simplified sewer, solid free sewer or conventional sewer) for conveyance

4.1 Decentralized Wastewater Treatment Systems(DEWATS)

Decentralized wastewater management systems include all parts of a sanitation system. In comparison to centralized systems, these systems are located at or near the point of wastewater generation. DEWATS can be characterized and differentiated from centralized systems along the following lines.

- Volume: Decentralized systems treat relatively small volumes of water (typically 1 - 1,000 m³/day)
- Sewer type: Centralized systems typically use conventional gravity sewers, while decentralized systems typically use small-diameter gravity sewers, often employing intermediate settlers for solid-free sewers

4.2 Components of DEWATS

The components of DEWAT are presented in the schematic layout Figure 4.1

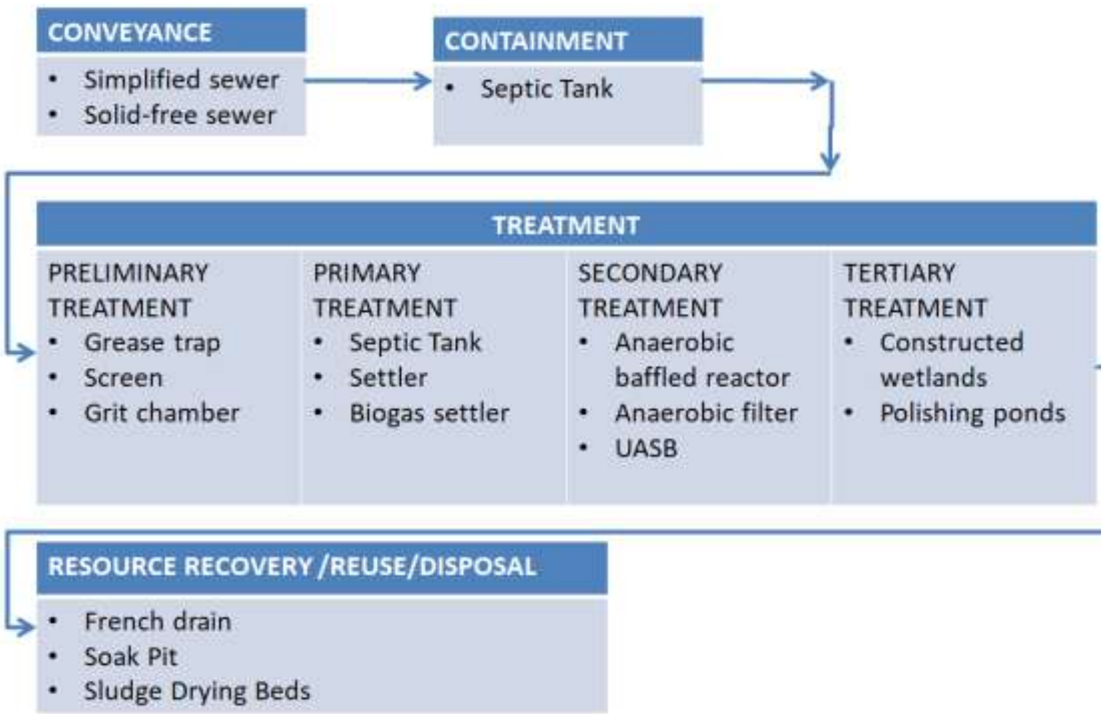


Figure 4.1: Treatment Flow Sheet for Components of DEWAT
(Source: MoW, 2018)

4.2.1 Containment

Containment technologies collect and store wastewater at the user interface on-site. Containment technologies are usually applicable for low-cost, non-sewered sanitation (faecal sludge) systems as intermediate storage, but can also serve as pre-treatment modules for small-scale wastewater treatment systems. The main containment technology applicable for wastewater treatment technologies is a septic tank (see Figure 4.2).

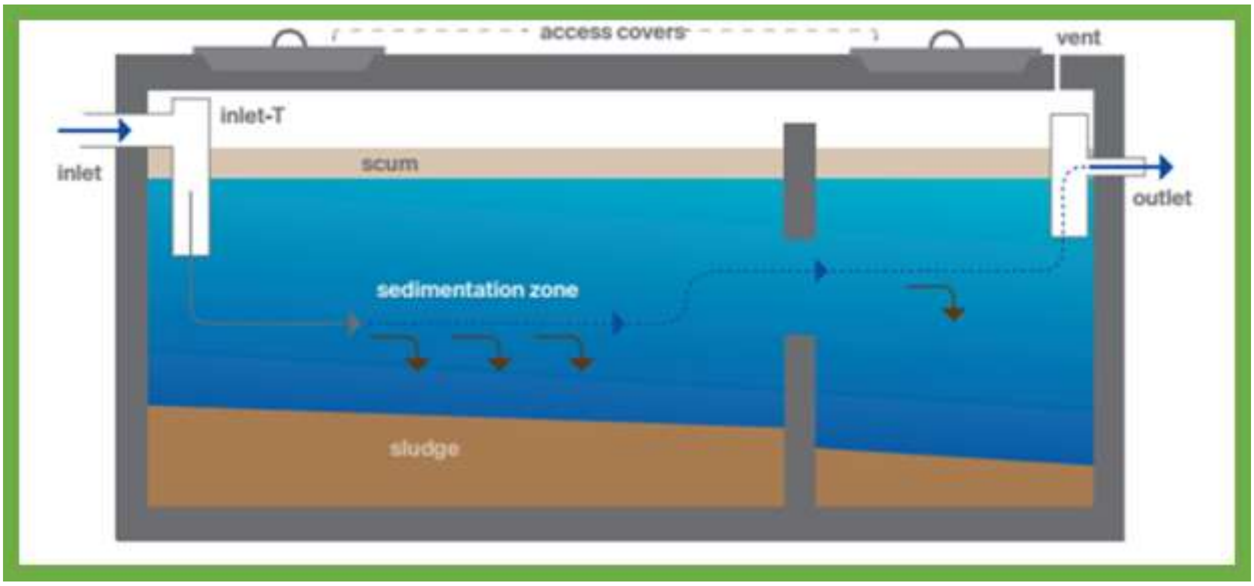


Figure 4.2: The cross section of a septic tank
(Source: MoW, 2018)

In the vast majority of situations, containment systems are already installed on-site but are often improperly designed, constructed and maintained, which poses severe environmental hazards. Apart from septic tanks providing some degree of pre-treatment, the effluent usually contains high concentrations of pollutants, which can carry severe public health and environmental burdens, especially in densely populated urban areas and in the vicinity of drinking water sources. Hence, proper sealing of containment options is crucial for environmental sanitation. Containment systems can also be implemented to buffer peak flows.

4.2.2 Design of Septic Tank

The capacity of septic tank depends on number of users and interval of sludge removal. Normally sludge should be removed every 2 years. The liquid capacity of the tank is taken as 130 litres to 70 litres per head. For small number of users 130 litres per head is sufficient.

A septic tank is usually provided with brick walls in which cement mortar [not less than 20cm (9 inch)] thick and the foundation floor is of cement concrete 1:2:4. Both inside and outside faces of the wall and top of the floor are plastered with minimum thickness of 12mm thick cement mortar 1:3 mix.

All inside corners of the septic tank are rounded. Water proofing agent (such as Impermo, Cem-seal or Accoproof etc.) is added to the mortar at the rate of 2% of the cement weight. Water proofing agent is to be added in similar proportion in to the concrete also for making the floor of the tank.

For proper convenience in collection and removal of the sludge, the floor of the septic tank is given a slope of 1:10 to 1:20 towards the inlet side. Which means that the floor of the outlet side will be on the higher elevation than the floor at the inlet side.

4.2.2.1 Dimensioning a Septic Tank

(A) Length, Width and Depth of Septic Tank

Width = 750mm(min)
Length = 2 to 4 times width
Depth = 1000 to 1300mm. (min below water level) + 300 to 450mm free board
Maximum depth = 1800mm + 450 mm free board
Capacity = 1 cubic metre (minimum)

(B) Detention period

Detention period of 24hrs (mostly) considered in septic tank design. The rate of flow of effluent must be equal to the rate of flow of the influent

(C) Inlet and outlet pipes

An elbow or T pipe of 100mm diameter is submerged to a depth of 250-600mm below the liquid level. For the outlet pipe an elbow or T type of 100mm diameter pipe is submerged to a depth of 200-500mm below the liquid level. Pipes may be of stone ware or asbestos or PVC.

(D) Baffle Walls of the Septic Tank

For small tanks, RCC hanging type scum baffle walls are provided in septic tanks. Baffle walls are provided near the inlet. It is optional near the outlet.
The inlet baffle wall is placed at a distance of L/5 from the wall, where L is the length of the wall. The baffle wall is generally extended 150mm above to scum level and 400-700mm below it.

Scum being light, generally floats at the water level in the tank. Thickness of the wall varies from 50mm to 100mm. For large tanks the lower portion has holes for flow of sludge.

(E) Roofing Slab of the Septic Tank

The top of the septic tank is covered with a RCC slab of thickness of 75-100mm depending upon the size of the tank. Circular manholes of 500mm clear diameter are provided for inspection and desludging. In case of rectangular opening clear size is kept as 600X450mm.

(F) Ventilation Pipe

For outlet of foul gases and ventilation purpose cast iron or asbestos pipe of 50-100mm diameter is provided which should extend 2m (min) above ground level. Top of the ventilation pipe is provided with a mosquito proof wire mesh or cowl.

4.2.3 Conveyance

Technologies presented in this section are sewer-based technologies, using water from waterborne toilets as a conveying medium.

4.2.3.1 Simplified Sewer

A simplified sewer describes a sewerage network that is constructed using smaller diameter pipes laid at a shallower depth and at a flatter gradient than conventional sewers (Figure 3.22).

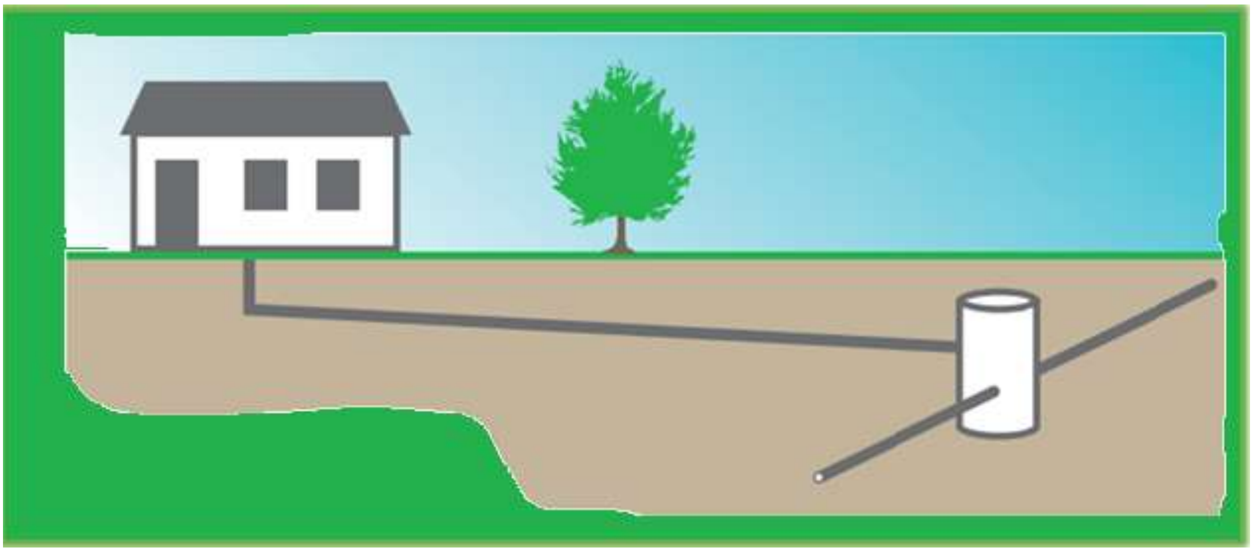


Figure 4.3: Sketch of a simplified sewer
Source: (MoW, 2018)

This sewer system generally does not apply pumping. For these reasons, simplified sewers allow for a more flexible design at lower costs. Simplified sewers can be installed in almost all types of settlements and are especially appropriate for dense urban areas where space for on-site technologies is limited. They should be considered as an option where there is a sufficient population density (about 150 inhabitants per hectare) and a reliable water supply (at least 60 L/capita/day).

4.2.3.1.1 Design considerations for simplified sewers

In contrast to conventional sewers that are designed to ensure a minimum self-cleansing velocity, the design of simplified sewers is based on a minimum tractive tension of 1 N/m^2 (1 Pa) at peak flow. The minimum peak flow should be 1.5 L/s and a minimum sewer diameter of 100 mm is required. A gradient of 0.5% is usually sufficient. For example, a 100 mm sewer laid at a gradient of 1 m in 200m will serve around 2,800 users with a wastewater flow of 60 L/person/day. PVC pipes are recommended to use. The depth at which they should be laid depends mainly on the amount of traffic. Below sidewalks, soil covers of 40 to 65 cm are typical. The simplified design can also be applied to sewer mains; they can also be laid at a shallow depth, provided that they are placed away from traffic.

Expensive manholes are normally not needed. At each junction or change in direction, simple inspection chambers (or cleanouts) are provided as can be seen on Figure 4.3. Inspection boxes are also used at each house connection. Where kitchen greywater contains an appreciable amount of oil and grease, the installation of grease traps (see grease traps) is recommended to prevent clogging. Greywater should be discharged into the sewer to ensure adequate hydraulic loading, but storm water connections should be discouraged. However, in practice it is difficult to exclude all storm water flows, especially where there is no alternative for storm drainage. The design of the sewers (and treatment plant) should, therefore, take into account the extra flow that may result from storm water inflows.

4.2.3.1.2 Design procedures for simplified sewers

Step 1. Estimation of the wastewater flow

Daily peak flows

The value of the wastewater flow used for sewer design is the daily peak flow. This can be estimated as follows:

$$q = \frac{k_1 k_2 P w}{86400} \quad \left[\frac{l}{s}\right] \quad ....(4.1)$$

where

q = daily peak flow, l/s

$\mathbf{k_1}$ = peak factor (= daily peak flow divided by average daily flow)

$\mathbf{k_2}$ = return factor (= wastewater flow divided by water consumption)

P = population served by length of sewer

A suitable design value for k_1 for simplified sewerage is 1.8 and k_2 may be taken as 0.85.

Thus equation 1 becomes:

$$q = 1.8 \times 10^{-5} P ....(4.2)$$

under consideration
w = average water consumption, litres per person per day
and 86 400 is the number of seconds in a day

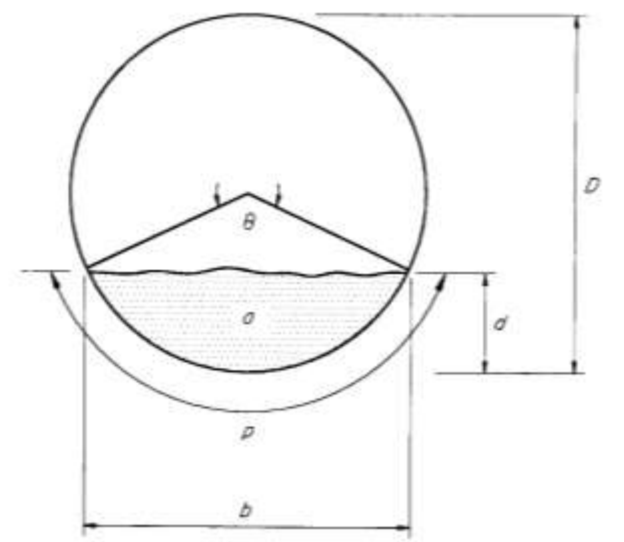
Variations in the value of k_2 have a much lower impact on design, except in middle and high-income areas where a large proportion of water consumption is used for lawn-watering and car-washing. In peri urban areas in Brazil a k_2 value of 0.85 has been used successfully, although other counties use a value of 0.65, even in low income areas and without any reported operational problems (Luduvice, 2000). However higher values may be more appropriate elsewhere – for example, in areas where the water supply is based on a system of public standpipes, values up to 0.95 may be used.

Step 2. Sizing of a simplified sewer
The flow in simplified sewers is always assumed to be an open channel flow – that is to say, there is always some free space above the flow of wastewater in the sewer. The hydraulic design of simplified sewers requires knowledge of the area of flow and the hydraulic radius. Both these parameters vary with the depth of flow.

From Figure 4.4 shows the trigonometric relationships can be derived for the following parameters:

i. The area of flow(*a*), expressed in m^2;
ii. The wetted perimeter (*p*), m;
iii. The hydraulic radius(r), m; and
iv. The breadth of flow(*b*), m.

The hydraulic radius (sometimes called the hydraulic mean depth) is the area of flow divided by the wetted perimeter.



Parameters i -iv above depend on the following three parameters:

(1) The angle of flow (θ), expressed in radians;
(2) The depth of flow (d), m; and
(3) The sewer diameter (D), m.

Figure 4.4: Definition of parameters for open channel flow in a circular sewer.
Source: Mara (1996).

If the angle of flow is measured in degrees, then it must be converted to radians by multiplying by (2п/360), since 360^o equals 2п radians.

The ratio *d*/*D* is termed the proportional depth of flow (which is dimensionless). In simplified sewerage systems the usual limits for *d*/*D* are as follows:

$0.2 < d/D < 0.8$

The lower limit ensures that there is sufficient velocity of flow to prevent solids deposition in the initial part of the design period, and the upper limit provides for sufficient ventilation at the end of the design period. The equations are as follows:

Angle of flow
$\theta = 2 \cos^{-1} [1 - 2 (d/D)]$ ……………………………………………………………………….(4.3)
Area of flow
$a = D^2 [(\theta - \sin \theta) / 8]$ ………………………………………………………………………….(4.4)
Wetted perimeter
$p = \theta D/2$ ………………………………………………………………………………………….(4.5)
Hydraulic radius (= a/p):
$r = (D/4) [1 - ((\sin \theta) / \theta)]$ ……………………………………………………………………….(4.6)
Breadth of flow:
$b = D \sin (\theta/2)$ …………………………………………………………………………………………….(4.7)
When d = D (that is, when the sewer is flowing just flow), then $a = A = \pi D2/4$; $p = P = \pi D$ and $r = R = D/4$.

The following equations for ''a'' and ''r'' are used in designing simplified sewers:

$a = k_a D^2$ ....*(4.8)*

$r = k_r D$ ....*(4.9)*

The coefficients k_a and k_r are given from equations 4.8 and 4.9 as:

$K_a = \frac{1}{8}(\emptyset - Sin\emptyset)$ ....(4.10)

$K_r = \frac{1}{4}[1 - ((sin\emptyset)/\emptyset)]$ ....(4.11)

When $a = A$ and $r = R$, then $k_a = \pi/4$ and $k_r = 0.25$.

Step 3. Velocity of flow

In 1889 Robert Manning (an Irish civil engineer, 1816-1897) presented his formula relating the velocity of flow in a sewer to the sewer gradient and the hydraulic radius(Manning, 1890). The formula is commonly, but improperly, known as the Manning equation; as pointed out by Williams (1970) and Chanson (1999), it should be known as the Gauckler-Manning equation since Philippe Gauckler (a French civil engineer,1826-1905) published the same equation 4.12 years earlier (Gauckler, 1867 and1868).

The Gauckler-Manning equation is

$V = (1/n) r^{2/3} i^{1/2}$ .... (4.12)

Where
V = velocity of flow at d/D, m/s
n = Ganguillet-Kutter roughness coefficient, dimensionless
r = hydraulic radius at d/D, m
i = sewer gradient, m/m (i.e. dimensionless)
Since flow = area × velocity
$q = (1/n) a r^{2/3} i^{1/2}$ ....(4.13)
Where q = flow in sewer at d/D, m^3/s
Using equations 4.8 and 4.9, equation 4.14 becomes:

$q = (1/n) k_a D^2 (k_r D)^{2/3}$ .... (4.14)

The usual design value of the Ganguillet - Kutter roughness coefficient, n is 0.013. This value is used for any relatively smooth sewer pipe material (concrete, PVC or vitrified clay) as it depends not so much on the roughness of the material itself, but on the roughness of the bacterial slime layer which grows on the sewer wall.

4.2.3.2 Solids-free Sewer

Solids-free sewers are also referred to as settled, small-bore, variable-grade gravity, or septic tank effluent gravity sewers. A precondition for solids-free sewers is efficient primary treatment at the household level. Figure 4.5 presents section of solids free sewers.

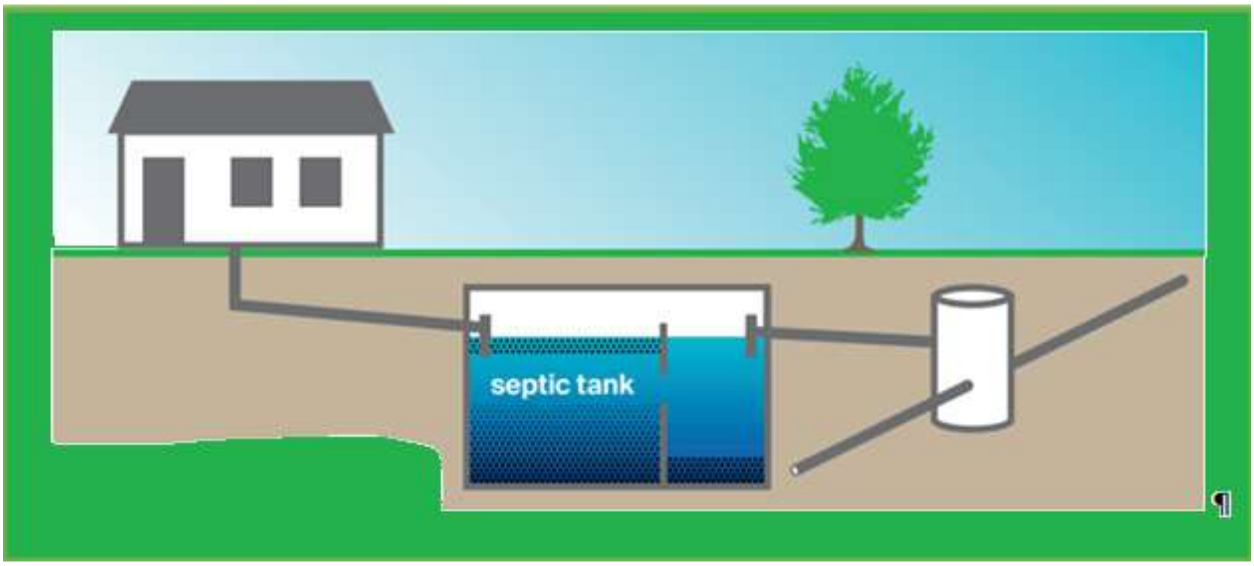


Figure 4.5: Section of solids free sewers
Source: (Tilley *et al.,* 2014)

An interceptor, typically a single-chamber septic tank, captures settleable particles that could clog small pipes. The solids interceptor also functions to attenuate peak discharges. Because there is little risk of depositions and clogging, solids-free sewers do not have to be self-cleansing, i.e., no minimum flow velocity or tractive tension is needed. They require few inspection points, can have inflective gradients (i.e., negative slopes) and follow the topography. When the sewer roughly follows the ground contours, the flow is allowed to vary between open channel and pressure (full-bore) flow.

4.2.3.2.1 Design considerations for solid free sewers

If the interceptors are correctly designed and operated, this type of sewer does not require self-cleansing velocities or minimum slopes. Even inflective gradients are possible, as long as the downstream end of the sewer is lower than the upstream end.

Solids-free sewers do not have to be installed on a uniform gradient with a straight alignment between inspection points. The alignment may curve to avoid obstacles, allowing for greater construction tolerance. At high points in sections with pressure flow, the pipes must be ventilated. A minimum diameter of 75 mm is required to facilitate cleaning.

Expensive manholes are not needed because access for mechanical cleaning equipment is not necessary. Cleanouts or flushing points are sufficient and are installed at upstream ends, high points, intersections, or major changes in direction or pipe size. Compared to manholes, cleanouts can be more tightly sealed to prevent storm water from entering. Storm water must be excluded as it could exceed pipe capacity and lead to blockages due to grit depositions. Ideally, there should not be any storm- and groundwater in the sewers, but, in practice, some imperfectly sealed pipe joints must be expected. Estimates of groundwater infiltration and storm water inflow must, therefore, be made when designing the system. The use of PVC pipes can minimize the risk of leakages.

4.2.3.2.2 Design steps for solids free sewers

Similar equation to simplified sewer can be used to design the size of the sewer in case of very large flow rate of the waste water. Design of the solids free sewer follows the following steps:

Step 1. Estimation of the waste water flow

Daily peak flows

The value of the wastewater flow used for sewer design is the daily peak flow. This can be estimated as follows:

$q = k_1 k_2 Pw / 86\ 400$ ---------------- *(4.15)*

where:

q = daily peak flow, l/s

$\mathbf{k_1}$ = peak factor (= daily peak flow divided by average daily flow)

$\mathbf{k_2}$ = return factor (= wastewater flow divided by water consumption)

P = population served by length of sewer under consideration

w = average water consumption, litres per person per day

and 86 400 is the number of seconds in a day

A suitable design value for k_1 for simplified sewerage is 1.8 and k_2 may be taken as 0.85.

Thus equation 4.14 becomes:

$q = 1.8 \times 10^{-5} P$ -------------------- (4.16)

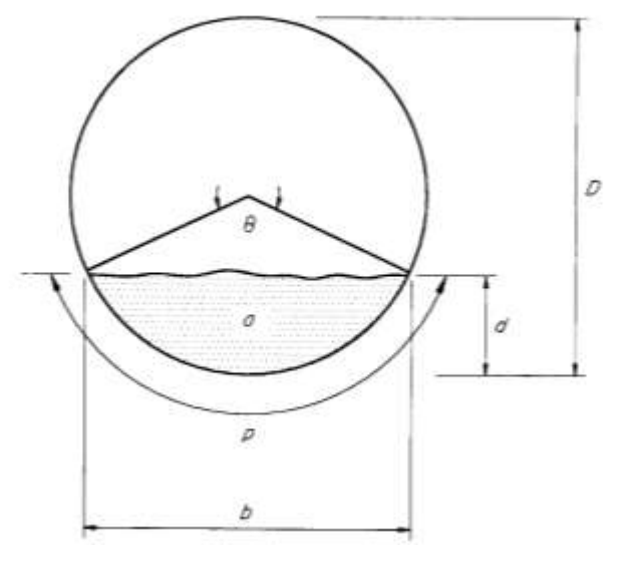
Variations in the value of k_2 have a much lower impact on design, except in middle and high-income areas where a large proportion of water consumption is used for lawn-watering and car-washing. In peri urban areas in Brazil a k_2 value of 0.85 has been used successfully, although other countries use a value of 0.65, even in low income areas and without any reported operational problems (Luduvice, 2000). However higher values may be more appropriate elsewhere – for example, in areas where the water supply is based on a system of public standpipes, values up to 0.95 may be used.

Step 2: Sizing of conventional gravity sewers

The flow in simplified sewers is always assumed to be an open channel flow – that is to say, there is always some free space above the flow of wastewater in the sewer. The hydraulic design of conventional gravity sewers requires knowledge of the area of flow and the hydraulic radius. Both these parameters vary with the depth of flow. From figure below trigonometric relationships can be derived for the following parameters:

(i) The area of flow (*a*), expressed in m^2;
(ii) The wetted perimeter (*p*), m;
(iii) The hydraulic radius (r), m; and
(iv) The breadth of flow (*b*), m.

The hydraulic radius (sometimes called the hydraulic mean depth) is the area of flow divided by the wetted perimeter.



Parameters i – iv above depend on the following three parameters:

a) The angle of flow (θ), expressed in radians;

b) The depth of flow (d), m; and

c) The sewer diameter (D), m.

Figure 4.6: Definition of parameters for open channel flow in a circular sewer.
Source: Mara (1996).

If the angle of flow is measured in degrees, then it must be converted to radians by multiplying by (2π/360), since 360^o equals 2π radians.

The ratio *d*/*D* is termed the proportional depth of flow (which is dimensionless). In simplified sewerage pipes the usual limits for *d*/*D* are as follows:

$0.2 < d/D < 0.8$

The lower limit ensures that there is sufficient velocity of flow to prevent solids deposition in the initial part of the design period, and the upper limit provides for sufficient ventilation at the end of the design period. The equations are as follows:

Angle of flow:

$\theta = 2 \cos^{-1} [1 – 2 (d/D)]$ …………………………………………………………………………(4.17)

Area of flow:

$a = D^2 [(\theta – \sin \theta) / 8]$ ………………………………………………………………………………(4.18)

Wetted perimeter :

$p = \theta D/2$ ……………………………………………………………………………………………(4.19)

Hydraulic radius (= a/p):

$r = (D/4) [1 – ((\sin \theta) /\theta)]$ ………………………………………………………………………(4.20)

Breadth of flow:

$b = D \sin (\theta/2)$ ………………………………………………………………………………………(4.21)

When d = D (that is, when the sewer is flowing just flow), then $a = A = \pi D2/4$; $p = P = \pi D$ and $r = R = D/4$.

The following equations for ''a" and ''r" are used in designing simplified sewers:

$a = k_aD^2$ ……………………………………………………………………………………*(4.22)*

$r = k_rD$……………………………………………………………………………………………*(4.23)*

The coefficients k_a and k_r are given from equations 4.22 and 4.23 as:

$K_a = \frac{1}{8}(\emptyset - Sin\emptyset)$……………………………………………………………………………(4.24)

$K_r = \frac{1}{4}[1 - ((sin\emptyset)/\emptyset)]$ ……………………………………………………………………(4.25)

When *a* = *A* and *r*= *R*, then k_a = π/4 and k_r = 0.25.

4.2.3.3 Conventional Gravity Sewer

Conventional gravity sewers are large networks of underground pipes that convey backwater, grey water and, in many cases, storm water from individual households to a (semi-)centralised treatment facility using gravity (and pumps when necessary). Schematic layout sketch of a conventional gravity sewer is presented in Figure 4.7;

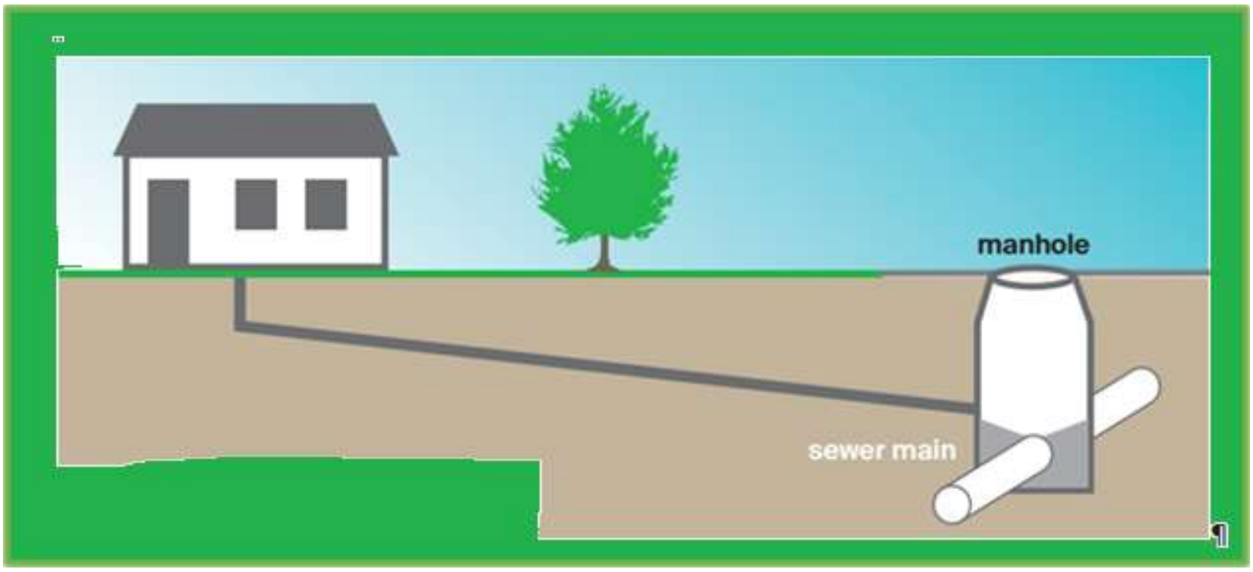


Figure 4.7: Schematic layout sketch of a conventional gravity sewer (MoW, 2018)

Because they can be designed to carry large volumes, conventional gravity sewers are very appropriate to transport wastewater to a (semi-) centralised treatment facility. Construction of conventional sewer systems in dense, urban areas is complicated because it disrupts urban activities and traffic. Conventional gravity sewers are expensive to build and a professional management system must be in place, as the installation of a sewer line is disruptive and requires extensive coordination between authorities, construction companies and property owners.

4.2.3.3.1 Design Considerations

Conventional gravity sewers normally do not require on-site pre-treatment, primary treatment or storage of the household wastewater before it is discharged. The sewer must be designed, however, so that it maintains self-cleansing velocity (i.e., a flow that will not allow particles to accumulate). For typical sewer diameter s, a minimum velocity of 0.6 to 0.7 m/s during peak dry weather conditions should be adopted. A constant downhill gradient must be guaranteed along the length of the sewer to maintain self-cleansing flows, which can require deep excavations. When a downhill grade cannot be maintained, a manhole must be installed. Primary sewers are laid beneath roads, at depths of 1.5 to 3 m to avoid damages caused by traffic loads. The depth also depends on the groundwater table, the lowest point to be served (e.g., a basement) and the topography. The selection of the pipe diameter depends on the projected average and peak flows. Commonly used materials are concrete, PVC, and ductile or cast iron pipes.

Access manholes are placed at set intervals above the sewer, at pipe intersections and at changes in pipeline direction (vertically and horizontally). Manholes should be designed such that they do not become a source of storm water inflow or groundwater infiltration.

4.2.3.3.2 Design steps for conventional gravity sewers

The design steps for the conventional gravity sewers should be the same as those for simplified and solids free sewers. In addition, the design for the system should allow for weir for discharge measurements. Please refer to these sections for the design of conventional gravity sewers. However, for conventional gravity sewer lines, the following should be observed on the pipe sizes to be applied because of potential abuse by users by introducing solids into the sewer lines:

1. Any new sewer connection should use plastic pipes of diameter not less than 150mm or (6") for further extensions (limited number of connected customers not more than 5 in number for domestic use only).

2. Lateral sewers, incorporating more than 5 sewer connections and that may need further extensions in future should involve plastic pipes of diameter not less than 200mm or (8").

3. Commercial and public sewer connections at lodges, hotels, business centres, institutions, industries, apartments and others should use plastic pipes of not less than 200mm (8").

4. All main sewers should start with pipes not less than 200mm (8").

Note that: The previously design plastic pipes of more than 500mm were discouraged to provide room for concrete pipes from that diameter. From field practical experience concrete pipes have higher roughness than plastic pipes and are easily corroded by sewage. Plastic pipes of various diameters and appurtenances for application in sewerage lines are currently manufactured in Tanzania.

4.2.4 Wastewater Treatment

Wastewater treatment is a process used to remove contaminants from wastewater or sewage and convert it into an effluent that can be returned to the water cycle with minimum impact on the environment, or directly reused (https://en.wikipedia.org/wiki/Wastewater_treatment). The typical wastewater treatment flow sheet is presented Figure 4.8.

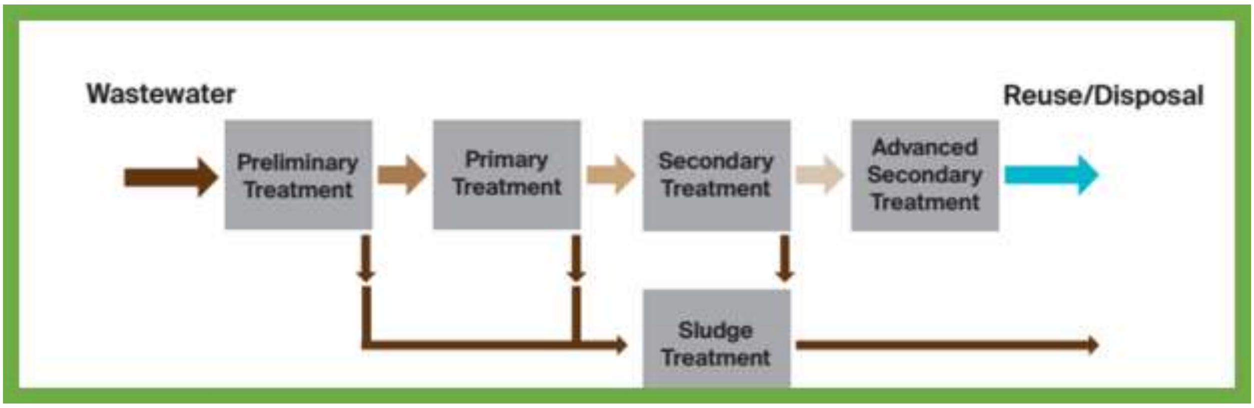


Figure 4.8: A typical flow sheet for a wastewater treatment plant (MoW, 2018)

4.2.4.1 Preliminary Treatment

4.2.4.1.1 Grease Trap

Fats, oils and grease are a major component of food stuffs. The term 'grease is commonly used and sometimes includes the fats, oils, waxes, and other related constituents found in waste water. Greases are solid products (as long as the temperature is sufficiently low) of animal or vegetable origin present in municipal waste water and in some industrial waste waters.

At municipal and industrial wastewater treatment plants where large quantities of grease and fat are to be removed, both aided and induced flotation systems are used to separate the grease and fat from the sewage. These systems involve the use of gas (normally air) bubbles to promote the separation of fat and grease particles from the liquid medium in which they are carried. The rising velocity of the gas bubble determines the efficiency of removal of grease and fat. Figure 4.9 shows section view of a grease trap.

This is sometimes calculated from Stokes equation which is as follows:

Stokes Equation	where:
$V = \frac{g}{18n}\left(P_i - P_g\right)d^2$ ....(4.26)	V = the rising velocity; d = diameter of air bubbles; P_g = density of the gas; P_i= density of the liquid: n= absolute viscosity; and g = gravitational acceleration

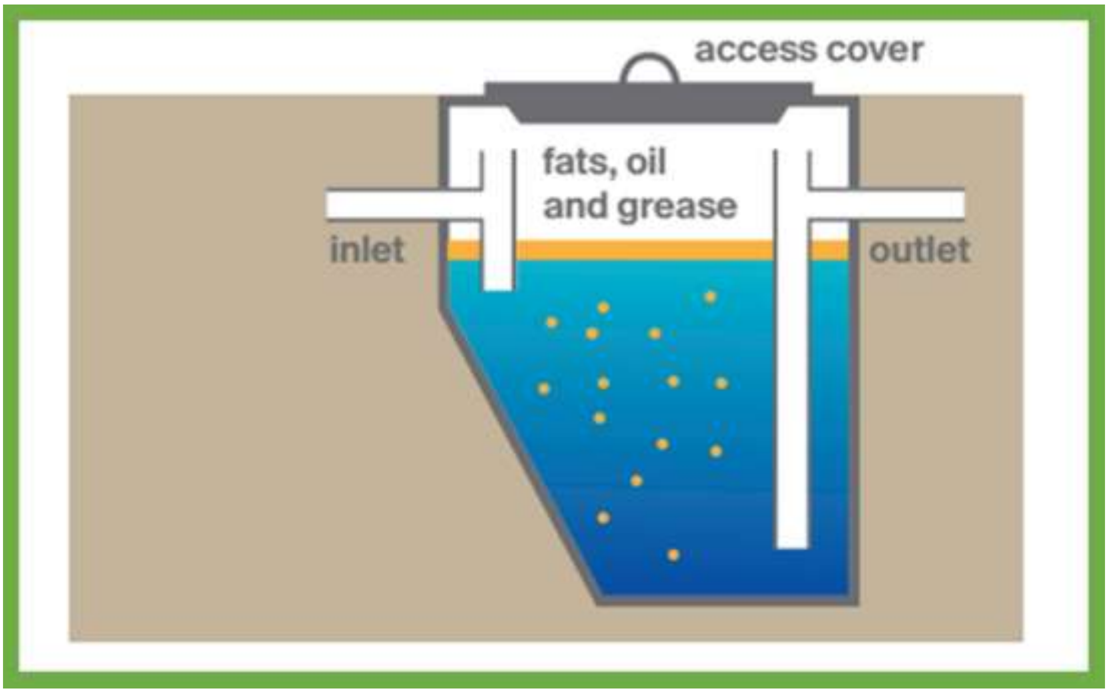


Figure 4.9: Section view of a grease trap
Source: (Tilley et al., 2014)

4.2.4.1.2 Design considerations for grease trap

The minimum requirements for grease trap design are

(i) Provide sufficient capacity to slowdown the passing wastewater, giving greasy waste, the opportunity to separate out. Check the size of an existing grease trap or determine the approximate size of a new grease trap

(ii) The length of the trap should be equal to between 1.3 and 2.0 times the total depth. Note that usually the grease trap contents occupy 2/3 of the total depth; the top 1/3 of the trap is head space. Do not include wall and cover thickness in the length and depth measurements if the grease trap is built of concrete.

(iii) The surface area of the trap (the length times the width in square millimetres) should be equal to between1000 and 2000 times the total depth measured in millimetres. Again, do not include wall and cover thickness in measuring a concrete trap.

(iv) Prevent wastewater entering the grease trap from mixing up the top greasy waste layer. A baffle should be present at the trap inlet to slow down the incoming wastewater and keep it separate from the top waste layer. The inlet pipe should end in a 90° downwards bend so that incoming wastewater enters the trap at least 100 mm below the water surface. The inlet pipe should not terminate above the liquid surface such that wastewater drops into the trap.

(v) Allow access to the trap for maintenance so that all covers can be lifted and accumulated material removed from both the top and bottom of the trap. Except for very large grease traps, the total depth of liquid should never exceed 1200 mm. A sampling hole with appropriate cover must also be provided if the opening for maintenance access does not also give access to the grease trap outlet.

(vi) Provide necessary safety features. All grease traps must be vented. Under-floor grease traps and grease traps with over 1000 litre capacity must be provided with a prominent sign to show location, to indicate both total and liquid depth, and the maximum allowable thickness of the greasy waste layer (30%).

4.2.4.1.3 Screens

Screening is the first unit operation used at wastewater treatment plants (WWTPs). Screening removes objects such as rags, paper, plastics, and metals to prevent damage and clogging of downstream equipment, piping, and appurtenances. Some modern wastewater treatment plants use both coarse screens and fine screens.

(A) Coarse Screens

Coarse screens remove large solids, rags, and debris from wastewater, and typically have openings of 6 mm (0.25 in) or larger. Types of coarse screens include mechanically and manually cleaned bar screens, including trash racks.

Table 4.1: Description of Coarse Screens

Screen Type	Description
Trash rack	Designed to prevent logs, timbers, stumps, and other large debris from entering treatment processes. Opening size: 38 to 150 mm
Manually cleaned bar screen	Designed to remove large solids, rags, and debris. Opening size: 30 to 50 mm Bars set at 30 to 45 degrees from vertical to facilitate cleaning. Primarily used in older or smaller treatment facilities, or in bypass channels
Mechanically cleaned bar screen	Designed to remove large solids, rags, and debris. Opening size: 6 to 38 mm. Bars set at 0 to 30 degrees from vertical. Almost always used in new installations because of large number of advantages relative to other screens.

(B) Fine Screens

Fine screens are typically used to remove material that may create operation and maintenance problems in downstream processes, particularly in systems that lack primary treatment. Typical opening sizes for fine screens are 1.5 to 6 mm. Very fine screens with openings of 0.2 to 1.5 mm placed after coarse or fine screens can reduce suspended solids to levels near those achieved by primary clarification.

(C) Screen Design Steps

Step 1: Selection

The specific screen to be selected will depend on the application. In general, the approach as set out in Table 4.2 is suggested.

Table 4.2: Screen Selection

Application	Aperture	Type
Large Pump houses	50 - 15 mm	Trash rack. R.B.I.
Small Pump houses	50 mm	Liftable cage. Bar screen
Small Wastewater Treatment Plants (Without Sludge Treatment)	15 - 25 mm.	Curved bar screen. Vertical bar screen. Inclined bar screen
Small Wastewater Treatment Plant (With Sludge Treatment)	5 - 10 mm.	Inclined bar screen. Vertical bar screen. Band screen.

Application	Aperture	Type
Medium Wastewater Treatment Plant (With Sludge Treatment)	5 - 10 mm.	Inclined bar screen. Vertical bar screen. Band screen. Screezer (V.D.S.). Rotomat. Contra-shear
Large Wastewater Treatment Plants (With Sludge Treatment)	15 - 50mm. (Before Fine Screen)	Vertical bar screen. R.B.I.
	5 - 10 mm	Band screen. Drum screen. Cup screen. Screezer (V.D.S.). Rotomat. Contra-shear.
Overflows (Retain Screenings in Foul Flow)	5 - 10 mm	Discreen. J&A Weir Mount

Source: Clay *et al.,* 1996

(D) Design Factor for Screens

The basic design of a bar screen should be such that the velocity through the screen would he sufficient for matter to attach itself to the screen without producing an excessive loss of head or complete clogging of the bars. At the same time, velocities in the channel upstream should be sufficient to avoid deposition of solids. In all cases the shape of the bar should be tapered from the upstream side so that any solids which pass the upstream face of the screen cannot be jammed in the screen, thereby causing a trip out of the raking mechanism. The Table 4.3 gives the design factors for bar screens:

Table 4.3: Bar Screen Design Factors

Item	Manually cleaned	Mechanically cleaned
Bar Size: Width (mm) **Depth (mm)**	5 - 15 25 - 80	5- 15 25 - 80
Aperture (mm)	20 - 50	5 - 80
Slope to Flow (Deg)	45^0 - 60^0	18° - 90°
Velocity Through Screen (m/s)	0.3 - 0.6	0.6 - 1.0 (Max. 1.4)

Source: Clay *et al.,* 1996

The following equations may be used for standard bar screens to calculate the width of channel required and the head loss through the screen:

Width of **Channel** $W = \frac{100Q}{V*D*S}$ .... (4.27) **Head Loss** for clean or partially clogged screens $HL = 1.43\left(\frac{V^2-v^2}{2g}\right)$ ....(4.28) for clean screens $HL = b\left(\frac{B}{A}\right)h_v sinq$...................................... (4.29) for fine perforated plate screens $HL = \frac{1}{2g}\frac{1}{C}*\left(\frac{Q}{A}\right)^2$ ....(4.30)	Where Q = Maximum Flow (m^3Is) V = Velocity Through Screen (mis) v = Velocity in Upstream Channel (m/s) D = Depth of Flow (m) W = Width of Channel (m) S = % Screen Open Area. HL = Head Loss Through Screen (m) g = 9.81 m/s^2 (gravity). h = Head on Screen Upstream (m) A = Submerged Aperture Area (mm^2) B = Bar Width (mm) ɵ = Angle of inclination of bars. C = Coefficient which should be checked with the manufacturer. ß= Bar Shape Factor.

The values of bar shape factors for clean rack are summarised as presented in Table 4.4.

Table 4.4: Bar shape factor

Bar type	Bar shape factor
Sharp-edged rectangular	2.42
Rectangular with semi- circular upstream face.	1.83
Circular.	1.79
Rectangular with semi- circular upstream and downstream faces.	1.67
Tear shape.	0.76

4.2.4.1.4 Comminutors and Grinders

Processing coarse solids reduces their size so they can be removed during downstream treatment operations, such as primary clarification, where both floating and settleable solids are removed. Comminuting and grinding devices are installed in the wastewater flow channel to grind and shred material up to 6 to 19 mm in size.

Comminutors consist of a rotating slotted cylinder through which wastewater flow passes. Solids that are too large to pass through the slots are cut by blades as the cylinder rotates, reducing their size until they pass through the slot openings.
Grinders consist of two sets of counter-rotating, intermeshing cutters that trap and shear wastewater solids into a consistent particle size, typically 6 mm (0.25 in). The cutters are mounted on two drive shafts with intermediate spacers. The shafts counter-rotate at different speeds to clean the cutters.

The chopping action of the grinder reduces the formation of rag "balls" and rag "ropes" (an inherent problem with comminutors). Wastewaters that contain large quantities of rags and solids, such as prison wastewaters, utilize grinders downstream from coarse screens to help prevent frequent jamming and excessive wear.

Caution: A designer must satisfy oneself as to why the materials need to be shredded down into small pieces and eventually design how to remove them from the wastewaters.

4.2.4.1.5 Grit Chamber

Grit consists of sand, gravel, stones, soil, cinders, bone chips, coffee grounds, seeds, egg shells, glass fragments, metals and other materials present in wastewater which do not putrefy. In general, grit as defined above has a specific gravity between 1.5 and 2.7 as opposed to a specific gravity for organics of approximately 1.02. In addition, grit settles as discrete particles, rather than as flocculant solids which is the case with organics.

Grit consists of discrete particles which settle independently of one another with a constant velocity. When a discrete particle is left alone in a liquid at rest, it is subjected to a settlement force of gravity and to a resistance resulting from the viscosity of the fluid and inertia. For any given size and density of particle, there is a particular settling velocity. This settling velocity is changed somewhat when the liquid in which the particle is contained is subjected to a horizontal velocity.
Grit settlement is generally regarded as following Stokes' Law which may be stated as:

Stokes Law

$$V_n = \frac{g}{18n}(l_s - l_i)d^2 \quad \text{..} \quad (4.31)$$

Where:
V_n = settling velocity (m/s):
g = gravitational acceleration (m/s^2):
n= viscosity of liquid (kg/ms): Table 4.13 Comparison of LRTF and HRTF
l_s = density of particle (kg/m^3):

l_l= density of liquid (kg/m^3): and
d = diameter of particle (m)

4.2.4.2 Primary Treatment

4.2.4.2.1 Septic Tanks

Please refer to the section 4.2.2 on the design of septic tank as a primary treatment

4.2.4.2.2 Settler/Clarifier/Sedimentation Tank

The main purpose of a settler is to facilitate sedimentation by reducing the velocity and turbulence of the wastewater stream. Settlers are circular or rectangular tanks that are typically designed for a hydraulic retention time of 1.5-2.5 h. Less time is needed if the BOD level should not be too low for the next biological step. The tank should be designed to ensure satisfactory performance at peak flow. In order to prevent eddy currents and short-circuiting, as well as to retain scum inside the basin, a good inlet and outlet construction with an efficient distribution and collection system (baffles, weirs or T-shaped pipes) is important.

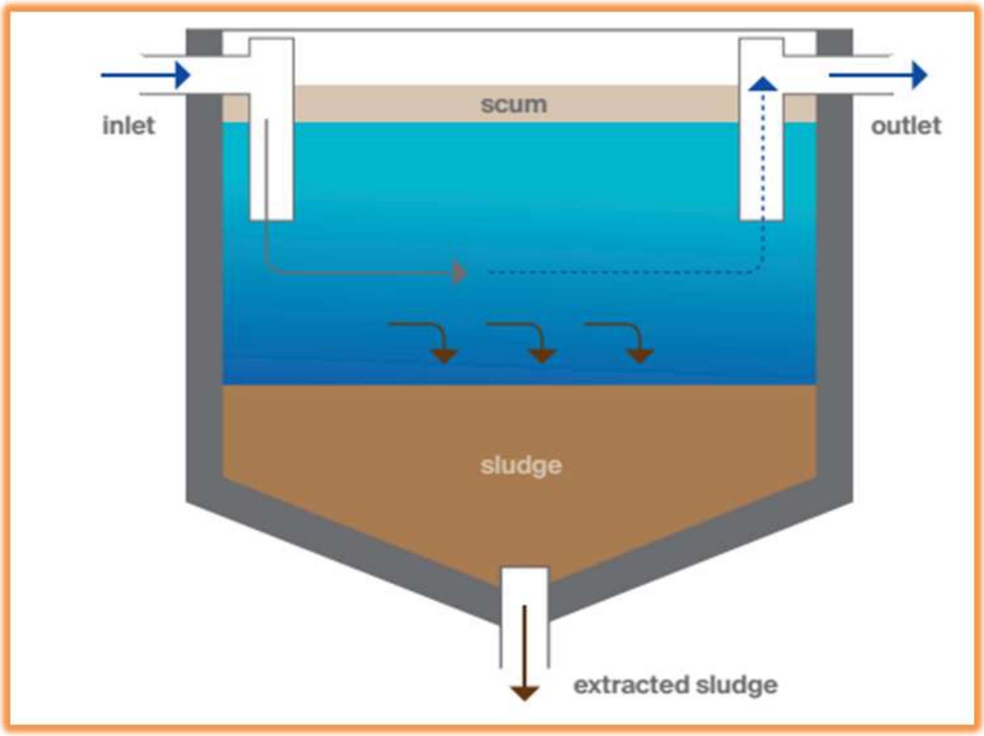


Figure 4.10: Section view of a settler
(adapted from Tilley, et al., 2014)

Depending on the design, desludging can be done using a hand pump, airlift, vacuum pump, or by gravity using a bottom outlet. Large primary clarifiers are often equipped with mechanical collectors that continually scrape the settled solids towards a sludge hopper in the base of the tank, from where it is pumped to sludge treatment facilities. A sufficiently sloped tank bottom facilitates sludge removal. Scum removal can also be done either manually or by a collection mechanism.

Figure 4.10 presents a typical cross section through the settler. The design considerations and procedures should follow like those ones for grit chamber

4.2.4.2.3 Biogas Settler

A DEWATS Biogas Settler is usually a gas- and watertight dome-shaped sub-surface structure. It is typically constructed with bricks or cement mortar/plaster. The primary function of the settler is to separate the incoming wastewater into liquid and solid components, allowing the digestion of organic solids.

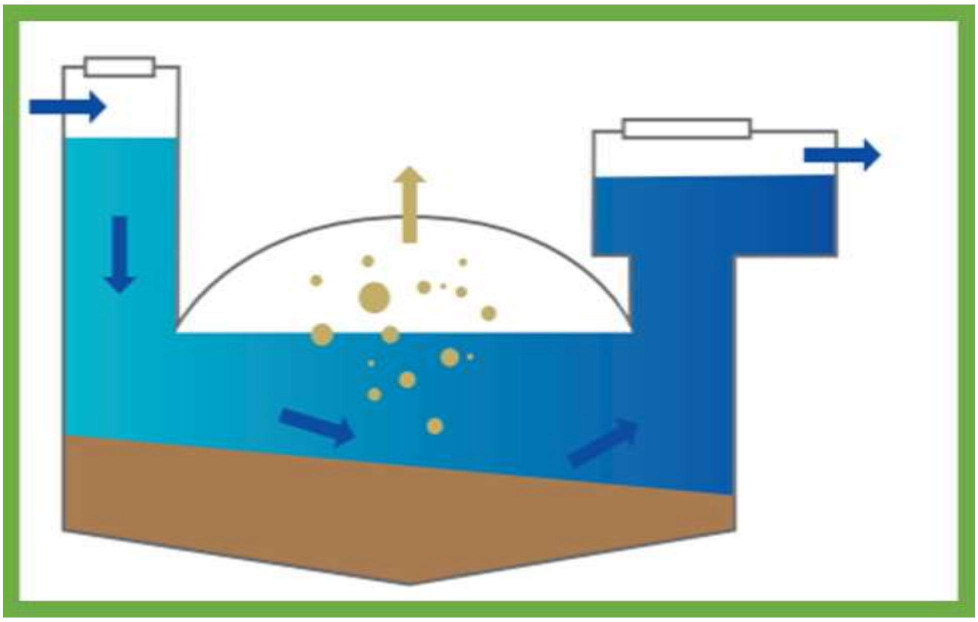


Figure 4.11: Biogas Settler (Tilley et al., 2014)

The microbial digestion process occurs under anaerobic conditions (without oxygen) and results in the generation of biogas. The by-products of this treatment process are (a) a digested slurry (digestate) that is stabilised and thus can be used as a soil amendment and (b) biogas that can be used for energy. Biogas is a mix of methane, carbon dioxide and other trace gases which can be converted to heat, electricity or light. Figure 4.11 presents a schematic sketch of a biogas settler.

4.2.4.2.4 Design principles of a biogas settler

Biogas settlers are similar in construction and design as fixed-dome or floating drum biogas plants. However, in opposition to biogas reactors, biogas settlers are designed for the retention of biomass and are thus typical high-rate biogas reactors. Other high-rate biogas plants are Anaerobic Baffled Reactors (ABRs); Anaerobic Filters (AF); and Up-flow Anaerobic Sludge Blanket Reactor (UASB). High-rate biogas reactors are characterized by a mixed flow regime: the liquid (e.g. flushing, anal cleansing or greywater) flows through (continuous flow), while the sludge (e.g. faeces, paper etc.) is retained (batch) and treated over a long time until it is removed and used as fertilizer. Thus, biogas settlers are characterized by relatively short hydraulic retention times (HRT) for the liquor and high sludge retention times (SRT) for the solid fraction (organics and inorganic). The settled sludge is transformed into biogas by anaerobic digestion). Gas bubbles to the top of the reactor are collected for use.

At this point it is important to note that much of the design details will be refined through greater experience and empirical data. The following instructions are only a suggestion of design techniques brought together from a number of published articles.

Digester (including gas holder)
The size of the digester largely depends on the amount of waste to be added. Digester shape should enable a minimum surface area: volume ratio to be reached to reduce heat loss and construction costs. Hemispherical digesters with a conical floor often work best (CAMARTEC design of …….). To calculate the required digester volume (VD) use Equation 4.32:

VD = VB x HRT ………………………………………………………………………………….(4.32)
Where:
VD = Volume of the digester (m^3)
VB = Volume of biomass added per day (m^3/day)
HRT = Retention time required (days)

The amount of human waste produced varies from person to person but generally lies in the region of 0.2-0.4kg (solid) and 1-1.3kg (liquid) per day (depending on diet, health, etc). If other waste (animal dung, organic food waste, etc) is added then this should also be taken into account. Clearly it is almost impossible to control the rates of waste input (especially in the case of latrines) so some discretion and common sense should be used when dealing with the numbers.

The volume of the gas holder VG depends on the relative rates of gas production and consumption. To calculate the daily gas production (G) either Equation 4.33 or Equation 4.34 can be used (it may be good to use both and take an average since data for Gyvaries greatly):

G = VB x Gy(moist mass) ……………………………………………………………………(4.33)
G = LSU x Gy(species) ……………………………………………………………………….(4.34)

Where:
G = Daily gas production rate (m^3/day)
MB = Mass of biomass added per day (kg/day)
LSU = Number of livestock units (number)
Gy(moist mass) = Gas yield per kg of excreta per day (m^3/kg/day)
Gy(species) =Gas yield per kg of livestock unit per day(m^3/kg/day)

The gas holder must be designed to cover the peak consumption rate (VG1) (if the primary reason for construction is based on biogas demand) and the longest period of zero consumption (VG2) (if the primary reason for construction is safe excreta treatment/disposal). The larger of these 2 volumes should be used to specify the gas holder volume with an additional 20% safety margin. The following equations should be used to calculate $VG_1$and VG_2

VG1 = Gcmax x Tcmax ....(4.35)
VG2 = G x Tczero ....(4.36)

Where:
VG_1= Gas holder volume 1 (m^3)
VG_2= Gas holder volume 2 (m3)
Gcmax= Maximum rate of gas consumption (m^3/day)
Tcmax= Maximum time of gas consumption (days)
G = Daily gas production rate (m3/day)
Tczero= Maximum time of zero gas consumption (days)

According to experience the ratio of digester volume: gas holder volume (i.e. VD:VG) usually lies in the range 3-10:1. Since the hemispherical design of the fixed-dome generator combines the digester volume (VD) with the gas holder volume (V_G) the total volume of the hemispherical dome (VH) can then be calculated:

VH = V_D V_G..(4.37)

The final part of the calculation is to determine the required radius (r) of the hemisphere. This can be done using Equation 4.38:

$r=((3V_h)/(2\ ))^{1/3}$ ....(4.38)

NB: Any calculated value should be taken as only an estimate – there are so many variables in the inputs (Hydraulic Retention Time (HRT), waste addition rate, gas consumption rate, climate, etc) that the value should be used with caution.

Displacement tank –There are a number of different options for the design (size, shape, etc) of the displacement tank. The tank could be a fully buried hemispherical structure (much the same as but smaller than the digester), a simple column tank or a large open drying bed. Available materials, workforce skills level, safety and space are factors which need assessing before choosing a design. The primary functions of the displacement tank are to provide a buffer for the pressure of the gas inside the digester and to allow digested slurry to be removed. The main parameters of the design are volume of the tank and height of the slurry overflow. The required size largely depends on the fluctuation in gas volume/pressure over time (e.g. 1 day). If the gas volume fluctuates a large amount then a large tank is required to prevent too much slurry being lost through the overflow during times of high gas pressure (which will cause a low pressure of the next batch/collection of gas). If the gas volume hardly fluctuates at all (e.g. rates of gas production/use are the same) then in theory a displacement tank may not be needed at all (which is unlikely).

According to experience the volume of the displacement tank should be roughly equal to that of the gas holder. However, there is a lot of variance between designs since the shape of the displacement tank can vary so much (from a simple self-contained tank with an overflow to a large drying bed structure).

4.2.4.3 Secondary Treatment

4.2.4.3.1 Anaerobic Baffled Reactor

An ABR (Figure 4.12) is a modified septic tank with a series of baffles under which the wastewater is forced to flow. The increased contact time with the active biomass (sludge) results in improved treatment. The up-flow chambers provide enhanced removal and digestion of organic matter. BOD can be reduced by 70% to 90%, which is far superior to its removal in a conventional septic tank. The main function of an ABR is the conversion of particulate matter into soluble BOD, as well as a certain percentage of soluble BOD into Methane (CH_4). This is achieved by de-coupling HRT from Solids Retention Time

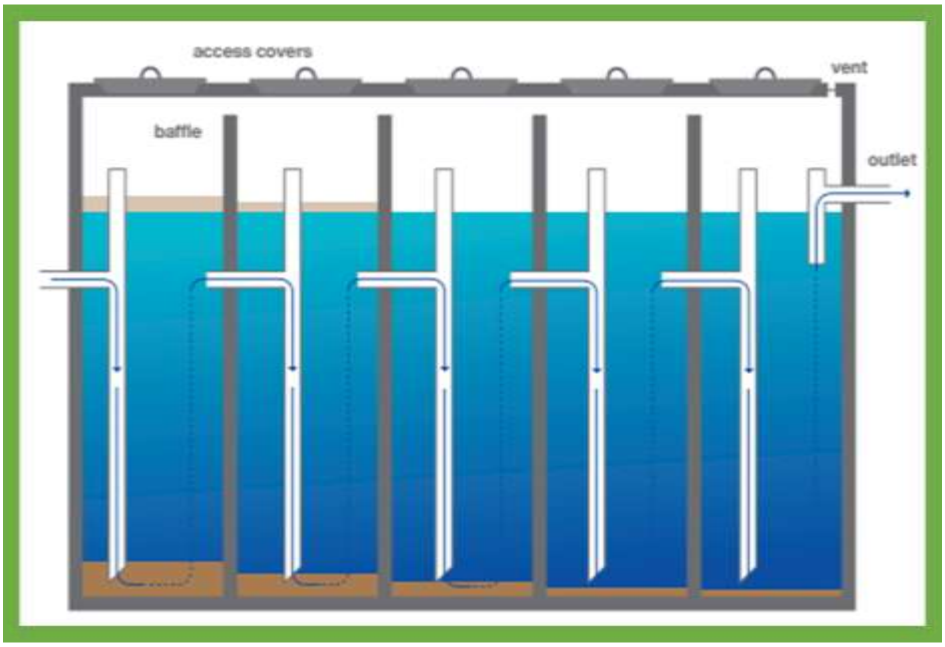


Figure 4.12: Section view of an ABR (Tilley, 2014)

(A) Design Parameters

The classic ABR process design consists of a number of equally dimensioned compartments. For a specific wastewater flow, the design is fully specified by fixing the following six independent parameters:

(i) Design hydraulic retention time,
(ii) Number of compartments,
(iii) Peak up-flow velocity,
(iv) Compartment width to length ratio,
(v) Reactor depth and
(vi) Compartment up-flow to down-flow area ratio.

The civil design of the reactor interior also requires values for hanging baffle clearance, headspace height, baffle construction and inlet and outlet construction. All other internal features such as length and width individual compartments dimensions are dependent on the first six parameters.

Fixing the design

Table 4.5 presents recommended ranges for values for the design parameters for an ABR treating domestic wastewater. Although the limits of operation have not been fully tested, these values have been selected based on experiences gained through 5 years of observing laboratory- and pilot-scale reactors in operation.

Table 4.5: Recommended ranges for parameters in the design of an ABR

Parameter	Symbol	Unit	Recommended parameter range or equation
Flow rate	F	m^3/d	-
Hydraulic Retention Time	HRT	h	20 to 60 But 40 to 60 during start-up
Reactor working volume	V_W	m^3	F x HRT/24
Peak up-flow velocity	V_p	m/h	0.54
Design up-flow velocity	V_d	m/h	$V_p/1.8=0.30$
Number of compartments	N	-	4 to 6
Hanging baffle clearance	d_h	m	0.15 to 0.20
Compartment up-flow area	A_U	m^2	$F/(V_D x 24)$
Up-flow to down-flow area ratio	$R_{U:D}$	m^2/m^2	2 to 3
Compartment width to length ratio	$C_{W:L}$	m/m	3 to 4
Total compartment area	A_c	m^2	$A_U x (1+R_{U:D})/R_{U:D}$
Reactor depth	r_D	m	1 to 3 (The reactor depth will largely be governed by the cost of excavation)
Reactor width	r_W	m	$\sqrt{\frac{V_W.C_{W:L}}{N.r_D}}$
Reactor length	r_L	m	$NxrW/C_{W:L}$

Source: Foxon *et al.*, 2004

4.2.4.3.2 Anaerobic Filter (AF)

An AF (Figure 4.13) is a fixed-bed reactor in an anaerobic contact process, with one or more filtration chambers in series. As wastewater flows through the filter, particles are trapped and organic matter is degraded by the active biomass that is attached to the surface of the filter material. Filter material can be gravel, rocks or specially formed plastic pellets. To reduce costs, locally available material shall be used. For example, in Tanzania, coconut husks can be used or in Indonesia volcanic rock might be a good solution. Good filter material provides $90m^2$ to $300m^2$ surface area per m^3. With this technology, TSS and BOD removal can be as high as 90%, but typically ranges between 50% and 80%. Nitrogen removal is limited and normally does not exceed 15% in terms of total nitrogen (TN).

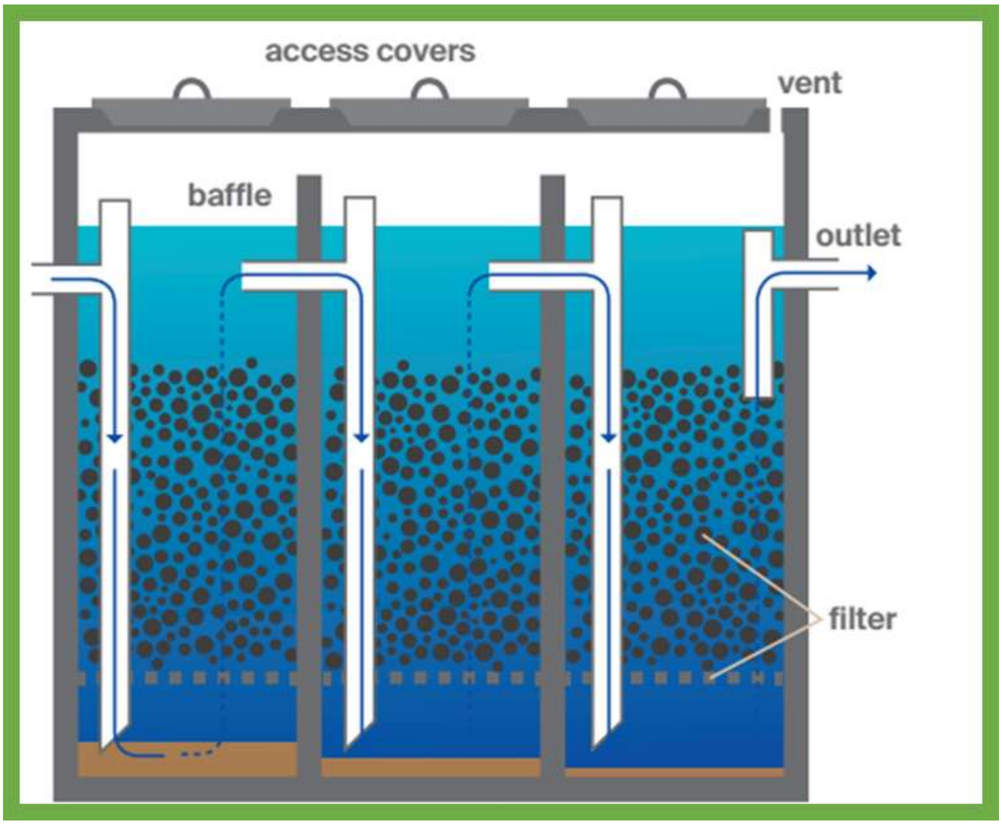


Figure 4.13: Section of an Anaerobic Filter (AF)
Source:Tilley, 2014)

(B) Design criteria and procedures

The use of anaerobic filters for the treatment of domestic waste water has been intended mainly for the polishing of effluents from septic tanks and UASB reactors. In this configuration, the main design consideration is described below:

(C) Hydraulic detention time

The hydraulic detention time refers to the average time of residence of the liquid inside the filter, calculated by the following expression:

$$t = \frac{V}{Q} \quad \text{..........(4.39)}$$

Where:
t=hydraulic detention time (hour)
V=volume of the anaerobic filter (m^3)
Q= average influent flowrate (m^3/d)

(D) Temperature

Anaerobic filters can be satisfactorily operated at temperatures ranging from 25 to 38^{o}C. Usually, the degradation of complex wastewater, whose first stage of the fermentation process is hydrolysis, requires temperature higher than 25^{o}C. Otherwise, hydrolysis may become the limiting stage of the process.

(E) Packing medium height

Based on the Brazilian experience it is recommended for most applications that the packed bed height should be between 0.8 and 3.0 m**.** The upper height limit of the packed bed is more appropriate for reactors with lower risk of bed obstruction, which depends mostly on the flow direction, on the type packing material and on the influent concentrations. Amore usual value should amount to approximately 1.5m.

(F) Hydraulic Loading rate

The hydraulic loading rate to the volume of wastewater applied daily per unit area of the filter packing medium, can be calculated by equation 3.60.

HLR=Q/A ....(4.40)

Where:

HLR=hydraulic loading rate ($m^3/m^2.d$)

Q= average influent flowrate (m^3/d)

A= surface area of the packing medium (m^2)

(G) Organic Loading rate

The volumetric organic loading rate refers to the load of organic matter applied daily per unit volume of the filter or packing medium, as calculated by Equation 4.41

$$L_v = \frac{QxS_o}{v} \quad \text{..(4.41)}$$

Where:

Lv= volumetric organic loading rate (kgBOD/m^3.d or kgCOD.m^3.d)

Q= average influent flowrate (m^3/d)

So=influent BOD or COD concentration (kgBOD/m^3 or kgCOD/m^3)

V= total volume of the filter or volume occupied by the packing medium (m^3)

Effluent distribution and collection systems

A very important aspect of the design of Anaerobic Filters concerns the detailing of the wastewater inlet and outlet devices, since the efficiency of the treatment system depends substantially on the good distribution of the flow on the packing bed, and this distribution is subject to the correct calculation of the inlet and outlet devices.

In case of Upflow Anaerobic Filters, once flow distribution tube has been used for every 2.0 to 4.0m^2 of filter bottom area. Figure 4.14 shows the wastewater distribution device, through perforated tubes, and the effluent collection launder.

	
Sewage distribution device at the bottom of an AF	**Effluent collection launder on the top of AF**

Figure 4.14: (a) and (b) Sewage distribution device at the bottom of an AF and Effluent collection launder on the top of AF

(H) Efficiency of Anaerobic Filters

The expected efficiencies for anaerobic filters can be estimated from the performance relationship presented in equation 3.63

$$E = 100x(1 - 0.87xt^{-0.50}) \quad \text{(4.42)}$$

Where:

E =efficiency of the anaerobic filter (%)

t = hydraulic detention time (hours)

0.87 = empirical constant (coefficient of the system)

0.50 = empirical constant (coefficient of the packing medium)

In situations in which the anaerobic filters are used as post-treatment units for effluents from septic tanks and UASB reactors, the BOD removal efficiency expected for the system as a whole varies from 75% to 85%.

From the efficiency expected for the system, the COD or BOD concentration in the final effluent can be estimated as follows

$$C_{effl} = S_o - \frac{E x S_o}{100} \qquad (4.43)$$

Where:
C_{effl}= effluent total BOD or COD concentration (mg/L)
So = influent total BOD or COD concentration (mg/L)
E = BOD or COD removal efficiency (%)

Table 4.6: Design Criteria for anaerobic filters applied to the post-treatment of effluents from anaerobic reactors

Design Criteria/parameter	Range of values, as a function of the flowrate		
	for $Q_{average}$	for $Q_{daily-maximum}$	for $Q_{hourly-maximum}$
Packing medium	stone	stone	stone
Packing bed height (m)	0.8 to 3.0	0.8 to 3.0	0.8 to 3.0
Hydraulic detention time (hour)	5 to 10	4 to 8	3 to 6
Surface loading rate (m^3/m^2.d)	6 to 10	8 to 12	10 to 15
Organic loading rate (kgBOD.m^3.d)	0.15 to 0.50	0.15 to 0.50	0.15 to 0.50
Organic loading in the packed bed (kgBOD/m^3.d	0.25 to 0.75	0.25 to 0.75	0.25 to 0.75

Source: Tilley et al., 2014

4.2.4.3.3 Up-flow Anaerobic Sludge Blanket Reactor (UASB)

The UASB (Figure 4.15) is a single-tank process. Wastewater enters the reactor from the bottom and flows upward. A suspended sludge blanket filters and treats the wastewater as the wastewater flow through it.

A UASB is not appropriate for small or rural communities without a constant water supply or electricity. The technology is relatively simple to design and build, but developing the granulated sludge may take several months. The UASB has the potential to produce higher quality effluent than septic tanks and a Biogas Settler and can do so in a smaller reactor volume. Although it is a well-established process for large-scale industrial wastewater treatment and high organic loading rates up to 10 kg BOD/m^3/d, its application to domestic sewage is still relatively new.

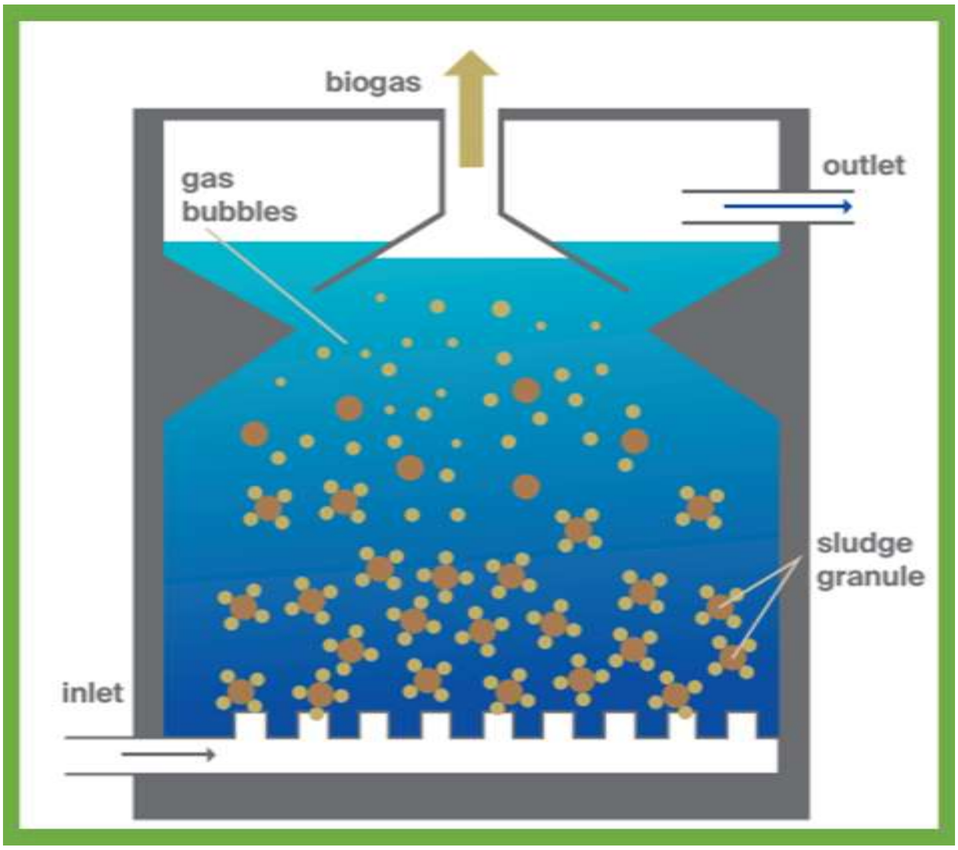


Figure 4.15: Section view of an Upflow Anaerobic Sludge Blanket reactor
Source: (Tilley et al., 2014)

Design procedures for UASB Reactor

Determine nominal volume of UASB Reactor

$$V_n = \left[\frac{Q \times S_o}{L_{org}}\right] \text{..(4.44)}$$

Where:
V_n is nominal/effective liquid volume of reactor (m^3), Q is influent flow rate (m^3/d),
S_o is influent COD
L_{org} is organic loading rate (kgCOD/m^3.d) and f_R is a fraction of COD removed in a particular reactor.

$L_{org} = \left[\frac{S_o}{\tau}\right]$, where: τ=hydraulic retention time (days)…………………(4.45)

Determine total liquid volume of UASB Reactor

Consider factor of effectiveness = 0.80

$V_L = \left[\frac{V_n}{E}\right]$………………………………………………………(4.46)

Where:

V_L, total liquid volume of the reactor (m^3), E the effectiveness factor (unit less).

Find out an area of UASB Reactor

$A = \left[\frac{V_L m^3}{H_L m}\right]$, ………………………………………………………………………(4.47)

H_L is water level height,

Calculate Diameter of UASB Reactor

$A = \left[\frac{\pi D^2}{4}\right]$…………………………………………………………………………(4.48)

Find out the new volume of reactor

$A = \left[\frac{V_L m^3}{H_L m}\right]$, H_L is water level height,

$V_L = A \times H$ ……………………………………………………………………… (4.49)

Determine the total Liquid Height of the UASB Reactor

The gas collection volume is additional to the reactor of volume and adds an additional height of 2.5m-3m; hence the total height of the reactor is given by:

$H_T = [H_L + H_G]$ ……………………………………………………………………… (4.50)

Where H_T is the total height of the reactor (m) and H_G is the total height of the gas collection and storage (m).

Calculate hydraulic Retention Time (HRT) in UASB Reactor

The hydraulic retention time, τ is given by:

$\tau = \left[\frac{V_L}{Q}\right]$…………………………………………………………………………………(4.51)

The Sludge Retention Time (SRT) in UASB

The value of SRT can be estimated by assuming that all the wasted biological solids are in the effluent. The design approach is to assume that the given effluent VSS concentration consists of biomass (Metcalf and Eddy, 2004).

The Solid Wasted

$$QX_e = P_{x,VSS} = \frac{Solids\ wasted}{day} \qquad (4.52)$$

Where:

Q flow rate m^3/d and X_e is particulate COD.

$$P_{x,VSS} = \left[\left(\frac{QY(S_0 - S}{1 + (k_d SRT)}\right) + \left(\frac{f_d k_d YQ(S_0 - S)}{1 + (k_d SRT)}\right) + (QnbVSS)\right] \qquad (4.53)$$

Where:

y = Biomass yield M of cell formed per M of substrate consumed

k_d = Endogenous decay coefficient

f_d = Fraction of cell mass remaining as cell debris = VSS cell debris/g VSS biomass decay

nbVSS = non-biodegradable volatile suspended solids, mg/L

So = Initial substrate concentration (COD) at time t = 0, mg/L,

S = Substrate concentration (COD) at time t, mg/L

SRT = Sludge Retention Time, d

μ_m = Maximum growth rate

Data for Determining SRT

Effluent COD (S) at COD removal efficiency

$$S = ((1 - \xi)S_o) \qquad (4.54)$$

Where:

S is effluent COD, mg/l, S_o is influent COD, mg/l and ξ is reactor COD removal efficiency

The effluent VSS concentration

Consider that efficiency (ε) 50% of influent VSS is degraded (Metcalf and Eddy, 2004).

$$VSS = (YS_0) \qquad (4.55)$$

The particulate COD (Xe)

Consider at efficiency (ε) 50% of COD degraded (Metcalf and Eddy, 2004).

$$pCOD = \ (\varepsilon(S_0 - S)) \qquad (4.56)$$

Solve for SRT

$QX_e = P_{x,VSS} = 150\frac{m^3}{d} \times 296\frac{g}{m^3}$ = 44,400g/d

$$P_{x,VSS} = \left[\left(\frac{QY(S_0 - S}{1 + (k_d SRT)}\right) + \left(\frac{f_d k_d YQ(S_o - S)}{1 + (k_d SRT)}\right) + (QnbVSS)\right] \quad (4.57)$$

Effluent COD due to calculated SRT

$$S = \left[\left(\frac{K_s(1+(k_d)SRT)}{SRT(Yk - k_d) - 1}\right)\right],\ k = \left[\left(\frac{\mu_m}{Y}\right)\right] \quad (4.58)$$

Check for adequacy of computed SRT

$$Fraction of COD_{infl} in effluent = \left[\left(\frac{S}{S_0}\right)\right] 100\% \quad (4.59)$$

If the fraction of COD_{infl} in effluent is higher than 15% remaining in effluent, then the process SRT is inadequate.

Check for concentration in biomass zone of the UASB reactor (XTSS)

The recommended range is 50-100g/L at bottom of reactor and 5-40g/L in a more diffuse zone at the top of the UASB sludge blanket (Metcalf and Eddy, 2004).

$$X_{TSS} = \left[\left(\frac{QX_e SRT}{V_L}\right)\right] \quad (4.60)$$

Effluents BOD_5

Since, the ratio of COD to BOD_5 for industrial wastewater can be approximated as 2-2.5 (COD = 2.25BOD_5), hence;

$$COD = (2.25 BOD_5)$$

$$BOD_{5\ effl} = \left(\frac{COD_{eff}}{2.25}\right) \quad (4.61)$$

4.2.4.4 Tertiary Treatment

4.2.4.4.1 Horizontal Subsurface Flow Constructed Wetland

A Horizontal Sub-Surface Flow Constructed Wetland (HSSF-CW) (Figure 4.16) also known as Planted Gravel Filter is a large gravel and sand-filled basin that is planted with wetland vegetation. As wastewater flows horizontally through the basin, the filter material filters out particles and micro-organisms degrade the organics. The filter media acts simultaneously as a filter for removing solids, a fixed surface upon which bacteria can attach, and a base for the vegetation. Although facultative and anaerobic bacteria degrade most organics, the vegetation transfers a small amount of oxygen to the root zone so that aerobic bacteria can colonise the area and degrade organics there as well. The plant roots play an important role in maintaining the permeability of the filter. This technology has been intensively researched at University of Dar es Salaam since early

1990. (IWSA Conference Proceedings, Vol I and II, 2002). Several systems have been installed in Tanzania (Figure 4.17).

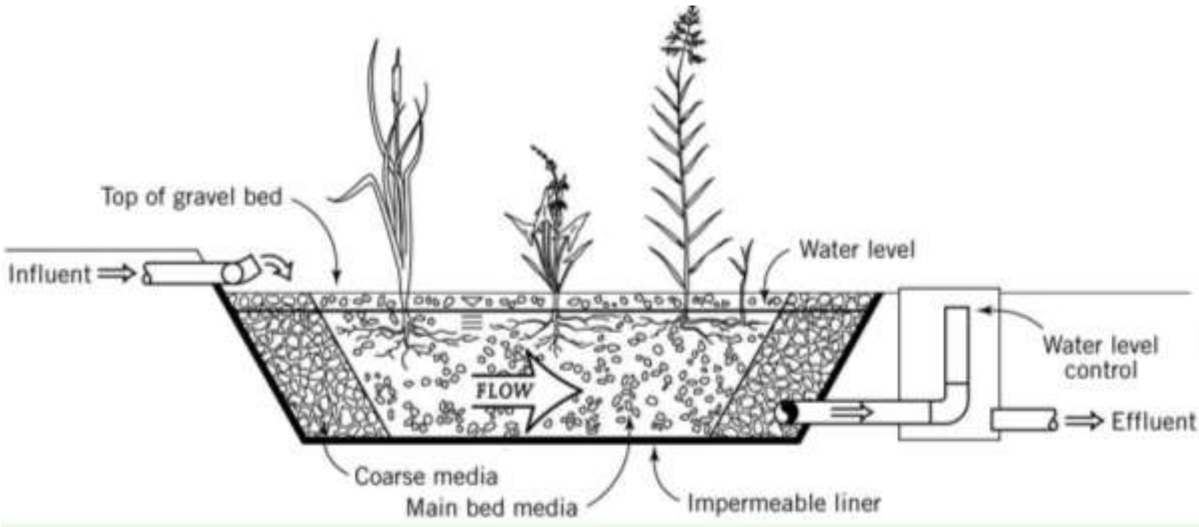


Figure 4.16: Section view of a Horizontal Sub-surface Flow Constructed Wetland
Source: (Kadlec, 2008).

(b) Design Considerations for the Constructed Wetlands

The design work should be based on good engineering practice and standards. It must also integrate the local practices and economics. The design should consider the following technical and environmental factors:

- The design has to adopt first order, plug flow reaction kinetics for BOD removals and check for compliance with Total Suspended Solids (TSS), Nitrate (NO_3), Ammonia (NH_3), Total Phosphorus (P) and Faecal Coliforms.
- Considering sensitivity of the location (bordered by a natural wetland), the old Reed approach(Reference) which considers temperature based pollutant removal rate constant and provide for maximum surface area enough to carry the treatment, was used.
- The systems should be designed to meet the local and international discharge limits as the treatment goals.

(c) Design procedures for CW

Determine Design Population

Estimation and projection of the population to be served is the first step in estimation of the quantity of the waste water to be treated.

Determine Water Demand

Literature suggests that 80% of the water consumption will be waste water, therefore it is important to measure or estimate the water demand based on the per capita water demand and the population to be served

Figure 4.17: Constructed Wetland polishing the wastewater from aeration unit of Mwanza City Abattoir wastewater treatment system.

Source: Courtesy of Prof. Karoli N. Njau, NM-AIST, Arusha.

Determine Design Flow and wastewater characteristics

The wastewater flows form the basis on which the CW and sewer sizes are determined. The wastewater flow rates are based on the existing water consumption and are derived by multiplying the water consumption rates by a factor less than unity, referred to as the ‘reduction factor’. The reduction factor takes into account the water that is supplied to the users but does not eventually end up as wastewater in the treatment system. A reduction factor of 20% is applied to estimate the quantity of wastewater generated.

Determine the design Equations

Constructed Wetland can be designed to remove
Biological Oxygen Demand (BOD) Total Suspended Solids (TSS) Nitrogen (ammonia, nitrates) Phosphorus (P) Pathogens and Heavy metals

BOD removal

$$C_e = C_o e^{(-k_T t)} \qquad \text{(4.62)}$$

$$K_T = k_{20}\theta^{T-20} \qquad \text{(4.63)}$$

Where

Ce = the effluent parameter mg/l

Co= influent parameter mg/l

K= First order removal rate constant (d^{-1})

t = is Residence time ranges from 4-15 days for wetland.

T = Temperature (^oC)

k_{20} is a parameter specific.

The Table 4.7 presents k_{20} values for different parameters

Table 4.7: Values of k_{20} for different parameters

Parameter	BOD (removal)	Nitrification (NH_4 – removal)	Denitrification (NO_3 – removal)	Pathogen (removal)
For sub-surface flow wetlands				
$K_{20}(day^{-1})$	1.104	K_{NH}	1.00	2.6
θ	1.06	1.048	1.15	1.19

Source: Cooper et al., 1996

Determine Surface area of a constructed wetland

Assumptions

Porosity=0.33 for coarse aggregate (adopted)

Total depth=1m

Effective depth (h) =0.6m

Freeboard=0.4m

$$A_s = Q\frac{(\ln C_o - \ln C_e)}{K_T h \varepsilon} \qquad (4.64)$$

$$HLR = \frac{Q}{A_s} \qquad (4.65)$$

Where

HLR Hydraulic Loading Rate should not exceed 5 cm/day.

However Tanzanian experience reveals that hydraulic loading of up to 20cm/day provides sufficient wastewater treatment.

NOTE: If the HRL does not fall within the specified limit, a new area is calculated by substituting the limit HRL.

Total Suspended Solids removal

$$C_e = C_o(0.1058 + 0.0011HLR) \qquad (4.66)$$

TP removal

$$C_e = C_o e^{\left(\frac{-K_p}{HLR}\right)} \qquad (4.67)$$

K_p = First order phosphorous removal rate constant = 2.73 cm/day

Pathogen Removal

$$N_e = \frac{N_o}{(1+\tau K_T)^n} \qquad (4.68)$$

Check organic loading rate

$$OLR = \frac{Q(C_o - C_e)}{A_s} \qquad (4.69)$$

The BOD loading for HSSF-CW should not exceed 133 kg/ha. day (Metcalf and Eddy, 1991).

(d) Layout and Configurations

Inlet Zone

The CW is designed to receive wastewater from a source through an inlet pipe/sewer. At an inlet zone it comprises of the pipes and the inspection chambers

Macrophyte and Substrate Zone

This zone includes substrates (clean and graded granitic aggregates 1.3-1.9 cm) well packed, 65cm thick), plants (diverse), a water column, invertebrate and vertebrates, and an aerobic and anaerobic microbial population. The water flow is maintained at 50cm above the bed surface. Within the water column, the stems and roots of wetland plants significantly provide the surface area for the attachment of microbial population. This zone therefore, is designed to provide the substrate with high hydraulic conductivity; to provide surface for the growth of Biofilm to aid in the removal of fine particles by sedimentation or filtration; to provide suitable support for the development of extensive root and rhizome system for the emergent plants.

Outlet Zone

It encompasses the following main components:

- (i) An outlet pipe to collect effluent water and control the depth of the water without creating dead zones in the wetlands.
- (ii) Boulder stones (50 – 100 cm diameter size) to ensure for even collection of treated water across the full width of the CW
- (iii) Wash out pipe to cater for flushing purposes during blockage and other functional problems
- (iv) An outlet chamber to provide access for sampling and flow monitoring
- (v) A sewer line to the disposal area.

(a) Geometrical and Hydraulic Data

The CW unit is commonly built as a trapezoidal structure. However systems with vertical walls have also been designed and constructed and built in Tanzania especially when using bricks and cement blocks for the walls. The overall depth is 1.05m whereby substrate level is 0.65m and free board is 0.4m. The depth of water in the constructed wetland will be maintained at 0.6m from the bottom of the bed.

Table 4.8: Geometrical, Hydraulic, Structural and Functional features for the CW

Parameter	Provisions	Remarks
Number of CW units	unit	Can be divided into more than one cells
Design flow	m^3	Normally it is projected to meet future demand
Shape and configuration	Trapezoidal or Rectangular	
Effective Length	m	Aspect Ratio 1:3 (W:L)
Effective Width	m	
Substrate depth	0.65m	
Effective (water) depth	0.5	
Free Board	0.4m	
Hydraulic retention time	2-5 days	
Substrate materials	Clean, graded, hard and high quality granitic aggregates 1.3-1.9 cm	Should be sourced from local environment
Wetland plants	Diverse CW i.e. Reeds (*phragmites mauritianus*); *Cyperus papyrus Cyperus involucratus,Typha domingensis, (Cattails spp.).*	Locally available plants are preferred
Plant density	3 plants/m^2	Plant cuttings or nursery pre-planted for large wetland

Source: Modified from (Kadlec and Wallace, 2009)

4.2.4.5 Treatment of Sludge from DEWATS

After desludging a Biogas Settler or Anaerobic Baffled Reactor (ABR), the sludge should be treated in drying beds where pathogens are killed off through exposure to oxygen and UV-radiation. In addition, dewatering (or "thickening") of sludge is an important treatment objective, as sludge contains a high proportion of liquid, and the reduction in this volume will simplify and greatly reduce the costs of subsequent treatment steps. Environmental and public health treatment objectives are achieved through pathogen reduction, stabilisation of organic matter and nutrients, and the safe end use or disposal of treatment end-products.

4.2.4.5.1 Unplanted Sludge Drying Beds (USDB)

An unplanted drying bed (USDB) is a simple, permeable bed that, when loaded with sludge, collects percolated leachate and allows the sludge to dry by evaporation. Approximately 50% to 80% of the sludge volume drains off as liquid or evaporates. Figure 4.18 presents section view of an Unplanted Sludge Drying Bed.

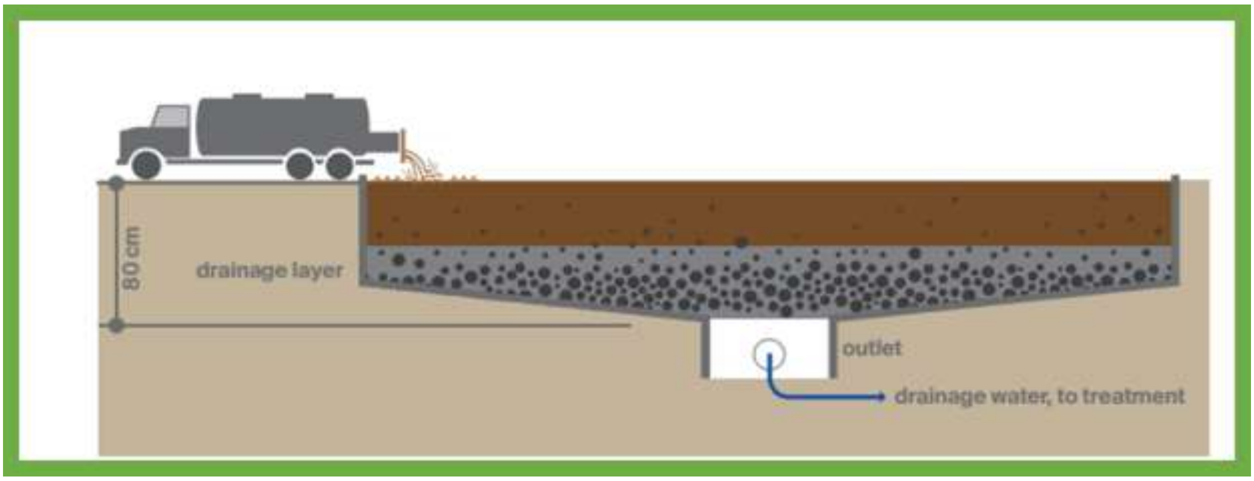


Figure 4.18: Section view of an Unplanted Sludge Drying Bed (Tilley, et al., 2014)

Design Considerations

The requirements for the design of the sludge drying bed are; volume of the sludge, climate, temperature and location. The bottom layer shall be of uniform gravel, and layer of clean sand lay over. Under-drains lay over the gravel layer for drainage of percolated liquid through these layers (Chatterjee, 1996).

The bottom of the drying bed is lined with perforated pipes to drain away the leachate that percolates through the bed. On top of the pipes are layers of gravel and sand that support the sludge and allow the liquid to infiltrate and collect in the pipe. These layers should not be too thick (maximum 20 cm), or the sludge will not dry effectively. The final moisture content after 10 to 15 days of drying should be approximately 60%. When the sludge is dried, it must be separated from the sand layer and transported for further treatment, end-use or final disposal. The leachate that is collected in the drainage pipes must also be treated properly, depending on where it is to be reused or disposed off.

Design procedures and steps for unplanted sludge drying beds

The design procedures for unplanted sludge drying beds is the same as for similar unit for FSM. The user of this manual is instructed to consult that section 3.2.7.2

4.2.4.5.2 Planted Sludge Drying Beds

The design procedures for the Planted Sludge Drying Bed see section 3.2.7.3.

4.3 Simplified Sewerage System (Condominial System)

Simplified sewerage is an off-site sanitation technology that removes all wastewater from the household environment. Conceptually, it is the same as conventional sewerage, but with conscious efforts made to eliminate unnecessarily conservative design features and to match design standards to the local situation. Simplified sewerage, also known as condominial system, is an important sanitation option in peri-urban areas of developing countries, especially as it is often the only technically feasible solution in the high-density areas. It is a sanitation technology widely known and used in Latin America. However, it is much less well known and applied in Africa and Asia and particularly Tanzania.

4.3.1 Key features of a simplified sewerage system

Key features of the condominium system include the following:

(a) **Layout**: *in-block* system (Figure 4.19), rather than – as with conventional sewerage – an in-road system. The key feature of an in-block system is that sewers are routed in private land, through either back or front yards. This in-block or back-yard system of simplified sewerage is often termed condominium sewerage in recognition of the fact that tertiary sewers are located in private or semi-private space within the boundaries of the `condominium'.

(b) **Depth and diameter** : simplified sewers are laid at shallow depths, often with covers of 400 mm or less. The minimum allowable sewer diameter is 100 mm, rather than the 150 mm or more that is normally required for conventional sewerage. The relatively shallow depth allows small access chambers to be used rather than large expensive manholes.

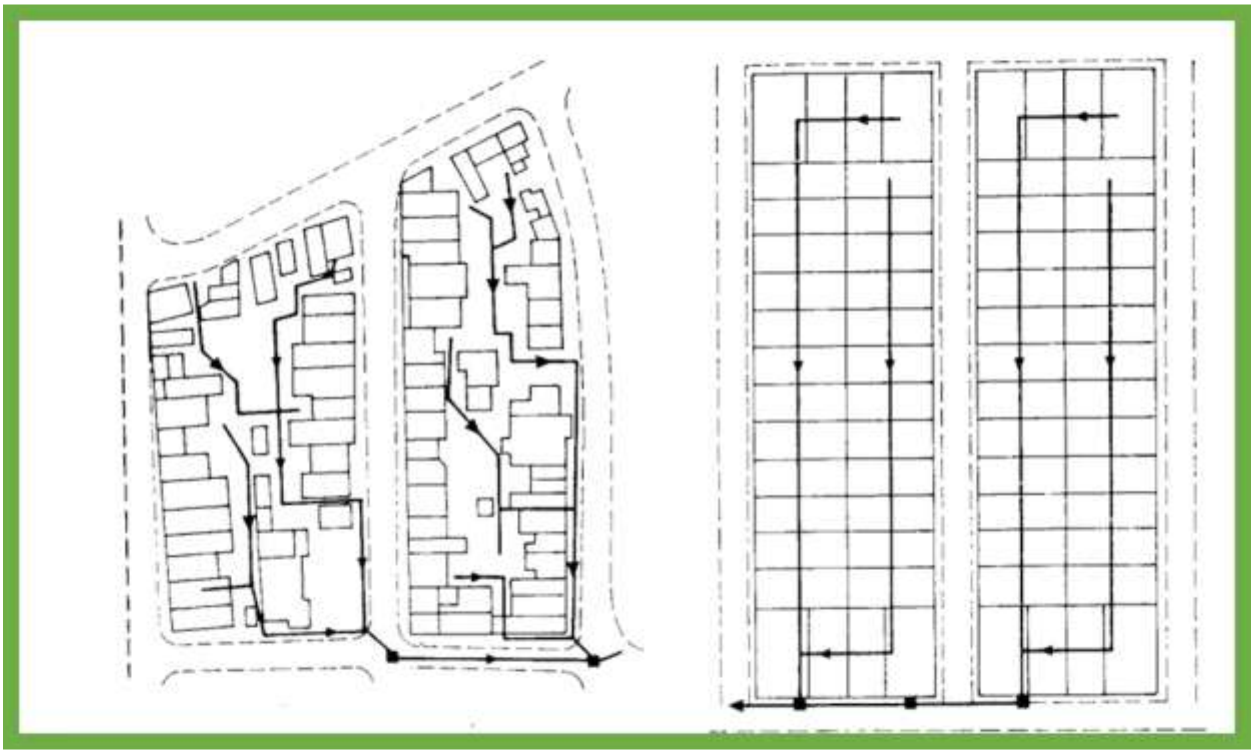


Figure 4.19: Layouts of in-block simplified (condominial) sewerage for unplanned and planned peri-urban housing areas (Sinnatamby, 1983).

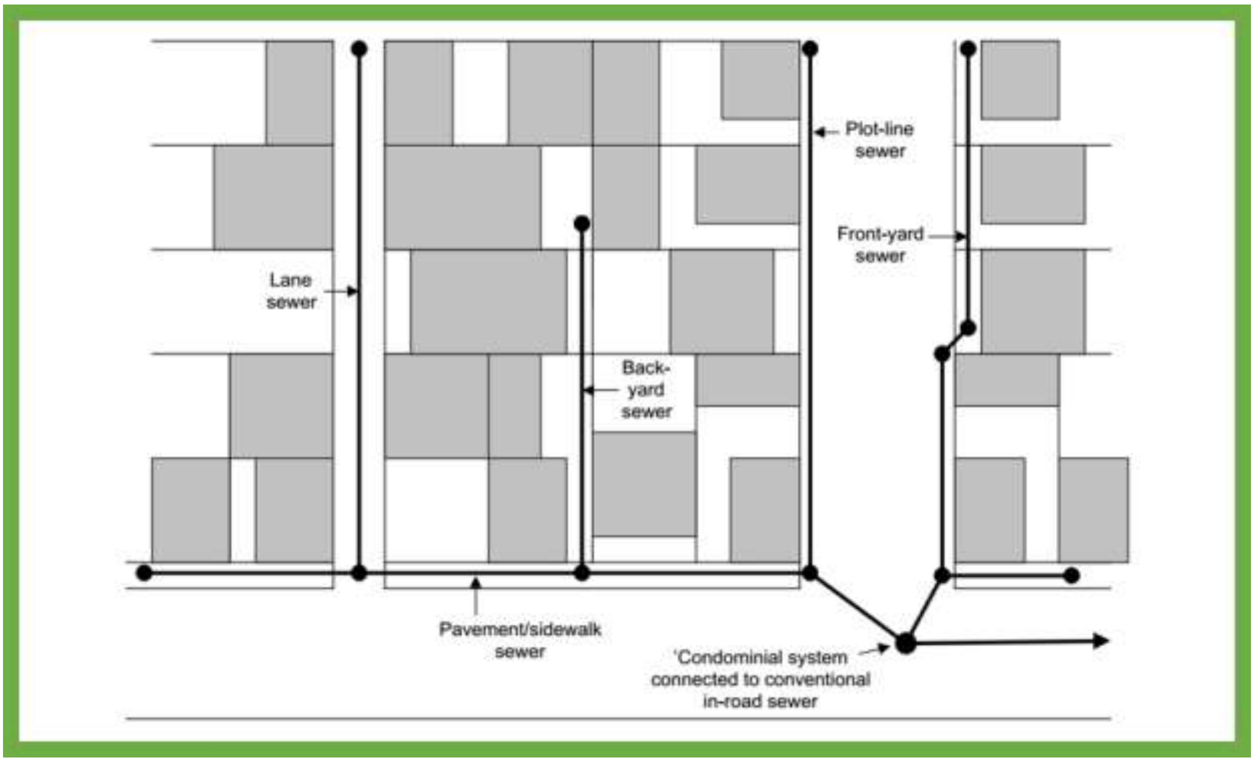


Figure 4.20: Alternative routes for simplified sewers

4.3.2 The planning for simplified sewerage system

The section is subdivided into two parts: Part one is concerned with the initial assessment of sanitation options. The assessment of technical options is explained and the issues relating to the management options for simplified sewerage are explored. Part two sets out the sewerage planning process, from the decision to adopt simplified sewerage system to the development of the overall sewerage layout. It explains what information is needed for the planning process and explores the factors that will influence the area to be included in a sewerage scheme. This leads in to the development of a draft sewerage plan.

(A) Initial assessment of Sanitation Options

Two basic questions should be asked at the beginning of the planning process. These are:

- What sanitation options are feasible in the local situation? and
- Assuming that simplified sewerage is feasible, what arrangements are possible for managing the construction and subsequent operation and maintenance of the local condominial systems?

(i) Technical options

This is the stage at which the decision to use simplified sewerage will be made. Simplified sewerage should only be considered where a reliable water supply is or can be made available on or near each plot so that total water use is at least 60 litres per person per day.

Other factors to be considered are

- population density,
- the arrangements for effluent disposal and the preferences of the local people; for evaluating on-site sanitations options the plot size,
- the infiltration capacity of the soil and
- the potential for groundwater pollution should also be considered (see Franceys *et al.*, 1992; Cotton and Saywell, 1998; and GHK Research and Training, 2000).

(ii) Management options

It is important to consider the possible management options for any proposed sanitation system from the very beginning of the planning process. In general, the more small-scale and local a sanitation system is the better the prospects for local management. So, it would appear that on-plot sanitation systems such as pit latrines and pour-flush toilets discharging to leach pits can be managed by individual householders, while city-wide sewage disposal systems must be managed at the municipal level.

(B) Planning for simplified sewerage systems

This section describes the steps to be taken during planning and adoption of a simplified sewerage system. These steps can be summarized as follows:

- (i) Collect existing information, focusing particularly on maps and plans of the area to be sewered and adjacent areas,
- (ii) Determine the area to be included in the sewerage plan, based on topography, the location of existing sewers and the limits of existing and future development,
- (iii) Develop a draft sewerage plan, showing the routes of the main collector sewers and the approximate areas of the various condominial systems,
- (iv) Undertake additional surveys as required to allow sewer routes and the areas of condominial systems to be confirmed, so that detailed design can be carried out, and
- (v) Finalise the overall sewerage plan and plot the sewer routes at an appropriate scale or scales

(C) Collection of existing information

The first task in the planning process is to collect all available information on the area to be sewered. In particular, existing topographical maps and any maps showing the routes of any existing drains and sewers should be collected, as these are needed to define the area to be sewered and to determine the overall sewer layout. This information may be available on a number of maps and plans; if this is the case, as much information as possible should be transferred to one base plan. Information on the existing management arrangements and responsibilities also needs to be collected.

(D) Areas to be included

The next task is to decide the area to be included in the scheme. There are two possible situations. The first is that the design is for an exclusively local system, which can be connected to a local treatment facility or an existing collector sewer. The second is that there is a need to look at the sewerage needs of a wider area, including both local condominial sewers and public collector sewers. In the first case, the decision on the area to be included in the scheme is relatively straightforward.

(E) Development of a draft sewerage plan

It should now be possible to develop a draft sewerage plan. The first step is to decide the routes of the main public collector sewers and then consider how local condominial systems can be joined to them. In general, public collector sewers should be designed to include flows from all parts of the drainage area that are or are likely to be sewered.

Failure to do this will mean that the sewers will be undersized, if not immediately then certainly in the future.

(F) Physical and social surveys

If accurate survey information is not available, detailed physical and social surveys are generally required. Each is briefly considered in turn below.

(G) Physical surveys

Physical surveys are required in order to determine sewer routes and levels. If existing plans exist, it may be possible to use them, at least for preliminary design.

(H) Social surveys

Simple social surveys should be used to provide information on household sizes and incomes, existing sanitation and water supply facilities, attitudes to sanitation and user preferences. Questionnaire surveys are useful for providing quantitative information. Semi-structured interviews and focused group discussions are more likely to provide information on attitudes and preferences.

(I) Final sewer routes

Once good survey information has been obtained, it can be recorded on suitable plans and detailed design of the system can commence. Minor changes to the routes of collector sewers may be required as a result of improved survey information. More substantive changes may be necessary in condominial systems as a result of the findings of both the physical and social surveys.

4.3.3 Detailed Design Considerations and Procedures

The design procedures for simplified sewerage system follows that on conveyance section under DEWAT section 4.2.

4.4 Centralized Wastewater Treatment

4.4.1 Overview

(A) A centralized system uses a series of sewer pipes, tunnels, and pumps to collect wastewater and to transport it to a central treatment plant. The sewer pipes can be combined (including storm water runoff) or separate. The sewer is the pipe or conduit for carrying sewage. It is generally closed and flow takes place under gravity (Atmospheric Pressure). There are two types of sewers for central systems, central system and simplified sewerage system also known as condominial system.

4.4.2 Design Consideration for Central Sewer

4.4.2.1 Sewage flow

It is flow derived from sewage that is the raw water from these industries and houses, Also it means it has direct relation with the amount of water consumed. Generally 80 to 90 % of the water consumption is taken as sewage or waste water flow.

(B) Variation in sewage flow

Like water supply, sewage flow varies from time to time. Since sewers must be able to accommodate Maximum Rate of Flow, the variation in the sewage flow must be studied. Generally Herman Formula is used to estimate the ratio of Maximum to Average Flow.

$$\text{Peak Factor} = M = \frac{Q_{max}}{Q_{avg}} = 1 + \left(\frac{14}{4+\sqrt{P}}\right) \quad \text{..........................(4.70)}$$

P is population in thousands.
Design considers the following relationship for sewer design:

Table 4.9: Relationship between average sewage flow and peak factor

Average Sewage Flow (m^3 /day)	Peak Factor
≤ 2500	4.0
2500 – 5000	3.4
5000 – 10000	3.1
10000 – 25000	2.7
25000 – 50000	2.5
50000 – 100000	2.3
100000 – 250000	2.15
250000 – 500000	2.08
> 500000	2.0

Source: Tchobanoglous et al, 2003

(C) Infiltration

It is amount of water that enters into the sewers through poor joints, cracked pipes, walls and covers of manholes.

- It is non-existent during dry weather but increases during rainy season.
- During wet season, the following infiltration rates for the design of sewer system are recommended.

Table 4.10: The relationship between sewer diameter and infiltration

Sewer Diameter	Infiltration
225 mm to 600 mm	5 % of Avg. Sewage Flow
> 600 mm	10 % of Avg. Sewage Flow

Source: Tchobanoglous et al, 2003

4.4.2.2 Design Procedures

(A) Design Flow

Calculate the average sewage flow on the basis of water consumption and the population at the end of the design period. i.e at the full development of the area. Then the design flow for sanitary sewer and partially combined sewers can by calculated by using the following formulae.

For sanitary sewer

Q_{design}= Peak sewage flow + infiltration

For partially combined sewer (WASA Criteria)

Q_{design} = 2xPeak sewage flow + infiltration

(B) Determine flow velocity using design Equation

Manning's Equation is used for sewers flowing under gravity

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{..........(4.71)}$$

Where:

V = Velocity of flow in m/sec

R = Hydraulic mean depth (A/P) = D/4 when pipe is flowing full or half full

S = Slope of the sewer

n = Coefficient of roughness for pipes

(C) Flow velocity selection

(i) Minimum (Self Cleansing) Velocity

Sewage should flow at all times with sufficient velocity to prevent the settlement of solid matter in the sewer. Self Cleansing Velocity is the minimum velocity that ensures non settlement of suspended matter in the sewer.

The following minimum velocities are generally employed

- Sanitary sewer = 0.6 m/sec
- Storm sewer = 1.0 m/sec
- Partially combined sewer = 0.7 m/sec

(ii) Maximum velocity

The maximum velocities in the sewer pipes should not exceed more than 2.4 m/sec. This max velocity in the sewer should not exceed this limit of 2.4 m/sec. It is to avoid the excessive sewer abrasion and also to avoid steep slopes.

(D) Minimum Sewer Size

225mm is taken as the minimum sewer size. The reason being that, the choking does not take place even with the bigger size particles, which are usually thrown into the sewer through manholes.

(E) Minimum Cover of Sewer

1m is taken as the minimum cover over the sewers to avoid damage from live loads coming on the sewer.

(F) Spacing of Manhole

For (Sewer Size) 225mm to 380mm - spacing not more than 100m
For (Sewer Size) 460mm to 760mm spacing not more than 120m
For (Sewer Size) greater than 760mm spacing not more than 150m

(G) Direction of Sewer Line

Sewer should flow, as for as possible the Natural Slope.

(H) Design of the Sewer

(i) Size of Sewer

Use the following relation to find the diameter of sewer

$Q_f = A \times V$ ....(4.72)

(ii) Slope of Sewer

Select the minimum velocity value and use the Manning's formula

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$ ....(4.73)

(iii) Invert Level

The lowest inside level at any cross-section of a sewer pipe is known as invert level at that cross-section.

Invert Level = NGSL/Road Level – Depth of Sewer – Thickness of Sewer – Diameter of Sewer

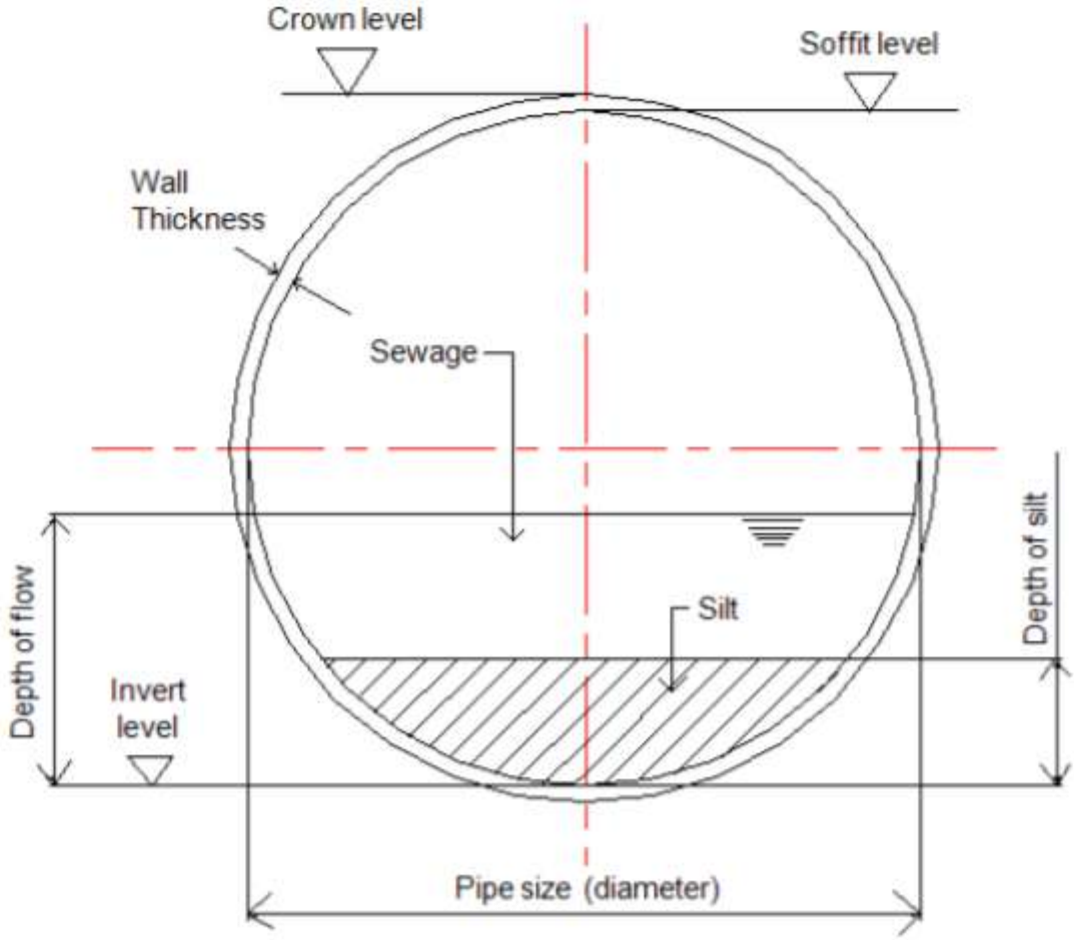


Figure 4.21: Invert level of sewer pipe.
(Adapted from http://www.hkius.org.hk)

(iv) Joints in Sewers

- Bell & Spigot Joint
- Tongue &Groove Joint

4.4.2.3 Typical Steps of Sewage Treatment system

Figure 4.22 provides a flow diagram of a typical wastewater treatment system

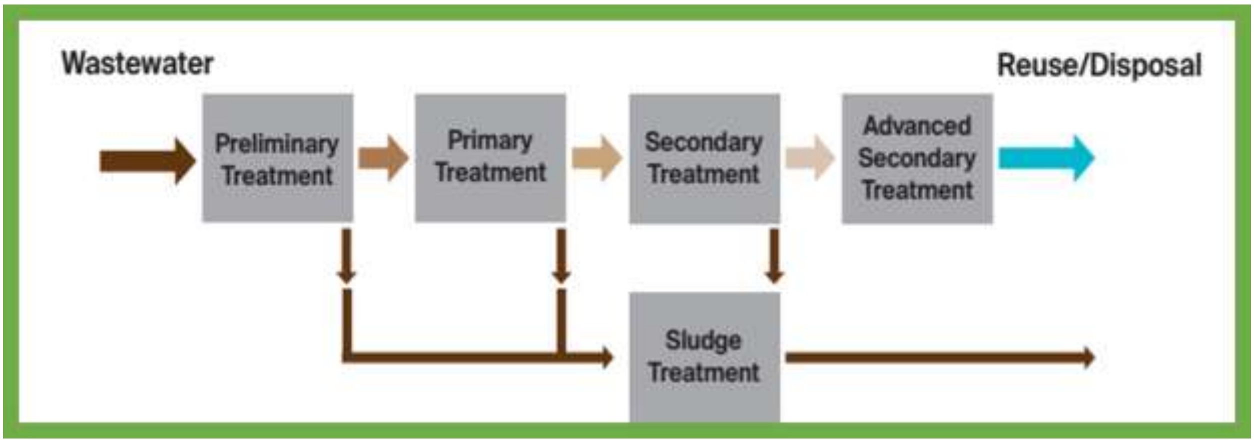


Figure 4.22: Flow diagram of a typical wastewater treatment system

4.4.2.4 Preliminary treatment

Under this category, the units involved include Screens, Grit chamber, FOG trap etc.. The design procedures and considerations follow those presented under DEWAT section 3.3.1. Use of comminattors is not be encouraged in practices.

4.4.2.5 Primary Treatment

Examples of primary treatment units are septic tanks, primary sedimentation, anaerobic ponds. With exception of roughing filters and anaerobic ponds (which is presented under WSP section) the other units follow the design procedures as the ones presented in section 3.3.1 of DEWAT.

4.4.2.6 Secondary treatment

The design procedures and considerations for other systems are provided in secondary treatment system other than the DEWAT. It is only trickling filters, activated sludge systems and waste stabilization ponds which are presented in this section.

4.4.2.6.1 Waste Stabilization Ponds

Waste Stabilization Ponds (WSPs) are large, shallow basins in which raw sewage is treated by entirely natural processes involving both algae and bacteria. They are used extensively for sewage treatment in moderate and tropical climates, and represent one of the most cost-effective, reliable and an easily operated process for the treatment of domestic and industrial wastes. WSP are very effective on the removal of faecal coliform, which is the indicator of pathogenic organism. Sunlight energy is the only requirement for its operation. It requires minimum supervision for its daily operation by cleaning the outlets and inlet works.

(A) Types of Waste Stabilization Ponds and Their Specific Uses

Waste stabilization pond systems comprise a single series of anaerobic, facultative and maturation ponds or several such series in parallel. Figure 4.23 presents the schematic layout of types of WSPs.

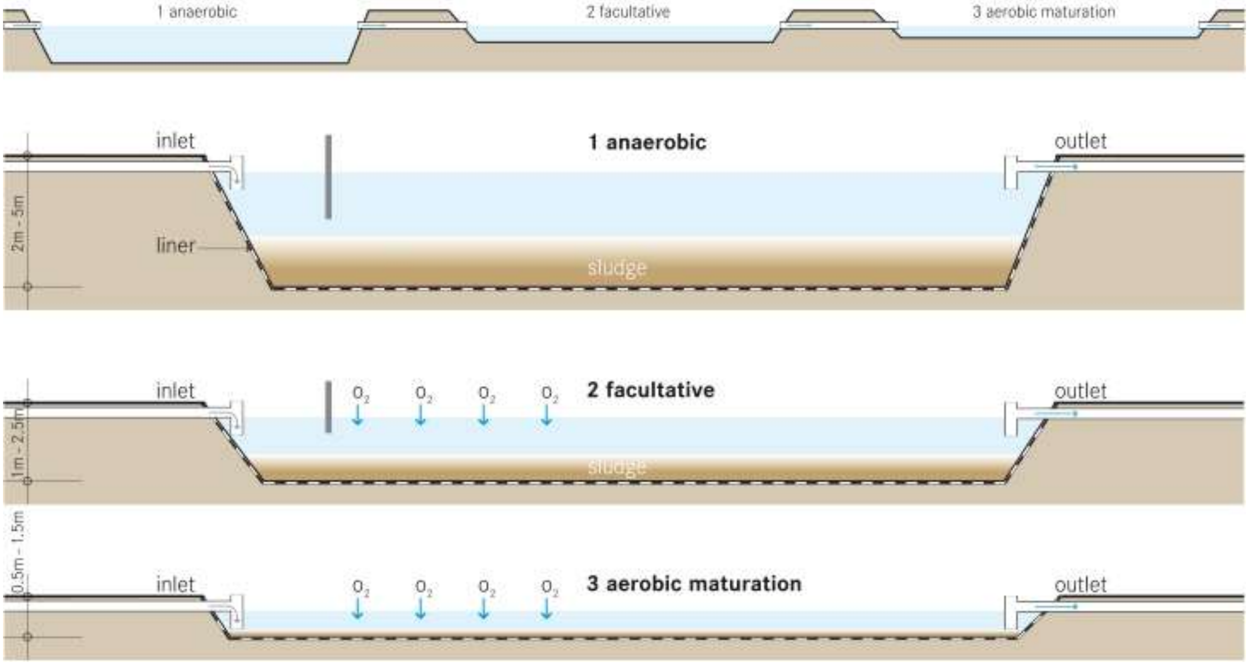


Figure 4.23: Schematic layout of types of WSP

Source:https://commons.wikimedia.org/w/index.php?sort=relevance&search=waste+stabilization+pond

In essence, anaerobic and facultative ponds are designed for BOD removal and maturation ponds for pathogen removal. Some BOD removal occurs in maturation ponds and some pathogen removal in anaerobic and facultative ponds (Mara, 1987). Maturation ponds are required only when the effluent is to be used for unrestricted irrigation and has to comply therefore the WHO guideline of >1000 faecal coliforms per 100 ml.

(B) Estimation of design flow and BOD concentration

There are four most important design parameters for WSP; temperature, net evaporation, flow and BOD. Faecal coliforms and helminth egg numbers are very important if the final effluent is to be used in agriculture or aquaculture.

The mean flow should be carefully estimated, since this has direct effect on the size of the pond and costs of construction. A suitable design is 85% of the in-house water consumption. The BOD may be measured if the wastewater exists based on 24 hour flows. Alternatively, the BOD may be estimated from the following equation;

$$L_i = 1000B/q \qquad (4.74)$$

Where:
L_i is wastewater BOD (mg/l),
B is BOD contribution (g/cap.day),
Q is wastewater flow (L/cap.day). Values of B vary between 30 and 70g per cap per day with rich communities producing more BOD than the poor communities (Campos and von Sperling, 1996).

In medium sized towns, a value of 50g per cap. per day is more suitable (Mara and Pearson, 1987). A typical design figure for an urban area in a developing country would be 40 to 50 grams BOD_5/cap.day (Arthur, 1976). A BOD_5 contribution per capita of 40 grams/day with a wastewater contribution of about 100 litres/cap/day is probably a reasonable initial estimate where there is a household water supply, although flows may be considerably less. The usual range of faecal coliforms in the domestic wastewater is $10^7 - 10^8$ faecal coliforms per 100 ml, and a suitable design value is 5 x 10^7 per 100 ml.

(C) Design of anaerobic ponds

Anaerobic pond is designed based on volumetric loading (λv, g/m^3/d), which is given by:

$$\lambda_v = L_i Q/V_a \qquad (4.75)$$

Where:
L_i is influent BOD (mg/l),
Q is flow rate (m^3/day),
V_a is anaerobic pond volume (m^3).

Meiring et al., 1998 recommends that the loading should be between 100 and 400g/m^3.d in order to maintain anaerobic conditions.

The hydraulic retention time is then calculated using equation.

$t_{an} = V_a / Q$ ....(4.76)

The retention time of less than one day should not be used for anaerobic ponds, if it happens then a retention time of one day should be used and a volume of the pond should be recalculated. Table 4.11 shows the permissible loading to the anaerobic ponds.

Table 4.11: Design value of permissible volumetric BOD loadings on and percentage BOD removal in anaerobic ponds at various temperatures

Temperature (°C)	Volumetric loading ($g/m^3.d$)	BOD removal (%)
< 10	100	40
10 – 20	20T – 100	2T+20
20-25	10T+100	2T+20
>25	350	70

T = temperature °C

(Source: Mara and Pearson (1986) and Mara *et al.,* (1997))

(D) Design of facultative ponds

Facultative ponds may be designed based on kinetic or empirical models. Use is made of kinetic models for design of facultative ponds as follows

Mathematically is as shown in equation

$dL/dt = -k_1 L$ ....(4.77)

Where:

L is the amount of BOD remaining (=organic matter to be oxidized) at time "t" and

k_1 is first order rate constant for BOD removal (day^{-1}).

The rational equation for the design is as shown in equation;

$L_e / L_i = 1/(1 + k_1 t)$ ....(4.78)

Rearranging the equation

$t = (L_i / L_e - 1)(1/k_1)$ ....(4.79)

Where: t is the retention time (days).

The mid-depth area of the pond is calculated using equation:

$$A = Qt/D \qquad \text{.....(4.80)}$$

Where:
Q is the volumetric flow rate (m^3/day),
D is the pond depth (m) and
A is the mid-depth area (m^2).

Substituting "t" from equation 4.79 into equation 4.80 the mid-depth area of the pond will be:

$$A = (Q/Dk_1)(L_i/L_e - 1) \qquad \text{.....(4.81)}$$

The value for k_1 at 20°C was found to be 0.3 day^{-1}, while the value of k_T are calculated using equation 4.82. Note that the rate k_1 is a gross measure of bacterial activity and in common with almost all parameters describing a biological growth process, its value is strongly temperature dependent.

Its variation with temperature is usually described by an Arrhenius equation

$$k_T = k_{20}\theta^{(T-20)} \qquad \text{.....(4.82)}$$

Where: θ is the Arrhenius constant whose value is usually between 1.01 and 1.09. However the typical values of θ for the design of waste stabilization ponds ranges between 1.05 and 1.09. Note that the temperature should be taken as the mean temperature of the coldest month.

Empirical models for design of facultative pond

Although there are several methods available for designing facultative ponds Mara, 1976, recommended that facultative ponds should be designed on the basis of surface loading (with the reasons stated in sections above, λ_s, kg/ha.day) which is given by equation (4.83)

$$\lambda_s = 10L_iQ/A_f \qquad \text{.....(4.83)}$$

Where:
L_i is the concentration of influent sewage (mg/l),
A_f is the facultative pond area, (m^2).

The selection of permissible design value of λ_s is usually based on the temperature.

The earliest relationship between λ_s and temperature was given by McGarry and Pescode (1970), and later on by Mara (1976). The Mara (1976) equation is as shown in equation (4.84)

$$\lambda_s = 20T - 120 \qquad \text{.....(4.84)}$$

However a most appropriate λ_s and temperature relationship was presented by Mara (1987) and is termed as a global design equation;

$$\lambda_s = 350[1.107 - 0.002T]^{(T-20)} \qquad \text{.....(4.85)}$$

Once the surface loading has been selected then the area of the facultative pond is calculated from equation (3.105) and its retention time (θ_f, day) is calculated from equation (4.86)

$$\theta_f = A_f D / Q_m \qquad \text{.....(4.86)}$$

Where;

D is the pond depth (usually 1.5m), Q_m is the mean flow (m^3/day).

The mean flow is the mean of the influent and effluent flows (Q_i and Q_e), the latter being the former less net evaporation and seepage. Thus equation (4.86) becomes

$$\theta_f = \frac{A_f D}{\left[\frac{1}{2}(Q_i + Q_e)\right]} \qquad \text{.....(4.87)}$$

If seepage is negligible, Q_e is given by

$$Q_e = Q_i - 0.001 A_f e \qquad \text{.....(4.88)}$$

Where:

e is net evaporation rate, mm/day. Hence equation (4.88) becomes:

$$\theta_f = \frac{2A_f D}{\left[2Q_i - 0.001 A_f e\right]} \qquad \text{.....(4.89)}$$

A minimum value of retention time of 5 days should be adopted for temperature below 20°C, and 4 days for temperature above 20°C. This is to minimize hydraulic short-circuiting and to give algae sufficient time to multiply (i.e. to prevent algal washout).

(E) Design of Maturation ponds for faecal coliforms removal

The method of Marais (1974) is generally used to design a pond series for faecal coliforms removal assuming first order kinetic model.

$$N_e = \frac{N_i}{(1 + k_T \theta)} \qquad \text{...(4.90)}$$

Where; N_e and N_i is the number of FC per 100 ml in the effluent and influent, k_T is the first order rate constant for FC removal, d^{-1}; and θ is a retention time, (day).
For a series of anaerobic, facultative and maturation ponds, equation (4.90) becomes:

$$N_e = \frac{N_i}{\left[(1 + k_T \theta_a)(1 + k_T \theta_f)(1 + k_T \theta_M)^n\right]} \qquad \text{..(4.91)}$$

Where; the sub-scripts a, f and m refer to the anaerobic, facultative and maturation ponds; and *n* is the number of maturation ponds. It is assumed in equation (3.110) that all the maturation ponds are equally sized, this is the most efficient configuration (Marais, 1974), but may not be topographically possible (in which case the last term of the denominator in equation (4.91) is replaced by; $[(1+k_T\theta_{m1})(1+k_T\theta_{m2})........(1+k_T\theta_{mn})]$).

The value of k_T is highly temperature dependent. Marais (1974) found that:

$k_T = 2.6\ (1.19)^{T-20}$ ....(4.92)

(F) Helminth eggs removal

Helminth eggs are normally removed by sedimentation and the process occurs in anaerobic or primary facultative ponds. If the final effluent is to be used for restricted irrigation, then it is necessary to ensure that it contains no more than one egg per litre. Analysis of eggs removal in the pond has yielded the following relation reported by Ayres et al. (1992).

$$R = 100[1 - 0.14 \exp(-0.38\theta)] \qquad \text{...(4.93)}$$

Where, R is percentage egg removal, θ is a retention time (day). The equation corresponding to lower 95 percent confidence limit of equation (4.93) is

$$R = 100[1 - 0.41 \exp(-0.49\theta + 0.0085\theta^2)] \qquad \text{..(4.94)}$$

Equation (4.83) or Table 4.12 is recommended to be used for the design. See Annex A on how to design a waste stabilization pond for the removal of helminth eggs.

Table 4.12: Design values of helminth egg removal (R%) in anaerobic, facultative or maturation ponds at various hydraulic retention times

θ	R(%)	θ	R	θ	R	
1	74.67	4	93.38	10	99.29	
1.2	76.95	4.2	93.66	10.5	99.39	
1.4	79.01	4.4	93.40	11	99.48	
1.6	80.87	4.6	94.85	12	99.61	
1.8	82.55	4.8	95.25	13	99.7	
2	84.08	5	95.62	14	99.77	
2.2	85.46	5.5	96.42	15	99.82	
2.4	87.72	6.0	97.06	16	99.86	
2.6	87.85	6.5	97.57	17	99.88	
2.8	88.89	7.0	97.99	18	99.90	
3.0	89.82	7.5	98.32	19	99.92	
3.2	90.68	8.0	98.60	20	99.93	
3.4	91.45	8.5	98.82			
3.6	92.16	9.0	99.01			
3.8	92.80	9.5	99.16			

Source: Ayres et al. 1992

(G) Design of WSP for nutrient removal

Design equation for nutrient removal in WSP is based on the equation developed in North America and designers should use these with precaution that it might not accurately predict the performance as expected. The equation for ammoniacal nitrogen (NH_3 + NH^+_4) removal in individual facultative ponds was presented by Pono and Middlebrooks (1982). The equation for the temperatures below 20°C is as follows:

$$C_e = C_i / \{1 + [(A/Q)(0.0038 + 0.000134T)\exp((1.041 + 0.044T)(pH - 6.6))]\} \quad \text{.....................(4.95)}$$

For temperatures above 20°C;

$$C_e = C_i / \{1 + [5.035x10^{-3}(A/Q)\;[\exp(1.540x(pH - 6.6))]]\} \quad \text{...(4.96)}$$

Where; C_e is ammoniacal nitrogen concentration in the pond effluent (mg N/L), C_i is ammoniacal nitrogen concentration in the pond influent, (mg N/L), A is pond area (m^2), and Q is influent flow rate (m^3/day).

The removal of total nitrogen in the individual facultative and maturation ponds was presented by Reed (1995) as follows;

$$C_e = C_i \exp\{-[0.0064(1.039)^{T-20}\;[\theta + 60.6(pH - 6.6)]]\} \quad \text{...(4.97)}$$

Where, Ce and Ci is the total nitrogen concentration in the pond effluent and influent, respectively (mg N/L), T is temperature (^{o}C range 1-28^{o}C) and θ is retention time (days; range 5 to 231 days). The pH values used in the above equations may be estimated as follows

$$pH = 7.3\exp(0.0005A) \quad \text{.....................}(4.98)$$

Where:
A is influent alkalinity, mg $CaCO_3$/L.

4.4.2.6.2 Activated Sludge

Activated Sludge Treatment is a biological wastewater treatment process which speeds up waste decomposition by adding activated sludge into wastewater, and the mixture is aerated and agitated for a specified amount of time thereby allowing the activated sludge to settle out by sedimentation and is disposed of (wasted) or reused (returned to the Aeration Tank - see Figure 4.24).

The activated sludge process has the advantage of producing a high quality effluent for a reasonable operating and maintenance costs. The activated sludge process uses micro-organisms to feed on organic contaminants in wastewater, producing a high-quality effluent. The basic principle behind all activated sludge processes is that as micro-organisms grow, they form particles that clump together. These particles (flocs) are allowed to settle to the bottom of the tank, leaving a relatively clear liquid free of organic materials and suspended solids.

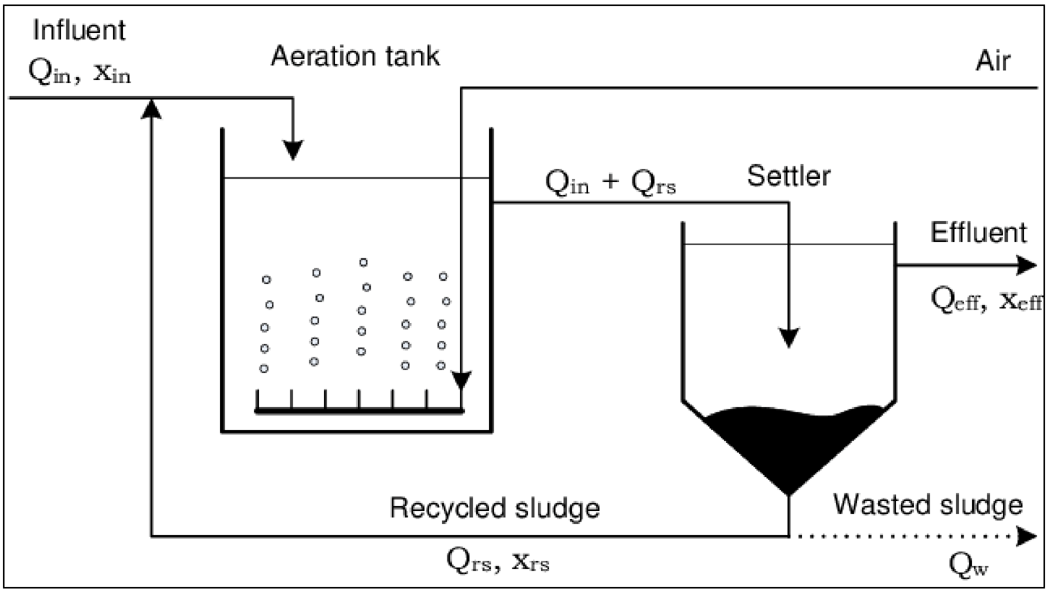


Figure 4.24: Schematic diagram of a typical activated sludge process (Chai, 2006).

Design Considerations

The items for consideration in the design of activated sludge plant are;

i. aeration tank capacity and dimensions,
ii. aeration facilities,
iii. secondary sludge settling and
iv. recycle and excess sludge wasting or re-use.

4.4.2.6.3 Aeration Tank

The **volume of Aeration Tank** is calculated for the selected value of q_c by assuming a suitable value of Mixed liquor Suspended Solids – (MLSS) concentration, X;

$$VX = \frac{YQq_c(S_o - S)}{1 + k_d q_c} \qquad \text{.......(4.99)}$$

Alternately, the tank capacity may be designed from;

$$F/_M = QS_o/_{XV} \qquad \text{.......(4.100)}$$

Design steps for Aeration Tank

(i) Choose a suitable value of $\mathbf{q}_c$ ***(or F/M)***

$\mathbf{q}_c$ depends on

a) the expected weather temperature of mixed liquor,
b) the type of reactor,
c) expected settling characteristics of the sludge and
d) the nitrification required.

The choice generally lies between 5 days in warmer climates to 10 days in temperate ones where nitrification is desired along with good BOD removal, and complete mixing systems are employed.

(ii) Select two interrelated parameters **HRT, t and MLSS concentration**

It is seen that economy in reactor volume can be achieved by assuming a large value of X. However, it is seldom taken to be more than 5000 g/m^3. For typical domestic sewage, the MLSS value of 2000-3000 mg/l if conventional plug flow type aeration system is provided, or 3000-5000 mg/l for completely mixed types.

Considerations which govern the upper limit are:

- initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in small reactor volume,
- increased solids loading on secondary clarifier which may necessitate a larger surface area,
- design criteria for the tank and minimum HRT for the aeration tank.

The **length** of the tank depends upon the type of activated sludge plant. Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. The **width** and **depth** of the aeration tank depends on the type of aeration equipment employed. The depth controls the aeration efficiency and usually ranges from 3 to 4.5 m. The width controls the mixing and is usually kept between 5 to 10 m. **Width-depth ratio** should be adjusted to be between 1.2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m.

Oxygen Requirements

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of the micro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required (g/d)} = \frac{Q(S_o - S)}{f} - 1.42 Q_w X_r \quad \text{.........} (4.101)$$

Where:

f = ratio of BOD_5 to ultimate BOD and 1.42 = oxygen demand of biomass (g/g).

The formula does not allow for nitrification but allows only for carbonaceous BOD removal.

Aeration Facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the wastewater against a specific level of dissolved oxygen in the wastewater.

Secondary Settling

Secondary settling tanks, which receive the biologically treated flow undergo zone or compression settling. **Zone settling** occurs beyond a certain concentration when the particles are close enough together that inter particulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as a single layer or "blanket" of sludge. The rate at which a sludge blanket settles can be determined by timing its position in a settling column test.

Compression settling may occur at the bottom of a tank if particles are in such a concentration as to be in physical contact with one another. The weight of particles is partly supported by the lower layers of particles, leading to progressively greater compression with depth and thickening of sludge. From the settling column test, the limiting solids flux required to reach any desired underflow concentration can be estimated, from which the required tank area can be computed.

The solids load on the clarifier is estimated in terms of (Q+R)X, while the overflow rate or surface loading is estimated in terms of flow Q only (not Q+R) since the quantity R is withdrawn from the bottom and does not contribute to the overflow from the tank. The secondary settling tank is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. Beyond an MLSS concentration of 2000 mg/l the clarifier design is often controlled by the solids loading rate rather than the overflow rate. Recommended design values for treating domestic sewage in final clarifiers and mechanical thickeners (which also fall in this category of compression settling) are given in Eddy and Metacalf, 2004).

Sludge Recycle

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settleability and thickening in the secondary sedimentation tank.

$$\frac{Q_r}{Q} = \frac{X}{X_r - X} \qquad \text{......(4.102)}$$

Where:

Q_r = Sludge recirculation rate, m^3/d

The sludge settleability is determined by sludge volume index (SVI) defined as volume occupied in mL by one gram of solids in the mixed liquor after settling for 30 min. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_r = 10^6/SVI$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids. The X_r value may not be taken more than 10,000 g/m^3 unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value.

Excess Sludge Wasting

The sludge in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter is preferred as the sludge concentration is fairly steady in that case. The excess sludge generated under steady state operation may be estimated by;

$$q_c = \frac{VX}{Q_W X_r} \quad \text{or} \quad Q_W X_r = YQ\,(S_O - S) - k_d\,XV \qquad \text{......(4.103)}$$

Where;

F:M–Food to microbe ratio

Floc –clumps of bacteria

Flocculation –agitating wastewater to induce the small, suspended particles to bunch together into heavier particles (floc) and settle out.

Loading -a quantity of material added to the process at one time

MLSS –mixed-liquor suspended solids

MLVSS –volatile mixed-liquor suspended solids

Mixed liquor –activated sludge mixed with raw wastewater Package plant –pre-manufactured treatment facility small communities or individual properties use to treat wastewater

SRT –solids retention time

Sludge –the solids that settle out during the process

Supernatant –the liquid that is removed from settled sludge. It commonly refers to the liquid between the sludge on the bottom and the scum on the surface.

TSS –total suspended solids

Wasting –removing excess microorganism's small package plants being used today

4.4.2.6.4 Trickling Filters

Trickling filter is an **attached growth process** i.e. process in which micro-organisms responsible for treatment are attached to an inert packing material. Packing material used in attached growth processes include rock, gravel, slag, sand, redwood, and a wide range of plastics and other synthetic materials Figure 4.25 presents high rate trickling filter which has been adapted from https://edurev.in

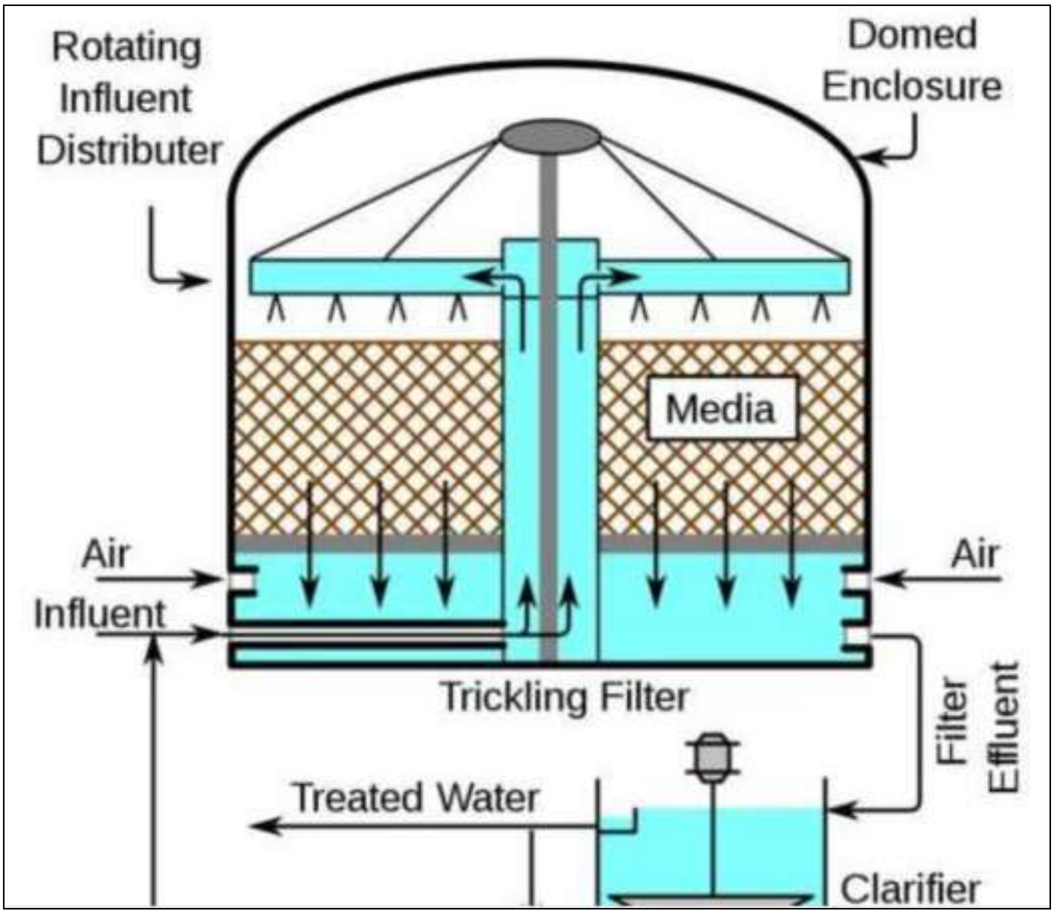


Figure 4.25: High Rate Trickling Filter

Types of Filters

Trickling filters are classified as high rate or low rate, based on the organic and hydraulic loading applied to the unit. Table 4.13 presents comparison of low and high rate filters.

Table 4.13: Comparison of LRTF and HRTF

S.No.	Design Feature	Low Rate Filter	High rate Filter
1	Hydraulic loading, $m^3/m^2.d$	1 – 4	10 - 40
2	Organic loading, kg BOD / $m^3.d$	0.08 – 0.32	0.32 – 1.0
3	Depth, m.	1.8 – 3.0	0.9 – 2.5
4	Recirculation ratio	0	0.5 – 3.0 (domestic wastewater) up to 8 for strong industrial wastewater

Source: Tchobanoglous et al, 2003

Key features

(i) The hydraulic loading rate is the total flow including recirculation applied on unit area of the filter in a day, while the organic loading rate is the 5 day 20°C BOD, excluding the BOD of the recirculant, applied per unit volume in a day.

(ii) Recirculation is generally not adopted in low rate filters.

(iii) A well operated low rate trickling filter in combination with secondary settling tank may remove 75 to 90% BOD and produce highly nitrified effluent. It is suitable for treatment of low to medium strength domestic wastewaters.

(iv) The high rate trickling filter, single stage or two stage are recommended for medium to relatively high strength domestic and industrial wastewater. The BOD removal efficiency is around 75 to 90% but the effluent is only partially nitrified.

(v) Single stage unit consists of a primary settling tank, filter, secondary settling tank and facilities for recirculation of the effluent. Two stage filters consist of two filters in series with a primary settling tank, an intermediate settling tank which may be omitted in certain cases and a final settling tank.

Process Design

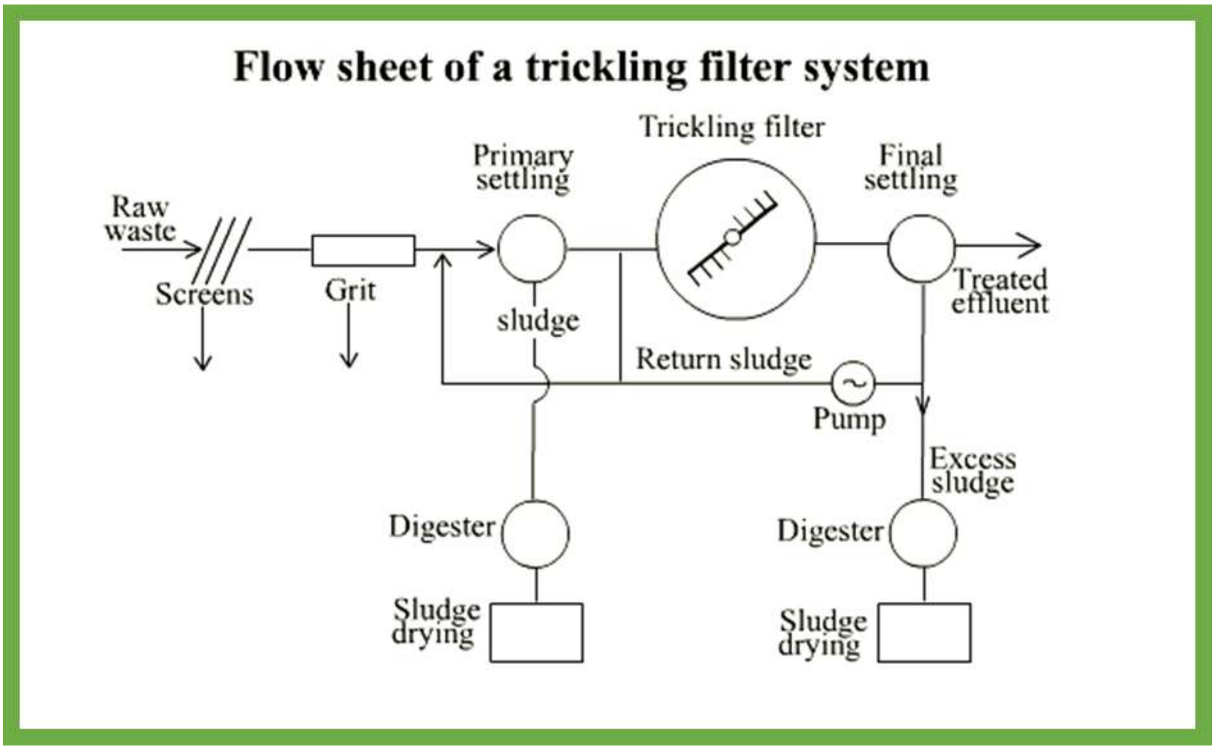


Figure 4.26: Flow sheet of a trickling filter system
(Source: https://www.chegg.com)

Generally trickling filter design is based on empirical relationships to find the required filter volume for a designed degree of wastewater treatment.

Types of equations:

i. NRC equations (National Research Council of USA)
ii. Rankins equation
iii. Eckenfilder equation
iv. Galler and Gotaas equation

NRC and Rankin's equations are commonly used. NRC equations give satisfactory values when there is no re-circulation, the seasonal variations in temperature are not large and fluctuations with high organic loading. Rankin's equation is used for high rate filters.

NRC equations: These equations are applicable to both low rate and high rate filters. The efficiency of single stage or first stage of two stage filters, E_2 is given by

$$E_2 = \frac{100}{1+0.44\left(F_{1.BOD}/V_1.Rf_1\right)^{1/2}} \quad \ldots\ldots (4.104)$$

For the second stage filter, the efficiency E_3 is given by

$$E_3 = \frac{100}{\left[(1+0.44)/1-E_2\right]\left(F_{2.BOD}/V_2.Rf_2\right)^{1/2}} \quad \ldots\ldots (4.105)$$

Where:

E_2= % efficiency in BOD removal of single stage or first stage of two-stage filter,
E_3=% efficiency of second stage filter,
$F_{1.BOD}$= BOD loading of settled raw sewage in single stage of the two-stage filter in kg/d,
$F_{2.BOD}$= $F_{1.BOD}$(1- E_2)= BOD loading on second-stage filter in kg/d,
V_1= volume of first stage filter, m^3;
V_2= volume of second stage filter, m^3;
Rf_1= Recirculation factor for first stage,
R_1= Recirculation ratio for first stage filter,
Rf_2= Recirculation factor for second stage,
R_2= Recirculation ratio for second stage filter.

Rankins equation: This equation also known as Tentative Method of Ten States USA has been successfully used over wide range of temperature. It requires the following conditions to be observed for single stage filters:

- Raw settled domestic sewage BOD applied to filters should not exceed 1.2 kg BOD_5/day/ m^3 filter volume.

- Hydraulic load (including recirculation) should not exceed 30 m^3/m^2 filter surface-day.
- Recirculation ratio (R/Q) should be such that BOD entering filter (including recirculation) is not more than three times the BOD expected in effluent. This implies that as long as the above conditions are satisfied efficiency is only a function of recirculation and is given by:

$$E = \frac{(R/Q)+1}{(R/Q)+1.5} \quad \text{(4.106)}$$

4.4.2.7 Tertiary Treatment

The treatment units that fall under this treatment level are Constructed wetlands, Sludge dewatering beds, Roughing filters. The design steps for Constructed wetlands and sludge dewatering bed follow those described under DEWATS section 4.2

4.5 Design guidelines for School WASH facilities

4.5.1 Overview

The Schools Water, Sanitation and Hygiene (SWASH) mapping survey conducted in 2009) in all primary and secondary schools in 16 Districts of Tanzania indicated that the water, sanitation and hygiene situation is very poor. Only 11% of the schools surveyed met the national standard of 20 girls and 25 boys per drop hole. Twenty percent of the schools have more than 100pupils per drop hole and 6% of schools have no latrines at all. It was also found that 96% of schools do not have facilities that are suitable or accessible to children with disabilities. Furthermore, around 40% of latrines have doors (however, these do not always guarantee privacy) and very few have hygienic facilities such as soap (1%) or sufficient water for hand-washing (8%) and just 7% of the latrines were free from smell or soiling. Regarding water supply, 62% of the schools in these Districts reported to have access to piped or other protected water supply options. However, some schools that reported having access to piped water or other protected water supply options do not have water on a regular basis and not all of these schools actually have the water supply sources within the school premises.

The overall picture from the SWASH mapping indicates that most schools are characterized by a non-existent or insufficient water supply, poor sanitation and lack of hand-washing facilities. In other cases, facilities do exist but many are broken, unhygienic or unsafe. Moreover, SWASH facilities (e.g. latrines) in most schools do not reflect the needs of girls, pre-primary school children and children with disabilities. There is a risk of low school attendance of girls (during their menstruation period) due to poor sanitation and hygiene facilities, denying them the necessary privacy and the right of getting education like their counterpart (boys).

4.5.2 Design Considerations and Procedures

This design manual is recommending anyone who wish to design WASH in schools to refer to the National Guideline for Water, Sanitation and Hygiene For Tanzania Schools which was published by the Ministry of Education, Science and Technology in 2016 available at https://www.unicef.org/tanzania/reports/national-guideline-water-sanitation-and-hygiene-tanzania-schools

4.6 Design guidelines for Health care WASH facilities

4.6.1 Introduction

Recently, the provision of improved water, sanitation and hygiene (WASH) services in health care facilities (HCFs) has attracted the attention of governments, Development Partners (DPs) and the international public health institutions. This is due to the fact that, although HCFs provide essential medical care to the sick, most of them especially in developing countries lack basic WASH services and thus compromising their ability to provide quality health care and consequently posing serious health risks not only to people who seek treatment but also to health care workers (HCWs) and careers.

There are numerous consequences of poor WASH services in HCFs. Several studies have revealed that, due to inadequate provision of WASH services, patients are potentially at higher risk of developing health care associated infections (HCAIs). The risk of infection is particularly high in newborns leading to sepsis which in most cases is fatal. The risks associated with sepsis are reported to be 34 times greater in developing countries. Further, lack of adequate WASH services may discourage women from giving birth in HCFs or causing delays in care-seeking. Therefore, addressing the inadequate provision of WASH services in HCFs will not only improve the quality of care but also attract many people to seek care including delivery services to pregnant women and most importantly contribute in the prevention of HCAIs

4.6.2 Design considerations and procedures

The user of this manual who would wish to design, construct and operate and maintain WASH in health care facilities is encouraged to consultant the National Guidelines for Water, Sanitation and Hygiene in Health Care Facilities which was prepared and published by Ministry Of Health, Community Development, Gender, Elderly and Children (MoHCDGEC) of the United Republic of Tanzania in 2017. The guidelines may be accessed through https://washmatters.wateraid.org/publications/national-guidelines-for-wash-services-in-health-care-facilities-in-tanzania

Overall, these guidelines have put in place a uniform and harmonized approach in the provision of WASH services in public and private HCFs all over the country. Specifically, they offer practical guidance for planning and budgeting as well as technical designing and construction of recommended WASH facilities, operation and maintenance (O&M), and monitoring of the performance of the services.

4.7 Design guidelines/options for small towns or emerging towns

4.7.1 Characteristics of small towns in Tanzania

i. Start as settlements and grow uncontrollably
ii. Not Planned
iii. Combination of small businesses and mainly agrarian economy
iv. Low capital to implement large projects
v. Low level infrastructure for water and sanitation
vi. More of community supply systems than in-house
vii. Combination of sources (shallow wells, surface sources and piped water systems)
viii. Mainly pit latrines of different kinds
ix. Higher percentages of dry type of sanitation than wet sanitation (Babati TC has 61.9% dry and 32.9% wet facilities[1])

4.7.2 Guidelines for water supply in small towns

i. Identify water supply schemes that are community based and demand driven
ii. Provide appropriate and affordable technology. Remember that cost of technology includes also the operation and maintenance costs of the technology.
iii. Standardize technology
iv. Supplement water supply with alternatives such as rainwater harvesting (Refer to **Volume 1** Chapter 3 sections 3.2.1, 3.6.1 and Chapter 9 section 9.1.6)

[1]Sanitation and hygiene practices in small towns in Tanzania: The case of Babati (submitted for publication in AJTMH)

4.7.3 Guidelines for sanitation services in small towns

i. Provide guidance on sitting of sanitation facilities in relation to water sources. Consider the geology of the area, soil type)
ii. Plan and provide systems for dealing with on-site sanitation facilities (sludge collection from septic tank, pit latrines, transportation of sludge and acceptable disposal facility).
iii. Designate appropriate areas for managing faecal sludge
iv. Provide faecal sludge treatment facility that is reasonably placed to minimize sludge transport costs.(Refer **section 3.2.6**)
v. **Consider decentralised wastewater treatment systems (DEWATS) section 4.2**
vi. Include possibilities of recovery of useful materials (biogas, nutrients, briquettes made from dried sludge and water)

4.8 Sanitation Resources Recovery and Reuse

4.8.1 Introduction

Ideally, both wastewater and faecal sludge should be seen as a resource that can be recovered, rather than a waste that needs to be managed and disposed of. With adequate collection and treatment, wastewater and faecal sludge can be transformed into treatment products that can be sold and utilized. For example, the water, organic matter, and nutrients in faecal sludge can be beneficial for soil properties and plant growth. The organic matter is beneficial for water retention, which can increase water-holding capacity and reduce the effects of drought, reduce soil erosion, and benefit the soil microbial community. It is important to consider the protection of public health, as well as public perception, with the use of treatment products. For safe resource recovery by end-users, it is important that pathogen levels are adequately controlled for the intended end-use, for example commercially available compost versus industrial fuel. Similarly, considering social acceptance of the product by the intended market is critical, for example selling faecal sludge briquettes to individuals as a household cooking fuel, as opposed to industrial customers. Figure 4.27 shows a typical process flowsheet of wastewater treatment with resource recovery. The units are defined in general terms for example the bio-reactor can be any of the biological contacting systems discussed in the previous sections such as biodigesters for biogas, activated sludge process, oxidation ponds etc. The type of actual units depend on the feed concentration of organics, what needs to be recovered and the use of the recovered components.

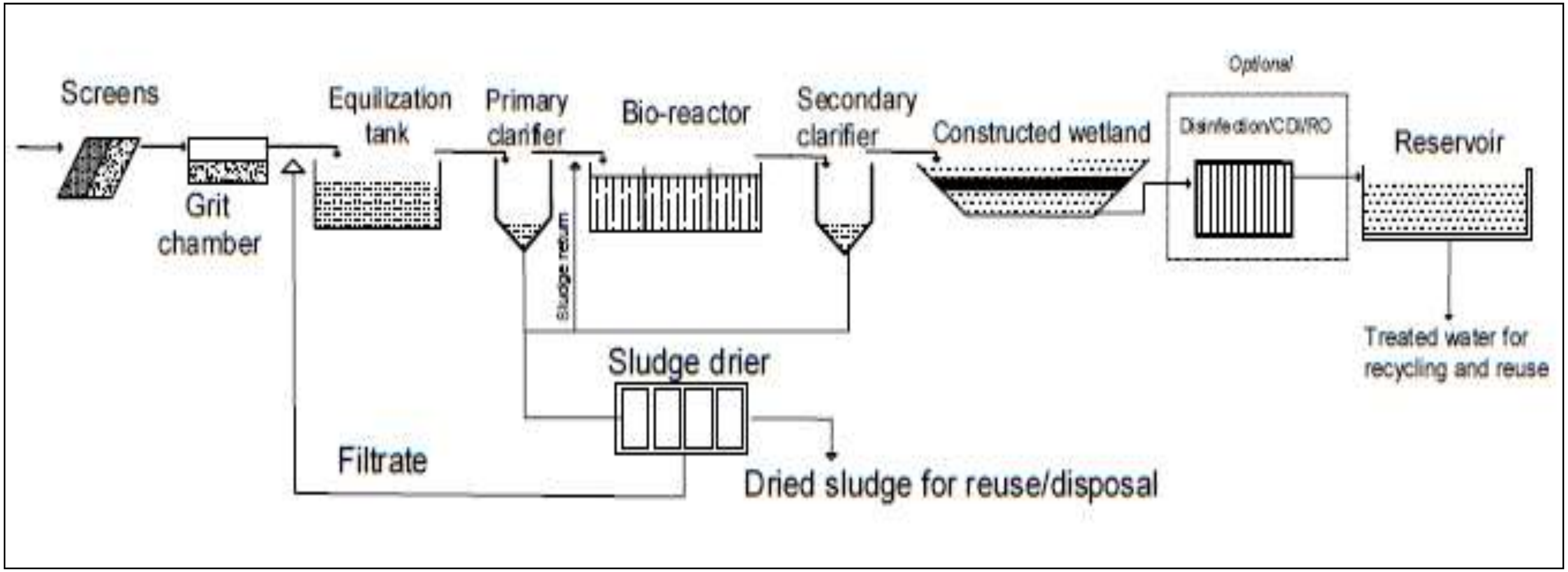


Figure 4.27: Example of Wastewater treatment process flowsheet with resource reuse/recovery

4.8.2 Treatment products

Resource recovery can be from both the solid and liquid fractions of faecal sludge. The types of treatment products will depend on the initial characteristics and on the treatment technologies. Examples of established forms of resource recovery include dewatered or dried sludge produced from unplanted drying beds for land application; co-composting of faecal sludge and organic solid waste; plants from planted drying beds (see section 3.2.2 on Faecal Sludge Management -FSM); deep-row entrenchment of untreated faecal sludge; or effluent from waste stabilization ponds used for irrigation or in aquaculture. Transferring and innovative treatment products include biogas from the anaerobic digestion of faecal, larvae from the treatment with black soldier fly and carbonization of faecal sludge.

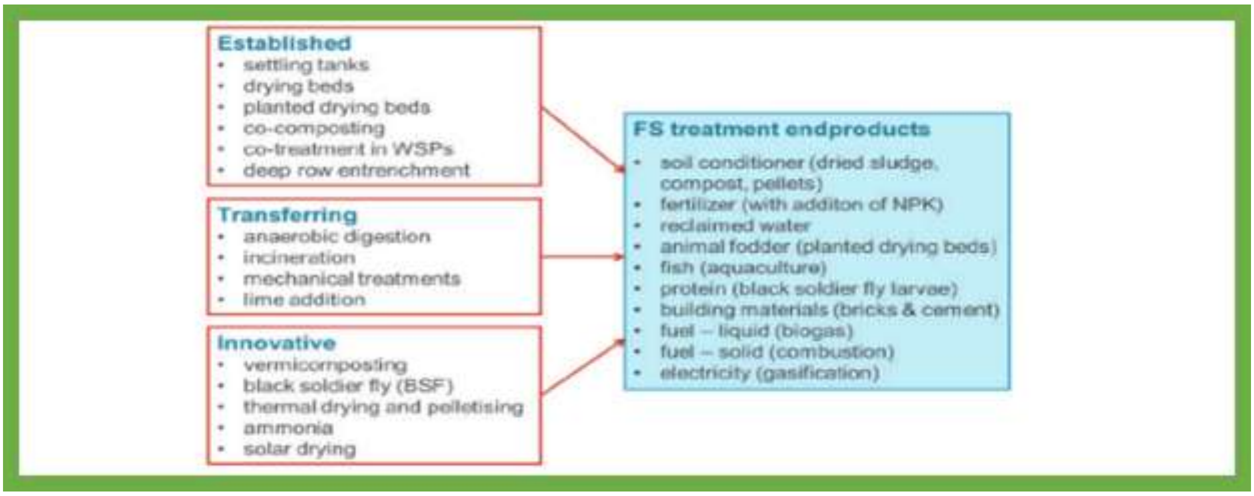


Figure 4.28: Level of establishment of faecal sludge treatment technologies and their treatment end-products

Source: Linda and Miriam, 2018

4.8.3 Application

Another way to think about resource recovery is by the actual resource that it provides, which is especially useful when thinking about the potential market demand. Treatment products can also be further processed to increase their market value. For example, further processing of char into briquettes that are suitable for recovery of its energy in institutions, or pelletizing of compost or dewatered (dried) sludge for easier transport.

Table 4.14: Summary of the potential resources and the treatment products

Resource	Treatment product	Product type
Energy	Solid fuel	Pellets, briquettes, powder
Energy	Gas fuel	Biogas
Energy	Electricity	Conversion of biogas, or gasification of solid fuel
Food	Protein	Black soldier flies, fish meal
Food	Animal fodder	Plants from drying beds, dried aquaculture plants
Food	Fish	Grown on effluent from faecal sludge treatment
Material	Building materials	Additive to bricks, road construction
Nutrients	Soil conditioner[1]	Compost, pellets, digestate, black soldier fly residual
Nutrients	Fertilizer[2]	Pellets, powder
Nutrients	Soil conditioner[3]	Untreated sludge, dewatered sludge from drying beds
Water, nutrients	Reclaimed water	Effluent from faecal sludge treatment

[1] With different levels of pathogen removal based on endues
[2] Addition of NPK to fulfil nutrient needs of a fertilizer
[3] No pathogen removal

Source: Schoebitz, l., et.al. 2016

4.8.4 Suitable faecal sludge characteristics for treatment products

Different characteristics of wastewater and faecal sludge will affect the quality of the treatment end-product, and will need to be evaluated to ensure that they meet market needs, and also to protect public and environmental health. Therefore, the multi-barrier approach (see Guidelines on sanitation and health. WHO, 2018 and MoW Water Safety Guidelines, 2015) can be used to protect public and environmental health when using faecal sludge as a treatment product. Relevant characteristics to consider include the following:

4.8.4.1 Land application

Nutrients such as nitrogen, phosphorus and potassium are essential for plant growth and important for the use of effluent for irrigation, and as a soil conditioner, compost or fertilizer. Heavy metals, such as cadmium, lead and zinc, and salinity are important as they can be toxic to plants and people. Indicators of pathogens for both liquid and solid streams to ensure that resource recovery adequately protects public health. Figure 4.30 shows an example of banana field irrigated by treated water.

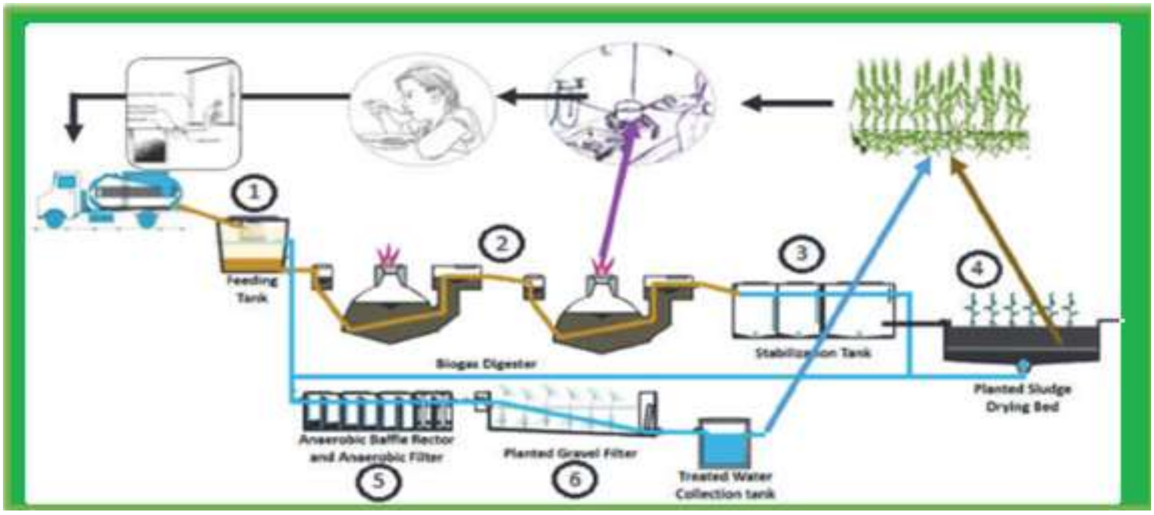


Figure 4.29: Process flow diagram of the faecal sludge treatment
Source: (Englund and Strande, 2019)

Key:
1 Feeding Tank
2. Biogas Digester
3. Stabilization Tank
4. Planted Sludge Drying Bed
5. Anaerobic Baffle Reactor and Anaerobic Filter
6. Planted Gravel Filter

4.8.4.2 Solid fuels

The calorific value is a measure of the energy content of a fuel, and is important for the characterization of solid fuel. Ash content is a metric of the non-combustible, inorganic fraction contained in faecal sludge, and it does not contribute to the calorific value. It needs to be disposed of, or used for phosphorus recovery. Indicators of pathogens are important depending on the final end-use, and risks need to be managed with a multi-barrier approach.

4.8.4.3 Biogas

Fractions of methane and carbon dioxide are important parameters for biogas, as higher methane and lower carbon dioxide concentrations increase the fuel potential.

4.8.4.4 Animal feed

The protein, fat and mineral contents are important for the use of insect larvae and plants as animal feed. Indicators of pathogens are important to ensure that pathogens are not transmitted to animals.

4.8.5 Treatment technologies for sanitation resource recovery and re-use

Figure 4.30: Banana field irrigated by treated wastewater at NM-AIST, Arusha Tanzania (Courtesy Prof. K.N. Njau)

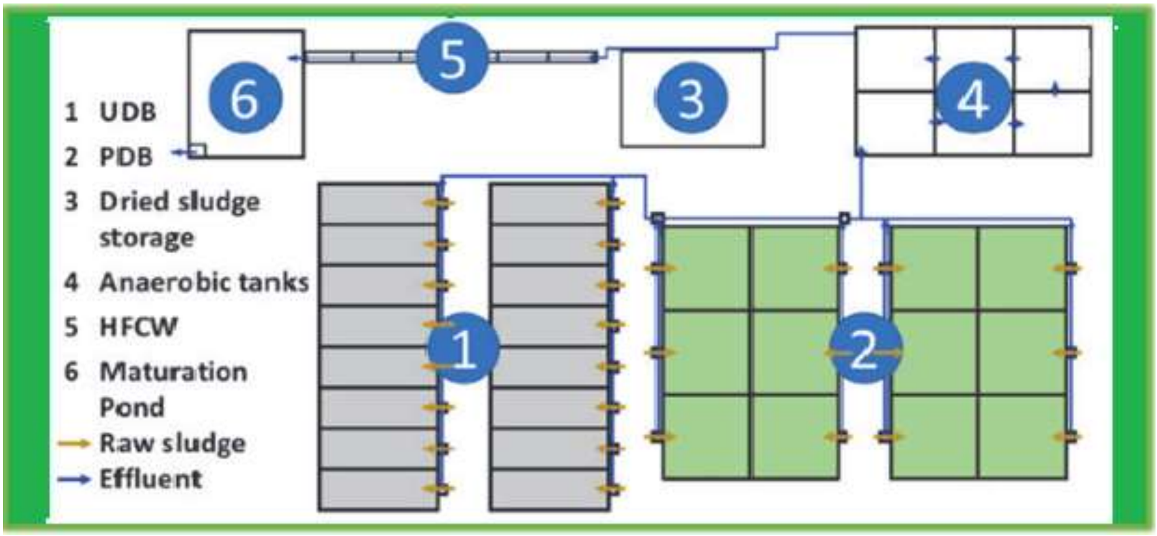


Figure 4.31: Faecal sludge treatment plant layout and flow diagram
Source: (Linda et al., 2018)

Figure 4.32: Preparation of the dewatered sludge cakes in Faridpur, Blangladesh, prior to transportation to the nearby co-composting plant
Source: (Linda et al., 2018)

4.8.6 Design procedures for treatment technologies for the resource

The design procedures for treatment technologies meant for sanitation resource recovery and re-use follow those ones under FSM and DEWAT sections this manual. The user of this manual is thus advised to refer to these sections for design procedures.

4.9 Safeguarding of sanitation infrastructure

Given the reality on impacts of climate change to WASH infrastructure and the fact the 3rd Edition of Design Manual for climate resilience issues to the same, this current edition has attempted to factor in the design of WASH infrastructure. The design manual is taking into account the dynamic of weather extreme events as articulated in section 2.4. Specifically, the manual is taking into account of climate issues into WASH infrastructure design into the following components; intake location (when considering combined systems) and its operation, the sitting and construction of wastewater and faecal sludge treatment facilities as well as collection/transmission lines from the waste generation points. It is envisaged that the user of this manual will find use it useful and they are encouraged to critically and thoroughly to take on board the climate change issues into design of WASH infrastructure.

4.10 Application software

4.10.1 Recommended Application Software

Common software being used in sanitation projects include the following:

(a) SewerCAD - is an easy-to-use sanitary sewer modeling and design software product that thousands of municipalities, utilities, and engineering firms around the world trust to design, analyze, and plan wastewater collection systems.

(b) STELLA - stands for Structural Thinking Experimental Learning Laboratory with Animation. Is the ecological definitive modeling tool to create professional simulations. Seamlessly create, design and publish models to share with anyone, anywhere, anytime. Is a modeling software package that diagrams, charts, and uses animation help visual learners discover relationships between variables and helps simplify model building.

(c) PC-based simplified sewer design – is a program aid the design of simplified sewerage systems. It seeks to do this by:

- Automating – and thus speeding up the necessary design calculations;
- Providing a tool for analyzing different design permutations/configurations; and
- Being suitable for training/learning purposes.

4.11 Technical spreadsheets

This section has been adapted partly to **pages 241-246** in the book Decentralised Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries 2009, (Gutterer, 2009). A practical guide useful in assisting designers to use spreadsheets for sizing some of the unit operations discussed in Chapter 4. The purpose of this section is to provide the engineer with an example of a tool to produce his or her own spreadsheets for sizing DEWATS in any computer programme that is familiar with. The exercise of producing one's own tables will compel engineers to deepen their understanding of design. Computerised calculations can be very helpful, particularly if the formulas and the input data are correct. Flawed assumptions or wrong data, on the other hand, will definitely result in worthless results. It is the duty of the design engineer to ensure that the assumptions made are reasonable and the data entered is correct. For detail explanations on the use of spreadsheet calculations and the limitations the engineers are directed to make reference to chapter 10 of this book (Gutterer, 2009).

Example: Domestic wastewater quantity and quality

The spread sheet shown in Table 4:15 helps to define domestic wastewater production and quality in terms of the number of people and the wastewater they discharge. BOD and water-consumption figures vary widely from place to place and, therefore, should be obtained for each site. These figures have been matched with figures used in Tanzania.

Formulas of spreadsheet "domestic wastewater production":
E5 = A5 x C5 / 1000
F5 = A5 x B5 / E5
G5 = D5 x F5

Table 4.15: **Spreadsheet for calculation of quantity and quality of domestic-wastewater production**

	A	B	C	D	E	F	E
	Wastewater Production and concentration of organic matter						
	Users	**BOD_5 per User**	**water Consumption per user per day**	**COD/BOD_5 ratio**[2]	**daily flow of wastewater**	**BOD_5 concentration**	**COD concentration**
1	*given*	*given*	*given*	*given*	*calculated*	*calculated*	*approximated*
2	number	g/day	litres/day	mg/l	m^3/day	mg/l	mg/l
3	80	55	80	2	5.1	859	1719
4	Range	40-65	50-100		80% of water consumed[3]		

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[3] In Tanzania we assume that 80% of water used for domestic purposes ends up as wastewater. In other countries a factor of 1 is used.

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CHAPTER FIVE
DESIGN STANDARDS AND SPECIFICATIONS

5.1 Design Standards

A Standards is a limit of the 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 agency standards 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 projects will be from Tanzania Bureau of Standards, for construction works British standards shall be used.

A list of institutions whose standards are recommended to be used in design is shown below:

i. Tanzania Bureau of Standards,
ii. British Standards (BS),
iii. American Society for Testing and Materials (ASTM)
iv. Deutsches Institut für Normung (DIN); German institute for standardisation,
v. American Association of State Highway and Transportation Officials (AASHTO),
vi. European Standards(ES),

Table 4.1 describes standards codes of practise and relevant area for application construction works.

Table 5.1: Common Standards used in Water projects

No	Name	Institution	Use of standard
1	BS 8110 of 1997	British Standard	Code of practise for design and construction
2	BS 812 : Part 2 : 1995	British Standard	Testing aggregates- Methods of determination of density
3	ISO 1167-1 Part 1	International Standard	Thermoplastics pipes, fittings and assemblies for the conveyance of fluids — Determination of the resistance to internal pressure
4	ISO 1452-2 Part 2	International Standard	Thermoplastics pipes, fittings and assemblies for the conveyance of fluids — Determination of the resistance to internal pressure
5	BS 6399 : Part 1 : 1996	British Standard	Code of practice for dead and imposed loads

No	Name	Institution	Use of standard
6	BS 1377-9: Part 9: 1990	British Standard	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 requirement that are not covered by recommended standards, the designer should seek approval from the Tanzania Bureau of Standards.

5.2 Specifications

Specifications is a detailed description of how work is to be perfumed or requirements to be achieved, dimensions to be met, materials to be used, standards to be followed and tests carried for the 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 and sanitation projects standard specifications have been prepared for various woks as follows:

i. Standard Specifications for Civil Works,
ii. Standard Specifications for Electrical works,
iii. Standard Specifications for Mechanical works and
iv. General Specifications.

These documents can be downloaded from Ministry's Website customized to fit the needs of particular work.

5.3 Materials

5.3.1 Building Materials

Building material is any material used for construction purpose such as materials for structures. Wood, cement, aggregates, metals,sand, bricks, concrete, clay are the most common type of building material used in construction.The choice of these are based on their quality and cost effectiveness for building projects.

5.3.2 Materials Testing

Before a material is to be used for construction work, it is imperative to conduct appropriate tests as per applicable standards. The following are minimum tests proposed to be conducted on various construction materials.

5.3.2.1 Aggregates

Test of aggregates explained below includes both fine and coarse aggregates.

Flakiness index test
Flaky particles are those whose least dimension is 0.6 times lesser than the mean size. Thickness of these particles are comparatively smaller than the other two dimensions.

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 purpose.

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, also one dimension is 1.8 times greater than the other two dimensions. 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 purpose.

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 high ratio of surface area to volume. The presence of flaky and elongated particles also may 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. Test for abrasion is conducted based on BS 812: Part 113: 1990.

5.3.2.2 Organic Impurities Test
Sand should be checked for presence of organic impurities such as decayed vegetation, humus, and coal dust as these affect the quality of concrete. Test for organic impurities should be conducted as per. BS 812: Part 4: 1976

5.3.2.3 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. 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 111

5.3.2.4 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

5.3.2.5 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

5.3.2.6 Absorption test

Water absorption is measure of the porosity of aggregate, it gives indication of the strength of 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 to BS 812-120:1989.

5.3.2.7 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

5.3.2.8 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 of steel reinforcement .Corrosion of steel affects serviceability and strength of concrete structures . The test to determine the content of chemicals in aggregates is conducted as per BS 812-Part 117 & 118:1988

5.3.3 Water

5.3.3.1 Impurities Test

Water for washing of 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.

5.3.3.2 Chemical content such as Chloride,PH values, Sulphate

Samples of the water being used or which it is proposed to use for mixing concrete and shal 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.

5.3.4 Cement

5.3.4.1 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 procedure explained in the AASHTO T 131 and ASTM C 191: Time of Setting of Hydraulic Cement.

5.3.4.2 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 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, 33 grade of cement. The test procedure to is as per code of practice BS EN 196-1:2005.

5.3.5 Concrete Works

Tests conducted for concrete includes:

5.3.5.1 Slump test

Concrete slump test or slump cone test is done to determine the workability or consistency of concrete mix prepared at the laboratory or the construction site during the progress of the work. Concrete slump test should be carried out from batch to batch to check the uniform quality of concrete during construction.The slump is carried out as per procedures mentioned in ASTM C143 in the United States, and EN 12350-2 in Europe.

5.3.5.2 Compressive strenght test:

Compressive strenght of concrete is the measure of Compressive strength is the ability of material or structure to carry the loads on its surface without any crack or deflection. Standard test method for Compressive Strength of Cylindrical Concrete Specimens is carried out from procedure as stated in American Society for Testing Materials ASTM C39/C39M.

5.3.5.3 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 determined by the procedures described in the code AASHTO T 269.

5.3.6 Steel

5.3.6.1 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.

5.3.7 Other materials

Testing for materials used in construction such as sands, bricks/blocks, etc should be done according to the recommended standards specified in volume I of DCOM or as may be recommended for a given project.

CHAPTER SIX

STAKEHOLDERS PARTICIPATION IN DESIGN OF SANITATION PROJECTS

6.1 The Stakeholders

Part of the job of a sanitation planner and designer involves mobilizing local resources to improve the sanitation situation in the community. This means helping to develop partnerships and collaborations among the stakeholders, for example, by organizing a focused group meeting with them. But who are the stakeholders of sanitation projects and how do one identify them?

A stakeholder is any person, organization or group with an interest (stake) in something, such as a particular situation, intervention, project or programme. The stakeholders depend on the type and scale of the sanitation project, the local context, the local institutional set-up and the socio-cultural conditions.

When considering a specific sanitation project and wishing to identify the stakeholders involved (Mathur et al., 2007), as a designer, one should consider those who:

i. are responsible for the project and its different components (including funders, WASH officials from different sector offices, managers, employees, etc.),
ii. are intended users or beneficiaries including those who register the CBWSOs,
iii. May be negatively affected by the project but may not be in a position to say so,
iv. might threaten the success of the project through their opposition or lack of co-operation,
v. could represent the interests of people unable to participate,
vi. have unique knowledge related to an aspect of the project,
vii. are form responsible ministries and agencies.

6.2 Key Stakeholders

Some of those among the wide group of possible stakeholders can be identified as key stakeholders. A key stakeholder is a person or a group of people with significant influence over a programme or who will be significantly impacted by it. For the programme to be successful, their interests and influences must be recognized.

Key stakeholders may include:

- individuals,
- organizations and
- businesses in the public, private and non-profit sectors.

These could be:

- local community representatives,
- municipal sector offices (for example, water resources, health and education), and
- development partners including, non-governmental organizations (NGOs), community-based water supply organizations (CBWSOs) and private sector groups.

Sometimes, new stakeholders may emerge during the lifetime of the operation of a sanitation system. For instance, a group of households using the same decentralised wastewater treatment system, may need to understand likely sources of conflicts associated to their infrastructures (eg. condominial systems). Areas for the passage pipelines for simplified sewerage, clogging/leakage and delays in fixing failures may cause personal or community complains. Area for treatment of wastewater and sludge is another potential problem both in securing the area in urban setting as well as neighbours to accept wastewater treatment systems in their neighbourhood. In view of these, stakeholders involvement in planning stages of the sanitation options has to be emphasised to avoid later conflicts. If such a stakeholder was not involved during the planning stage, then conflicts are likely to be more serious and require resolution. It is much better to try to identify any unintended users at an early stage as this will enable them to feel some sense of ownership and reduce the likelihood of future conflicts.

6.3 Aim of Stakeholders Engagement

Stakeholder engagement is the process by which the organizers of a project involve the stakeholders so they can influence its decisions and implementation. Some stakeholders may support the decisions, while others may oppose them. Some may be influential in the organization or community in which they operate and hold official positions. Others may be affected in the short or long term by the outcomes of the sanitation project. The underlying principle of stakeholder engagement is that stakeholders have the opportunity to influence the decision-making process. This differentiates stakeholder engagement from communication processes, which just share and explain decisions that have already been made.

The aim of stakeholder engagement is to:

i. hear what stakeholders have to say to establish what issues matter most to them,
ii. develop an understanding and agree how best to deal with issues of concern to the stakeholders,

iii. ensure project sustainability by involving stakeholders in planning, implementation and monitoring,
iv. improve decision making and accountability.

Through working together, key stakeholders can identify common concerns, develop common goals and reap the benefits of the impact of the sanitation project. Some stakeholders may also become involved in technical aspects, contributing to implementation, designing solutions and providing technical advice. Involving stakeholders in this way ensures more effective outcomes.

As a designer of a sanitation project, one may be involved in arranging and facilitating discussions with stakeholders. This means encouraging people to participate. For this one will need to develop the communication skills so as to succeed:

i. ensure involvement of all stakeholders, including vulnerable groups of the society and the marginalized individuals and households,
ii. understand their demand for service options and their willingness to pay or contribute,
iii. create a sense of ownership among users and beneficiaries,
iv. help to achieve a common understanding between the implementing organization, user community and the relevant stakeholders.

It is important to involve stakeholders throughout the planning and implementation process. This brings benefits through:

i. opening the planning process to the public, making it more transparent and equitable,
ii. allowing stakeholders to participate in budget setting and sanitation tariff payment mechanisms,
iii. ensuring the needs of the whole community are considered, thus making the projects more effective,
iv. helping to overcome resistance and mistrust by enlisting their support,
v. It may also increase efficiency if stakeholders contribute their labour and resources.

Stakeholder engagement improves communication and leads to better project understanding. The benefits will depend on the context, but may include increased community confidence, which comes from co-operating over project development. It can also encourage a culture of innovation and learning, which enables participants to make better-informed decisions. It builds trust, through open discussions of issues that are difficult to resolve, can bridge cultural gaps and helps to reduce potential conflicts. It can also enhance partnerships, for example, between the community and industry, increasing efficiency and so reducing future costs.

6.4 Identifying and Mapping of Local Stakeholders

Key stakeholders can be identified based on their:

i. influence in decision making,
ii. responsibility,
iii. involvement in day-to-day operations,
iv. direct or indirect dependency on the project and
v. representation in the community.

6.4.1 Identifying Key Stakeholders

Representation from all the stakeholders is a priority in a multi-stakeholder WASH engagement project. Some less obvious stakeholders may be excluded from the usual decision-making processes; this should be avoided. Local institutions such as schools, health centres, mosques and churches are considered important stakeholders. These are important strategic institutions for promoting community-based sanitation interventions. While at school, children gain knowledge that influences them and informs their attitude and practice. In addition school children, via their teachers and WASH clubs, can educate their families and relatives when they return home. By this route, they can serve as *agents of change* to their communities.

It is important to identify all stakeholders from the community including women, children and marginalized people. Marginalised people are those on the edges (margins) of society who are treated as insignificant or not important. There may be people in a community who find it difficult to come to meetings, for example because of their work pattern or because they have a disability. It is particularly important to ensure that such groups have a voice and are listened to. Excluding less obvious stakeholders from the usual decision-making processes is an easy mistake to make and may have serious social or economic costs. It can lead to unsustainable projects and no overall improvement in conditions.

A systematic approach to defining and identifying all relevant stakeholders during early planning stages is therefore essential for ensuring the effectiveness and sustainability of WASH initiatives covering both rural and urban areas including school and health facilities in the respective area.

6.4.2 Stakeholder Mapping

Stakeholder mapping is the process of systematically identifying and analyzing the relevant stakeholders, their relationship to each other, their level of interest, and their roles and responsibilities in relation to the power they hold. Mapping the levels of interest of different stakeholders in relation to their interest or power can be done using the diagram shown in Figure 6.1. Their relative power and interest is categorized into four groups:

i. those with high interest but little power (A),
ii. high interest and high power (B),
iii. low interest but high power (C) and
iv. low interest and little power (D)

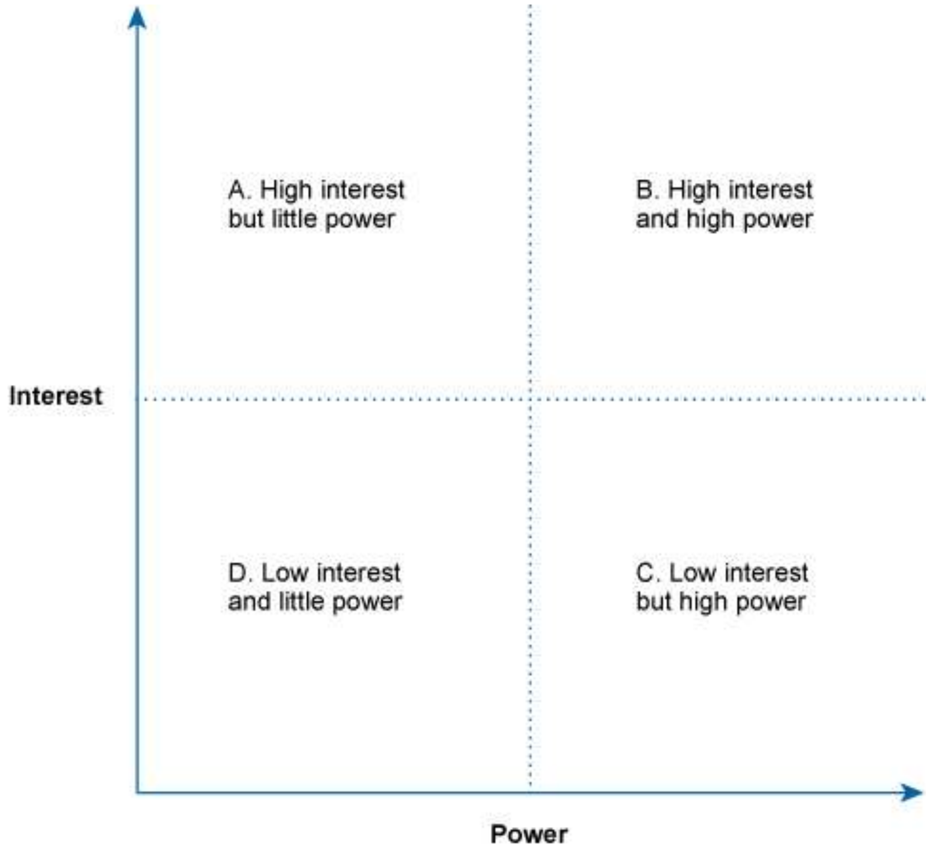

Figure 6.1: Mapping stakeholders on a power/interest grid
(Source: Adapted from DfID, 2003).

Stakeholder mapping can help to fully understand a situation and see the relationships between the stakeholders and their role in the project or programme. This can be useful when developing a plan for stakeholder engagement. Such a plan should outline:

- objectives (what is one trying to achieve?)
- scope (who and what is included?)
- methods (how will one put the plan into action?).

The methods used will vary for different stakeholders and will depend on several factors including how actively they are involved. For example, for users and beneficiaries, mediated discussions with service providers could be appropriate. For other, less engaged stakeholders, printed leaflets or other methods for providing information could be considered.

6.5 Key points of Stakeholders Participation in Design of Sanitation Projects

The following is the summary of stakeholders participation in sanitation projects:

i. It is important to identify and characterize the stakeholders involved when planning sanitation projects so that all interests can be considered.
ii. The planning and implementation stages of sanitation projects needs effective communication with stakeholders so that their knowledge and resources can be included.
iii. Engaging stakeholders helps to improve decision making and accountability and ensure sustainability of the sanitation projects.
iv. Stakeholder mapping is a useful tool for defining the level of interest and power of each stakeholder.
v. In the past, the approach has been fragmented with a lack of coordination between organizations responsible for sanitation projects.
vi. It is important to understand the advantages of working across disciplinary and sector boundaries. Teamwork involving a variety of people with different skills and knowledge will bring more effective and sustainable results.

SUBJECT INDEX