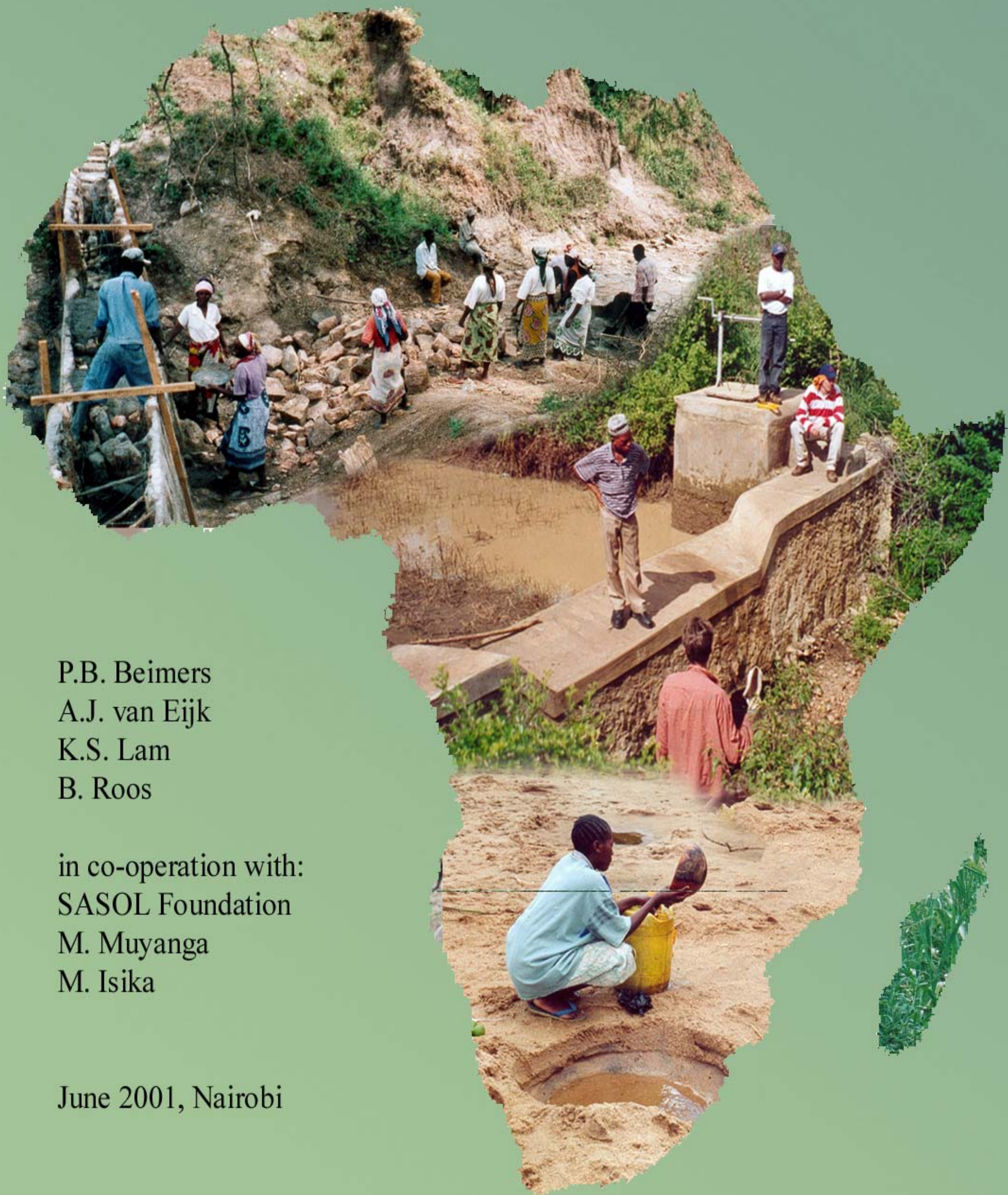
IMPROVED DESIGN SAND-STORAGE DAMS

KITUI DISTRICT, KENYA

PROJECT REPORT



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in co-operation with:
SASOL Foundation
M. Muyanga
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June 2001, Nairobi

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COLOPHON

Delft University of Technology is not responsible for consequences of any kind, resulting from applying data, calculations and conclusions to be found in this report.

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PREFACE

As part of our study, Civil Engineering at Delft University of Technology, we have been involved in a project concerning the construction of sand-storage dams in Kitui District, Kenya. We have spent 15 weeks in Kenya (April 1st - July 15th). This project can be seen as an experiment for Delft University of Technology, sub-faculty of Civil Engineering, because it is combined with a practical work period. The information that was gained from the practical work (six weeks) has been used for the project. Our group consists of four students with different specialisation. One student specialises in water management; the other three specialise in hydraulic engineering.

This project report is one of the four reports that have been written during the past 16 weeks. One report deals with the practical work period, one is a manual on location choice, design, construction and maintenance of sand-storage dams and the third one is a proposal for a detailed hydrologic study to determine the impact on the surrounding of the dams.

The project has been initiated by our counterpart SASOL and the UNESCO chair of the University of Nairobi. SASOL is trying to know more about the hydrologic impact and possible constructional improvements of the sand-storage dams that are built in Kitui District. This is the first time a group of students of TU Delft has worked with SASOL Foundation.

During the project, we stayed in Kitui and Nairobi. In Kitui we visited and evaluated 50 dams. The report writing took place at the Apostles of Jesus Youth Technical Institute in Nairobi.

We would like to thank Ir. W.J. Dijk, Ir. M.W. Ertsen, Prof. ir. R. Brouwer and Prof. ir. L.A.G. Wagemans of Delft University of Technology for accompanying us and having confidence in this special combination of project and practical work. Also thanks for Prof. J.M. Bahemuka of the UNESCO chair of the University of Nairobi arranging our internship. Maartje van Westerop made arrangements for our stay in Kenya and provided a lot of useful contacts, thanks for this. Gerard Pichel and Wim Spaans provided useful information and gave advice about the construction of dams in Kenya.

We also extend our thanks to Prof. C.G.M. Mutiso, chairman of the SASOL board, for the general co-ordination of the project and accompanying us during the field visits to the dams. We would also like to thank Milu Muyanga and Mutua wa Isika for their contribution and for revising this report.

Thanks to father Joseph and the other fathers for accommodating us in Langata and driving us around in Nairobi.

Last but not least, we would like to thank our sponsors for their financial support. Without their support this project would not have been possible.

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Nairobi, June 2001

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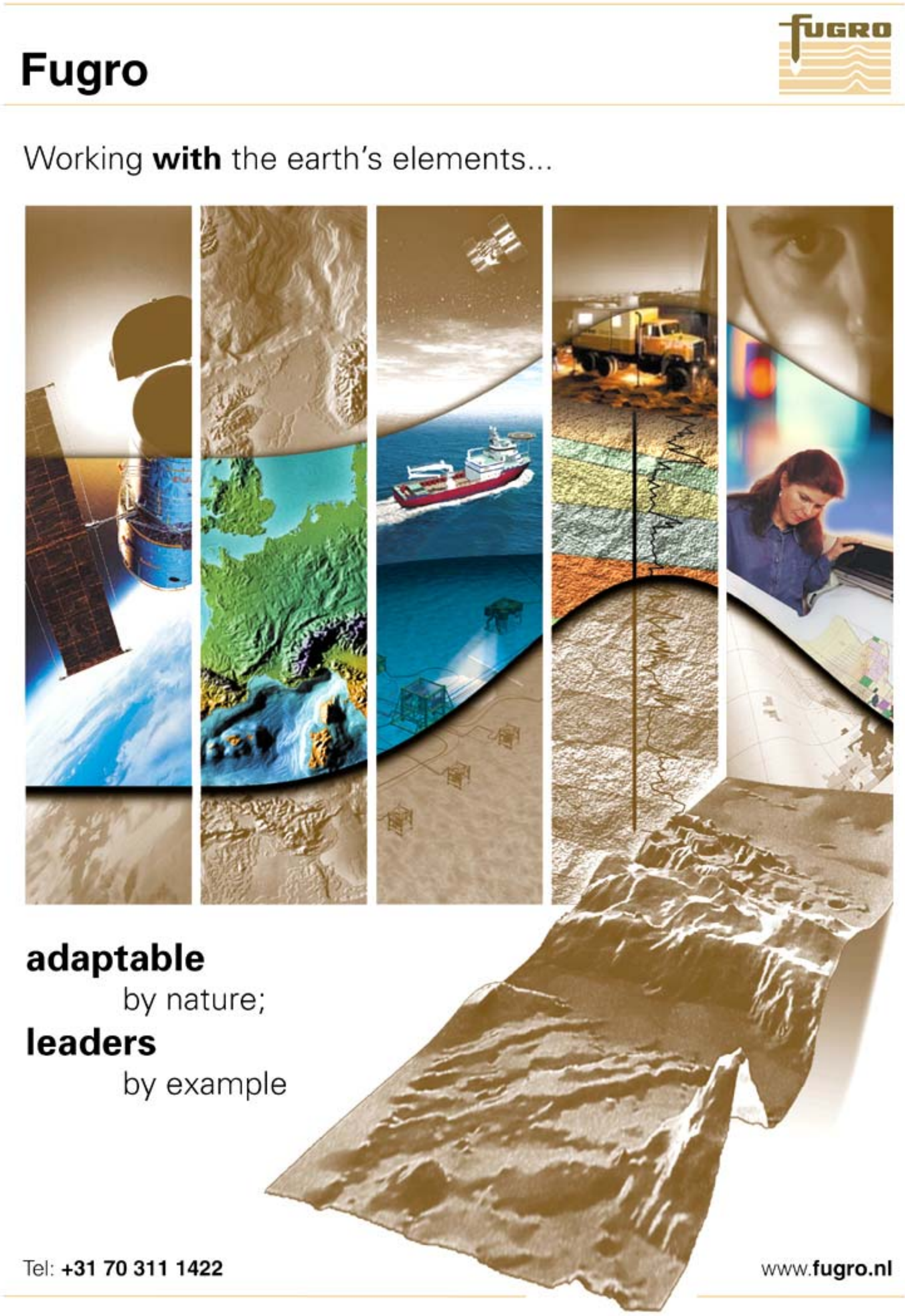
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SUMMARY

Kitui District is located in Eastern Kenya. The climate is hot and dry for most of the year as in other arid and semi-arid areas. The rainfall is highly erratic and the rate of evaporation is high. The main problem in the area is inadequate water for a large part of the population as there are only a few water sources such as rivers and springs.

SASOL is a non-governmental-organisation and focuses on satisfying the important basic need, water. The main objective of SASOL is to prove that simple, low-cost technology for water infrastructure can improve the lives of people in arid and semi-arid areas. By April 2001, SASOL had already built 216 dams in different rivers in an area of 600 km^2 in the Kitui District. SASOL uses a bottom-up approach when implementing the projects by letting the local community members define their own problem, setting their own priorities and making their own decisions on how to solve it.

Problem definition:

- There is a lack of knowledge on the technical background of sand-storage dams built by SASOL. This lack of knowledge hampers SASOL from optimise the dam design.
- There is a lack of documentation about the design and the building process of sand-storage dams so that copying of this technology for water infrastructure for other development organisations or governments is difficult.
- Many effects of series of sand-storage dams are claimed, but none of them has been proved scientifically. Not much is known about the characteristics of the reservoirs of sand-storage dams, although it is an important aspect of sand-storage dams.

The **objectives** of the project are:

- To optimise the design of sand-storage dams.
- To make a manual on location choice, design, construction and maintenance with emphasis on the specific problems in ASALs.
- To set up a proposal for a hydrological research on the effects of the sand-storage dams.

The most **important demands** on the dams are:

- The construction should be built only using simple equipment.
- No skilled workers should be needed to build the construction.
- The construction method must be appropriate for other areas of Kitui District.
- The building process has to be suitable for a community-based approach.

There are many different options for the storage of water in ASALs: sub-surface dams, sand-storage dams and roof and compound catchments. The roof and compound catchments are expensive per m^3 of storage. The sub-surface dams and the sand-storage dams are more economical for water-storage. The sub-surface dams and sand-storage dams are applied in different natural preconditions:

Sub-surface dams	**Sand-storage dams**
• Low topographical gradient of the slope of the river bottom • Simple design, no spillway • Uses storage capacity below surface only • Wide gentle rivers	• High topographical gradient of the slope of the river bottom • Spillway is needed • Increase of storage volume possible • Narrow river, steep banks

By means of a multi-criteria-analysis the most suitable dam type is determined for Kitui District. The stone-masonry sand-storage dams as well as the stone-masonry sub-surface dams have the best score on the criteria of costs, maintenance and suitability for community-based building. Other options, like a plastic sheet sub-surface dam and a stone sand-storage dam are interesting and worth trying.

During the first two weeks of the project 50 SASOL sand-storage dams have been evaluated. For this purpose an evaluation form was drawn up. The inventory concerned aspects like: construction, used materials, area characteristics, abstraction system, user characteristics and costs. The goal of the inventory is to get a broad overview of the different possibilities and problems and eventually to optimise the SASOL design. Dams from three different catchment areas were visited and evaluated. These areas have different characteristics.

The SASOL sand-storage dams show weak points as shown in the table below.

	Area A		**Area B**		**Area C**		**Total Areas**	
Condition	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
weak points								
erosion	16	62%	8	62%	9	82%	33	66%
stability	3	12%	0	0%	0	0%	3	6%
settlement	1	4%	0	0%	0	0%	1	2%
cracks	3	12%	1	8%	0	0%	4	8%
seepage	5	19%	0	0%	2	18%	7	14%
bank	5	19%	3	23%	0	0%	8	16%
abutment	8	31%	5	38%	0	0%	13	26%
downstream toe	1	4%	2	15%	1	9%	4	8%
protection	7	27%	6	46%	5	45%	18	36%
draw-off system/well	3	12%	0	0%	0	0%	3	6%
others	4	15%	3	23%	4	36%	11	22%

The most important findings from the evaluation are:

- The main weak point, erosion, is caused by the way the river flows across the dam. If the discharge is high, the water flows across the wings and causes erosion nearby the wings, the banks and the toe of the embankment.

- The lack of a stilling basin that prevents the downstream toe of the dam gives the erosion full play to damage the foundation of the dam.

- Planting vegetation prevents erosion. Napier-grass has proved its suitability. The non-permanent rivers are wild and have a high flow velocity. Planting Napier-grass can fix the course of the river.

- The reservoirs are not yet protected against contamination by cattle or other contagion sources. To prevent contamination of the water in the reservoirs cattle should not be allowed to walk in the reservoir.

- Although a few dams have cracks and some of them have seepage, the dams are still able to retain water in their reservoirs.

To be able to design the spillway for a sand-storage dam the peak-discharge must be determined. Methods based on peak levels, such as the Slope-Area Method, are often used in areas where no data is available. This method is simple and gives a quite good approximation of the peak discharge. The Slope-Area Method is the most suitable method to estimate the peak discharges, because it is simple and generally applicable.

- Building in stages is recommended, since building in stages gives better sediment in the reservoir and thus more storage capacity and a better extractability of the water from the reservoir.
- The erosion of the banks and the soil directly behind the wings is caused by wrong spillway dimensions. The dimensions of the spillway are calculated using the design-discharge from the Slope-Area method.

- The stilling basin protects the downstream side of the construction against erosion. The dimensions of the stilling basin depend on the design discharge and the height of the spillway crest about the downstream riverbed level.

- The width of the dam is determined for different heights of the spillway above the river, by checking the internal and external stability of the dam. The dams that are founded on rock are schematised as gravity-wall-structures, the dams that are founded on clay or murram are considered as restrained structures. From a constructional point of view reinforcement in horizontal as well as in vertical direction is not required.

With this optimised design, the practical work report, the collected literature and the acquired impressions during the duties in Kitui District enough information was gained to write a detailed manual on design, location choice, construction and maintenance of sand-storage dams in arid and semi-arid areas.

A proposal for a hydrologic study on the SASOL sand-storage dams has been drawn up with this information.

TABLE OF CONTENTS

GLOSSARY OF TERMS

ASALs	Arid and Semi-Arid Lands.
Base rock	Rock that is present underneath the soil top-layer and can be used for the foundation of sand-storage dams.
Bottom-protection	Protection of the soil on the river bottom with stones or mortar.
Building records	Records of the building process.
Catchment area	The land area upstream of the dam that takes in all streams and rivers that supply the dams.
Centre point of the river	The middle of the river-bottom in the concerned cross-section.
Committee rules	Rules about presence during construction and the use of the stored water behind the dam. After completion the site committee draws up these rules.
Community record	Record of the number of people and activities of the community.
Community-based approach	In this approach the community plays a major role to solve their own problems.
Crest length dam	The length between the two points where the crest level of the dam intersects both banks.
Crest level dam	The height of the crest of the dam above the river bottom in the centre point of the river.
Crest level spillway	The height of the spillway crest above the river bottom in the centre point of the river.
Dam crest	The top of the dam.
Dam height	The distance between the crest of the dam and the centre point of the river bottom.
Design discharge	The discharge that is used to determine the dimensions of the dam.
Domestic water point	An abstraction point for water for human consumption.
Erosion	The loss of soil and rock by natural agencies such as rainfall, river and flood flows, undermining or gravity. The opposite of sedimentation.
Evaporation	The process by which water is changed from the liquid state into water vapour through the transfer of heat energy.
Extractability	The quantity of water that can be extracted from a saturated sand volume, expressed as a percentage of the total volume.
Flash floods	Abnormal river flows following excessive rainfall and/or run-off.
Flood marks	Remaining materials on the banks that have been carried with the flow, or traces of erosion on the banks.
Flood model	A model to predict discharges and water levels.
Gross freeboard	The difference in height between the dam crest and the spillway crest. Estimated at the design stage and with the correct width of spillway must be sufficient to allow for the safe carrying away of flood flows when the basin is filled up.
Groundwater dam	Groundwater dams store water in the sand below the surface.

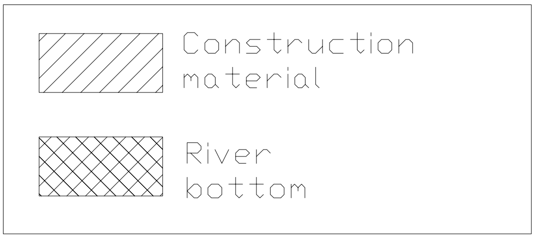
Hard-core	Broken stone.
Human workday	The work one human produces in one day.
Impermeable	Watertight.
Key	A screen that is used to prevent seepage/piping.
Length dam	Crest length dam and the length of the wing walls together.
Masonry	Construction work carried out with stone and mortar.
Mixing place	A place, where the cement is mixed (made out of cement).
Mortar	Mix of water, cement and sand, mixed with a certain rate of the ingredients.
Murram	Sticky soil with stones.
Napier-grass	Type of vegetation that is very suitable for protecting banks and the prevention of erosion.
Non-uniform reach	The velocity in this part of the river differs from place to place, but is constant in every cross-section.
Peak discharge	Highest discharge during the last X years.
Peak level	Highest water level during the last X years.
Piping	Erosion under the dam due usually to uncontrolled seepage.
Plaster layer	A layer of a sand-cement mix to make the dam watertight.
Porosity	Water holding capacity as a percentage of the total volume.
Safety factor	A multiplication factor that is used to cover uncertainties from the calculations.
Sand-storage dam	A sand-storage dam is constructed above the ground level and collects the water in the sediments that settle behind the dam.
Sedimentation	Settling of particles in a reservoir, the opposite of erosion.
Seepage	Water moving through or under the dam is said to be 'seepage'.
Siltation	Filling up with sediment.
Site committee	Committee elected by the community that supervises the construction, operation and maintenance of the dam.
Slope-Area Method	A method to calculate the design discharge with the slope of the river and the cross-sectional area.
Spillway	The overflow section of the dam.
Spillway crest	The top of the spillway.
Spillway height	The height of the spillway crest above the river bottom in the centre point of the river.
Stilling basin	A structure to prevent erosion on the downstream side of the dam. The energy surplus of the water is dissipated in this basin.
Sub-surface dam	A sub-surface dam is constructed below ground level and arrests the groundwater in a natural aquifer.
Throwback	The distance between the dam and the upstream apex of water stored in the reservoir at the spillway crest level.
Uniform reach	The velocity in this part of the river is the same and constant in every cross-section.
Water abstraction	The way of draw-off for the reservoirs.

Water Storage Capacity (WSC)	The amount of water that can be stored in the reservoir behind a sand-storage dam.
Wing wall	Anchoring wall of a sand-storage dam in the banks.
Yield of the reservoir	Volume of extractable water.

KEY OF SYMBOLS

Symbols for Chapter 3

Symbols for Chapter 6



1. Introduction

1.1 Project context

Two obligatory subjects in the curriculum of Civil Engineering at Delft University of Technology are the reason of the undertaking of project group CF 599 in Kenya. In the last four months of the fourth year of the curriculum everyone has to execute a six week project and a practical work for a period ranging between six to ten weeks. When the project is done abroad, six weeks is rather a short period to gain good insight into the project. Because of this, CF 599 has decided to combine the project and the practical work period.

The goal of the practical work period was to gain insight into the construction process of sand-storage dams built by SASOL. Construction plays a major role in the design of the dams. The construction method applied in building has a great influence on costs, quality and planning. In the field a good impression was gathered about the needs and possibilities of the future dam users and the organisational aspects of the counterpart, SASOL. The practical part also gave the opportunity to study the 50 dams that already had been visited, in more details. In other words: the practical work period served as a period of collecting information that can be used in the project.

1.2 Objective of this report

The objective of this report is to optimise and to give guidelines for the design of sand-storage dams.

1.3 Contents project report

In the next chapter a description of the problems and circumstances in Kitui District is given. The problem approach is presented together with the programme of demands and a comparison between Kitui District and other arid and semi-arid areas (ASALs). Chapter 3 deals with the different possibilities of storing water in ASALs. By means of a multi-criteria-analysis the most suitable dam type for Kitui District is determined.
In chapter 4 the 50 dams that have been documented are evaluated on different aspects. From this evaluation weak points of the SASOL sand-storage dams are found. Chapter 5 gives a method to determine the design discharge, on which the dams should be dimensioned. Improvements of the design are made in chapter 6. Finally conclusions and recommendations are given in chapter 7.

2. Backgrounds

2.1 Kitui District, Kenya

2.1.1 General

Kitui town is located in Eastern Kenya. It is the administrative centre for Kitui District, which is one of the twelve districts making Eastern Province. The district covers an area of 20,556 km^2, including 6,369 km^2 of the Tsavo National Park inhabited by wildlife. It is divided in eight administrative divisions. An overview map of Kenya, including the location of Kitui District, is shown in figure 2.1.

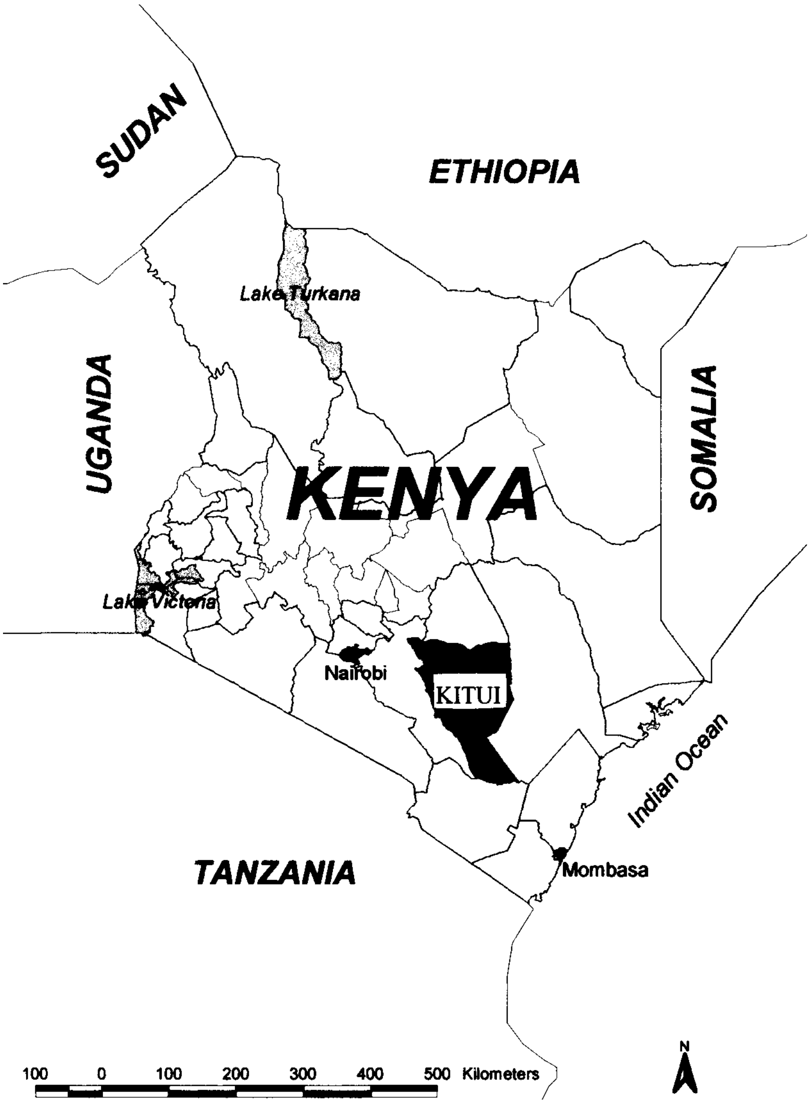


Figure 2.1: Location of Kitui District in Kenya [29]

Kitui District lies between 400 and 1,800 m above sea level. At the south-eastern side of the district lies the Yatta Plateau between the rivers Athi and Tiva. The eastern side is almost plain with shallow widely spaced valleys and with some hills. In the higher part of the district there is more rainfall and therefore more productive areas. The main crops in the district are corn and beans. Livestock production is a major economic activity. The majority of the rural households keep cattle either for meat, milk, pulling carts and/or ploughing.

The climate is hot and dry for most of the year as in arid and semi-arid areas. The rainfall is highly erratic and the rate of evaporation is high. One in three rainy seasons is a total failure. In the district there are two rain seasons, from April to June and from November to December. Dry periods are between June to September and January to February. The amount of rainfall depends on the part of the district and the topography. The higher areas receive 500 – 760 mm per year and the lower part less than 500 mm per year. The minimum temperature is 20 ºC and the maximum over 30 ºC. The climate makes intensive use of land hard. It is important to note that the poor and rich households draw 77 % and 22 % respectively of their incomes from agriculture. Irrigation potential along rivers has been only minimally exploited. With more use of the water in the (seasonal) rivers, a lot of cultivation would be done to increase food production in the district. The district normally experiences food deficit due to recurrent drought episodes. The little harvest is supplemented by relief food from donor agencies.

The population of Kitui District was estimated to be more than 574,000 in 1999 (Census Republic of Kenya, 1999) with a density of 213 persons per km^2 and is growing at a rate of 3.3 % per year. Sixty per cent of the households in Kitui are female headed and this has impact on the household human capital endowment. This is because of various reasons including men working outside the district, single parenthood and widowhood.

The main problem in the area is inadequate water for a large part of the population as there are only a few water sources such as rivers and springs. Since seepage is slow, there are long waiting hours at the water source to draw enough water. The major sources of water are perennial rivers. To access water, people and animals have to travel for long distances especially during the dry season. In some areas people walk for 25 - 30 km in search to water. Most rainwater is not harnessed. It finds its way to the Indian Ocean. If this rainwater can be stored, it can be used during the dry season.

2.1.2 Social and economic impact of the sand-storage dams project

By Milu Muyanga & Mutua Isika

The following assessment is based on observations, comments of the beneficiaries and the emerging results of a broad social and economic study being carried out by the authors. The definition “sand-storage dam” will be explained in section 3.3.

Impact on community

- In the past, people had difficulty extracting water from scoop-holes in the sand; both for their own use and for animals as the dry season progressed. As the water levels dropped, holes and scoop-holes got deeper risking the collapse of their sidewalls further risking trapping people and livestock.

- Construction of sand-storage dams impedes downstream flow and recharges the riverbanks, from which water returns as the dry season proceeds. This has the effect of maintaining a steady water level for long.

- If there is no sand-storage dam, there are additional problems in times of drought due to the slow rate at which water infiltrates into the scoop holes. This can mean that women have to queue to get access to the water. In some situations, women have been forced to queue overnight and men have been assigned to protect them from possible attack by wild animals or thugs.

- When the usual source of water dries up, people must go further and further a field to find a river where water is still available. The time and energy employed in fetching water-diverted attention from other tasks that could have been carried out in the dry season, such as terracing. It is not uncommon for women to spend three or more hours in a day fetching water in the dry season.

- When adverse scarcity of water, girls are often withdrawn from school to help fetch it. This has serious implications for their education. The education of girls and women is reckoned to have a greater long-term impact in reducing population growth than any other single factor. However, it has not empirically proved that improving water resources will contribute to slowing the rate of the population growth.

- The impact of sand-storage dams on health has already been recognized by some people but will be more noticeable as wells are constructed and the risk of pollution is reduced. When this is coupled with the wider adoption of pit latrines, boiling water for drinking and education on the causes of sickness, there will undoubtly be significant improvements in health. Furthermore, improvement in water supplies allows women to wash clothes in the dry season, a task that was difficult or impossible before.

- Apart from the direct impact of improving the supply of water, the project has an indirect impact on the morale of the community. First, PRAs have enabled members of the community to take a look at their own situation in ways they have never done before. It has encouraged people to think, to question, to analyse and to reflect. PRA has shown the importance of local knowledge and local observations. Through the PRAs, the communities have been able to assess their strength and weakness and to plan together for measures that will improve their livelihood.

- Second, the work of constructing sand-storage dams and wells by the community with the help of an artisan has shown community members what they can do with the resources of labour and materials that they already have. It has shown them that they do not have to wait for someone else to take the initiative and that there is much that they can do themselves to improve their own situation, using their own tradition of cooperative work. This has the effect of empowering the people and especially the women, who have in the past borne the biggest burden of producing the food and maintaining the family.

Mzee Malonza Ndunga (an elderly man):

"Before, we went to Nzeeu river in dry season for water. The women, children and donkeys went to Nzeeu from May to November. They left in the morning each day, one hour to go and one hour to come back. Before, this river (the Kiindu) was dry but now water is on standby full-time".

Juma Muthami (a 15 year boy):

"The water is coming up now more than before. Before, no one was growing vegetables but after the construction people are growing vegetables and fishing. Before, the sand was down (in the river bed) and the banks were high but now the banks are near (when you stand in the river). To build the construction, it was our mother and father who did it and they did it well".

Mama Mboovi Malonza (a woman):

"By now Kiindu river is good (better) than before. It is good because the water is high, people are growing the vegetables around. Before, water stayed (in the Kiindu river) January, February, March, April and May. In June it got dry and then we went to the Nzeeu river. From here to the Nzeeu it took one hour to go and one hour to come back. We went many people; we waited in line, next to next (sic), for 30 minutes (awaiting our turn to scoop the water). Children around 12 years helped me. They went to school at 7.00am and after school in the evening they went to fetch water from Nzeeu on their backs, reaching home at 7.00 in the night when it was already dark. I am happy because of this new mud brick house which we made since the water is near".

Adapted from 'Where There is No Water- a story of community water development and sand-storage dams in Kitui District, Kenya'

Figure 2.2: Comments of local people after installation of sand-storage dams on the Kiindu River

Impact on agriculture

- Tree nurseries have been established close to the dams and wells so that seedlings are now available. The identification of the most useful trees during PRA will ensure that trees that are useful and also well adapted to the area will be given priority.

- The time saved from fetching water is now available for other activities (e.g. terracing- which leads to conservation rainfall and improvement in producing the main food crops).

- The production of vegetables in plots close to the river can provide income especially in the dry season, and lead to improvements in nutrition. Increased planting in riverbeds of sweet potato varieties that can be grown and harvested in the long dry season, from June to October, will also improve food supply.

- Improvement in the water will reduce the time that livestock used to trek to watering points and the risk of diseases associated with this trek. It has also led to improvement in supply of forage due to the extensive planting of Napier grass at the river edges.

Impact on environment

- Some of the impacts already mentioned such as trees, terracing, and growing vegetables and fodder grass close to the sand-storage dams imply changes in the environment.

- Other more direct impacts include rising the bed level of the river by installing dams reduces the erosion of the riverbanks and of the watercourses leading into the river. Also, raising the water table encourages vegetation along the riverbanks and improves the stability of the banks. Improved water retention has encouraged the planting of Napier grass alongside the riverbanks. Where both sides are planted in this way, the river is confined to the centre of the channel, but if only one side is planted, it may force the river towards the opposite side thus culminating in riverbank erosion.

- Preliminary data seem to show that there is more water down stream than in the past as the overall flow is slowed. It is likely that after a long dry season when much of the water in the sand-storage dams had been extracted, a small rainstorm in the upper part of the catchment will recharge only the upper dams. However, where there is more general rainfall, the dams should be recharged by runoff from adjacent areas as well as by water coming down the river from the upper part of the catchment.

- It might be thought that sand-storage dams would lead to flooding of the land adjacent to the river and thus damage crops. This has not been a significant problem and can be explained by the relatively steep gradient of the riverbeds. On a few sites, there is already evidence of lateral ground recharge as new water demanding vegetation has appeared. These plants provide fodder for animals. Where wetlands are being created, there are new opportunities for growing specialized crops such as vegetables.

- Sand-storage dams not only provide storage for water, but for sand too. During times of flood, the lightest soil particles tend to be carried on down the river while the heaviest- the sand- are deposited wherever storage has been created. Sand is a resource for building, but its use must be controlled to avoid negative consequences like cleaning out the riverbeds.

2.2 SASOL Foundation

SASOL is a non-governmental-organisation (NGO) and was founded in 1990. The name SASOL stands for Sahelian Solutions. Sahelian is an Arabic word for Arid and Semi-Arid Lands (ASALs). The population in ASALs all over the world is growing or will be growing in the next decades. The high potential areas cannot supply the needs of the growing populations, so people have to move to less favourable areas. These are the areas where SASOL focuses on. The major need in these areas is water. If the lack of water in these areas is solved, opportunities arise for the population by increasing the productivity. Not only the livestock and crop production, but also the industrial production that is now absent in these areas.

SASOL focuses on satisfying the important basic need, water. With the funding from WaterAid, a NGO from the UK, SASOL started a pilot project on the Kiindu River in 1995. This pilot enabled SASOL to test the initial thinking and the practicality of instituting the system. So the concept of building dams for water supply could be approved and the pilot project was followed by "25 sand dam projects" on the same Kiindu River catchment and was funded by WaterAid too. In 1998, WaterAid was wound up and ended the funding on dams. In 1994 the Swedish International Development Authority (SIDA) and in 1996 the United Kingdom Department for International Development, (DFID) started to fund dams in different rivers in the Kitui District.

By April 2001, SASOL had already built 216 dams in different rivers in an area of 600 km^2 in the Kitui District. Although not all dams are working (a few have been broken down by El Niño in 1997), most dams have a great impact on the local communities.

The main objective of SASOL is to prove that simple, low-cost technology for water infrastructure can improve the lives of people in arid and semi-arid areas, so that other development organisations or governments can copy this technology for water infrastructure. To prove this, SASOL's target is to supply water within 2 km of every home throughout the year in the Kitui District compared to the 10 km women and children used to walk before to fetch water.

The approach of SASOL is to develop a catchment in total. Construction of sand-storage dams is the base on which the community builds other activities. The catchment approach depends on the co-operation by the community in developing sequential sand-storage dams in their dry rivers coupled with terracing and tree planting on individual plots. The projects are community driven and managed.

SASOL uses a bottom-up approach when implementing the projects by letting the local community members define their own problem, set their own priorities and make their own decisions on how to solve it. SASOL provides the facilities, resources and, if necessary, the required funding.

The projects are run by:
- 1 Field Manager
- 1 Technical Manager
- 1 Community Manager
- 1 Administration Assistant
- 2 Group leaders for the masons
- 15 Masons

In 1999 SASOL's turnover was approximately Ksh 9.6 million (about NLG 320.000,-).

In the future, SASOL plans to build dams in the southern divisions of Kitui District, bordering the Tsavo East National Park. The population density in this region is lower and the soil differs from the situation in the present project area. It is a challenge for SASOL to prove that this technology for water infrastructure can also work in this area. To finance these dams, SASOL is likely to get funds from the European Union for the coming year (2002).

2.3 Problem description

2.3.1 Problem analysis

The construction of sand-storage dams by SASOL is mainly based on experience. Before construction some minor calculations are done, but the determination of dimensions and location is still a matter of trial and error. Because SASOL wants to build dams more efficiently and economically, a research on construction methods is necessary.
SASOL is very careful with its reputation. Mobilising the local people to build a dam is not very easy in most cases. The construction of a dam is a lot of work, which, most of the times, is carried out by women. Because of this effort, failures are not acceptable.
Mobilising an entire community for the second time after the collapse of a dam is difficult. The reputation of SASOL will suffer from these failures and people will have less faith in the dams. The result is, that SASOL is conservative in constructing the dams. The dams are often over-designed.

To be able to construct more efficiently and thus build more dams using the same amount of money, knowledge is required about the technical background of the dams.

Another reason to learn more about the technical background of the dams is the wish of SASOL to extend its activities to other areas. When more is known about the technical aspects the knowledge can be spread and used in other ASALs.

SASOL also wants to know what the effects of the dams are on the surrounding areas. The influence of the dams on the height of the groundwater table is yet not known. The efficiency of the dams is still difficult to determine, since the reservoir capacity can hardly be determined.

Now the concept of sand-storage has proved itself, SASOL is interested in other dam types and alternative construction methods.

2.3.2 Problem definition

- There is a lack of knowledge on the technical background of sand-storage dams built by SASOL. This lack of knowledge hinders SASOL from optimise the dam design.
- There is a lack of documentation about the design and the building process of sand-storage dams so that copying of this technology for water infrastructure for other development organisations or governments is difficult.
- Many effects of series of sand-storage dams are claimed, but none of them has been proved scientifically. Not much is known about the characteristics of the reservoirs of sand-storage dams, although it is an important aspect of sand-storage dams. For projects in the near future SASOL wants to know more about the availability of groundwater around the dams.

2.3.3 Objectives

The objectives of the project are:
- To optimise the design of sand-storage dams.
- To make a manual on location choice, design, construction and maintenance with emphasis on the specific problems in ASALs.
- To set up a proposal for a hydrologic research on the effects (of a series) of sand-storage dams

In this report the design for sand-storage dams is optimised.

2.3.4 Problem approach

To pursue the objective a thorough literature search was done in the Netherlands. This literature search was meant to gain insight in the different options for storing water, hydrological models and constructional theories.
After this 50 dams were visited and evaluated in Kitui District. The evaluation gave information about strong and weak points of the current design of the SASOL sand-storage dams. During the field trips a good insight was gained of the circumstances in Kitui and the effects of the dams on the surrounding. The evaluation gave information about technical aspects of the dams. Also information about users, draw-off systems and the area of the dam was gathered.
To be able to see the actual construction of the dams, a period of 6 weeks was spent in Mangina, a small town in the centre of a SASOL project area. During this stay in the countryside the design, the construction process and general aspects about building sand-storage dams were documented and evaluated. Because the design strongly depends on the possible construction methods, this information is very useful to the project. Also the necessary knowledge for a construction plan was obtained from the Mangina-period.
In this report an overview is given of the different options of sand-storage techniques and the most suitable technique for Kitui District is determined. The evaluation forms are studied, so that the weak points of the dams can be found and, if possible, approved. The dimensions of the dams depend on the discharge and the height of the water table during design conditions. A method has to be found to determine this design discharge.

A manual on location choice, design, construction and maintenance containing an overall construction plan, maintenance plan and user plan is made using all information that is gathered from the field trips, the literature search and the practical work period.

Finally, a proposal for a further hydrologic research is made.

2.4 Restrictions

2.4.1 Boundary conditions

Technical
- The construction should be built only using simple equipment.
- No skilled workers are needed to build the construction.
- The construction should be designed and dimensioned in such a way that it can withstand the force of overflowing water.
- The construction has to store water.

Functional & community
- The construction method must be appropriate for other areas of Kitui District.

Financial & economical
- The life expectancy of the dams should be at least 25 years.

2.4.2 Starting points

Technical
- The strength of mortar and aggregate stones differs considerably from site to site.
- Constructions are built whole year through, because the masons are hired on annual basis.
- The construction has to be watertight.
- No or little maintenance should be needed.
- The suitability of the different construction types is determined for Kitui District, but the construction should also be suitable for arid and semi-arid areas with the same characteristics.
- The materials used should be as durable as possible.
- Vulnerable materials should be avoided as much as possible.
- Materials that are available in the surrounding of the construction site should be used to construct the dam with, because they will lower the cost.

Functional & community
- Seepage should be as less as possible.
- The building process has to be suitable for a community-based approach.

Financial & economical
- The dam must be constructed at the lowest possible price per m^3 storage.

2.4.3 Assumptions

Functional & community
- There will be enough people to build the dam.
- Community workers are not paid.
- The community provides the needed materials for the construction, like stones, sand and water.
- After completion, the community is responsible for the construction.
- The community and a craftsman will look together for the best location to build the construction.

2.5 Comparison problems and circumstances Kitui District with similar areas

It is estimated that 35 - 40 % of all Kenyans live in the ASALs (Arid and Semi Arid Lands). Arid means that the potential evaporation in an area is higher than the precipitation. All over the world arid and semi-arid areas are found. Many developing countries are located in climatic regions where rainfall is seasonal and highly erratic.
Supplying water in these regions is to a large extent a matter of storing water from the rainy season to the dry season, just like in Kitui District.
One method of storing water is to use sub-surface and sand-storage dams. It is a very old technology and it has frequently been used in Kenya during the colonial period. Many dams have recently been built in Kenya, 216 by SASOL in Kitui District and 115 near Machakos, but the technology has also been used in the USA, India, East and North Africa and Brazil. Figure 2.2 shows potential dam construction areas.

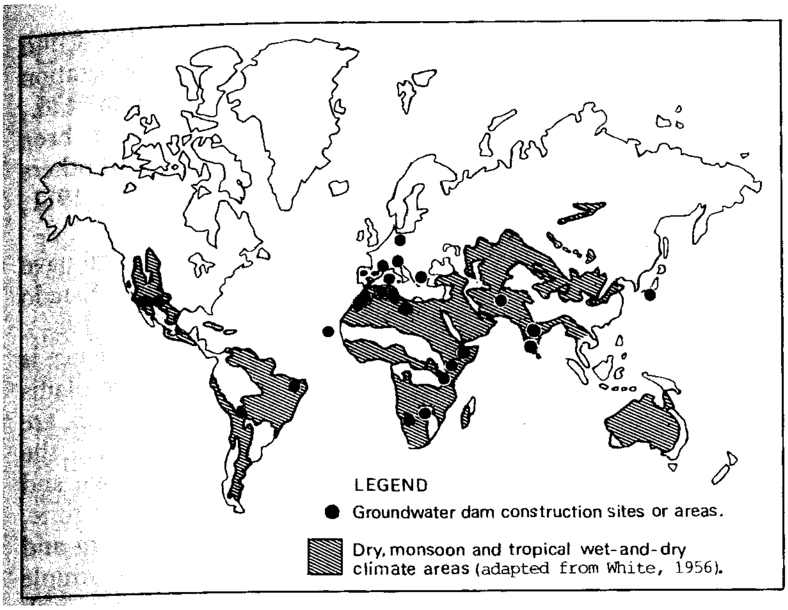


Figure 2.3: Identified groundwater dam construction areas [20].

It is one of the objectives of SASOL to prove that the sand-storage dam technology is very effective. The project area of SASOL shows a wide variation of circumstances on community organisation, topography and geology to test the technology.
According to SASOL the sand-storage technique can be useful for (estimated) 30 % of Kenya.

Some features of these areas are given as well as the suitability of SASOL's construction approach.

Climate
Sand-storage dams and shallow wells are likely to have greatest application in the transitional zone that lies between the sub-humid and the semi-arid zones and has a mean annual rainfall between 700 mm in the drier and 900 mm in the wetter areas. In wetter areas greater opportunities exist for trapping springs and streams for distribution of water, because the water table is higher and shallow wells can produce enough for water.

Topography and geography
In hilly zones the run-off is high. This makes it very hard to hold the precipitation from the rainy season. Sand-storage dams are suitable, because base rock is found near the surface. Also narrow valleys or riverbeds can be found, which reduces costs and controls possible

seepage to the banks. On the other hand it might be difficult to find an acceptable relation between height of the dam and storage volumes in too steep riverbeds.
Important for sub-surface dams is the depth and lateral extent of the deposits of the riverbeds. The optimum relation between these factors is generally found on the gentle slopes in the transition zone between hills and plains. Finding suitable places to build the dam is harder when the river is wider. For an efficient reservoir it is important that impermeable beds or bedrock are underlying the reservoir [20]. SASOL argues that their design can be based on clay.
In general the topographical gradient of the construction sites is between 0.2 - 4 %, but in extreme cases dams have been constructed on slopes of 10 - 16 % [20].

Water quality
Water quality is important. Rivers from towns and trade centres can be polluted due to industries and urban waste. The Kalundu River, which passes through Kitui town, is not included in SASOL's sand-storage dam programme for this reason. Economical growth and intensive agriculture can also increase the use of pesticides that pollute the water.

Community
SASOL's approach is community-based. This calls for a great involvement of the people and it is likely to succeed only where people have a tradition of working together to solve communal problems. [29] Due to out-migration of a large section of the able-bodied men in Kitui in search of employment, communal work will be harder to organise, where population densities are less than 100 persons per km^2. [29]

Construction
'Stones, sand and water can be found in the surrounding' and 'the community delivers 'free' labour', are boundary conditions for the SASOL approach. Materials and labour are not used very effective, but the advantage is that only one craftsman is needed. When one or more of these materials are expensive or not available or when labour is expensive, another construction method could be useful.

The information that is used comes from '*Groundwaterdams for small-scale water supply* ' [20], '*Where there is no water* ' [29] and from field experience.

3. Suitable dam types for Kitui District

3.1 Introduction

In this chapter different groundwater dam types will be discussed. The focus will mainly be on groundwater dams, since these dams have a lot of advantages compared to surface dams. The water in these dams is stored below the surface. There are two types of groundwater dams, sub-surface dams (see section 3.2) and the sand-storage dams (see section 3.3).

Advantages of groundwater dams are:

- Evaporation is confined to the upper layer of a sand reservoir. As the water level sinks, the evaporation is reduced and even completely stopped, when the water level sinks to about 60 cm below the sand surface. [12]
- Contamination of the water by insects and animals cannot take place because the water is not visible at the sand surface. Health hazards such as musquito breeding are avoided.
- When conventional surface storage is used, it means that land is occupied for the reservoir; in the case of groundwater dams the land above the stored water can be used for other purposes.
- Siltation is not a problem for groundwater dams, in contrast with surface dams, which would silt up quickly.
- Recharge of water reservoirs in sand rivers takes place as soon as rainfall has produced sufficient run off, to flood a sand river. Run-off depends on the kind of surface as shown in table 3.1.
- Maintenance is simple.
- Construction costs are relatively low.

Disadvantages are:

- The size of the voids between the sand grains determines the capacity of the basin. When the particle size is small, only 5 % of the water can be extracted (see table 3.2).
- The slopes of the banks alongside the river determine the size of the sediment that will be trapped in the reservoir.
- Survey, design and construction processes call for trained persons so as to avoid possible failures.
- Deep-rooted vegetation must be avoided in the reservoir, because evapo-transpiration will increase by the deep-rooted plants.

Table 3.1: Run-off factors from various types of catchments [24]

Catchment:	**Roofs**	**Rocks**	**Roads**	**Hills**	**Eroded lands**	**Conserved lands**
Run-off	95 %	90 %	40-70 %	20-40 %	20-70 %	0-10 %

Table 3.2: Extractable water from silt, soil, sand and gravel [24]

Material	Silt	Fine Sand	Medium sand	Coarse sand	Fine gravel	Gravel
Size (mm)	<0.5	0.5-1	1-1.5	1.5-5	5-19	19-70
Sample (l)	4.00	4.00	4.00	4.00	4.00	4.00
Saturate (l)	1.52	1.58	1.63	1.80	1.87	2.05
Porosity (%)	38.0	39.5	40.8	45.0	46.8	51.3
Extract (l)	0.18	0.75	1.00	1.40	1.65	2.00
Extractability (%)	5	19	25	35	41	50

Recharging of groundwater dams
Underground reservoirs in sand-rivers are recharged by rainwater from flash floods, which originate in catchment areas with higher elevation. A single and short-lived flash flood may fully recharge a reservoir with water.
Upon full saturation of the reservoir, the remaining flash floods will pass over the dam without any more infiltration. A few hours after a flash flood has passed over a dam, the surface of the sand-river may look dry again, but water will have been stored in the reservoir that can be drawn for many months. [23]

Because groundwater dams and small-scale water supply systems seem to be the most suitable solutions for Kitui District, these systems are discussed in this chapter.

3.2 Sub-surface dams [20], [21]

General principle
A sub-surface dam is constructed below ground level and arrests the flow in a natural aquifer. Before construction of a sub-surface dam, a trench is dug, reaching down to an impervious layer. An impervious wall is constructed in the trench, and finally the trench is filled up again with the excavated material. The actual stored volumes of sub-surface dams range from 100 m^3 to 1.10^6 m^3. The depth of sub-surface dams is usually 3-6 m. Nine alternatives for sub-surface dams are presented in the following sections.

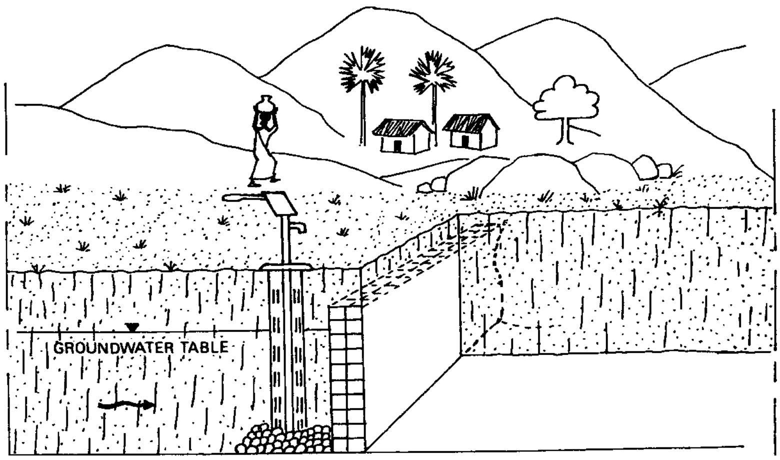


Figure 3.1: General principle of a sub-surface dam [20]

3.2.1 Clay dike

Advantages: - No need for skilled labour
- Materials are often nearby and cheap

Disadvantages: - Risk of erosion due to groundwater flow (clay not properly compacted)
- Risk of cracking
- Only suitable for limited depth because of the use of simple construction methods

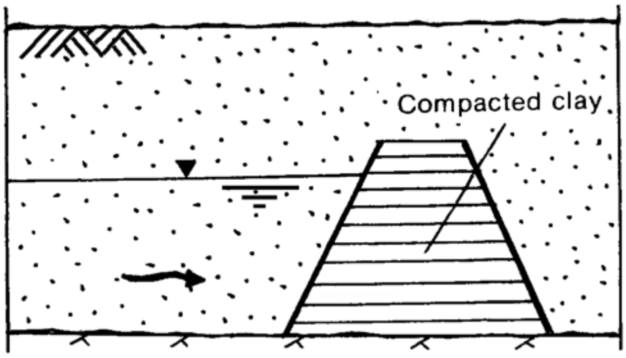


Figure 3.2: Clay dike (for key of symbols see page XV) [20]

3.2.2 Concrete dam

Advantages: - Possibility to raise the dam easily above the riverbed for further sand accumulation.
- Good watertight solution

Disadvantages: - Use of formwork
- Availability cement, sand and gravel in the area
- High costs
- Skilled labour needed

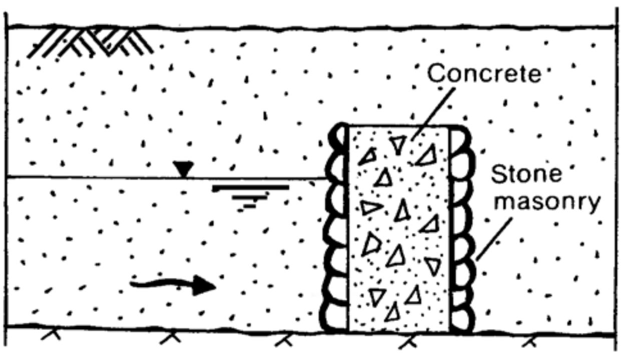


Figure 3.3: Concrete dam (for key of symbols see page XV) [20]

3.2.3 Stone-masonry dam

Advantages: - Possibility to raise the dam easily above the riverbed for further sand accumulation.
- Relatively low costs

Disadvantages: - Availability cement, sand and stones in the area
- Skilled labour needed, especially for the stone-masonry work

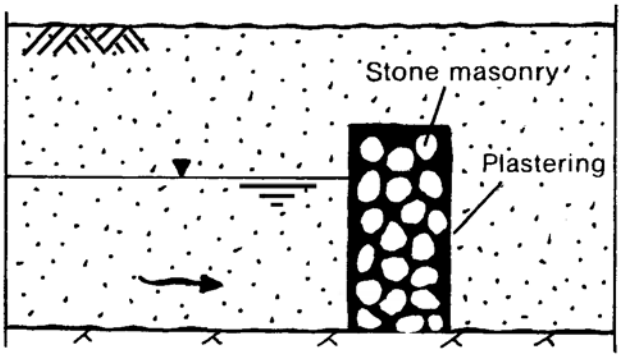


Figure 3.4: Stone-masonry dam (for key of symbols see page XV) [20]

3.2.4 Ferroconcrete dam

Advantages: - Little material is needed to achieve a strong wall

Disadvantages: - Use of steel (expensive)
- Use of formwork
- Availability cement, sand and gravel in the area
- Good anchoring demanded
- Skilled labour needed
- High costs

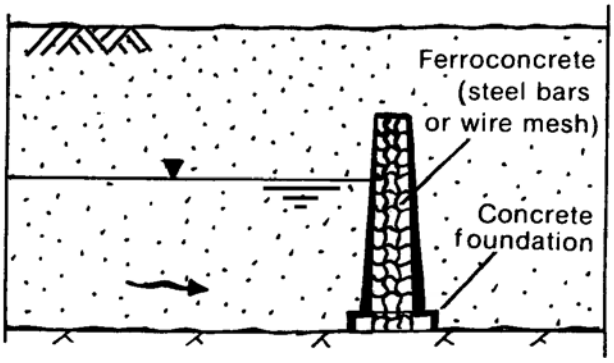


Figure 3.5: Ferroconcrete dam (for key of symbols see page XV) [20]

3.2.5 Brick wall

Advantages:
- Bricks generally available/manufactured from local clay
- Simple building procedure
- Possibility to raise the dam above the riverbed and further sand accumulation.

Disadvantages:
- Relatively high costs of bricks
- Stability is poor because the low shearing strength

Note: When clay is not available, sandcement blocks could be used. Sandcement block is a brick manufactured from a mixture of cement and sand (1 cement : 10 sand).

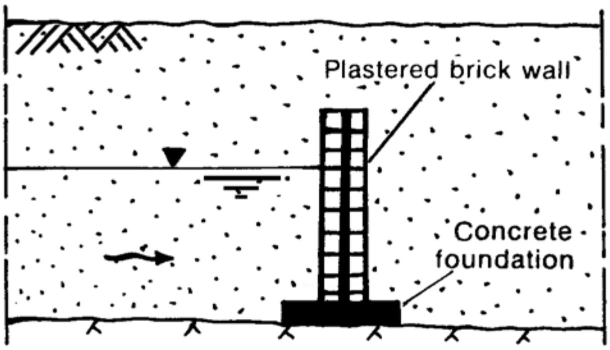


Figure 3.6: Brick wall (for key of symbols see page XV) [20]

3.2.6 Plastic sheet

Using thin sheets of impermeable materials such as tarred felt or polyethylene.

Advantages:
- Low material cost
- Simple building procedure

Disadvantages:
- Complicated construction process
- Material highly sensitive during the erection and refilling of the trench
- Minor rip will cause leakage problems
- Joints between sheets become weak points for seepage
- Material is vulnerable

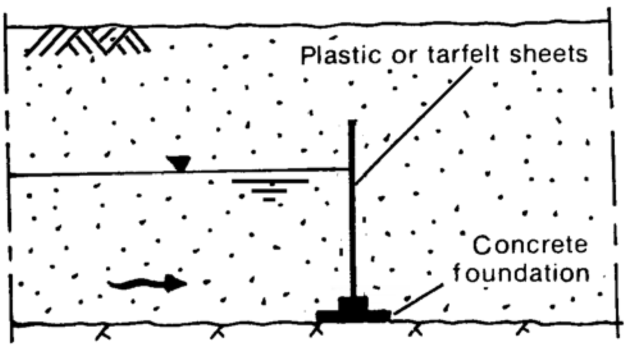


Figure 3.7: Plastic sheets (for key of symbols see page XV) [20]

3.2.7 Steel sheets (corrugated iron)

Advantages:
- Low material cost
- Simple building procedure
- Sturdy and impermeable construction
- No need for pumping groundwater during construction
- Possibility to raise the dam above the riverbed and further sand accumulation.

Disadvantages:
- Sensitive to corrosion
- Joints between sheets are vulnerable to seepage

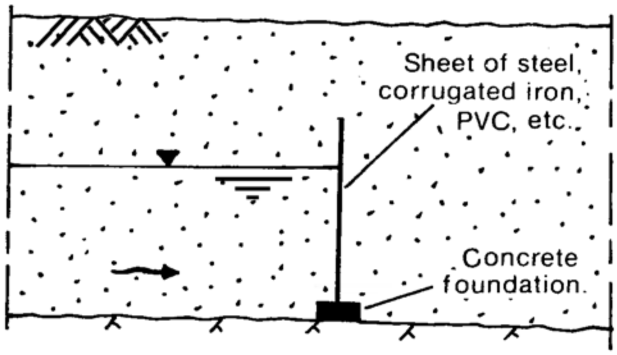


Figure 3.8: Sheets of steel (for key of symbols see page XV) [20]

3.2.8 Injection screen

Advantages:
- Injection can be carried out without draining aquifer
- Feasible for large deep aquifers

Disadvantages:
- Expensive solution
- Complicated construction process
    - Skilled labour needed

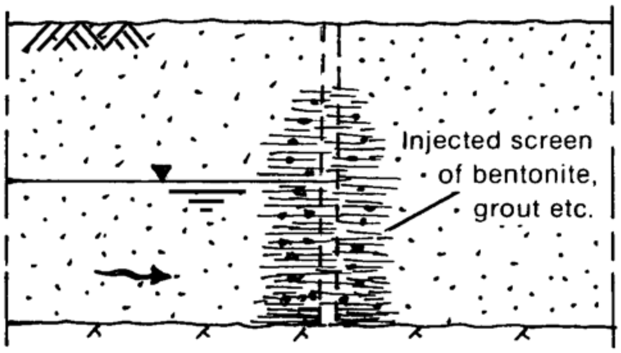


Figure 3.9: Injection screen (for key of symbols see page XV) [20]

3.2.9 Natural sub-surface dam

Of course the natural sub-surface dam is the simplest and cheapest form of sub-surface dams (see figure 3.10). The natural sub-surface dam exists of base rock.

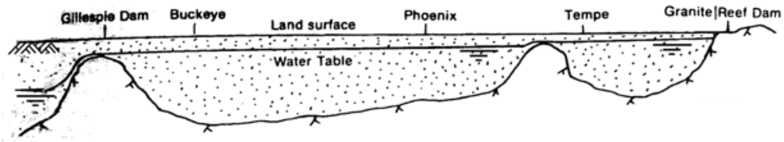


Figure 3.10: Natural sub-surface dam with water hole (for key of symbols see page XV) [23]

3.3 Sand-storage dams [20]

General principle
By the construction of a dam across the riverbed, sand carried by flow during the rainy season, will settle in front of the dam and gradually the reservoir in front of the dam will fill up with sand. The sand is used to store water from the rainy season for use in dry periods. Sometimes the dam is built up in different stages. The idea is to keep a sufficiently high velocity in the reservoir so that light particles cannot settle. The stored volume of a sand-storage dam ranges from 100 m^3 to 50,000 m^3. The typical height of a sand-storage dam is 1-4 m.

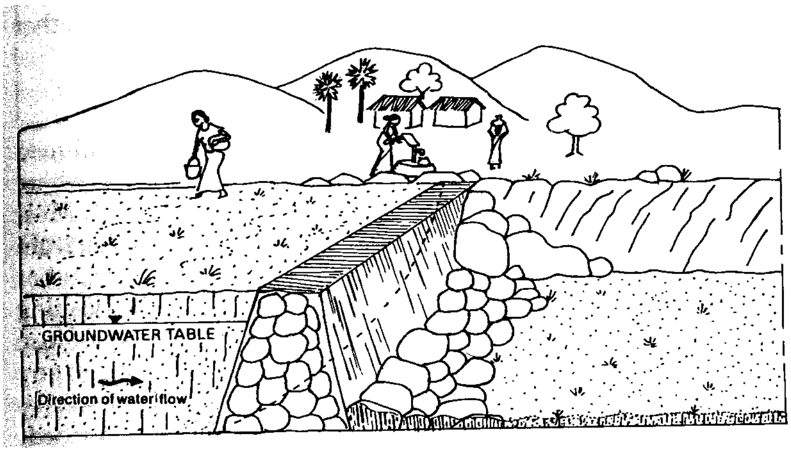


Figure 3.11: General principle sand-storage dam (Note: Danger of downstream erosion at the toe of the dam) [20]

In the next sections some types of groundwater dams are shown. The advantages and disadvantages of these alternatives depend mainly on the availability of the used materials.

3.3.1 Concrete sand-storage dam

Advantages: - Good watertight solution

Disadvantages: - Needs formwork (timber)
- Availability cement, sand and gravel in the area
- High costs (much cement is needed)
- Skilled labour needed

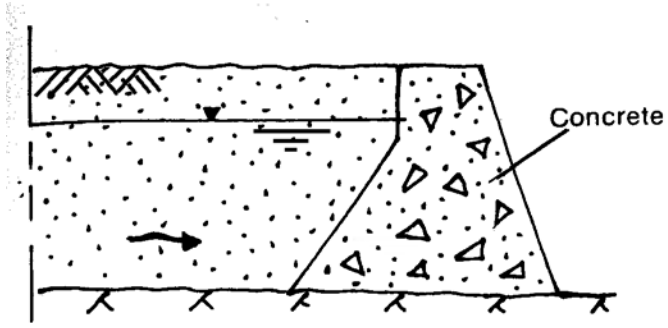


Figure 3.12: Concrete sand-storage dam (for key of symbols see page XV) [20]

3.3.2 Stone-masonry sand-storage dam

Advantages: - Good watertight solution
- Relatively low costs

Disadvantages: - Availability cement, sand and stones in the area
- Skilled labour needed, especially for the stone-masonry work

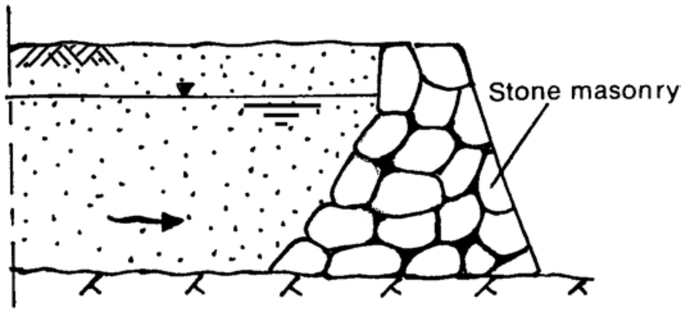


Figure 3.13: Stone-masonry sand-storage dam (for key of symbols see page XV) [20]

3.3.3 Gabion sand-storage dam with clay cover

Advantages: - Relatively low costs
- Skilled labour not needed

Disadvantages: - Availability clay and stones in the area
- Clay cover has to make the dam watertight
- Gabions must be heavy enough to withstand current velocity

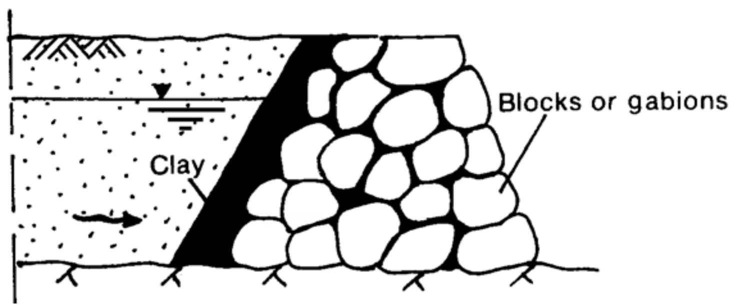


Figure 3.14: Gabion sand-storage dam with clay cover (for key of symbols see page XV) [20]

3.3.4 Gabion sand-storage dam with clay core

Advantages:
- Relatively low costs
- Skilled labour not needed

Disadvantages:
- Availability clay and stones in the area
- Clay core has to make the dam watertight
- Gabions must be heavy enough to withstand current velocity

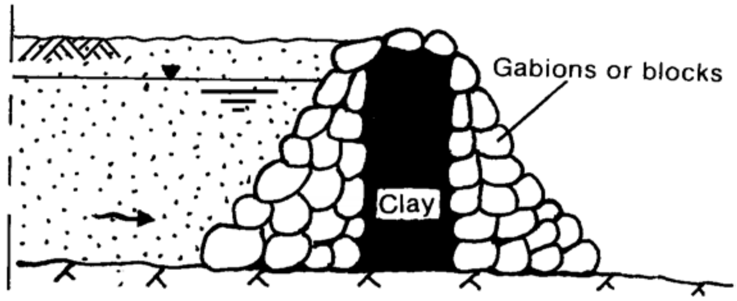


Figure 3.15: Gabion sand-storage dam with clay core (for key of symbols see page XV) [20]

3.3.5 Stone-fill concrete sand-storage dam

Advantages:
- Concrete walls for stability and tightness
- Can be used as bridge
- Good watertight solution

Disadvantages:
- Availability cement, sand and stones in the area
- Needs formwork (timber)
- Skilled labour needed
- Complicated construction process

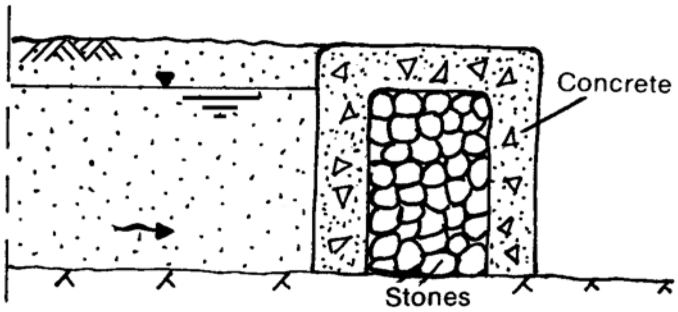


Figure 3.16: Stone-fill concrete sand-storage dam (for key of symbols see page XV) [20]

3.3.6 Stone sand-storage dam

Advantages:
- Skilled labour is not needed
- Relatively low costs

Disadvantages: - Availability flat stones in the area to pile up
- Seepage through dam

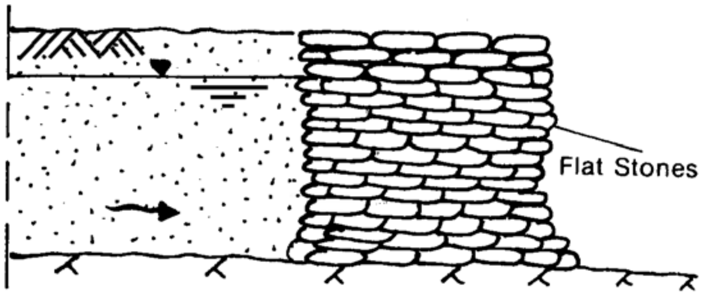


Figure 3.17: Stone sand-storage dam (for key of symbols see page XV) [20]

3.3.7 Concrete arch dam

Advantages: - Good watertight solution

Disadvantages: - Requires rock abutments
- Difficult to bend reinforcement bars
- Needs formwork (timber)
- Availability cement, sand and gravel in the area
- Skilled labour required
- Only economically suitable for high dams in narrow rock gorges

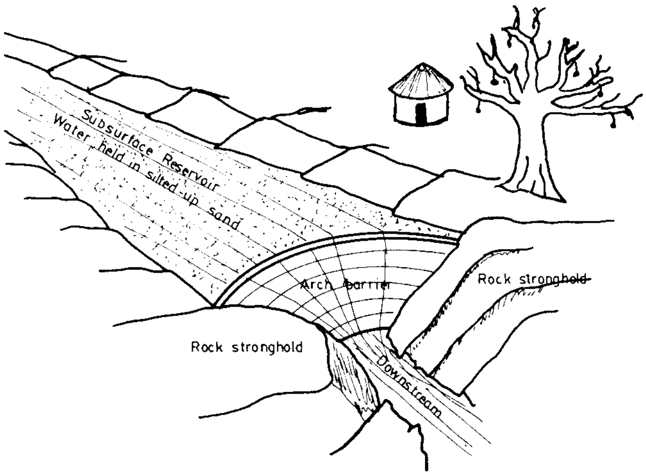


Figure 3.18: Concrete arch dam (Note: Downstream toe of the arch dam should be well protected or founded on base rock) [21]

3.3.8 Dam and crossing

Advantages: - Road is situated behind the spillway

Disadvantages: - Road is exposed to high current velocities and axle loads
- Problems can be expected (erosion downstream side construction)

Figure 3.19: Dam and crossing

3.3.9 Sand-storage dam enlarged from an initial sub-surface dam

Option when reservoir capacity is not sufficient

3.4 Alternative storage possibilities

In this section two other storage possibilities are given for water storage avoiding the presence of surface water.

3.4.1 Roof catchment

A roof catchment can provide clean water throughout the year. The water can be used for domestic use. The water is stored in a tank near the house. Disadvantage of the storage is the relatively high cost of a water tank. Another disadvantage is the house must have a suitable roof, which is very expensive if it is not present.
If more water is needed, one could also think about a compound catchment.

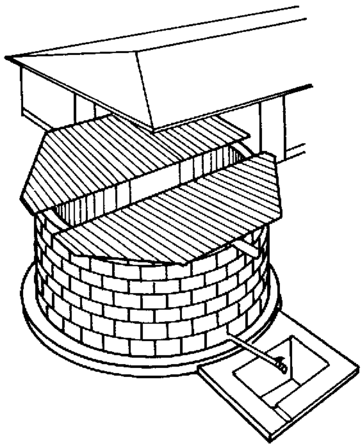


Figure 3.20: Roof catchment [21]

3.4.2 Compound Catchment

A compound catchment can collect about half of the annual run-off. The compound around the house or roads can be used. The water can be used for live stock and irrigation. Water does not need to be stored in a sophisticated water tank. It can just be stored in a waterhole with a clayish surface.

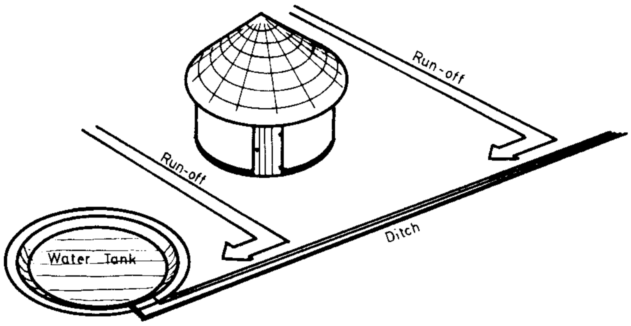


Figure 3.21: Compound catchment [21]

3.5 Multi-Criteria-Analysis

3.5.1 Dam criteria

To be able to find the most suitable dam-type for Kitui a Multi-Criteria-Analysis is used. This analysis is meant to get insight in the suitability of the construction. The solution of the analysis is not strict, but is a tool to determine the most suitable dam type in a restricted situation and area. For other situations or areas the outcome of the analysis can be different. Since it is not possible in this case to determine the scores of the alternatives with quantitative facts the rewarding is subjective. The Multi-Criteria-Analysis is just a simple method to get insight in the suitability of the different dam types in Kitui.
First of all the criteria have to be formulated. The criteria arise from the starting points (2.4.2). All criteria have their own weight factor, according to their importance to SASOL.

Criteria	*Weight factor Sub-surface dams*	*Weight factor Sand-storage dams*
1. Costs	25	25
2. Maintenance	10	15
3. Durability	10	10
4. Vulnerability	10	10
5. Suitability Kitui District	10	10
6. Construction	20	20
7. Extendable	10	5
8. Water tightness	5 +	5 +
Total	100	100

The different criteria are clarified below:

1. Costs
   The first criterion is about the relation between construction costs and the volume of stored water. A high score means low costs per volume stored water, a low score means an expensive solution.

2. Maintenance
   This criterion is about the maintenance that is needed during the lifetime of the construction. A high score means little or no maintenance, a low score means much maintenance to keep the structure in a good condition.

3. Durability
   The durability criterion gives the durability of the materials that are used in the construction. Materials that tend to wear quickly get a low score, wear-resistant materials get a high score.

4. Vulnerability
   The vulnerability criterion describes the reliability of the construction and its response to exposed forces. Strong structures like concrete dams get a high score, vulnerable structures get a low one.

5. Suitability Kitui District
   Suitability Kitui District deals with the presence of the needed construction materials in Kitui District. Dams that are built with materials that are found in Kitui District will get a high score. Dams that are built with materials from outside the district or even Kenya will get a low score.

6. Construction
   This criterion describes the suitability of the construction technique for community based building. Most techniques that are simple and require a lot of manpower are good for community building and get a high score. Construction techniques that are complicated and require craftsmen will get a low score.

7. Extendable
   This criterion indicates the suitability of the alternative to raise the dam after it has been finished. This criterion is more important for sub-surface dams, than for sand-storage dams since sand-storage dams are mostly built to the maximum height at once.

8. Water tightness
   Good water tightness will get a high score, poor water tightness gets a low score.

3.5.2 Dam type analysis

First of all a choice has to be made between the different dam types. After that the design and the material-use can be determined. A choice has to be made between sub-surface dams (see section 3.2), sand-storage dams (see section 3.3) and the two other types of storage (see section 3.4).

The roof and compound catchments are relatively expensive. The price for a m^3 of stored water stored is high, compared to groundwater storage. For example: a 45 m^3 tank in Kitui would cost Ksh 90,000 to construct (1999), while a typical sand-storage dam would cost Ksh. 225,000 and stores approximately 4,000 m^3. [28]

The focus is now on sub-surface dams and sand-storage dams. The general principle is about the same. The difference between the two types is the height above the river bottom level. In general sand-storage dams are used when the topographical gradient of the slope of the river bottom is high and sub-surface dams are used when the topographical gradient of the slope is low. In the case of Kitui most dams are built in hilly areas with rather steep slopes.
If the impervious layer is near the surface, the storage capacity of the basin will be low. A sand-storage dam will enlarge the storage, while a sub-surface dam only uses the storage capacity that is already present under the surface. When the river is narrow and has high banks the storage capacity of the dams can easily be enlarged. The banks give a good anchoring possibility.

An advantage of sub-surface dams over sand-storage dams is the simple design. Water does not flow through the dam and no spillway is needed. The sub-surface dam is not exposed to the forces of flowing water as a sand-storage dam.

In table 3.3 the different qualities of the two dam-types are listed.

Table 3.3: Qualities of sub-surface and sand-storage dams

Sub-surface dams	Sand-storage dams
• Low topographical gradient of the slope of the river bottom • Simple design, no spillway • Uses storage capacity below surface only • Wide gentle rivers	• High topographical gradient of the slope of the river bottom • Spillway is needed • Increase of storage volume possible • Narrow river, steep banks

It is clear that different circumstances ask for different solutions. As can be seen from table 3.3, the sand-storage dam is much more applicable in Kitui District. Nevertheless some times the sub-surface technique is used by SASOL.

A total of 18 different sub-surface and sand-storage alternatives have been summed up in sections 3.2 and 3.3. An overview is given in table 3.4.

Table 3.4: Different storage options for Kitui District.

Sub-surface options Kitui District	Sand-storage options Kitui District
• Clay dike • Concrete dam • Stone-masonry dam • Ferroconcrete dam • Brick wall • Plastic sheet • Sheets of steel • Injection screen • Natural sub-surface dam	• Concrete sand-storage dam • Stone-masonry sand-storage dam • Gabion sand-storage dam with clay cover • Gabion sand-storage dam with clay core • Stone-fill concrete sand-storage dam • Stone sand-storage dam • Concrete arch dam • Dam and crossing • Sand-storage dam enlarged from an initial sub-surface dam

Because of the boundary conditions some options are not possible. For example: An injection screen is not realistic because of the high construction costs and the skilled labour that is needed. A selection of the realistic options for Kitui District is stated below in table 3.5.

Table 3.5: Selected options Kitui District

Sub-surface options Kitui District	Sand-storage options Kitui District
1. Clay dike 2. Concrete dam 3. Stone-masonry dam 4. Ferroconcrete dam 5. Brick wall 6. Plastic sheet 7. Sheets of steel	8. Concrete sand-storage dam 9. Stone-masonry sand-storage dam 10. Gabion sand-storage dam with clay cover 11. Gabion sand-storage dam with clay core 12. Stone-fill concrete sand-storage dam 13. Stone sand-storage dam 14. Concrete arch dam

The different options that are considered get a score for the different dam criteria (see 3.5.1) between 1 and 10. The worst alternative is rewarded 1, the best one gets 10 points. The scores are multiplied by a weight factor and the total of this gives the total score of each option. The result of these calculations can be seen from table 3.6 and 3.7. Also the total score of each option without weight factor is given.

The first column gives the individual score of each option for all 8 criteria. The second column is the result of the multiplication of the individual score with the weight factor for a criterion (see section 3.5.1). The total score without weight factor can be seen from the total of the first column. The total score with weight factor is given in the second column. The last row gives the ranking of the different options. The best option is ranked 1, the worst one 7.

Table 3.6: Individual scores sub-surface dams

Option	1		2		3		4		5		6		7	
Criteria	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	10	250	3	75	4	100	5	125	3	75	10	250	5	125
Maintenance	2	20	10	100	9	90	5	50	5	50	4	40	5	50
Durability	2	20	8	80	9	90	5	50	5	50	4	40	4	40
Vulnerability	2	20	10	100	9	90	5	50	5	50	2	20	8	80
Suitability Kitui	4	40	8	80	8	80	6	60	3	30	8	80	6	60
Construction	10	200	2	40	7	140	3	60	3	60	5	100	3	60
Extendability	1	10	10	100	10	100	1	10	1	10	1	10	10	100
Water tightness	5	25	10	50	8	40	8	40	4	20	8	40	8	40
Total	**36**	**585**	**61**	**625**	**64**	**730**	**38**	**445**	**29**	**345**	**42**	**580**	**49**	**555**
		3		2		1		6		7		4		5

Explanation
a = the individual score of each option
b = the result of the multiplication of the individual score with the weight factor (See also section 3.5.1)
The last row gives the ranking of the different options.

Table 3.7: Individual scores sand-storage dams (Note:For explanation see Table 3.6)

Option	8		9		10		11		12		13		14	
Criteria	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	2	50	4	100	4	100	4	100	4	100	10	250	1	25
Maintenance	10	150	9	135	1	15	3	45	10	150	5	75	10	150
Durability	10	100	10	100	5	50	6	60	8	80	1	10	10	100
Vulnerability	10	100	10	100	1	10	3	30	10	100	4	40	8	80
Suitability Kitui	5	50	8	80	6	60	6	60	8	80	5	50	1	10
Construction	4	80	8	160	10	200	10	200	4	80	10	200	1	20
Extendability	10	50	10	50	1	5	1	5	5	25	10	50	1	5
Water tightness	10	50	8	40	3	15	3	15	7	35	1	5	9	45
Total	**61**	**630**	**67**	**765**	**31**	**455**	**36**	**515**	**56**	**650**	**46**	**680**	**41**	**435**
		4		1		6		5		3		2		7

After the multi-criteria-analysis the weight factors have been changed to test the sensibility of the scores to the weight factors. The final results remained about the same (see Appendix 1). This means that the outcome of the multi-criteria-analysis does not depend too much on the weight factors.

3.5.3 Conclusions and recommendations

SASOL builds stone-masonry dams right now. According to the multi-criteria-analysis this is the best solution for Kitui District. In case SASOL wants to build a sub-surface dam, the stone-masonry sub-surface dam also has the best score.
For sub-surface dams the plastic sheet method is an interesting option. This method does not have a high score, mainly because of its lack of durability and its vulnerability. When strong plastic sheets can be found this method is worth trying.
For the sand-storage dam the stone sand-storage dam is a very interesting option. This option will only give good results when seepage is not a problem. The seepage through this dam is likely to be very high.

SASOL can keep on experimenting with the dam design and the use of materials. In the rest of this report the stone-masonry sand-storage dam will be worked out, while no attention is paid to the sub-surface dams, since these dams are not frequently build by SASOL.

4. Inventory Groundwater Dams Kitui District

4.1 Inventory aspects

The inventory of the SASOL dams was carried out in detail for all aspects that influence the functioning of the dams. For this purpose a general evaluation form for drinking water dams was drawn up. The reason for this generality was that it was not possible to determine inventory-circumstances in advance.
This general evaluation form could be used to evaluate more drinking water dams in the future (see Appendix 2).
To determine the aspects that influence the functioning of the dams, literature about small dams was consulted and experts on construction of small dams in developing countries were interviewed. Besides this, professional literature was used to get information on aspects like reservoir-, rainfall- and river characteristics.

The different inventory aspects concern (see Appendix 2):

- Construction
- Used materials
- Reservoir
- Abstraction systems
- River and banks
- Surroundings
- Rainfall and evaporation
- Subsoil
- Users
- Costs

The functioning and the condition of the dams are determined by investigating these aspects.

The dams are evaluated to find weak points of the current design and construction of the dams. The inventory is also meant to gain insight in the effect of the dams on the surroundings.

4.2 Inventory scheme

4.2.1 Recording

Fifty dams were recorded during the first two weeks of April 2001. SASOL's technical manager and other SASOL-staff accompanied us during the visits. For the evaluation three catchment areas were selected (see figure 4.1 and Appendix 3). These three catchment areas have different backgrounds; the geological properties, fluvio-morphology and community organisation differ from area to area. The dams in these areas were also built in different periods, while the design of the dams has changed in the course of time.

In catchment area A the dams are built in the beginning of the SASOL Dam Project. The design in this catchment is quite simple. The dams were long and low, almost no spillway has been used. The rivers are wide at the places where the dams have been built.
In catchment area B the river width is small and the velocity of the flood is fast. These dams are a few years old. Some experiments have been made on the design of the sand-storage dams.

The dams in catchment C are the recent built of the three catchment areas. In this catchment area the rivers are also narrow and the flood is fast. In the design of the dams improvements have been made. The spillway is different for different dams.

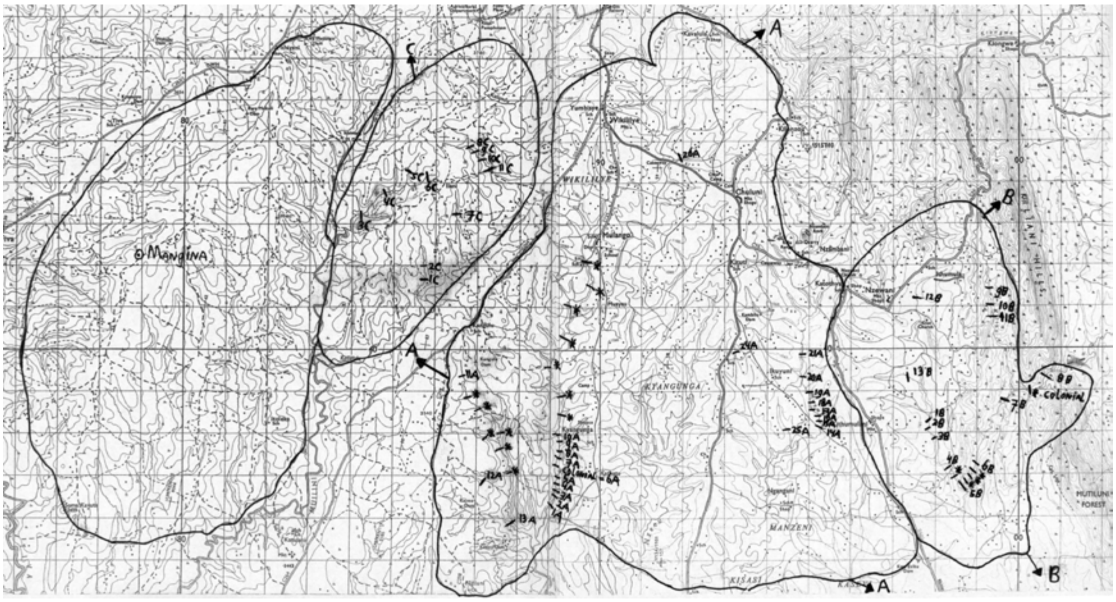


Figure 4.1: Investigated catchment areas

More than 50 dams were visited, while only 50 dams were recorded. The reason for this is that recording dams cost a lot of time. A selection of the dams that were visited was made to be able to see more different dams. A diversity of dams gives a better understanding of the different problems and construction-possibilities. Because common dams were neglected, the recorded data is not random anymore. This is not a problem since the goal of the inventory was to get a broad overview of the different possibilities and problems, so that the SASOL design can be optimised.

At each recorded dam all aspects were talked through with the technical manager and other SASOL staff members. Few users or people from the surroundings were questioned, while especially the questions about the user- and the surrounding aspects were supposed to be answered by the users.
Except the aspects that were clearly visible, such as the technical state of the dam and the used materials, information about the other aspects was got from the technical manager or the SASOL staff members. This means that the objectivity is biased on different points. Still the information is useful to get insight in the use and functioning of the dams. The dam dimensions were also measured during the visits.

Some of the aspects were difficult to quantify, like river slopes, catchment areas, topographical and geological conditions, peak water levels, rainfall, evaporation and soil characteristics. For these aspects only a rough estimate could be made.

For all dams, except for one that was built in the colonial period, records are available. SASOL keeps track on the material use, labour and the design in small notebooks. The information from these records is also taken along in the inventory.

Figure 4.2: Recording the dams

4.2.2 Documentation

The recorded dams were documented on the evaluation form. All main information on the different aspects was entered in the form. Besides that a sketch of the dam is given. Pictures of weak and interesting points were taken during the dam visits.
These data will all be bunched together in a folder with a description of the pictures. In the future this folder can be used to compare the conditions of the dams during the course of time.

The information gathered at the recording is also entered in a spreadsheet program for further processing.

4.3 Inventory evaluation

The gathered information was evaluated in two parts. The first part of the evaluation deals with the construction of the 50 dams and was carried out during the practical work period. In this evaluation the use of labour, the duration of the construction, the materials and the costs were evaluated and compared to the dimensions of the dams and the reservoirs.
The results of this evaluation are described in the Practical Work Report 'Building Sand-Storage Dams, SASOL Foundation, Kitui District, Kenya'.[6]

The second part of the evaluation deals with the dams in general, especially about the weak points and the interesting aspects of the dams. This part of the evaluation is described in this report.

SASOL dams show similarities:

- SASOL's technical manager designs the dams together with the mason, who is in charge of the construction of the dam, while the community is organised to build the dam.

- The dams are all stone-masonry sand-storage dams and built in non-perennial rivers.

- The reservoirs behind the dams have no fences or other protection against contamination. Hedges protect the big abstraction points.

- Scoop-holes are preferred over wells by the communities. Communities think the wells are not hygienic and that diseases are spread by the wells, because everybody uses its own bucket in the well. In times of drought the wells are still used, since there is no other possibility to fetch water.

- The protection against erosion consists of Napier-grass or local present rock. No other erosion protection is used.

- The sediment in the reservoir consists of sand most of the time. The grading of the settled sand differs per dam.

- If the reservoir is not yet filled up a thin layer of silt settles on top of the sand. According to SASOL this thin silt layer flows away in the rainy season.

- The water in the reservoir is mainly used for domestic water supply and livestock. If there is enough water in the reservoir irrigation and other economic activities are undertaken.

Figure 4.3: Fetching water by using scoop-hole

Because few users have been questioned about the dam, the general opinion about the dam is not taken into account in this evaluation.

The occurrence of all the different aspects are listed in Appendix 4.

In the tables 4.1 to 4.6 the occurrence of the most important aspects are given for each catchment area. The recommendations that are given at the recording are shown in table 4.7. In figure 4.11 an overview of the main weak points is given.

Table 4.1: General information evaluated dams

	Area A		Area B		Area C		Total Areas	
General	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
total dams	26		13		11		50	
rebuilt of repaired	3	12%	0	0%	0	0%	3	6%

Figure 4.4: Dam rebuilt

Table 4.2: Occurrence of the embankment aspects

	Area A		Area B		Area C		Total Areas	
Embankment	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
stilling basin	2	8%	0	0%	1	9%	3	6%
impermeability	26	100%	13	100%	11	100%	50	100%

Figure 4.5: Stilling basin with a lot of erosion

Table 4.3: Occurrence of the soil types

	Area A		Area B		Area C		Total Areas	
Soil	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
river bottom								
base rock	12	46%	10	77%	2	18%	24	48%
clay	1	4%	1	8%	0	0%	2	4%
sand	13	50%	2	15%	8	73%	23	46%
others	3	12%	0	0%	1	9%	4	8%
banks								
base rock	10	38%	5	38%	1	9%	16	32%
clay	9	35%	0	0%	2	18%	11	22%
sand	5	19%	2	15%	0	0%	7	14%
others	17	65%	13	100%	9	82%	39	78%
foundation								
base rock	20	77%	12	92%	11	100%	43	86%
clay	2	8%	0	0%	0	0%	2	4%
sand	1	4%	1	8%	0	0%	2	4%
others	6	23%	1	8%	2	18%	9	18%

Table 4.4: Occurrence of the protection materials

	Area A		Area B		Area C		Total Areas	
Protection	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
materials								
vegetation	20	77%	9	69%	8	73%	37	74%
others	5	19%	0	0%	0	0%	5	10%

Figure 4.6: Napier-grass planted on both banks

Table 4.5: Occurrence of the place of the erosion

	Area A		Area B		Area C		Total Areas	
Erosion	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
place								
banks upstream	10	38%	4	31%	1	9%	15	30%
river bottom downstream	3	12%	1	8%	4	36%	8	16%
banks downstream	5	19%	6	46%	1	9%	12	24%

Figuur 4.7: Erosion at banks nearby the wing wall of the dam

Table 4.6: Occurrence of the weak points

	Area A		Area B		Area C		Total Areas	
Condition	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
weak points								
erosion	16	62%	8	62%	9	82%	33	66%
stability	3	12%	0	0%	0	0%	3	6%
settlement	1	4%	0	0%	0	0%	1	2%
cracks	3	12%	1	8%	0	0%	4	8%
seepage	5	19%	0	0%	2	18%	7	14%
bank	5	19%	3	23%	0	0%	8	16%
abutment	8	31%	5	38%	0	0%	13	26%
downstream toe	1	4%	2	15%	1	9%	4	8%
protection	7	27%	6	46%	5	45%	18	36%
draw-off system/well	3	12%	0	0%	0	0%	3	6%
others	4	15%	3	23%	4	36%	11	22%

Figure 4.8: The plate around the well is broken by settlement

Figure 4.9: Piping

Figure 4.10: Erosion at near by the dam

Table 4.7: Occurrence of the recommendations given at the recording

	Area A		Area B		Area C		Total Areas	
Recommendations	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*	*hits*	*per cent*
protection								
banks upstream	12	46%	4	31%	1	9%	17	34%
banks downstream	12	46%	9	69%	6	55%	27	54%
river bottom downstream	1	4%	1	8%	0	0%	2	4%
downstream toe	1	4%	1	8%	1	9%	3	6%
reparation								
cracks	1	4%	0	0%	0	0%	1	2%
decrease seepage	1	4%	0	0%	1	9%	2	4%
protection	5	19%	8	62%	7	64%	20	40%
landslide: recover/protect	0	0%	1	8%	0	0%	1	2%
improvement								
improve stability	1	4%	0	0%	0	0%	1	2%
adapt abutment	0	0%	1	8%	0	0%	1	2%
others	5	19%	1	8%	1	9%	7	14%

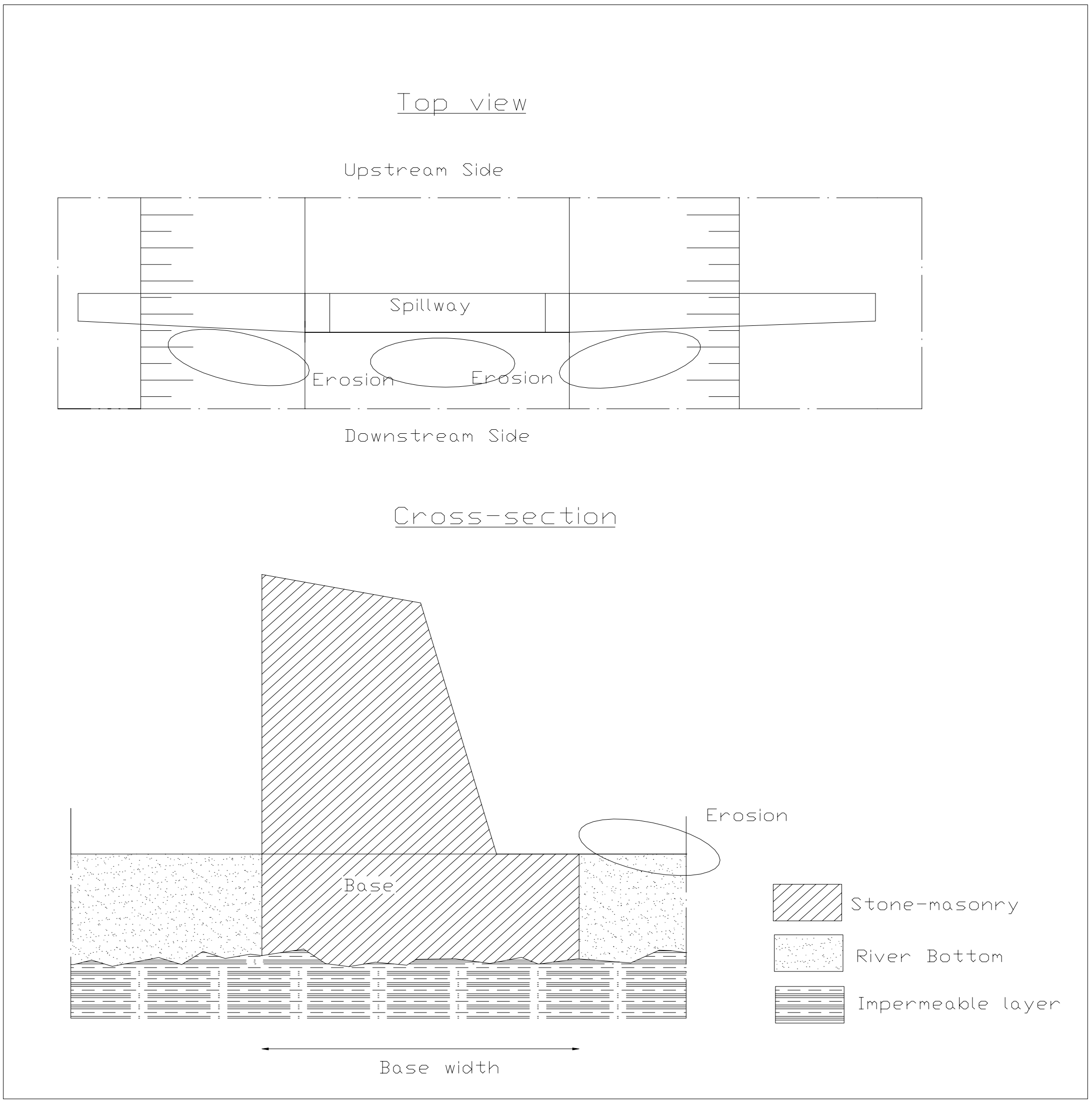


Figure 4.11: Overview of the main weak points

From the tables and the overview can be seen that:

- Most dams do not have a stilling basin, while erosion and protection are the main weak points of the evaluated dams.

- The banks upstream as well as downstream suffer from erosion, especially nearby the dam at the wings and at the toe. Recommendations given during the recording also show that the erosion protection in the form of vegetation is present at most dams, but not sufficient and should be improved. The reason for the repair or rebuilding of damaged dams was erosion.

- The water in the reservoir is used or can be used at all dams. Some dams have cracks or seepage but they still hold water.

- The dams are often based on base rock. In some cases the subsoil consists of sand, leck (solid clay) or murram.

In general the condition of the dams is fairly good. On a scale from 1 to 5 the dams are given an average 4.0 (good). For the quantity of available water an average grade of 4.6 is given, while the accessibility of the dams is graded 3.7 (see table 4.8).
On the last 2 grades a comment has to be made: Some grades have been given by SASOL instead of by the users.

Table 4.8: Grading of the evaluated dams

	Average grade	Median	Minimum value	Maximum value
Condition (1)	4.0	4.0	1.0	5.0
Accessibility (1)	3.7	4.0	1.0	5.0
Available water quantity (2)	4.6	5.0	2.0	5.0

explanation (1)		explanation (2)	
	1 = very bad		1 = shortage
	2 = poor		2 = insufficient
	3 = acceptable		3 = even
	4 = good		4 = enough
	5 = exellent		5 = plenty of water

4.4 Conclusions and recommendations

- The main weak point, erosion, is caused by the way the river flows across the dam. If the discharge is too high, the water flows across the wings and causes erosion nearby the wings, the banks and the toe of the embankment. The spillway should prevent this. The spillway must direct the river. In combination with erosion protection the water should not be able to harm the construction, the river or the banks. In section 6.5.3 the way to design the spillway is described.

- The lack of a stilling basin that prevents the downstream toe of the dam gives the erosion full play to damage the foundation of the dam. In section 6.7.3 the function of the stilling basin and the way to use the stilling basin as an erosion protection will be described.

- Planting vegetation prevents erosion. Napier-grass has proved its suitability. The non-permanent rivers are wild and have a high flow velocity. Planting Napier-grass is recommended along the whole river not only for protection against erosion, but planting Napier-grass can also fix the course of the river.

- Although the stone-masonry sand-storage dams need little maintenance the dams should be inspected from time to time. Weak points that threaten the functioning of the dam can be eliminated before they harm the construction seriously. Repairs or improvements should be carried out immediately to prevent further damage to the dam.

- The reservoirs are not yet protected against contamination by cattle or other contagion sources. To prevent contamination of the water in the reservoirs cattle should not be allowed to walk in the reservoir. This is hard to accomplish since rivers often serve as roads. Still everything should be done to minimise the possibility of contamination. Wells should be protected from contamination by using a lid.

- Most dams are founded on base rock. The locations that have been chosen for the dams have a good foundation layer and the banks and riverbeds often consist of rock. The location choice made by SASOL is correct.

- Although a few dams have cracks and some of them have seepage, the dams are still able to retain water in their reservoirs.

Finally, besides the erosion in the surroundings of the dams and a few small weak points, the evaluated dams function very well. Planting Napier-grass and improving the spillway and the protection of the downstream toe by a stilling basin makes the stone-masonry sand-storage dams an even more effective system to store water in dry areas.

5. Estimation of the design discharge

5.1 General

For the design of a sand-storage dam, the peak discharge is an important parameter. In this chapter, a choice is made between the different methods that can be used for the estimation of this peak discharge. The most suitable method for the design of sand-storage dams, the Slope-Area Method, is described in 5.3.

5.2 Methods to estimate the peak discharge

In small and rural catchment areas, where the sand-storage dams are generally built, discharge data of seasonal rivers will not be available most of the time. An estimation of the peak discharge must be based on other data. There are many different methods to determine the peak discharge, based on rainfall data, on the size of the catchment area or based on peak levels.

Best way would be to use rainfall-based flood models that are calibrated for the area. Only one of the problems is that reliable rainfall data and information about the rainfall/discharge relation are hard to get or will not be available at all in the project areas. If these data are available, the flood models based on rainfall are recommended, but still it requires much work for skilled personnel and the question is if this matches with a simple technology as that of sand-storage dams.

A simpler method is to determine the peak discharge on the basis of the size of the catchment areas. Some methods relate this two parameters, but they often give inaccurate results (see [9] and [26]), because they are too general and do not depend on the quantities of rainfall.
It would be better to fix a relation for a specific area, with the same geological and rainfall characteristics. This asks for skilled work too, but when the relation is fixed, only the surface of the catchment area has to be measured, to come to a peak discharge. Problem is however, that rainfall and geography can vary very much in hilly areas, such as northern Kitui District, which makes it difficult to relate it to larger areas.

Methods based on peak levels, such as the Slope-Area Method, are often used in areas where no data is available. Information about peak levels can be obtained from local inhabitants, flood marks or can be registered by data loggers. This method is simple and gives a quite good approximation of the peak discharge. The Slope-Area Method [31] is the most suitable method to estimate the peak discharges, because it is simple and generally applicable. The (little) work that has to be done fits the design process of the sand-storage dams.

5.3 Slope-Area Method

The Slope-Area Method is widely used to compute peak discharges after the passage of a flood. After selecting a river section, preferably a uniform channel, the flood peak level is determined on basis of observed peak levels by local people or flood marks of major floods. From this information, the water level slope, the wetted cross-sectional area and the hydraulic radius are derived. The discharge can be computed using the Manning formula or the Chézy formula.

For the estimation of the peak discharge, the Manning formula is used. The Manning formula has the practical advantage over the Chézy formula that it uses a roughness coefficient, which does not depend on the water depth.

$$Q_{sa} = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot s^{\frac{1}{2}} \tag{5.1}$$

Where:
Q_{sa} = Discharge [m^3/s]
n = Manning's roughness coefficient [$m^{1/3}/s$]
A = Wetted cross-section [m^2]
R = Hydraulic radius = Wetted cross-section (A) / Wetted parameter (P) [m]
s = Energy gradient [-]

The wetted cross-section can be schematised as shown in figure 5.1.

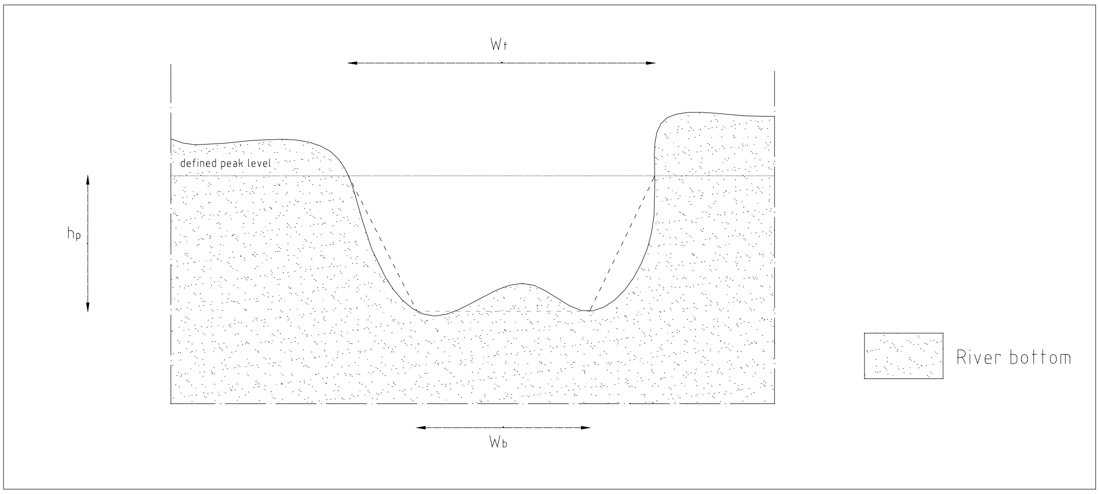


Figure 5.1: Schematic diagram of the wetted cross-section

A and R can be calculated as follows:

$$A = \frac{W_b + W_t}{2} \cdot d \tag{5.2}$$

$$R = \frac{A}{P} = \frac{A}{\left(W_b + 2 \cdot ((0.5 \cdot (W_t - W_b))^2 + h_p{}^2)\right)^{\frac{1}{2}}} \tag{5.3}$$

Where:
A = Wetted cross-section [m^2]
R = Hydraulic radius [m]
W_b = Width of the seasonal river at the bottom [m]
W_t = Width of the seasonal river at the definite peak level [m]
h_p = Distance between the defined peak level and the river bottom [m]

The best way to determine the peak level is to rely on information of the communities. It is likely that local inhabitants observed major peak flows (preferable the highest peak in the last 10 years) and know the peak level. Questioning local people makes it possible to estimate the peak level (h_p) during major peak flows.

Another way to determine h_p is to look for flood marks on the banks, such as slime, organic materials and floating debris

Uniform reach

The Manning formula is valid for uniform flow. The next criteria should be met for a uniform channel. The criteria are adjusted for the specific areas where sand-storage dams are built, to make them more applicable. [31]

a) The reach should have a uniform cross-section, be free from obstructions and from backwater effects;
Backwater effects are not likely in the middle of the reach, when there are no big obstructions within a short distance downstream and when the gradient is steep, as it is in the seasonal rivers of the sand-storage dams.
b) The length of the reach should be at least 75 times the mean depth, at least 5 times the mean width and preferable at least 300 m;
This comes down to a distance of about 100 m for the small channels, where sand-storage dams are built.
c) The water surface fall in the reach should be greater than the velocity head and be at least 0.15 m;
This condition is almost always met, because river-gradients are high most of the time.
d) The entire reach shall have either sub-critical flow or supercritical flow. The water level profile shall not cross the critical depth line in a hydraulic drop or a hydraulic jump.

The criteria can be simplified for the areas where sand-storage dams are usually built:
A river section of about 100 m with a uniform cross-section, free from obstructions and without big obstructions directly beneath the reach.

In a uniform channel the energy gradient is the same as the water level slope, because of a constant discharge. Both the energy gradient and the water level are parallel to the streambed. On the highest point of a flood wave (highest water level and highest discharge), the discharge is considered constant for a moment. Therefore the slope of the river can be used for the energy gradient (s).

Non-uniform channel

A different situation occurs, when the channel is not uniform. Since Manning is derived for uniform flow, an error is introduced by using this formula. Another problem is that the energy gradient is no longer parallel to the river bottom gradient. It is difficult, if not impossible, to determine this energy gradient accurately from field observations.

To come to a method to estimate the peak discharge, the flow is assumed to be uniform in the cross-sections. Figure 5.2 shows an example of a longitudinal profile with a changing bottom gradient. When a channel is uniform, the water level is equal to the equilibrium water depth (d_e) (d_e depends on the discharge). But in this non-uniform reach, the actual water level is lower. Assuming that the flow is uniform, means that the measured height (d_m) is used for the equilibrium water depth (d_e), which leads to an error (in this example a lower discharge).
To compensate this error, a higher safety factor is used ($S_f = 1.5$).

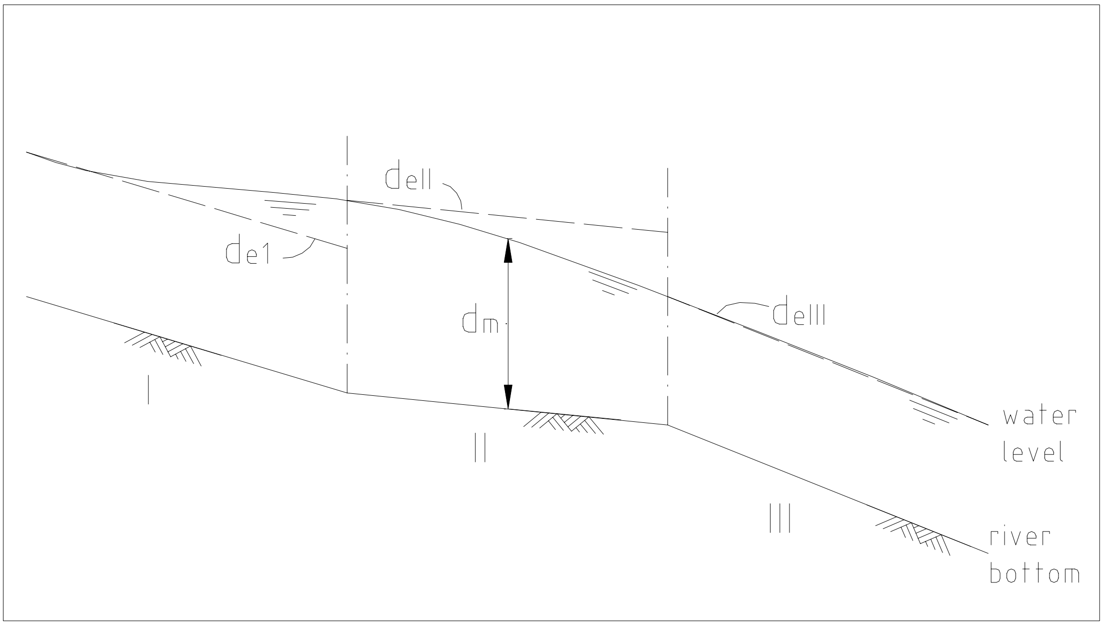


Figure 5.2: Water depth in river section with changing gradient

Method for uniform and non-uniform reaches

The method for uniform and non-uniform reaches becomes as follows:

- In the reach 3 cross-sections are selected and the peak level is determined (see figure 5.3)
- In each cross-section W_t, W_b and h are measured
- The slope (s) over the reach is measured.
- The discharges in these cross-sections are calculated as if the flow is uniform.
- The average value of these 3 discharges is calculated (5.4).

$$Q_{sa} = \frac{Q_{sa1} + Q_{sa2} + Q_{sa3}}{3} \tag{5.4}$$

The design discharge (Q_d) is calculated as follows:

$$Q_d = S_f \cdot Q_{sa} \tag{5.5}$$

Where:
Q_d = Discharge which is used for the further design [m^3/s]
Q_{sa} = Peak discharge found with the Slope-Area Method [m^3/s]
S_f = Safety factor; 1.3 for a uniform reach, 1.5 for a non-uniform reach [-]

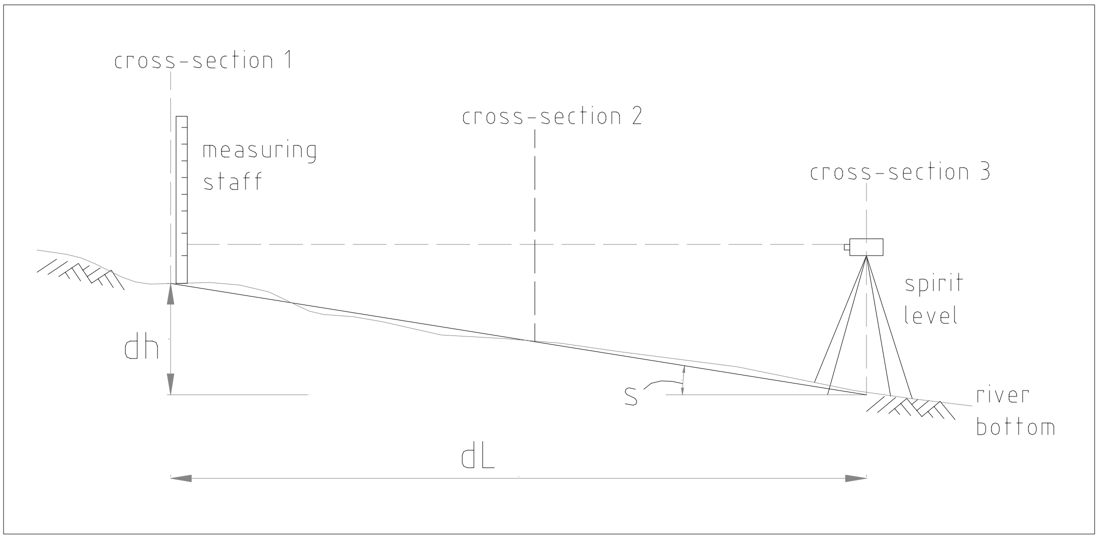


Figure 5.3: Measuring the reach

Roughness coefficient of Manning (n)

The Slope-Area Method is an approximate way of discharge calculation, mainly caused by the estimation of the channel roughness coefficient, which is very arbitrary. In table 5.1 values for the Manning coefficient are given.

Table 5.1: Manning's coefficient n as a function of profile, irregularity and vegetation.

Type of channel and description	**Manning's coefficient n [$m^{1/3}$/s]**		
	Minimum	**Normal**	**Maximum**
Excavated or dredged			
Earth type: straight and uniform			
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. with short grass, some weeds	0.022	0.027	0.033
Channels not maintained			
1. clean bottom, brush on sides	0.040	0.050	0.080
2. dense brush, high stage	0.080	0.100	0.140
Natural streams			
Minor streams, on plain, B>30 m			
1. clean, straight, regular profile.	0.025	0.030	0.033
2. clean, winding, some pools and shoals	0.033	0.040	0.045
3. same as 2, but irregular profile	0.045	0.050	0.060
Mountain rivers, moderate to steep slope			
1. gravel bottom with few boulders	**0.030**	**0.035**	**0.050**
2. cobbles with large boulders	**0.040**	**0.055**	**0.070**
Flood plains			
1. pasture, cultivated areas	0.025	0.035	0.050
2. brush and trees	0.035	0.075	0.160

6. Improved design sand-storage dams

6.1 General

In this chapter the SASOL sand-storage dam design is described and evaluated. When the design can be improved a new design is proposed. In this chapter attention is paid to building in stages (6.2). Section 6.3 deals with the foundation depth and 6.4 with the dam length. An important factor of the dams is the design of the spillway and the crest level. These issues are discussed in 6.5. In 6.6 the width of the dam is evaluated and optimised. The design of the stilling basin is discussed in 6.7. Finally the construction of sand-storage dams is evaluated in 6.8.

SASOL's technical manager and the mason, who will build the dam, design the dam together. The SASOL dams are designed on the basis of some rules of thumb. These rules are based on practical experience, information from congresses and literature from Erik Nissen-Petersen. SASOL's technical manager worked with Erik Nissen-Petersen from 1980 until 1991. The current SASOL design is documented during the past practical work period. The information for the evaluation of the different design aspects comes from the practical work period and information from the visits to the 50 dams.

6.2 Building in stages

6.2.1 General

In literature suggestions are sometimes made to build the dam in stages. The reason to build the dam in stages is that the reservoir can gradually fill up with sediment. Only coarse sediment will settle behind the dam because of the higher current velocities. Fine particles will just flow across the dam. Coarse sediment increases the storage capacity of the dam because coarse sediment has a higher porosity. Another factor, which is very important, is that fine particles in the upper layer will reduce the recharge rates considerably. Furthermore the abstraction of water from the reservoir is easier in case of coarse material.

6.2.2 SASOL construction method

Nevertheless SASOL has chosen not to build the dam in stages, because it is very difficult to mobilise communities for the second time. Most dams do not have problems concerning the sediment behind the dam. In general the settled sediment is coarse.

6.2.3 Evaluation of SASOL construction method

The best way to construct sand-storage dams is in stages. SASOL's problem with building in stages can be solved by building the spillway in stages and to ensure that all needed stones are already collected the first time. The wings of the dam and the embankments can already be built up till the final height the first time.

6.2.4 Possible new design

The height of each stage can be assessed by studying the extent of natural sedimentation in the stream, but most of the time a stage height of about 0.5 m is a good start. After one stage has been built, the subsequent accumulation of sediments can be studied and the height of the next stage can be determined accordingly. The spillway should only be raised with a new stage when floods have raised the level of coarse sand to the current level of the spillway. However, since a good rainy season can facilitate several floods capable of depositing sufficient sand, a spillway could be raised several stages during one rainy season, if the builders are prepared to do so.

6.3 Foundation depth

6.3.1 SASOL design

The final foundation depth depends on the fact whether and on which level base rock, clay or murram (sticky soil with small stones) is found. Base rock is the best foundation for the dam. Clay is the second best foundation. In general the clay in Kitui District is very compact and does not settle.

The foundation depth depends on the depth of the base rock, clay or murram.

6.3.2 Evaluation SASOL design

After some problems in the past, SASOL determines the final foundation depth on the level where base rock, clay or murram is found. This is done for the spillway, embankments and for the wings. In our opinion this is a good way of determining the foundation depth of the dam, because of the following three reasons:
1. A good base is provided
2. Seepage is prevented
3. Piping is prevented

Prevention of seepage and piping has to get more attention.

6.3.3 New design

At the location of the dam, the foundation material should be strong enough to carry the weight of the dam. The final foundation depth depends on the fact whether and on which level base rock, impermeable clay or murram or other unconsolidated formations with low permeability is/are found. This has to be done both for the whole dam, so also for the wings, and is a good base to found and prevents seepage and piping. Base rock is the best foundation for the dam.

Rock foundation
- When founded on rock, the foundation trench is dug until the rock.
- To prevent seepage, it is sensible to cut a groove along the length of the dam and make the rock-base rough enough for a good connection between the mortar and the rest of the dam, especially when the rock base is not deep.
- In case of a weathered rock formation, the profile should be completely excavated before the dam foundation is constructed. The presence of open fracture zones should also be investigated.

- If the presence of open fracture zones is expected, the rock surface should be cleaned and simple infiltration tests can be carried out by pouring water on the cleaned surface. If fractures are discovered, they need to be sealed, and expensive grouting (cement mixture) might be required.
- In case of permeable rock, the dam should be built against the rock to prevent seepage through the rock-base. The rock-base will also support the dam, so the width of the dam can eventually be adapted.

For rock base the foundation depth ($D_{foundation}$) is:

$$D_{foundation} = \text{Depth base rock} \tag{6.1}$$

Clay/murram foundation

- When good quality clay or murram is found, it is better to dig an extra 0.5 m in the clay/murram layer to prevent seepage.
- The depth of the foundation should be at least 1.0 m.
- Care should be taken in detecting the presence of any seepage-lines of sand, soil or gravel, since this can cause seepage and piping (washing out of small soil particles because of the difference in water level upstream and downstream). In general, for the type and size of dams under consideration, the risk of dam failure through piping is in general not very big, due to the low water heads.
- If seepage-lines are found in the foundation trench, the bottom of the dam must be extended down to a depth of 0.5 m below any seepage-lines.

For clay or murram the foundation depth ($D_{foundation}$) is:

$$D_{foundation} = \text{Depth clay or murram} + 0.5\text{ m} \quad (D_{foundation} > 1.0\text{ m}) \tag{6.2}$$

6.4 Dam length

6.4.1 SASOL design

The length between the two points where the crest level of the dam intersects both banks (crest length dam) together with the length of the wings becomes the final length of the dam (see figure 6.1). The height of the dam (distance between river bottom and the crest of the dam) depends on the gross freeboard of the spillway and the lowest bank (see section 6.5.3).

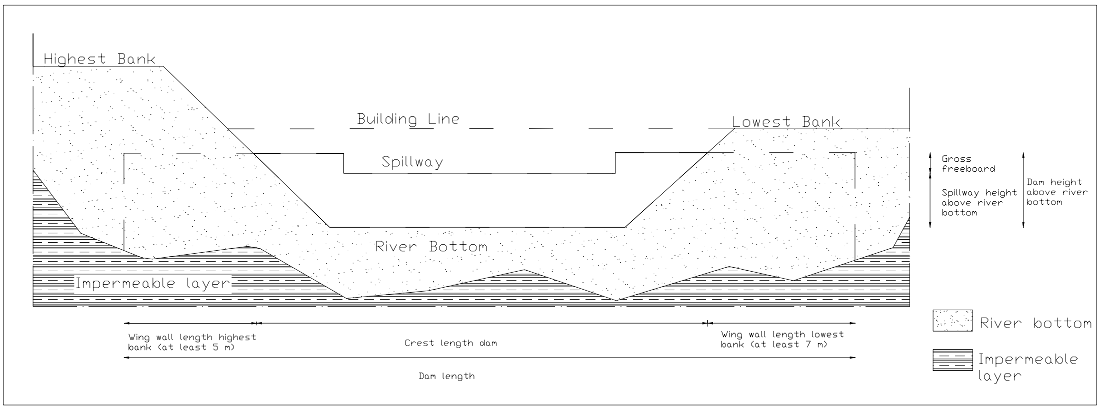


Figure 6.1: Dam length

Where the riverbanks are flat or consist of bad soil, wing walls are added to the main dam to prevent the river cutting around during a flood. After finishing the dam the highest bank above the wings will be covered with soil. The wing on the highest bank should be at least five metre long, and the wing on the lowest bank should be at least seven metre long (if the banks do not consist of base rock). These are minimum lengths, because the length of the wings can be increased to create more wetland. The wings function as a sub-surface dam. The formula for the length of the dam is:

$$L_{dam} = L_{crest\ dam} + L_{wing\ lowest\ bank} + L_{wing\ highest\ bank} \tag{6.3}$$

Where:
L_{dam} = Length of the dam
$L_{crest\ dam}$ = Length of the crest of the dam (distance between the two points where the crest level of the dam intersects both banks)
$L_{wing\ lowest\ bank}$ = Length of the wing on the lowest bank (see table 6.1)
$L_{wing\ highest\ bank}$ = Length of the wing on the highest bank (see table 6.1)

Table 6.1: Wing length

Bank	**Wing length**
Lowest bank	> 7 m
Highest bank	> 5 m

6.4.2 Evaluation SASOL design

After some problems in the past with water flowing round the dams, the length of the wings is extended. The present design length seems to be long enough to prevent water flowing around the dams.

6.5 Crest level and spillway dimensions

6.5.1 SASOL design

Crest level determination
The lowest bank of the river determines the final height of the crest of the dam. The level of the lowest bank is called the 'building line' of the dam. The height of the dam is the distance between the crest of the dam and the lowest point of the river bottom. The height of the dam depends on the height of the spillway above the river bottom (see table 6.5) and the gross freeboard of the spillway (distance between the crest of the dam and the top of the spillway).

$$\text{Dam height} = \text{Height of spillway above river bottom} + \text{gross freeboard} \tag{6.4}$$

Design spillway
Whether a dam is equipped with a spillway depends on the kind of river. When the river has a high discharge during the rainy season, a spillway is not applied at the dam. In SASOL's opinion it makes no sense to construct a spillway when a lot of water flows across the spillway and the dam.

At this moment SASOL applies two spillway types:
1. A rectangular cross-section
2. A trapezium cross-section

The length of the spillway depends on the kind of river. The downstream length of the spillway is smaller than the upstream length. The length of the upstream side of the spillway is determined by the width of the river bottom minus two times a certain distance from the banks. The length of the downstream side of the spillway is determined by the width of the river bottom minus two times a certain distance from the banks minus two times 0.15 m. So the width of the spillway downstream is 0.3 m smaller than upstream (see figure 6.2 and table 6.2). The reason for this is to direct the flow to the centre of the river, thus preventing erosion of the downstream banks and the downstream side of the dam.

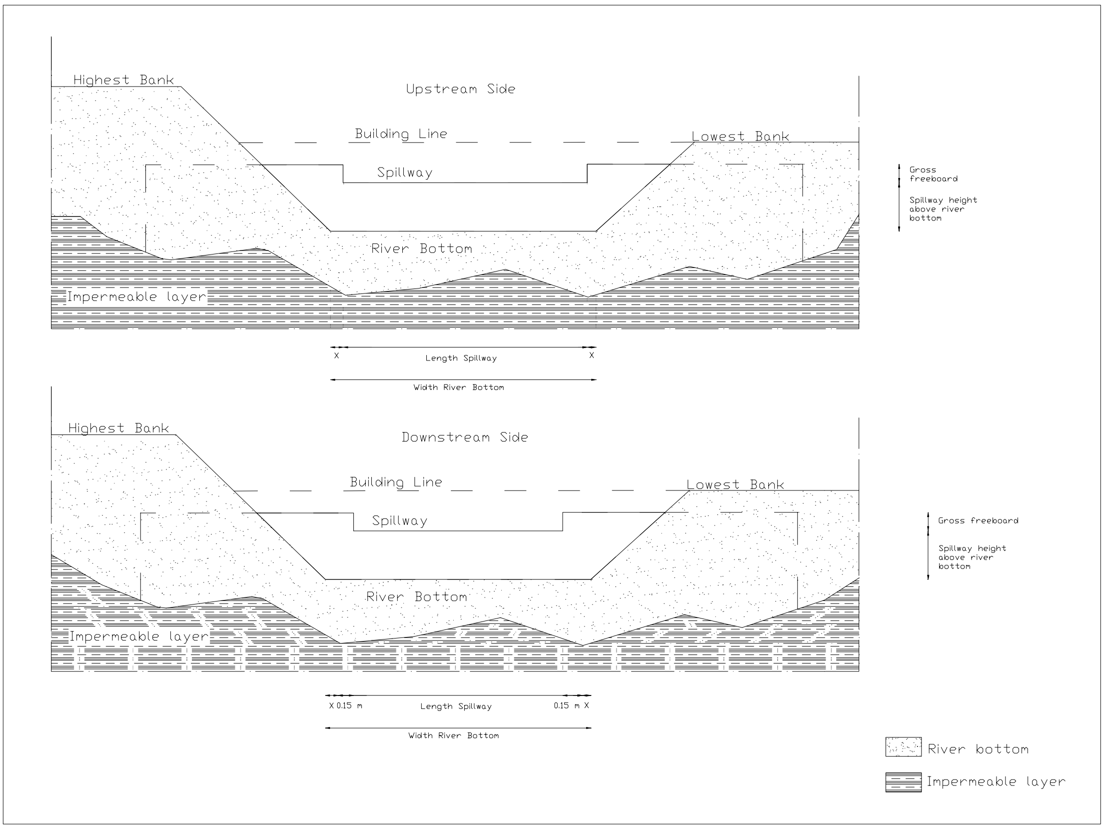


Figure 6.2: Rectangular spillway dimensions

Table 6.2: Rectangular spillway length

Kind of river	Width river bottom	Downstream length spillway	Upstream length spillway
Small	0 - 5 m	Width river bottom - 2 * 0.6 m - 2 * 0.15 m	Width river bottom – 2 * 0.6 m
Medium	5 - 10 m	Width river bottom - 2 * 0.6 m - 2 * 0.15 m	Width river bottom – 2 * 0.6 m
Large	> 10 m	Width river bottom - 2 * 2.0 m - 2 * 0.15 m	Width river bottom – 2 * 2.0 m

For the spillway with a trapezium cross-section two lengths are important. One at the crest level of the spillway and one at the crest level of the dam (see figure 6.3). The length of the spillway at the crest level of the dam is the same as the width of the river bottom. The length of the spillway at the crest level of the spillway is determined by the width of the river bottom minus two times a certain distance from the banks (see table 6.3).

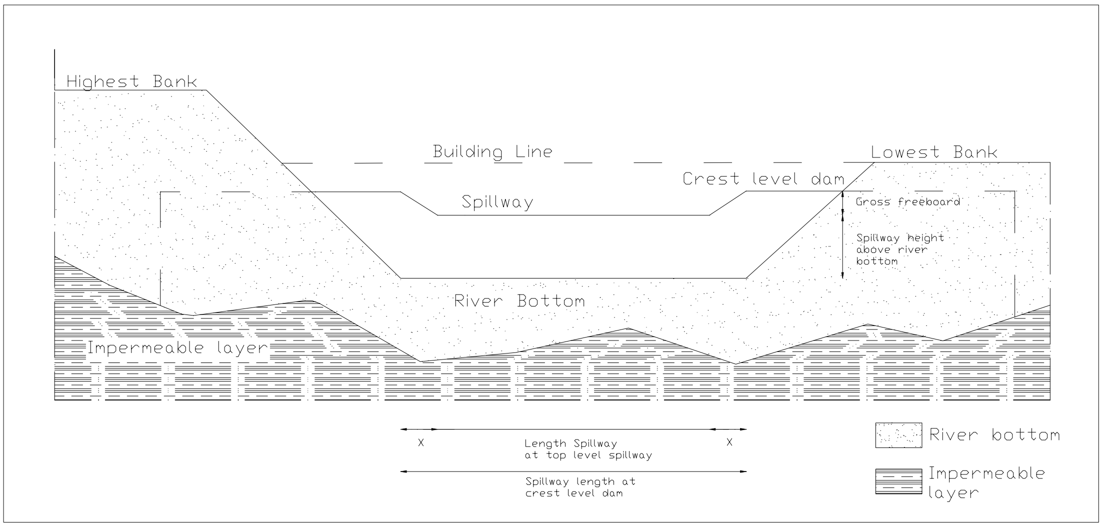


Figure 6.3: Trapezium spillway dimensions

Table 6.3: Trapezium spillway length

Kind of river	**Width river Bottom**	**Length spillway at crest level dam**	**Length spillway at top level spillway**
Small	0 - 5 m	Width river bottom	Width river bottom - 2 * 0.6 m
Medium	5 - 10 m	Width river bottom	Width river bottom - 2 * 0.6 m
Big	> 10 m	Width river bottom	Width river bottom - 2 * 2.0 m

The slopes of the trapezium cross-section are determined on the basis of the width of the river (see table 6.4).

Table 6.4: Slope of trapezium cross-section

Kind of river	**Width river bottom**	**Slope of trapezium cross-section**
Small	0 - 5 m	gross freeboard/0.6 m
Medium	5 - 10 m	gross freeboard/0.6 m
Big	> 10 m	gross freeboard/2.0 m

The gross freeboard (distance between the top of the spillway and the crest of the dam) depends on the width of the river, for both types of spillways. For all the dams SASOL is building at this moment, the gross freeboard is between 0.6 m and 0.75 m. A general rule does not exist however. The technical manager estimates the average water level in the river during the rainy seasons and determines the final height of the gross freeboard.

When the discharge is high the gross freeboard of the spillway will be high. A low discharge will result in a low gross freeboard.

The height of the spillway above the river bottom in the centre point (most of the time the lowest point) of the dam is determined by multiplying the distance between the river bottom and the building line with ¼ or ½ depending on the width of the river (see table 6.5). The height of the spillway is usually about 1.5 to 2 m above the river bottom, but dams up to 4 m have been constructed.

Table 6.5: Height of spillway above river bottom

Kind of river	Width river bottom	Height of spillway above river bottom in centre point
Small	0- 5 m	½ * distance between river bottom and lowest bank
Medium	5-10 m	½ * distance between river bottom and lowest bank
Big	> 10 m	¼ * distance between river bottom and lowest bank

The downstream side of the spillway is 15 cm lower than the upstream side. Because of this the crest of the spillway gets a downstream gradient. The slope will accelerate the water and this will prevent the water from eating the plaster layer at the downstream side of the spillway. Another reason for the slope is to prevent people walking over the spillway and damaging the dam.

The choice of the kind of spillway rests on discretion of the technical manager. Both cross-sections satisfy. The masons prefer a trapezium cross-section, because they say it results in less erosion.

6.5.2 Evaluation SASOL design

- SASOL has problems to determine the height of the spillway above the river bottom and the gross freeboard. Because the height of the dam is determined by these two factors (dam height = height of spillway in lowest point above river bottom + gross freeboard), the crest level cannot be determined adequately.
- The height of the spillway above the river bottom and the gross freeboard do depend on the width of the river and the discharge. The height of the dam only depends on the dimensions of the spillway and not on the quantity of water SASOL wants to store behind the dam (height of the reservoir).
- At this moment SASOL uses a guideline that is based on practical experience (trial and error). This guideline does not always give the right results. A guideline is necessary to determine the dimensions of the spillway more precisely. After that the height of the dam can be determined more precisely too.

The relations between spillway dimensions, height of the spillway, height of the dam and the height of the reservoir behind the dam will be considered closer in the next section.

6.5.3 New design

A rectangular cross-section is recommended for the spillway, because a longer length of the spillway at the crest level is required for the same flow section in case of a trapezium cross-section (see figure 6.4). A longer length of the spillway at the crest level is likely to give more erosion of the banks behind the dam.

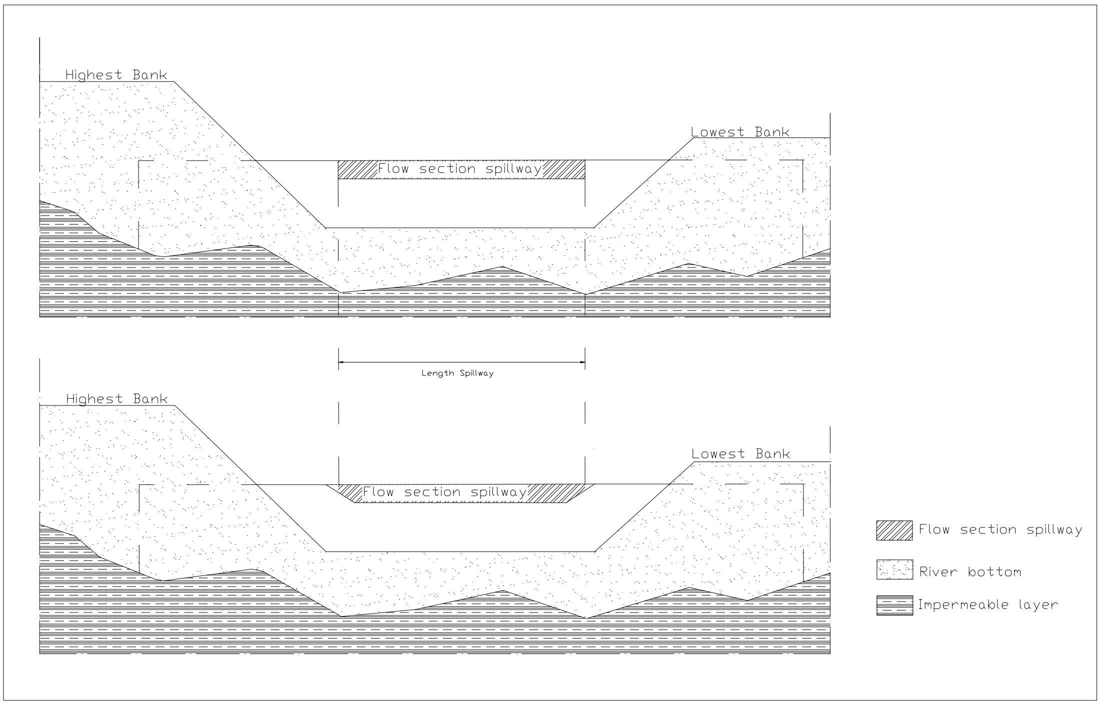


Figure 6.4: Rectangular and trapezium spillway dimensions

Length spillway

The length of the spillway depends on the kind of river. The downstream length of the spillway has to be the same as the upstream length, because the downstream velocities are higher in case of a narrowing spillway. Higher velocities cause more erosion and require a longer stilling basin downstream of the spillway. The length of the spillway is determined by the width of the river bottom minus two times a certain distance from the banks. The reason for this is to prevent the downstream banks from erosion and to make sure that the water flows into the stilling basin at the downstream side of the spillway.

Table 6.6: Spillway length

Kind of river	Width river bottom	Distance between spillway and the banks	Spillway length
Small	< 5 m	0.3 m	Width river bottom - 2 * 0.3 m
Medium	5 - 10 m	0.6 m	Width river bottom - 2 * 0.6 m
Large	> 10 m	0.9 m	Width river bottom - 2 * 0.9 m

Gross freeboard of the spillway

The gross freeboard of the spillway is dimensioned at the design discharge Q_d that is found with the Slope-Area Method (see chapter 5). The energy head above the crest of the spillway (H) with a rectangular control section under free flow during design discharge Q_d, can be calculated using the following basic depth-discharge relation:

$$Q_d = \frac{2}{3}\cdot\sqrt{\frac{2}{3}\cdot g}\cdot c_d\cdot L_s\cdot H^{\frac{3}{2}} = 1.71\cdot c_d\cdot L_s\cdot H^{\frac{3}{2}} = c\cdot L_s\cdot H^{\frac{3}{2}} \qquad (6.5)$$

Where:
Q_d = Design discharge found with the Slope-Area Method [m^3/s]
g = Gravitational acceleration [m/s^2]
c_d = Discharge coefficient [-]
c = Spillway coefficient [$m^{1/2}/s$]
H = Energy head above the crest of the spillway [m]
L_s = Length of the spillway [m]

The necessary gross freeboard (GF) of the spillway has to be equal to the energy head above the crest of the spillway, to prevent the water flowing across the wings at the corner between the wings and the banks (see figure 6.5). In that point the flow velocity is zero, so the piezometric level (water level) equals the energy head (see the equation of Bernoulli below).

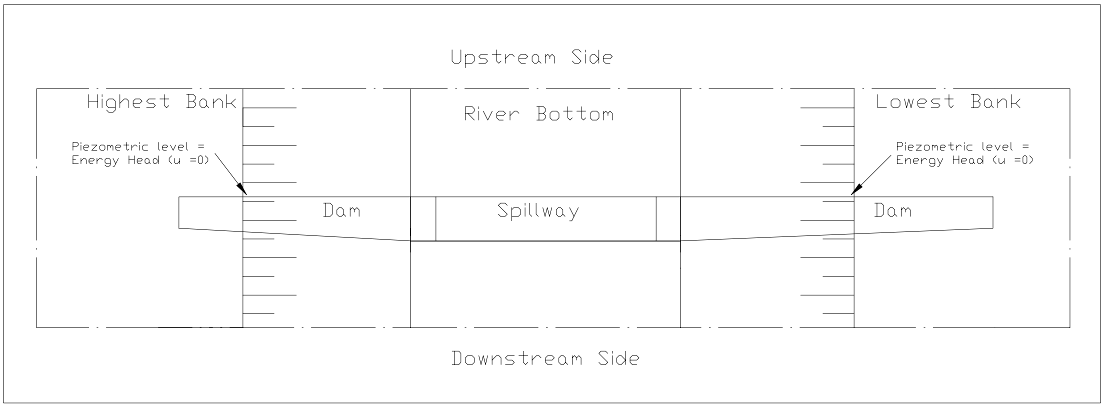


Figure 6.5: Places where piezometric level equals the energy head

Energy head = Piezometric level + Velocity head = *constant* [Bernoulli] (6.6)

Where:

Velocity head = $\frac{U^2}{2 \cdot g}$ [m]

Piezometric level = $h = z + \frac{p}{\rho \cdot g}$ [m]

or

$$H = z + \frac{p}{r \cdot g} + \frac{U^2}{2 \cdot g} = constant \quad \text{[Bernoulli]} \quad (6.6)$$

Where:
H = Energy head above the crest of the spillway [m]
z = Elevation head [m]
p = Pressure at height z [N/m^2]
ρ = Density [kg/m^3]
g = Gravitational acceleration [m/s^2]
U = Average current velocity in cross-sectional profile [m/s]

So:

$$GF = H \quad (6.7)$$

or

$$GF = \left(\frac{Q_d}{c \cdot L_s}\right)^{\frac{2}{3}} \qquad (6.8)$$

Discharge coefficient
The discharge coefficient depends on the radius of curvature of the streamlines above the crest, and thus on the kind of spillway. The sharp-crested spillway has curved streamlines above the crest. The rectangular 'broad-crested spillway' is an overflow structure with a horizontal crest, while the streamlines are straight and parallel (see figure 6.6).

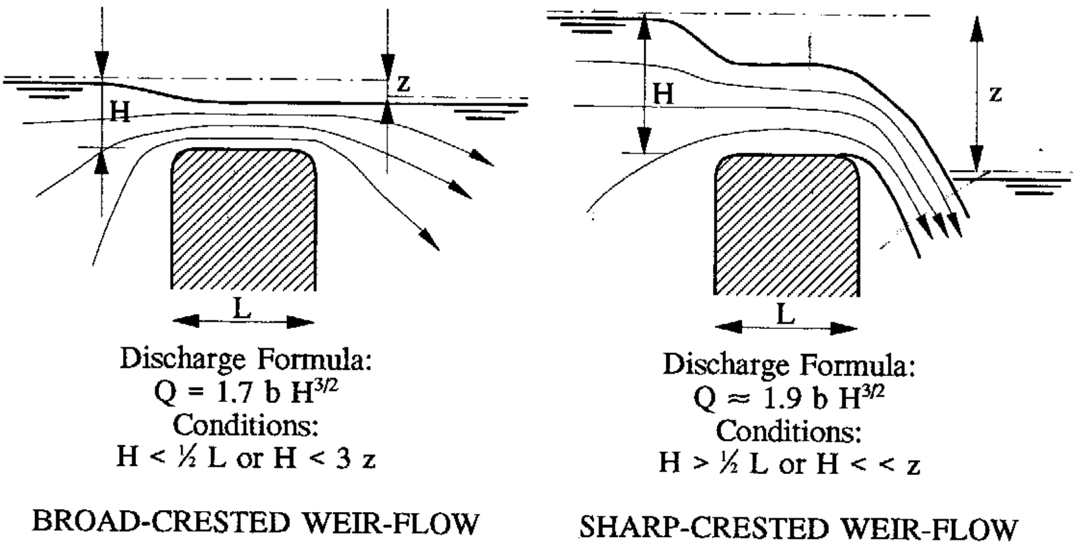


Figure 6.6: Broad-crested and sharp-crested spillway flow [4]

There are two possible types of sharp-crested spillways (see figure 6.7):
1. An unrounded sharp-crested spillway
2. A cylindrical sharp-crested spillway

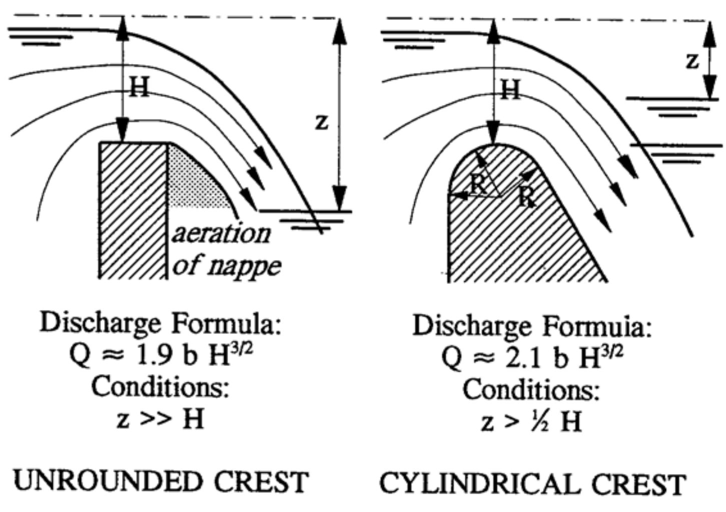


Figure 6.7: Types of sharp-crested spillways [4]

The radius R of cylindrical sharp-crests may not be taken too small, because of the sub-atmospherical pressure under the nappe. Usually a radius 0.3 H < R < 0.7 H for masonry spillways is used.
A lot of experience is required for the construction of a cylindrical sharp-crested spillway with a radius R between 0.3 * H and 0.7 * H. Because a good execution is not ensured, it is preferred to make a rounded sharp-crested spillway and to calculate the gross freeboard with the discharge coefficient for an unrounded sharp-crested spillway.

The discharge coefficient amounts $c_d = 1.0$ for a broad-crested spillway and $c_d = 1.1$ for an unrounded sharp-crested spillway. The spillway coefficient can be determined with the expression:

$$c = 1.71 \cdot c_d \qquad (6.9)$$

Spillway coefficient
Typical values for the spillway coefficient (c) are presented in table 6.7.

Table 6.7: Spillway coefficients

Spillway type	**Spillway coefficient c [$m^{1/2}/s$]**
Unrounded sharp-crested spillway	1.9
Broad-crested spillway	1.7

Tailwater level
The tailwater level may not rise too high because of the free flow-conditions. The free flow (non-submerged) conditions for a headloss *z* over the structure are described in table 6.8. Because the velocity head loss is small, for the total head loss only the piezometric level loss is taken (see also the equation of Bernoulli). For the headloss *z,* the following expression is assumed (see figure 6.7):

$$z = \text{piezometric level loss = crest level spillway} \qquad (6.10)$$

Table 6.8: Free-flow conditions

Spillway type	**Free-flow condition**	
Unrounded sharp-crested spillway	z >> H	Spillway height >> GF
Broad-crested spillway	z > 1/3 H	Spillway height > 1/3 GF

An unrounded sharp-crested spillway requires more headloss during free-flow conditions. The advantage is a shorter gross freeboard of the spillway, because of the higher spillway coefficient c. This lowers the cost of the spillway. An unrounded sharp-crested spillway is preferred.
The gross freeboard for an unrounded sharp-crested spillway can be calculated according table 6.9, if the two conditions are satisfied.

Table 6.9: Calculation gross freeboard of an unrounded sharp-crested spillway

Spillway type	**Gross freeboard (GF)**	**Condition 1**	**Condition 2**
Unrounded sharp-crested spillway	$\left(\frac{Q_d}{1.9 \cdot L_s}\right)^{\frac{2}{3}}$	GF > 1/2 W	GF << spillway height

Most of the time the spillway satisfies the conditions. A broad-crested spillway occurs when the gross freeboard (or the energy depth H) is less than half times the width of the top of the spillway or the height of the spillway is less than the gross freeboard (or energy depth H). The gross freeboard of a broad-crested spillway can be calculated according table 6.10, if the condition is met. The condition that the height of the spillway has to be more than $^1/_3$ times the gross freeboard will satisfy for almost all dams.

Table 6.10: Calculation gross freeboard of a broad-crested spillway

Spillway type	Gross freeboard	Condition
Broad-crested spillway	$\left(\frac{Q_d}{1.7 \cdot L_s}\right)^{\frac{2}{3}}$	GF < 3 * (spillway height)

In the manual a table is presented that gives the gross freeboard for different values of $\left(\frac{Q_d}{1.9 \cdot L_s}\right)$ and $\left(\frac{Q_d}{1.7 \cdot L_s}\right)$.

Dam height

The lowest bank of the river and the gross freeboard determine the final height of the crest of the dam. The distance between the crest of the dam and the centre point of the river bottom is called ‘dam height’. The gross freeboard is dimensioned at the design discharge of the river.

To ensure that a discharge higher than the design discharge does not flow across the banks of the river, it is recommended to lower the crest level of the dam half times the gross freeboard in respect of the lowest bank. This way the flow follows the riverbed and does not flow across the banks. It is important to protect the banks. Napier-grass can be used for this protection.

The crest of the dam can be determined with the height of the lowest bank above the centre point of the river bottom minus half times the gross freeboard (see figure 6.8):

$$\text{Dam height} = \text{Distance lowest bank to centre point river bottom} - 0.5 \cdot GF \quad (6.11)$$

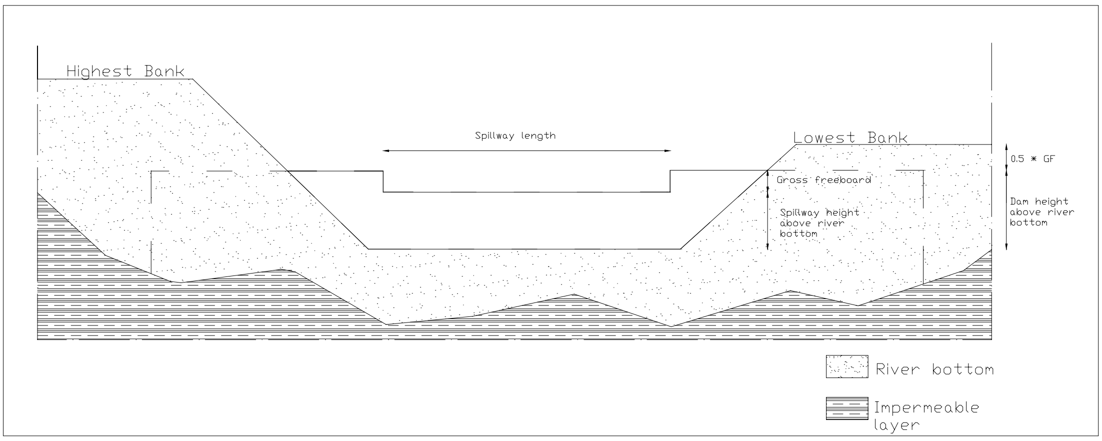


Figure 6.8: Dam height

Spillway height
The height of the spillway crest above the river bottom in the centre point of the river is called 'spillway height' and can be determined by subtracting the gross freeboard from the height of the dam (see figure 6.8).

$$\text{Spillway Height} = \text{Dam height - GF} \qquad (6.12)$$

The upstream and downstream side of the spillway crest are rounded to get a better flow over the spillway.

6.6 Width

6.6.1 SASOL design

The width of the dam depends on the size of the river. The width of the dam is determined, according table 6.11.

Table 6.11: Width of the dam

		Spillway/dam			Wings	
Kind of River	**Width river bottom**	**Base width**	**Width at river bottom**	**Crest width**	**Base width**	**Crest width**
Small	< 5 m	2.0 m	1.0 m	0.75 m	0.60 m	0.60 m
Medium	5 - 10 m	2.0 m	1.0 m	0.75 m	0.60 m	0.60 m
Large	> 10 m	2.0 m	1.5 m	0.75 m	0.60 m	0.60 m

These dimensions arise out of practical experience. The upstream side of the spillway is straight and the downstream side is obliquely, because of the stability of the dam (see figure 6.9). Both sides of the wings are straight.

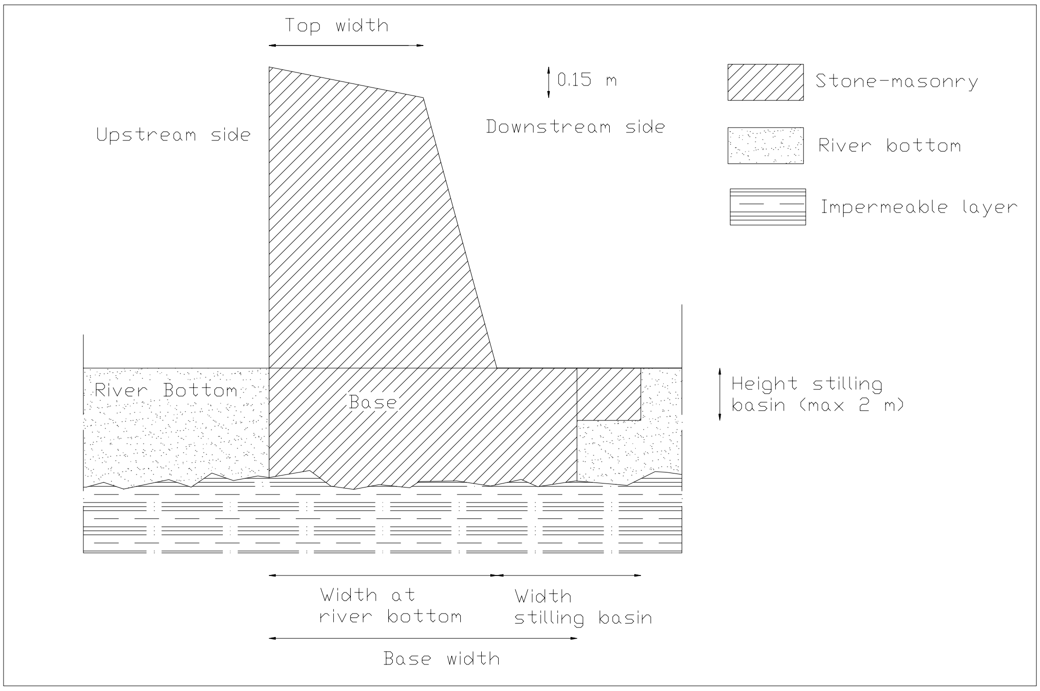


Figure 6.9: Cross-section sand-storage dam

6.6.2 Evaluation SASOL design

- The width of the dam is based on practical experience. The materials used have a wide variation in strength and the construction is not very strict. The minimum possible width of the dam depends on these facts. This makes it difficult to say something about it.
- At first sight the width of the dams is right, because of the quality of the hardcore, the mortar and the construction. The weakest point of the construction is the connection between the base and the spillway at the river bottom level.
- The minimum width of the wings is determined by construction. Because of this the minimum width of the wings is 0.60 m. Making the wings thicker makes no sense because the wings only have to change the direction of the water and the wings do not have a constructional function. Constructing thinner wings is not possible, because the workers cannot dig the foundation trench adequately.

In the next section the width of the dam is determined for different heights above the river bottom.

6.6.3 New design

The relation between the height and the width of the dam depends on the external and internal stability of the dam. *External* stability is assured if overturning and sliding of the dam or the subsoil cannot occur. *Internal* stability is assured when the strength of the construction itself is able to resist all the forces.

The external stability depends on the kind of foundation of the dam. There are two different types of foundation.
The first type is a foundation on solid subsoil like base rock. The dam can be anchored on the base rock with masonry and steel pins. The dam and the subsoil become one solid construction. Sliding and overturning of the dam will only occur if the ultimate shearing strength and tensile strength of the masonry is insufficient. Solid subsoil has no sliding surface, so sliding of the subsoil is not possible.
The second type is the natural foundation, like a foundation on clay and murram. Sliding depends on the slide resistance between the dam and the foundation. Overturning does not only depend on the possibility of overturning of the dam itself, but also on sliding of the subsoil through eccentricity of the resultant force.

The internal stability can be determined by checking the maximum occurring pressure and tensile forces. The strength of the stone-masonry has to resist these forces.

The calculations for checking the external stability are shown below in separate steps. If a step has different methods for the two foundation-types, both are presented.
After that, the internal stability is discussed. Finally a practical approach to use the results of these checks for the stability will be given. Tables are presented to determine the width of the dam for the different retaining heights of a sand-storage dam.

External stability

1. Determination of the horizontal forces and origin of the forces

The resultant horizontal force can be calculated by adding the individual resultant horizontal forces of water pressure and earth-pressure upstream as well as downstream of the dam (see figure 6.10). All forces are multiplied by a safety factor that depends on the fact if the force is favourable or unfavourable and permanent or variable (see table 6.12). For the passive or active earth-pressure coefficient is also applied (table 6.14).

Horizontal water force by upstream water level per running metre:

$$F_{w,up,hor} = \frac{1}{2} \cdot \gamma_w \cdot w_w \cdot (h_{w,up}{}^2 - y^2) \quad (6.13)$$

Horizontal water force by downstream water level per running metre:

$$F_{w,down,hor} = \frac{1}{2} \cdot \gamma_w \cdot w_w \cdot h_{w,down}{}^2 \quad (6.14)$$

Horizontal grain stress on the upstream side per running metre:

$$q_{w,up,hor} = \gamma_w \cdot w_w \cdot h_{w,up} \quad (6.15)$$

$$q_{g,up,ver} = \gamma_g \cdot w_{g,wet} \cdot h_{g,up} \quad (6.16)$$

$$q_{k,up,hor} = (q_{g,up,ver} - q_{w,up,hor}) \cdot K_{a,g} \quad (6.17)$$

$$F_{k,up,hor} = \frac{1}{2} \cdot q_{k,up,hor} \cdot h_{g,up} \quad (6.18)$$

Horizontal grain stress on the downstream side

$$q_{w,down,hor} = \gamma_w \cdot w_w \cdot h_{w,down} \quad (6.19)$$

$$q_{g,down,ver} = \gamma_g \cdot w_{g,wet} \cdot h_{g,down} \quad (6.20)$$

$$q_{k,down,hor} = (q_{g,down,ver} - q_{w,down,hor}) \cdot K_{p,g} \quad (6.21)$$

$$F_{g,down,hor} = \frac{1}{2} \cdot q_{k,down,hor} \cdot h_{g,down} \quad (6.22)$$

Resultant horizontal force:

$$\sum F_{hor} = \sum F_{n,hor} \quad (6.23)$$

Where:

F = Force [kN/m]
q_w = Distributed load of water pressure per m^2 [kN/m^2]
$q_{g,ver}$ = Distributed load of earth-pressure in vertical direction per m^2 [kN/m^2]
$q_{k,hor}$ = Distributed load of grain stress in horizontal direction per m^2 [kN/m^2]

γ = Safety factor [-] (see table 6.12)
w = Specific weight [kN/m^3] (see table 6.13)
K = Active or passive pressure coefficient [-] (see table 6.14)
h = Height of the water level above the foundation level of the dam [m]
y = Water level above the crest of the spillway (above crest level spillway) [m]

Table 6.12: Safety factors

	Favourable	**Unfavourable**
Permanent	0.9	1.2
Variable	0.9	1.5
Water	1	1

Table 6.13: Specific weight

	Specific weight
Water	10 kN/m^3
Wet sand	20 kN/m^3

Table 6.14: Active and passive earth-pressure coefficients

	Active pressure	**Passive pressure**
Sand	0.33	3

The origin of the resultant horizontal force can be determined by calculating the distance ($e_{F,n}$) between the origin of the different horizontal forces to the overturning point P of the construction (see figure 6.10).

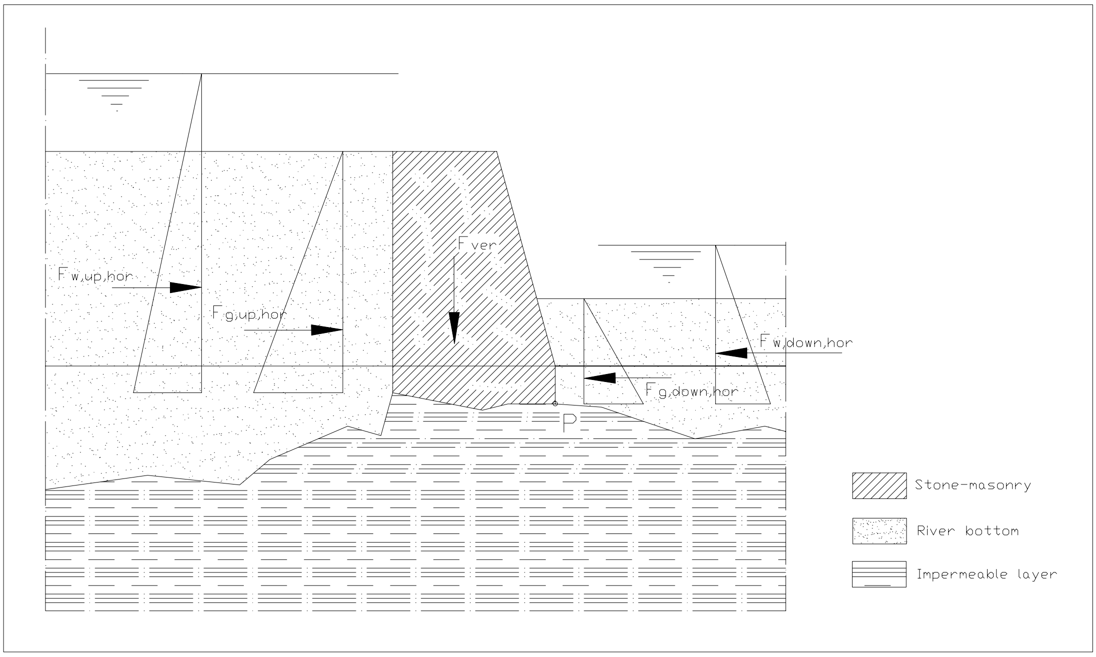


Figure 6.10: Overturning point and forces on the construction of the construction

The origin of the resultant horizontal force is calculated by multiplying the different horizontal forces ($F_{n,hor}$) with their calculated distances ($e_{F,n}$) to the overturning point P. Adding these products and dividing by the resultant horizontal force (ΣF_{hor}) gives the origin:

$$e_{\sum F,hor} = \frac{\sum F_{n,hor} \cdot e_{F,n}}{\sum F_{hor}} \tag{6.24}$$

Where:
e = Distance between the origin of the forces and the overturning point P [m]

2. <u>Determination of the vertical forces and origin of these forces</u>

The resultant vertical force is built up by the weight of the dam:

$$\sum F_{ver} = \sum \left(w_{dam} \cdot V_{dam} \right) \tag{6.25}$$

Where:
F = Force [kN/m]
w = Specific weight [kN/m^3] (see table 6.15)
V = Volume [m^3]

Table 6.15: Specific weight

	Specific weight
Stone-masonry (75% stones, 25% mortar)	24 kN/m^3

The origin of the resultant vertical force can be calculated the same way as the origin of the resultant horizontal force:

$$e_{\sum F,ver} = \frac{\sum \left(w_{dam} \cdot V_{dam} \cdot e_{dam} \right)}{\sum F_{ver}} \tag{6.26}$$

Where:
e = Distance between the origin of the forces and the overturning point P [m]

3. <u>Determination of the resultant overturning moment</u>

The resultant overturning moment can be calculated by adding the product of the resultant forces and the distances between the origin of the resultant forces and the overturning point P. The formula for the resultant overturning moment is given below:

$$\sum M_p = \sum \left(\sum F_n \cdot e_{\sum F,n} \right) \tag{6.27}$$

Where:
ΣM_p = Resultant overturning moment [kNm/m]
ΣF = Resultant force [kN/m]
$e_{\Sigma F}$ = Distance between the origin of the force and the overturning point P [m]

4. Determination of the origin of the resultant force at the foundation base

The origin of the resultant force at the foundation base can be found by dividing the resultant overturning moment by the resultant vertical force:

$$x = \frac{\sum M_p}{\sum F_{ver}} \tag{6.28}$$

Where:

x = Distance between the origin of the resultant force at the foundation base and the overturning point P [m]

ΣM_p = Resultant overturning moment [kNm/m]

ΣF_{ver} = Resultant vertical force [kN/m]

5. Check on overturning

Base rock foundation

In general, for gravity walls, the origin of the resultant force must be within the centre one-third part of the base to prevent sliding of the subsoil due to the eccentric resultant force:

$$\frac{1}{3} \cdot b \leq x \leq \frac{2}{3} \cdot b \tag{6.29}$$

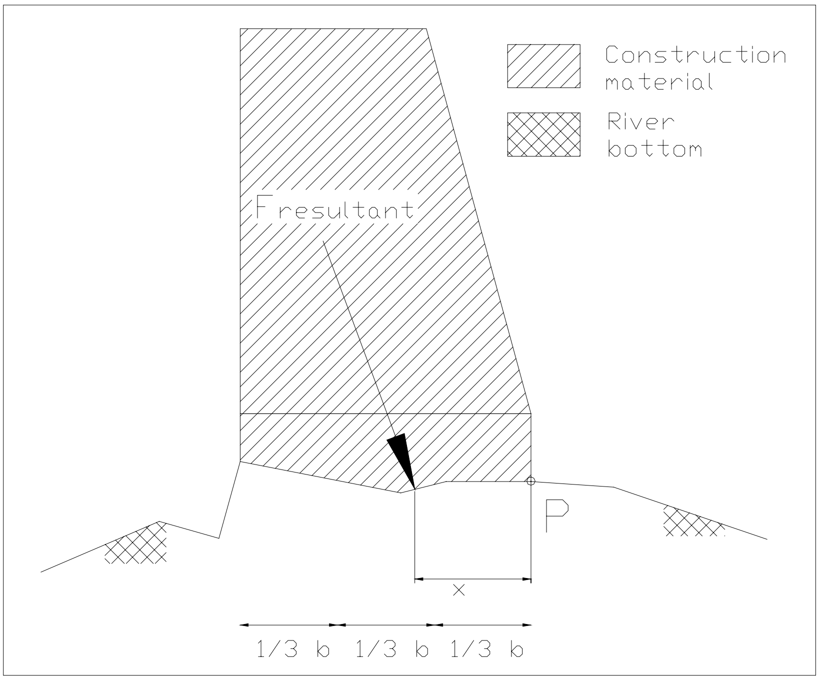


Figure 6.11: Distance between the origin of the resultant force at the base

Where:

x = Distance between the origin of the resultant force at the base and the overturning point P [m]

b = Width of the foundation base [m]

Because a base rock foundation cannot slide, overturning will be prevented if the origin of the resultant force is within the base:

$$0 \leq x \leq b \qquad (6.30)$$

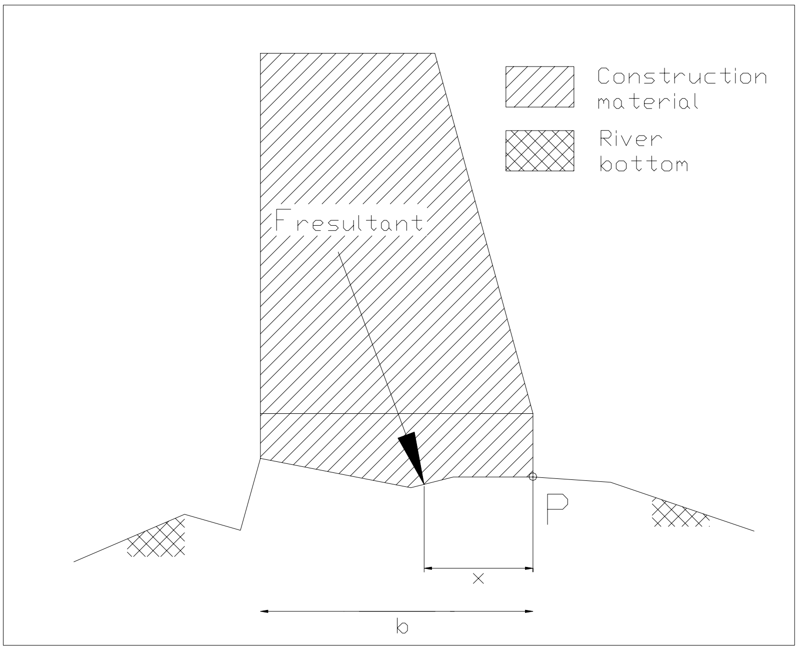


Figure 6.12: Distance between the origin of the resultant force at the base

NOTE: This method differs from the general engineering guidelines so the use of this method is restricted to the described situations and areas.

Clay or murram foundation

For clay and murram foundations the ordinary expression (6.29) for gravity walls is valid.

$$\frac{1}{3} \cdot b \leq x \leq \frac{2}{3} \cdot b \qquad (6.29)$$

Using this rule will make the dams quite big and cumbersome. The sand-storage dams made by SASOL will not satisfy this rule. SASOL builds slender dams.
The reason these slender dams are possible is that the dams are gravity walls as well as restrained constructions in the ground. For clay and murram foundations, the dams are founded at least 1 m below the surface. The rule for gravity walls is derived for constructions that are founded on surface level. The rule for gravity walls (6.28) is too strict for this situation, because the effect of the restrained end of the dam is not taken into account.

To reduce the needed dimensions by calculating the dam as a restrained construction, soil parameters should be known. Because the dams are low-tech, soil analysis is not likely to be done to retrieve soil parameters.
Optimising the dimensions of the dams by calculating them as restrained using soil parameters is not a practical solution.

Another method is used to check the dimension for overturning and sliding of the subsoil. This method is less accurate than calculations with soil parameters, but it can be an alternative if soil parameters are unavailable.

This method uses equation 6.30 for the origin of the resultant force:

$$0 \leq x \leq b \tag{6.30}$$

This rule does not exclude sliding of the subsoil. Another rule, the method of Blum, is used to prevent a sliding surface.

According to the method of Blum, restrained constructions are stable and have no sliding surface if the maximum retaining height is approximately two-third of the total height of the construction:

$$H_{retaining} < \frac{2}{3} \cdot H_{total} \tag{6.31}$$

$$H_{restraint} > \frac{1}{3} \cdot H_{total} \tag{6.32}$$

Where:
$H_{retaining}$ = Retaining height of the dam [m]
$H_{restraint}$ = Restrained height of the dam [m]
H_{total} = Total height of the dam [m]

This means that when the depth of the base of the dam is at 1 m below surface level, the retaining height of the spillway above surface level can maximally be 2 m.

Since this method is less exact the retaining height of the dam should be restricted. The recommendation is to limit the retaining height of the dam founded on clay or murram layers to 3 m.

NOTE: This method differs from the general engineering guidelines so the use of this method is restricted to the described situations and areas.

6. Check on sliding

Base rock foundation
To prevent sliding the construction has to satisfy the following equation:

$$\frac{\sum F_{hor}}{V_u} \leq 1 \tag{6.33}$$

ΣF_{hor} = Resultant horizontal force [kN/m]
V_u = Shearing strength of the stone-masonry [kN/m]

The shearing strength of the stone-masonry can be calculated with the formula:

$$V_u = \tau_1 \cdot b \cdot d \tag{6.34}$$

And:

$$\tau_1 \geq 0.4 \cdot f_b \tag{6.35}$$

Where:

f_b = $0.7 \cdot \dfrac{1.05 + 0.05 \cdot f'_{ck}}{1.4}$

τ_1 = Shear strength [N/mm^2]

b = Width of the sliding surface [m]

d = Length of the sliding surface ≡ 1 [m]

f_b = Tensile strength of concrete [N/mm^2]

f'_{ck} = Cube strength of concrete [N/mm^2]

Formula 6.34 is a safe approach of the shearing strength of the stone-masonry used for the construction of sand-storage dams.

The calculation for the check on sliding of the dam is done with a concrete strength of B5 (Dutch classification system for concrete). The f'_{ck} of B5 is 5 N/mm^2.

N.B.: This way of checking on sliding is only valid if the base rock is cut out until a clean and rough surface has been derived, at the place of the joint of the stone-masonry and the base rock. Reinforcement by placing steel pins into small holes of at least 5 cm deep to get a good connection between stone-masonry and the base rock is recommended. If these rules are followed, the horizontal forces from the construction can be carried to the base rock foundation.

Clay or murram foundation

The retaining height of dams founded on clay or murram layers is limited to two-third of the total height of the dam with a maximum of 3 m (see point 5). At least one-third of the total height of the dam is restrained and the dam will act as a sheet pile wall. Under this condition the dams will be stable and do not have a sliding surface in the subsoil (method of Blum).

7. General comments on external stability

The 6 steps to check the external stability only check the vertical section of the dam per m. They do not take account for the fact that the dam is also (partly) restrained in the banks. If that fact would be taken in account the dams could be dimensioned more optimal.
Because

- the dam is less wide at the place of the banks and
- a lot of factors (quality of construction, soil parameters, etceteras) can not be determined precisely and
- the calculations accounting for the restraining of the banks are more difficult, while the profit is small,

this method is not used.

Internal stability

In the previous part *External stability* overturning by the dam has been checked. For both foundations on base rock and on clay/murram is the method for checking the overturning of the dam different as in the literature for gravity-wall structures. The reason here for is that the sand-storage dams build by SASOL are not only gravity-wall structures but also joint on the base rock in case of a base rock foundation and restrained in the ground in case of a clay/murram foundation. By gravity-wall structures the internal stability is not normative if the resultant of the force is within the center one-third part of the base. In the case of sand-storage dams build by SASOL the internal stability can be normative because the structure differs from gravity-wall structures. In the below part this internal stability of the structure will be checked.

1. Determination of the bending moment

The length of the dam is much longer than the height of the dam most of the time. The width of the spillway increases from the top to the base. Most forces will be carried in vertical direction by the foundation and not in horizontal direction by the banks. The bending moment in vertical direction will be normative and not the bending moment in horizontal direction. Only the bending moment in vertical direction will be determined.

The bending moment in each point of the vertical section can be determined by the following formula (see also figure 6.13):

$$M_n = \sum \left(F_{n,m} \cdot e_{n,m} \right) \tag{6.36}$$

Where:
M_n = Bending moment at point n on the vertical section [kNm/m]
$F_{n,m}$ = Force m acting for the moment at point n [kN/m]
$e_{n,m}$ = Arm of the force from the origin of the force m to the point n [m]

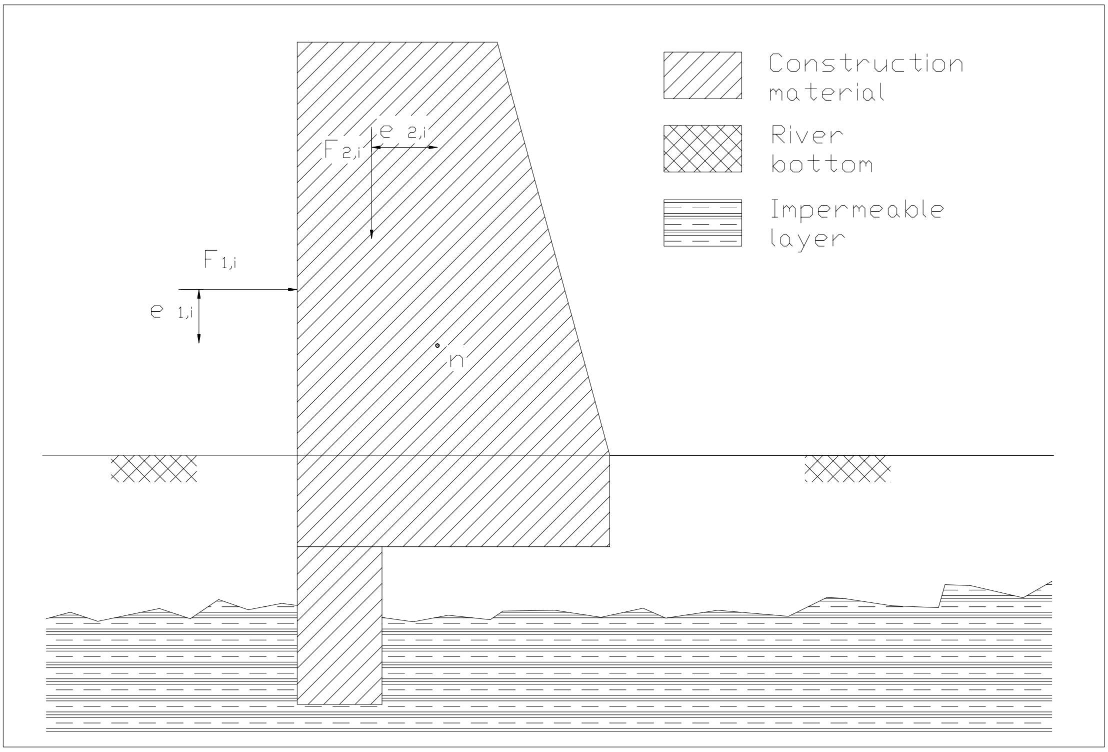


Figure 6.13: Determination bending moment vertical section

2. Determination of the maximum compressive and tensile stresses

Since the width of the dam differs from its crest, the maximum and minimum stresses must be calculated for each point in the vertical section to determine the maximum compressive and tensile stresses in the whole dam. Because the forces on the construction are composed of

non-uniform distributed loads, the moment line is a cubic curve while the width of the spillway increases linearly from the crest to the base (see figure 6.11).
The normative value of the stress is found at the lowest point of the construction (the base). The stress caused by the bending moment will be the highest at this level. Therefore only the stresses in the base have to be determined.

The stress is calculated with the next formula:

$$\sigma_{max} = \sigma_{bc} = -\left(\frac{\sum F_{b,ver}}{b_b \cdot 1}\right) \pm \left(\frac{M_b}{\frac{1}{6} \cdot 1 \cdot b_b^2}\right) \tag{6.37}$$

Where:
σ_{bc} = Maximum stress in the dam in base [$N/mm^2/m$]
$\Sigma F_{b,ver}$ = Resultant vertical force [kN/m]
b_b = Base width of the dam [m]
l = Length of the dam (calculation per metre dam, so l ≡ 1 m) [m]
M_b = Bending moment at the base on the vertical section [kNm/m]

A positive result means tensile stress and a negative result means compressive stress. Now the maximum compressive and tensile stresses of the dam have been determined.

3. Check on compressive and tensile stresses

To check the compressive stress, the dam has to satisfy the following equation:

$$\frac{\sigma'_{max}}{f'_b} \leq 1 \tag{6.38}$$

Where:
σ'_{max} = Absolute value of the maximum compressive stress of the dam [$N/mm^2/m$]
f'_b = Pressure strength of the dam [N/mm^2]

The pressure strength can be calculated with the formula 6.39:

$$f'_b = 0.6 \cdot f'_{ck} \tag{6.39}$$

Where:
f'_{ck} = Cube strength of concrete [N/mm^2]

In case of tensile stress the dam has to satisfy the next equation.

$$\frac{\sigma_{max}}{f_b} \leq 1 \tag{6.40}$$

Where:
σ_{max} = Absolute value of the maximum tensile stress of the dam [$N/mm^2/m$]
f_b = Tensile strength of the dam [N/mm^2]

The tensile strength can be calculated with formula 6.41:

$$f_b = 0.7 \cdot \frac{1.05 + 0.05 \cdot f'_{ck}}{1.4} \tag{6.41}$$

Where:
f'_{ck} = Cube strength of concrete [N/mm^2]

4. Check on shearing forces

In general constructions should be checked on shearing forces. In the case of sand-storage dams shearing forces are not be normative. The following calculations proof this.

Shearing force on horizontal cross-section
A fictive example of a dam with a retaining height of 10 m is used for the calculation. The water level upstream is 3 m above the spillway level. The water level downstream is neglected.

The transverse force in the dam at surface level can be determined by the following formulas.

Horizontal water force by upstream water level:

$$F_{w,up,hor} = \frac{1}{2} \cdot \gamma_w \cdot w_w \cdot (h_{w,up}{}^2 - y^2) \tag{6.13}$$

$$F_{w,up,hor} = \frac{1}{2} \cdot 1 \cdot 10 \text{ kN/m}^3 \cdot \left((13 \text{ m})^2 - (3 \text{ m})^2\right) = 800 \text{ kN/m}$$

Horizontal grain stress on the upstream side:

$$q_{w,up,hor} = \gamma_w \cdot w_w \cdot h_{w,up} \tag{6.15}$$

$$q_{w,up,hor} = 1 \cdot 10 \text{ kN/m}^3 \cdot 13 \text{ m} = 130 \text{ kN/m}^2$$

$$q_{g,up,ver} = \gamma_g \cdot w_{g,wet} \cdot h_{g,up} \tag{6.16}$$

$$q_{g,up,ver} = 1.2 \cdot 20 \text{ kN/m}^3 \cdot 10\text{m} = 240 \text{ kN/m}^2$$

$$q_{k,up,hor} = (q_{g,up,ver} - q_{w,up,hor}) \cdot K_{a,g} \tag{6.17}$$

$$q_{k,up,hor} = (240 \text{ kN/m}^2 - 130 \text{ kN/m}^2) \cdot 0.33 = 36.3 \text{kN/m}^2$$

$$F_{k,up,hor} = \frac{1}{2} \cdot q_{k,up,hor} \cdot h_{g,up} \tag{6.18}$$

$$F_{k,up,hor} = \frac{1}{2} \cdot 36.3 \text{ kN/m}^2 \cdot 10 \text{ m} = 181.5 \text{ kN/m}$$

And:

$$V_d = F_{g,up,hor} = F_{w,up,hor} + F_{k,up,hor} \tag{6.42}$$

$$V_d = 800 \text{ kN/m} + 181.5 \text{ kN/m} = 982 \text{ kN/m}$$

Where:
F = Force [kN/m]
V_d = Transverse force [kN/m]

To prevent shearing, the construction has to satisfy the equation 6.43:

$$\frac{V_d}{V_u} \le 1 \tag{6.43}$$

Where:
V_d = Transverse force [kN/m]
V_u = Shearing strength of the stone-masonry [kN/m]

The shearing strength of the stone-masonry is calculated with formula 6.34:

$$V_u = \tau_1 \cdot b \cdot d \tag{6.34}$$

And:

$$\tau_1 \ge 0.4 \cdot f_b \tag{6.35}$$

Where:
$f_b = 0.7 \cdot \frac{1.05 + 0.05 \cdot f'_{ck}}{1.4}$
τ_1 = Shear stress [N/mm^2]
b = Width of the sliding surface [m]
d = Length of the sliding surface [m]
f_b = Tensile strength of concrete [N/mm^2]
f'_{ck} = Cube strength of concrete [N/mm^2]

To satisfy equation 6.32 the shearing strength should be at least equal to the transverse force.

$$V_u \ge 982 \text{ kN/m}$$

Using B5 for the stone-masonry, the shear stress equals:

$$\tau_1 \ge 0.26 \text{ N/mm}^2 = 260 \text{ kN/m}^2$$

Because the transverse force (V_d) is calculated per running metre, the width of the dam should be at least 3.8 m. The relation between the height and width of the dam is 1 : 0.38. This is very small for dams, so the horizontal shearing forces are not normative, because the external stability demands a wider construction.

Shearing force on vertical cross-section
Shearing on a vertical surface is excluded because the construction has to slide with the subsoil or has to shear off on a horizontal surface too. Sliding of the subsoil is checked in the external stability and shearing on a horizontal surface is checked in the above part. Shearing forces are not normative in this situation either.
No reinforcement is needed to carry the shearing forces.

5. General comments on internal stability

Using reinforcement
If the dam satisfies the equation for the maximum tensile stress (see equation 6.40) in step 3 of the internal stability no reinforcement is needed at all. The required width has been determined for different heights. In these calculations the maximum tensile stress was never normative. The base of the dam is wide, so reinforcement is not required. It can be concluded that the external stability is normative for the dimensions of the dam, especially the width of the dam.

SASOL has built dams and is still building dams that are equipped with reinforcement, barbed wires in the horizontal direction and reinforcement columns in vertical direction. Both are not required as the calculations show. Putting reinforcement to support the construction is a waste of material, money and execution time. The amount of reinforcement that SASOL uses is even not enough to be effective. A simple calculation can prove this.

In general the cross-sectional area of reinforcement in a concrete construction should be at least 0.2 per cent of the cross-sectional area of concrete to be effective in any way. SASOL dams consist of about four layers of average ten strands of barbed wire. The strands of barbed wire are assumed to have a diameter of 3 mm. The cross-sectional area of one strand of barbed wire is 7.1 mm^2 and for 40 strands of barbed wire this is 283 mm^2. A vertical cross-section of a dam is at least 0.75 m wide and 2 m high, so the cross-sectional area is at least 1.5 m^2. The cross-sectional area of barbed wire is less than 0.02 per cent of the cross-sectional area of the stone-masonry and will not be effective.

In vertical direction reinforcement columns are more than 2 m apart. Each reinforcement column consists of four iron bars with a diameter of about 12 mm. The cross-sectional area of one bar is 113 mm^2, for four pieces this area is 452 mm^2. The horizontal cross-section of the dam between the centre-to-centre distance of two columns is at least 2 m long and at least 0.75 m wide, so the cross-sectional area is 1.5 m^2. The cross-sectional area of the iron bars is less than 0.03 per cent of the cross-sectional of the stone-masonry.

From a constructional point of view reinforcement in horizontal direction as well as in vertical direction is not required, but it does give the construction some more coherence. Besides this, the reinforcement has a psychological effect for the communities. By placing reinforcement the communities have more confidence in their dam.

Hence, the use of reinforcement is up to the community and constructors, but it is not recommended, because it only complicates the construction.

Practical approach
The mentioned methods give a practical approach for determining the width of the dam. For different retaining heights and different conditions the width of the dam is calculated. The results of these calculations are put in tables that can be used to determine the dimensions of a new dam. In figure 6.14 the cross-section of the sand-storage dams with the meaning of the different terms, used in the tables, is given.

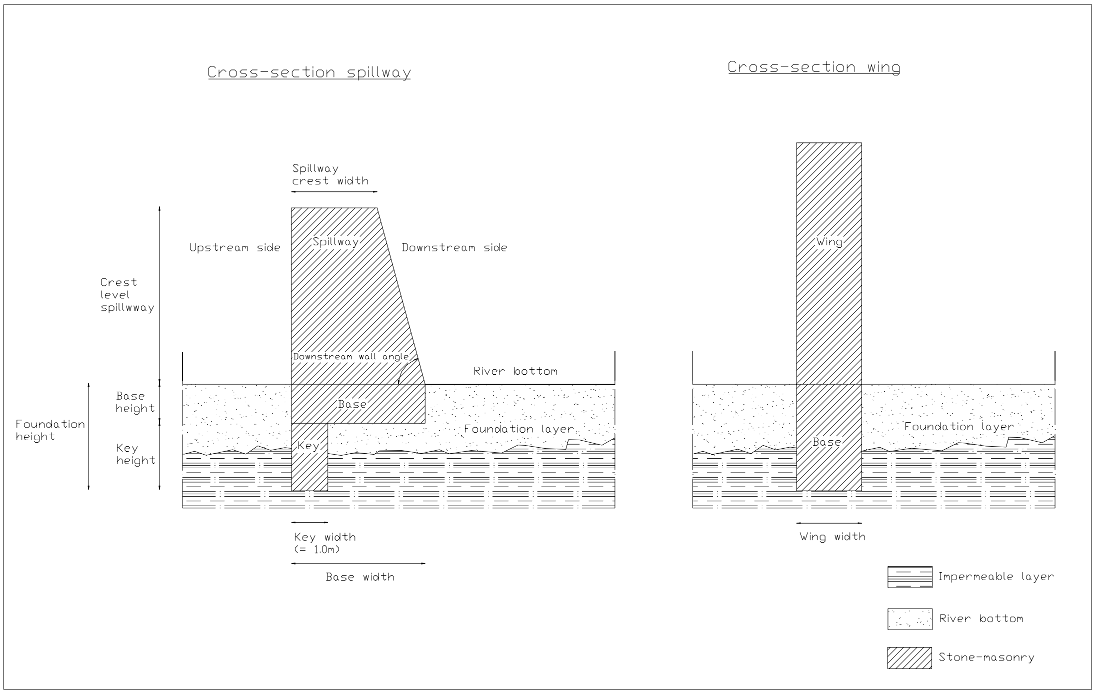


Figure 6.14: Cross-sections sand-storage dam

Instead of building the base of the dam down to the impermeable layer, a key is used. A key is a wall under the base and is built down to the impermeable layer. The key provides a watertight screen from the impermeable layer up to the base of the dam. The advantage of a key is that the width of the key is smaller than the width of base. This saves material and a smaller trench has to be dug out.

These tables for the two foundation-types are given below together with the prior conditions for using these tables.

Base rock foundation
Prior conditions:

- The dam is built with stone-masonry.
- The sand-storage dam must be well anchored in the base rock foundation. Therefore the base rock has to be cut out at the place of the joint of the stone-masonry and the base rock until a clean rough surface is derived. The connection is established by placing ribbed steel pins of at least 0.30 m into small holes of at least 5 cm deep. On 1 m^2 base rock there should be at least 9 pins spread over the area.
- The retaining height of the dam is between 1 and 10 m.
- Dams with a retaining height of more than 5 m should have strict conditions for the location choice. These conditions are as follows:
  - ✓ The site must consist of a wedge. A spillway is not required in this case.
  - ✓ The river should be small (riverbed smaller than 5 m).
  - ✓ The complete dam should be founded on base rock.
  - ✓ The dam has to be built in at least 2 stages to get good sediment in the reservoir.
  - ✓ The dam should be built carefully by an experienced constructor, so the quality of the construction is guaranteed.
- The foundation layer is between 0 and 3 m below the original river bottom.
- At the entire riverbed the dam should have the same width. Starting at the banks, not in the riverbed, the width can be reduced to the width of the wings.
- The wings have the same width as the width of the spillway crest for all dams.

- The width of the crest of the dam is equal to the width of the spillway crest for all dams.
- The highest water level above the spillway is 1.5 times the gross freeboard of the dam. The result should be round up.
- The highest water level above the spillway crest is not more than 3 m.
- If the water level above the spillway crest is different from the given values, the base width of the dam can be determined by adding 0.25 m on the base width per 0.5 m water level.

Table 6.16: Determination dam width for base rock foundations

water level =1.0 m above crest level spillway												
	foundation layer depth 0 m			foundation layer depth 1 m			foundation layer depth 2 m			foundation layer depth 3 m		
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle
m	*m*	*m*	*o*	*m*	*m*	*o*	*m*	*m*	*o*	*m*	*m*	*o*
1	0.80	0.6	78.7	1.10	0.6	63.4	1.10	0.6	63.4	1.10	0.6	63.4
2	1.40	0.6	68.2	1.70	0.6	61.2	1.90	0.6	57.0	1.90	0.6	57.0
3	2.00	0.6	65.0	2.30	0.6	60.5	2.50	0.6	57.7	2.70	0.6	55.0
4	2.50	0.6	64.6	2.90	0.6	60.1	3.10	0.6	58.0	3.30	0.6	56.0
5	3.00	0.6	64.4	3.40	0.6	60.8	3.80	0.6	57.4	4.10	0.6	55.0
6	3.50	1.0	67.4	3.90	1.0	64.2	4.30	1.0	61.2	4.60	1.0	59.0
7	4.00	1.0	66.8	4.40	1.0	64.1	4.80	1.0	61.5	5.10	1.0	59.6
8	4.50	1.0	66.4	4.90	1.0	64.0	5.30	1.0	61.7	5.60	1.0	60.1
9	5.00	1.0	66.0	5.40	1.0	63.9	5.80	1.0	61.9	6.10	1.0	60.5
10	5.50	1.0	65.8	5.90	1.0	63.9	6.30	1.0	62.1	6.60	1.0	60.8

water level =1.5 m above crest level spillway												
	foundation layer depth 0 m			foundation layer depth 1 m			foundation layer depth 2 m			foundation layer depth 3 m		
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle
m	*m*	*m*	°	*m*	*m*	°	*m*	*m*	°	*m*	*m*	°
1	1.05	0.6	65.8	1.35	0.6	53.1	1.35	0.6	53.1	1.35	0.6	53.1
2	1.65	0.6	62.3	1.95	0.6	56.0	2.15	0.6	52.2	2.15	0.6	52.2
3	2.25	0.6	61.2	2.55	0.6	57.0	2.75	0.6	54.4	2.95	0.6	51.9
4	2.75	0.6	61.7	3.15	0.6	57.5	3.35	0.6	55.5	3.55	0.6	53.6
5	3.25	0.6	62.1	3.65	0.6	58.6	4.05	0.6	55.4	4.35	0.6	53.1
6	3.75	1.0	65.4	4.15	1.0	62.3	4.55	1.0	59.4	4.85	1.0	57.3
7	4.25	1.0	65.1	4.65	1.0	62.5	5.05	1.0	59.9	5.35	1.0	58.1
8	4.75	1.0	64.9	5.15	1.0	62.6	5.55	1.0	60.4	5.85	1.0	58.8
9	5.25	1.0	64.7	5.65	1.0	62.7	6.05	1.0	60.7	6.35	1.0	59.3
10	5.75	1.0	64.6	6.15	1.0	62.8	6.55	1.0	61.0	6.85	1.0	59.7

water level =2.0 m above crest level spillway												
	foundation layer depth 0 m			foundation layer depth 1 m			foundation layer depth 2 m			foundation layer depth 3 m		
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle	base width	spillway crest / wing width	downstream wall angle
m	*m*	*m*	°	*m*	*m*	°	*m*	*m*	°	*m*	*m*	°
1	1.30	0.6	55.0	1.60	0.6	45.0	1.60	0.6	45.0	1.60	0.6	45.0
2	1.90	0.6	57.0	2.20	0.6	51.3	2.40	0.6	48.0	2.40	0.6	48.0
3	2.50	0.6	57.7	2.80	0.6	53.7	3.00	0.6	51.3	3.20	0.6	49.1
4	3.00	0.6	59.0	3.40	0.6	55.0	3.60	0.6	53.1	3.80	0.6	51.3
5	3.50	0.6	59.9	3.90	0.6	56.6	4.30	0.6	53.5	4.60	0.6	51.3
6	4.00	1.0	63.4	4.40	1.0	60.5	4.80	1.0	57.7	5.10	1.0	55.7
7	4.50	1.0	63.4	4.90	1.0	60.9	5.30	1.0	58.4	5.60	1.0	56.7
8	5.00	1.0	63.4	5.40	1.0	61.2	5.80	1.0	59.0	6.10	1.0	57.5
9	5.50	1.0	63.4	5.90	1.0	61.4	6.30	1.0	59.5	6.60	1.0	58.1
10	6.00	1.0	63.4	6.40	1.0	61.6	6.80	1.0	59.9	7.10	1.0	58.6

Clay and murram foundation
Prior conditions:

- The dam is built with stone-masonry.
- The retaining height of the dam is between 1 and 10 m.
- The foundation layer is between 0 and 3 m below the original river bottom.
- The base or key of the construction should be at least 0.5 m down in the foundation layer.
- The width of the dam is the same for the whole riverbed. Starting at the banks, not in the riverbed, the width can be reduced to the width of the wings.
- The wings have the same width as the width of the spillway crest for all dams.
- The width of the dam is equal to the width of the spillway crest for all dams.
- The highest water level above the spillway is 1.5 times the gross freeboard of the dam. The result should be round up.
- The highest water level above the spillway crest is not more than 2 m.
- The width of the key is 1 m.
- The minimum foundation depth should be 1 m.
- The height of the base of the construction is 1 m.

Table 6.17: Determination dam width for clay and murram foundations

water level =1.0 m above crest level spillway																								
	foundation layer depth 0 m						foundation layer depth 1 m						foundation layer depth 2 m						foundation layer depth 3 m					
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height
m	*m*	*m*	*°*	*m*	*m*	*m*	*m*	*m*	*°*	*m*	*m*	*m*	*m*	*m*	*°*	*m*	*m*	*m*	*m*	*m*	*°*	*m*	*m*	*m*
1	1.1	0.6	63.4	1.0	0.0	1.0	1.0	0.6	68.2	1.0	0.5	1.5	1.1	0.6	63.4	1.0	1.5	2.5	1.2	0.6	59.0	1.0	2.5	3.5
2	1.7	0.6	61.2	1.0	0.0	1.0	1.6	0.6	63.4	1.0	0.5	1.5	1.4	0.6	68.2	1.0	1.5	2.5	1.7	0.6	61.2	1.0	2.5	3.5
3													1.9	0.6	66.6	1.0	1.5	2.5	2.0	0.6	65.0	1.0	2.5	3.5

water level =1.5 m above crest level spillway																								
	foundation layer depth 0 m						foundation layer depth 1 m						foundation layer depth 2 m						foundation layer depth 3 m					
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height
m	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*
1	1.4	0.6	51.3	1.0	0.0	1.0	1.2	0.6	59.0	1.0	0.5	1.5	1.2	0.6	59.0	1.0	1.5	2.5	1.5	0.6	48.0	1.0	2.5	3.5
2	2.0	0.6	55.0	1.0	0.0	1.0	1.8	0.6	59.0	1.0	0.5	1.5	1.7	0.6	61.2	1.0	1.5	2.5	1.9	0.6	57.0	1.0	2.5	3.5
3													2.2	0.6	61.9	1.0	1.5	2.5	2.1	0.6	63.4	1.0	2.5	3.5

water level =2.0 m above crest level spillway																								
	foundation layer depth 0 m						foundation layer depth 1 m						foundation layer depth 2 m						foundation layer depth 3 m					
crest level spillway	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height	base width	spillway crest / wing width	downstream wall angle	base height	key height	foundation height
m	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*	*m*	*m*	°	*m*	*m*	*m*
1	1.7	0.6	42.3	1.0	0.0	1.0	1.5	0.6	48.0	1.0	0.5	1.5	1.4	0.6	51.3	1.0	1.5	2.5	1.8	0.6	39.8	1.0	2.5	3.5
2	2.2	0.6	51.3	1.0	0.0	1.0	2.1	0.6	53.1	1.0	0.5	1.5	1.9	0.6	57.0	1.0	1.5	2.5	2.0	0.6	55.0	1.0	2.5	3.5
3													2.4	0.6	59.0	1.0	1.5	2.5	2.4	0.6	59.0	1.0	2.5	3.5

6.7 Stilling basin

6.7.1 SASOL design

The downstream part of the base of the spillway (width = 2 m) serves partly as stilling basin too (see figure 6.15). The length of the stilling basin for all river types is 2 m (see table 6.18). The height of the stilling basin is at most 2 m, depending on the foundation depth.

Figure 6.15: The base serves as stilling basin

Table 6.18: Dimensions stilling basin

Kind of river	Spillway/dam Base width	Width at river bottom	Length stilling basin
Small	2.0 m	1.0 m	2.0 m
Medium	2.0 m	1.0 m	2.0 m
Large	2.0 m	1.5 m	2.0 m

The first metre of the stilling basin consists of hard-core. The top of the stilling basin consists of hard-core with mortar.

6.7.2 Evaluation SASOL design

The length of the stilling basin is 2.0 m and the same for all kind of rivers. According to us this is too short for large rivers. It is likely that flowing water will cause erosion directly behind the stilling basin. During the evaluation of the 50 dams a lot of dams with erosion on the downstream side of the stilling basin were found. The downstream end and the sides of the stilling basin are not protected. In the new design these weak points will have to be improved.

Weak points:
- Stilling basin is too short.
- Erosion directly behind the stilling basin.
- No protection on the sides of the stilling basin.

A new design for the stilling basin is presented in section 6.7.3.

6.7.3 New design

In case of a sand-storage dam the main function of the stilling basin is to protect the structure itself from downstream erosion. In the ideal situation a sand-storage dam is constructed on top of a rock bar that crosses the river as shown in figure 6.16. In this *ideal* case a stilling basin is not necessary at all, because the stability of the construction is not endangered by erosion downstream. Of course there will be erosion but this will not affect the foundation of the dam. When the stilling basin is too short or not dimensioned well, a failure mechanism as shown in figure 6.17 can occur.

Figure 6.16: Ideal location of a sand-storage dam

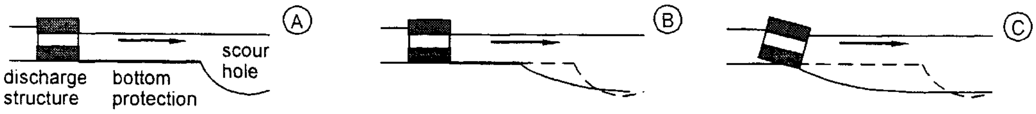


Figure 6.17: Stability of bottom protection [30]

If the ideal conditions are not present, a stilling basin has to be constructed to protect the construction from downstream erosion. The usual layout of a stilling basin is shown in figure 6.18.

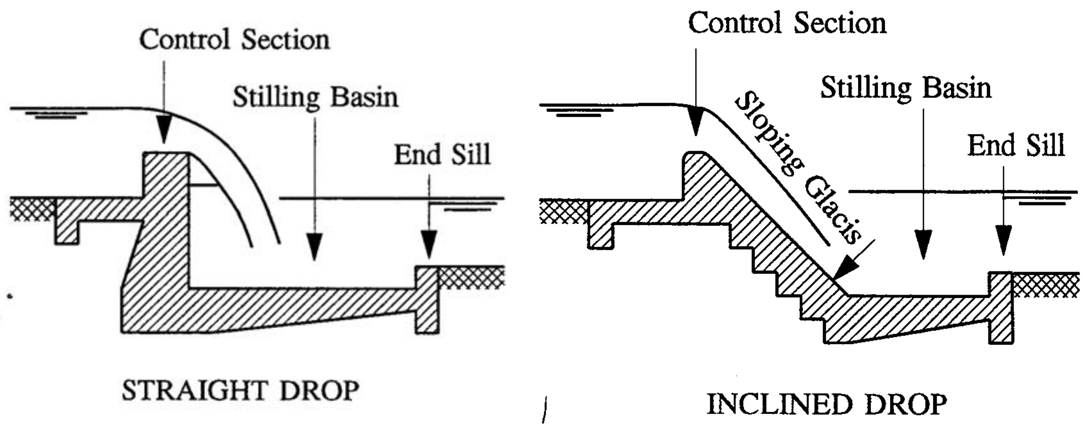


Figure 6.18: Design stilling basin (for key of symbols see page XV) [4]

Because of the simple construction a *straight-drop structure* is preferred above an *inclined drop structure.*

The *vertical-drop basins* are characterised by a free falling jet into the basin. The free-falling jet hits the basin floor and the water flows downstream. At the end of the basin an end-sill is constructed. Up to 50 % of the energy is dissipated by the impact of the jet and by the turbulent circulation in the pool beneath the jet. The remainder part is dissipated by the hydraulic jump downstream (see figure 6.19).

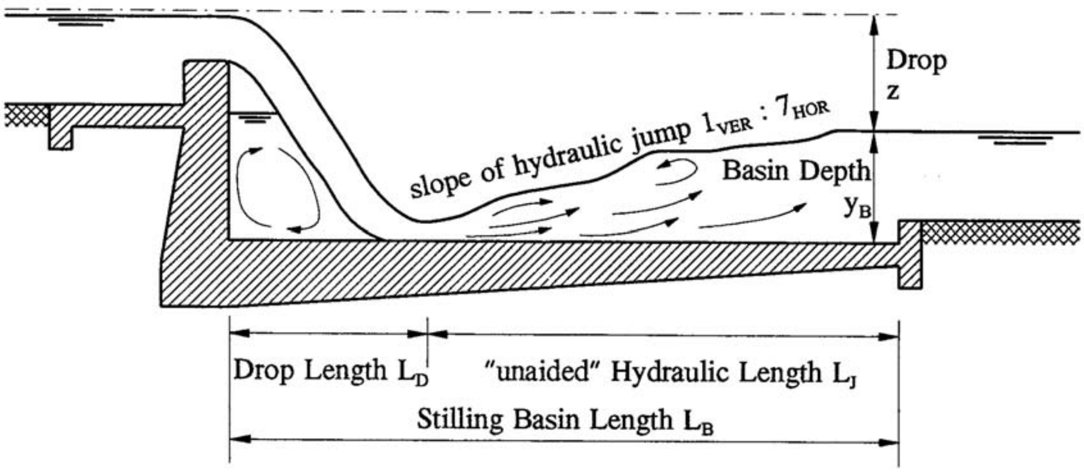


Figure 6.19: Straight drop structure with a 'vertical-drop' basin (for key of symbols see page XV) [4]

To determine the dimensions of the stilling basin, input data about the design discharge (Q_d), the length of the spillway (L_s), the height of the spillway above the original river bed (h_s), the downstream water depth (h_d) and the upstream water depth (h_u) is needed.

Length stilling basin
The length of the stilling basin (L_B) consists of a drop length (L_D) and a hydraulic length (L_J) [4]:

$$L_B = L_D + L_J \tag{6.44}$$

L_D is the horizontal distance covered by the dropping jet:

$$L_D = 5.14 \cdot z^{0.857} \cdot H_a^{0.143} \tag{6.45}$$

Where:

$$H_a = \left(\frac{q}{1.7}\right)^{\frac{2}{3}} \tag{6.46}$$

Where:

$$q = \frac{Q_d}{L_s} \tag{6.47}$$

The head difference over the structure (z) is assumed to be the same as the height of the spillway above the original bottom level (h_s).

In drainage channels it is usually necessary to dissipate all the energy in the stilling basin since erosion in drainage channels is not desirable. In the case of sand-storage dams erosion in the down-stream-section of the river is not a problem most of the times. For this reason the stilling basin will be as long as the distance covered by the dropping jet. This horizontal distance, calculated with formula 6.45, can be long (up to 5 m for a head difference of 1.0 m and a q of 1.4 m^2/s). In the table below the drop length (D_L) is given as a function of q and z.

q = 1.0		q = 2.0		q = 3.0		q = 4.0	
z	Ld	z	Ld	z	Ld	z	Ld
0.2	1.23	0.2	1.31	0.2	1.37	0.2	1.40
0.4	2.23	0.4	2.38	0.4	2.47	0.4	2.54
0.6	3.15	0.6	3.37	0.6	3.50	0.6	3.60
0.8	4.04	0.8	4.31	0.8	4.48	0.8	4.61
1.0	4.89	1.0	5.22	1.0	5.43	1.0	5.58
1.2	5.71	1.2	6.10	1.2	6.34	1.2	6.52
1.4	6.52	1.4	6.97	1.4	7.24	1.4	7.44
1.6	7.31	1.6	7.81	1.6	8.12	1.6	8.34
1.8	8.09	1.8	8.64	1.8	8.98	1.8	9.23
2.0	8.85	2.0	9.46	2.0	9.83	2.0	10.10
2.2	10.10	2.2	11.15	2.2	11.82	2.2	12.32
2.4	10.88	2.4	12.02	2.4	12.74	2.4	13.27
2.6	11.66	2.6	12.87	2.6	13.64	2.6	14.21
2.8	12.42	2.8	13.72	2.8	14.53	2.8	15.15
3.0	13.18	3.0	14.55	3.0	15.42	3.0	16.07

Table 6.19: Drop length, L_d [m], as a function of q [m^2/s] and z [m]

This method of determining the length of the stilling basin leads to long, uneconomical, lengths of the stilling basin. For the practical design of sand-storage dams this method is not appropriate.

Another simple method to determine the horizontal distance covered by the jet is to use the free fall equation. The horizontal distance is:

$$X_{min} = 0.96 \cdot \left(q^{\frac{1}{3}} \cdot h_s^{\frac{1}{2}} \right) \tag{6.48}$$

$$X_{max} = 0.96 \cdot q^{\frac{1}{3}} \cdot \left(h_s + 0.464 \cdot q^{\frac{2}{3}} \right)^{\frac{1}{2}} \tag{6.49}$$

In which q is the discharge per unit of width: $q = Q_d/L_s$ and h_s is the spillway height above the original bottom level (see figure 6.20 for a definition of the parameters). The deduction of these formulas can be found in Appendix 5.

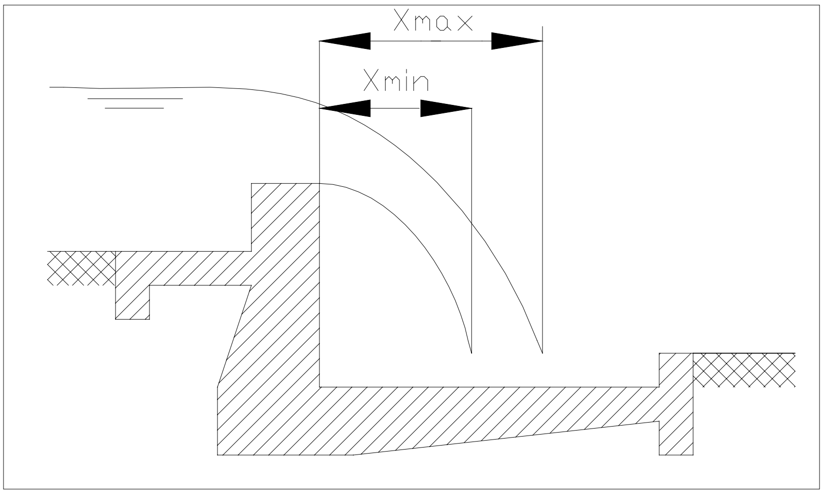


Figure 6.20: Definition of the design parameters(for key of symbols see page XV)

X_{max} determines the length of the stilling basin. In table 6.20 the length of the stilling basin is given as a function of q and h_s.

Table 6.20: Length of the stilling basin, X_{max} [m], as a function of q [m^2/s] and h_s [m].

q = 1.0		q = 2.0		q = 3.0		q = 4.0	
hs	Xmax	hs	Xmax	hs	Xmax	hs	Xmax
0.2	0.78	0.2	1.17	0.2	1.49	0.2	1.78
0.4	0.89	0.4	1.29	0.4	1.62	0.4	1.91
0.6	0.99	0.6	1.40	0.6	1.73	0.6	2.03
0.8	1.08	0.8	1.50	0.8	1.84	0.8	2.14
1.0	1.16	1.0	1.59	1.0	1.94	1.0	2.24
1.2	1.24	1.2	1.68	1.2	2.04	1.2	2.35
1.4	1.31	1.4	1.77	1.4	2.13	1.4	2.44
1.6	1.38	1.6	1.85	1.6	2.22	1.6	2.54
1.8	1.44	1.8	1.93	1.8	2.30	1.8	2.63
2.0	1.51	2.0	2.00	2.0	2.38	2.0	2.71
2.2	1.57	2.2	2.07	2.2	2.46	2.2	2.80
2.4	1.62	2.4	2.14	2.4	2.54	2.4	2.88
2.6	1.68	2.6	2.21	2.6	2.61	2.6	2.96
2.8	1.73	2.8	2.27	2.8	2.69	2.8	3.04
3.0	1.79	3.0	2.34	3.0	2.76	3.0	3.11

Depth of the stilling basin

An over depth of the stilling basin (d_s) of 0.3 x the downstream water (h_d) depth will be sufficient (see figure 6.21). [4] The downstream water depth is assumed to be the same as the upstream water depth (h_u).

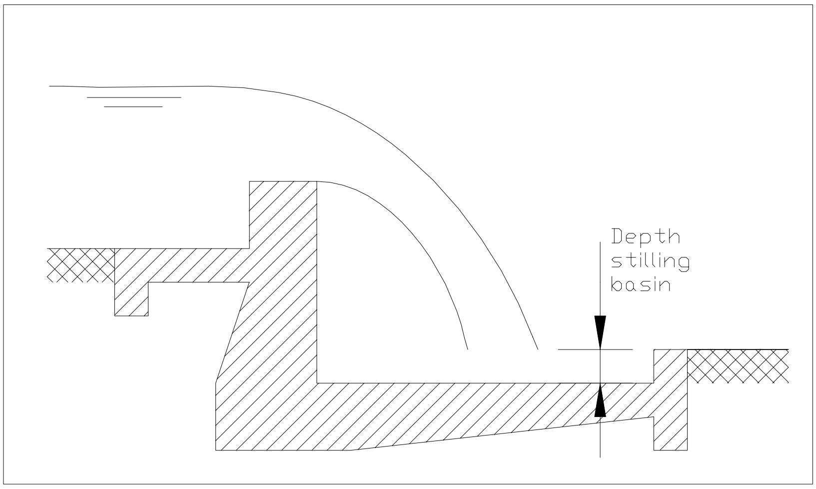


Figure 6.21: Depth of the stilling basin (for key of symbols see page XV)

Width of the stilling basin

The width of the stilling basin is the same as the width of the river bottom.

Bottom of the stilling basin

The foundation of the stilling basin is made out of hard-core mixed with mortar. The top layer is made rough to dissipate more energy in the stilling basin. The thickness of the bottom is 1m at the base of the dam and 0.6 m at the end-sill (see figure 6.22).

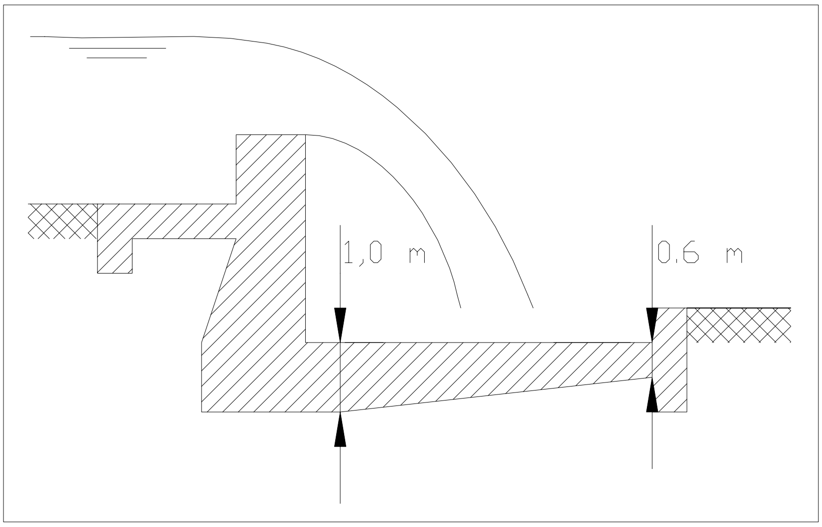


Figure 6.22: Bottom of the stilling basin (for key of symbols see page XV)

Details
For the width of the sides and the end-sill 0.3 m will do.

Underneath the end-sill coffers are constructed.
Coffers protect the floor against scouring-holes in the downstream bed. These scouring holes threaten the stability of the floor of the stilling basin. Coffers will also increase the critical seepage length.
The height of the coffers is 1 m and the width of the coffers is 0.3 m, the same as the width of the end-sill (see figure 6.23). The coffers are constructed over the whole width of the river bottom.

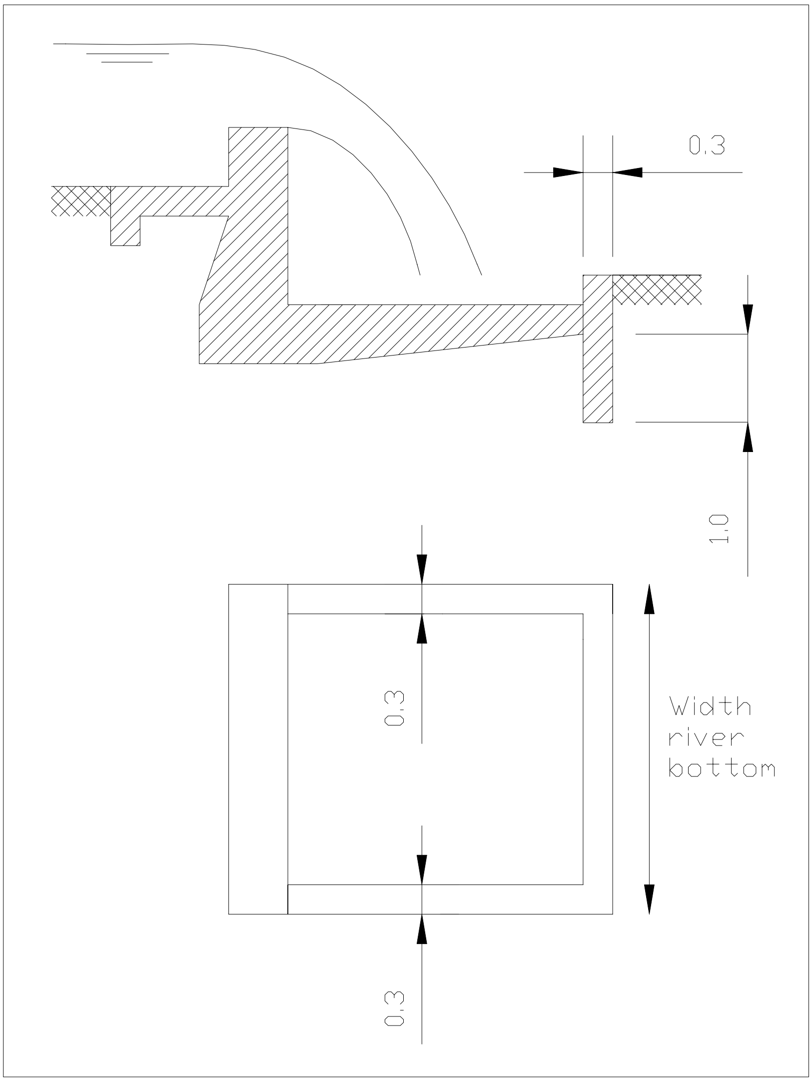


Figure 6.23: Details (for key of symbols see page XV)

The stilling basin will protect the foundation of the dam. Behind the stilling basin, heavy stones are placed to protect the soil underneath. These stones are placed to a length of approximately 4 times the downstream water depth. [4] The diameter of the stones should be heavy enough to withstand the force of the flowing water (see table 6.21).

Table 6.21: Stone diameters in relation to the average velocity and water depth

Waterdepth = 0.5 m		Waterdepth = 1.0 m		Waterdepth = 1.5 m		Waterdepth = 2.0 m		Waterdepth = 2.5 m		Waterdepth = 3.0 m	
Velocity [m/s]	Dn50 [m]	Velocity [m/s]	Dn50 [m]	Velocity [m/s]	Dn50 [m]	Velocity [m/s]	Dn50 [m]	Velocity [m/s]	Dn50 [m]	Velocity [m/s]	Dn50 [m]
0.5	0.003	0.5	0.003	0.5	0.002	0.5	0.002	0.5	0.002	0.5	0.002
1.0	0.023	1.0	0.017	1.0	0.014	1.0	0.013	1.0	0.012	1.0	0.011
1.5	0.090	1.5	0.055	1.5	0.045	1.5	0.039	1.5	0.036	1.5	0.033
2.0	0.317	2.0	0.145	2.0	0.109	2.0	0.092	2.0	0.082	2.0	0.075
		2.5	0.346	2.5	0.233	2.5	0.189	2.5	0.164	2.5	0.147
		3.0	0.914	3.0	0.475	3.0	0.358	3.0	0.300	3.0	0.264
				3.5	1.008	3.5	0.663	3.5	0.526	3.5	0.450
						4.0	1.267	4.0	0.912	4.0	0.746
								4.5	1.641	4.5	1.236
								5.0	3.516	5.0	2.139

The results from table 6.19 have been derived from the following equations [30]:

$$D_{n50} = u_c^2 / \Psi / \Delta / C^2 \tag{6.50}$$

$$C = 18 \cdot \log\left(12 \cdot \frac{h}{k_r}\right) \tag{6.51}$$

$$k_r = 2 \cdot D_{n50} \tag{6.52}$$

Where:
u_c = Critical (average and uniform) velocity on which the protection is dimensioned [m/s]
ψ = Shields factor, considered 0.03 (no motion of the stones at all) [-]
Δ = Relative density of the stones in water and is assumed 1.65 [-]
C = Chézy-factor, found through iteration of (6.50) and (6.51) [$m^{1/2}/s$]
h = Downstream water depth [m]
k_r = Bottom-roughness, assumed to be two times D_{n50} [m]

D_{n50} is, like C, found through iteration of (6.50) and (6.51). D_{n50} is the nominal diameter and equals the side of a cube with the same volume as the stone considered. D_{n50} indicates that 50 % of the weight of all grains is smaller or larger than that value.

The formulas are valid for uniform flow. Since the flow behind the stilling basin is turbulent, the D_{n50} from table 6.19 was adjusted by applying formula 6.53 [30]:

$$D_{n50} = K_v^2 \cdot D_{n50\ \text{uniform flow}} \tag{6.53}$$

K_v is a load increase factor and was chosen 1.5.

The dimensions of the bottom protection behind the dam are designed for a free-discharge weir. In case of a submerged weir the flow attack on the bottom protection will be less heavy and the bottom protection will also meet.

6.8 Dam construction

6.8.1 SASOL design

The dam is constructed out of the following components:

1. <u>Reinforcing columns</u>

Reinforcing columns are placed at intervals of about 2 - 3 m across the length of the dam (wings and spillway). The reinforcing columns consists of:

- 4 vertical round iron bars in the corners of the column
  - Diameter: 12.5 mm
  - Length: depending on height dam (foundation depth)
- Bent square iron bars (horizontal placed) every 0.45 - 0.6 m.
  - Diameter: 6.3 mm
  - Length sides square at the base of the spillway: 0.75 m (depending on depth of dam)
  - Length sides square at the base of the wings: 0.40 m
  - Length sides square at the top of the spillway: 0.45 m
  - Length sides square at the top of the wings: 0.4 m

The round iron bars of the columns are firmly grouted into holes of 5 cm deep that have been cut into the foundation rock.

2. First mortar layer

A layer of cement sand mortar (1:3 mix) is spread on the foundation to a depth of 5 cm. When there is no foundation rock the vertical iron bars are placed in the mortar layer.

3. First horizontal reinforcement layer

After the mortar layer 12 strands of barbed wire are evenly divided over the width of the wings and the spillway, are laid lengthways along it.

4. Second mortar layer

The wires are covered by 5 cm of mortar.

5. Masonry comprising hard-core and mortar

Building proceeds in stages by constructing the wall faces to a height of about 1 m and then filling the middle with masonry comprising clean hard-core (broken stone) and mortar (mostly 1:4 mix depending on quantity of clay in sand). The joints between the rocks are filled with about 25 mm of mortar (1:3 mix). The mortar for filling contains much more water to prevent seepage through the dam. The rocks might be permeable. That is why the dam is not constructed out of masonry only. The mortar will fill up the space between the stones.

6. Horizontal reinforcement layers

After each 0.45 m - 0.60 m strands of barbed wire, evenly divided along the width of the wings and the spillway, are laid lengthways as before. The number of strands of barbed wire reduces with two strands every layer, starting with 10 strands of barbed wire.

7. Plaster layer

The top of the dam is covered with a layer of plaster, around 1.5 cm thick, rounded at the downstream edge to prevent cavitation. The wall on the upper side is also plastered to ensure that the dam is watertight, and an extra thickness of plaster is put at the foot of the wall on either side for the same purpose. The wall on the downstream side is plastered for only 15 cm below the top of the spillway and the wings (see figure 6.24). The plastering of the downstream side is only to make the dam look nice, but it has no function.

Figure 6.24: Plastering the dam

6.8.2 Evaluation SASOL design

In section 6.6.3 it was concluded that no reinforcement for the dams is required at all. In these calculations the maximum tensile stress was never normative, because the base of the dam is wide enough. The use of reinforcement is up to the community and constructors, but it is not recommended, because it only complicates the construction.

6.8.3 New design

The dams can be constructed out of the following parts:

1. <u>First mortar layer</u>

A layer of cement sand mortar (1:3 mix) should be spread on the foundation (5 cm).

2. <u>Masonry comprising hard-core and mortar</u>

The dam consists of constructed wall faces. The joints between the rocks should be filled with about 25 mm of mortar (1 cement : 3 sand). The space between the wall faces is filled up with masonry comprising clean hard-core (broken stone) and mortar (1 cement : 4 sand).

3. Plaster layer

The top of the dam should be covered with a plaster layer of around 1.5 cm. The upstream and downstream edges should be rounded to prevent cavitation. The part of the dam above the surface on the upstream side should be plastered to ensure the water tightness.
The wall on the downstream side can be plastered for example only 15 cm below the top of the spillway and the wings (see figure 6.25).

Figure 6.25: Plastering the dam is the final part of construction

7. Final conclusions and recommendations

To collect water for household and production in Kitui District groundwater dams are the best option. Evaporation losses are reduced or even completely avoided, stored water is less susceptible to pollution and the water is filtrated through the sand.

Within the group of groundwater dams the sand-storage dam is preferred over the sub-surface dam because it stores more water in areas with an impervious layer or steep slopes.

Stone-masonry sand-storage dams are the best solutions for the Kitui District. The costs of the dams are relatively low, the materials are basic and widely available in Kitui, while the skilled labour that is needed for the stone-masonry work is limited. In SASOL's view community based approach fits the stone-masonry dams perfectly. The building of the dam is labour-intensive, maintenance is low and the technology is simple and easy.
The water tightness of the stone-masonry dams is secured and the dams increase the availability of water to the communities. Besides the stone-masonry sand-storage dam SASOL can keep on experimenting with the dam design and the use of materials. Practical experience will prove if theoretical solutions for the sand-storage dams will work. The stone-masonry sand-storage dams can be implemented in other arid and semi-arid lands, when the circumstances are comparable with the situation in Kitui.

Recording and documenting 50 sand-storage dams gave insight and enough information about the weak points of the current design and construction of the dams. The inventory was also useful to gain insight in the effects of the dams on the surroundings. It can be concluded that the dams are functioning properly. More water is available for the community even in the dry periods. But not all aspects of the dam are functioning well. A lot of dams have erosion problems at the downstream sides of the wings and the downstream toe of the construction. A wrong spillway design and the absence of a stilling basin cause this. This threatens the functioning of the dams and should be eliminated. A new design has been made for the spillway and stilling basin. Using this new design will prevent water flowing across the wings. The protection of the banks remains an important item, not only nearby the dam but also further downstream on the river. Planting Napier-grass or other vegetation by the communities will certainly help to decrease erosion.

For the new design of the spillway and stilling basin a design discharge is required. With the Slope-Area Method the design discharges can be estimated on a simple and generally applicable way. If rainfall or discharge data is available more sophisticated skilled methods can be used. In general reliable rainfall, discharge data or information is not available for these small dams because rainfall and geography vary very much within one and the same area.

For the determination of the width of the dam for different heights of the spillway above the river a practical approach using tables for the different foundation heights and conditions is recommended. In these tables a scientific relation between the height and the width of the dam is given. From a constructional point of view reinforcement in horizontal as well as in vertical direction is not required.

It is recommended to ensure that the foundation is dug deep enough in the foundation layer to prevent seepage and/or piping. For base rock foundation special attention has to be paid to the connection between the base rock and the construction.

With the improvements of the design of the SASOL sand-storage dams the design has been optimised as well as possible. Most important is the balance between an optimised design and the practical applicability of this low-skilled technique for community-based approach.

With this optimised design, the practical work report, the collected literature and the acquired impressions during the duties in Kitui District enough information was gained to write a detailed manual on design, location choice, construction and maintenance of sand-storage dams in ASALs. Also a proposal for a hydrologic study on the SASOL sand-storage dams has been drawn up with this information.

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APPENDIX 1: SENSITIVITY ANALYSIS

Results of the sensibility tests for the sub-surface dams are showed below. The weight factors have been changed for each test.

The first column gives the individual score of each option for all 8 criteria. The second column is the result of the multiplication of the individual score with the weight factor for a criterion (see section 3.5.1). The total score without weight factor can be seen from the total of the first column. The total score with weight factor is given in the second column. The last row gives the ranking of the different options. The best option is ranked 1, the worst one 7.

In the first table all criteria have got about the same weight, the second table has an emphasis on costs and maintenance and in the third table the costs and maintenance criteria do not have any weight at all.

Table A1.1: Sensitivity analysis for sub-surface dam 1

Criteria	**Weightfactor**	**Option**
Costs	10	1. Clay dike
Maintenance	15	2. Concrete dam
Durability	15	3. Stone masonry dam
Vulnerability	15	4. Ferro concrete dam
Suitability Kitui	15	5. Brick wall
Construction	10	6. Plastic sheet
Extendable	5	7. Sheets of steel
Water tightness	15	
Total	100	

Option / *Criteria*	1 a	b	2 a	b	3 a	b	4 a	b	5 a	b	6 a	b	7 a	b
Costs	10	100	3	30	4	40	5	50	3	30	10	100	5	50
Maintenance	2	30	10	150	9	135	5	75	5	75	4	60	5	75
Durability	2	30	8	120	9	135	5	75	5	75	4	60	4	60
Vulnerability	2	30	10	150	9	135	5	75	5	75	2	30	8	120
Suitability Kitui	4	60	8	120	8	120	6	90	3	45	8	120	6	90
Construction	10	100	2	20	7	70	3	30	3	30	5	50	3	30
Extendability	1	5	10	50	10	50	1	5	1	5	1	5	10	50
Water tightness	5	75	10	150	8	120	8	120	4	60	8	120	8	120
Total		**430**		**790**		**805**		**520**		**395**		**545**		**595**
		6		2		1		5		7		4		3

Explanation
a = the individual score of each option
b = the result of the multiplication of the individual score with the weight factor
(See also section 3.5.1)
The last row gives the ranking of the different options.

Table A1.2: Sensitivity analysis for sub-surface dam 2

Criteria	**Weightfactor**	**Option**
Costs	60	1. Clay dike
Maintenance	20	2. Concrete dam
Durability	5	3. Stone masonry dam
Vulnerability	5	4. Ferro concrete dam
Suitability Kitui	5	5. Brick wall
Construction	5	6. Plastic sheet
Extendable	0	7. Sheets of steel
Water tightness	0	
Total	100	

Option	1		2		3		4		5		6		7	
Criteria	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	10	600	3	180	4	240	5	300	3	180	10	600	5	300
Maintenance	2	40	10	200	9	180	5	100	5	100	4	80	5	100
Durability	2	10	8	40	9	45	5	25	5	25	4	20	4	20
Vulnerability	2	10	10	50	9	45	5	25	5	25	2	10	8	40
Suitability Kitui	4	20	8	40	8	40	6	30	3	15	8	40	6	30
Construction	10	50	2	10	7	35	3	15	3	15	5	25	3	15
Extendability	1	0	10	0	10	0	1	0	1	0	1	0	10	0
Water tightness	5	0	10	0	8	0	8	0	4	0	8	0	8	0
Total		**730**		**520**		**585**		**495**		**360**		**775**		**505**
		2		4		3		6		7		1		5

Table A1.3: Sensitivity analysis for sub-surface dam 3

Criteria	**Weightfactor**	**Option**
Costs	0	1. Clay dike
Maintenance	0	2. Concrete dam
Durability	10	3. Stone masonry dam
Vulnerability	10	4. Ferro concrete dam
Suitability Kitui	10	5. Brick wall
Construction	20	6. Plastic sheet
Extendable	25	7. Sheets of steel
Water tightness	25	
Total	100	

Option	1		2		3		4		5		6		7	
Criteria	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	10	0	3	0	4	0	5	0	3	0	10	0	5	0
Maintenance	2	0	10	0	9	0	5	0	5	0	4	0	5	0
Durability	2	20	8	80	9	90	5	50	5	50	4	40	4	40
Vulnerability	2	20	10	100	9	90	5	50	5	50	2	20	8	80
Suitability Kitui	4	40	8	80	8	80	6	60	3	30	8	80	6	60
Construction	10	200	2	40	7	140	3	60	3	60	5	100	3	60
Extendability	1	25	10	250	10	250	1	25	1	25	1	25	10	250
Water tightness	5	125	10	250	8	200	8	200	4	100	8	200	8	200
Total		**430**		**800**		**850**		**445**		**315**		**465**		**690**
		6		2		1		5		7		4		3

Results of the sensibility tests for the sand-storage dams are showed below. The weight factors have been changed for each test.

The first column gives the individual score of each option for all 8 criteria. The second column is the result of the multiplication of the individual score with the weight factor for a criterion (see section 3.5.1). The total score without weight factor can be seen from the total of the first column. The total score with weight factor is given in the second column. The last row gives the ranking of the different options. The best option is ranked 1, the worst one 7.

In table A1.4 the focus is on the extendibility and water tightness, while the cost and maintenance do not have any weight at all. In table A1.5 all criteria have got about the same weight and in table A1.6 the analysis has an emphasis on costs and maintenance.

Table A1.4: Sensitivity analysis for sand-storage dam 4

Criteria	Weight factor	Option
Costs	0	8. Concrete sand-storage dam
Maintenance	0	9. Stone masonry sand-storage dam
Durability	10	10. Gabion sand-storage dam with clay cover
Vulnerability	10	11. Gabion sand-storage dam with clay core
Suitability Kitui	10	12. Stone-fill concrete sand-storage dam
Construction	20	13. Stone sand-storage dam
Extendability	25	14. Concrete arch dam
Water tightness	25	
Total	100	

Option / *Criteria*	8 a	b	9 a	b	10 a	b	11 a	b	12 a	b	13 a	b	14 a	b
Costs	2	0	4	0	4	0	4	0	4	0	10	0	1	0
Maintenance	10	0	9	0	1	0	3	0	10	0	5	0	10	0
Durability	10	100	10	100	5	50	6	60	8	80	1	10	10	100
Vulnerability	10	100	10	100	1	10	3	30	10	100	4	40	8	80
Suitability Kitui	5	50	8	80	6	60	6	60	8	80	5	50	1	10
Construction	4	80	8	160	10	200	10	200	4	80	10	200	1	20
Extendability	10	250	10	250	1	25	1	25	5	125	10	250	1	25
Water tightness	10	250	8	200	3	75	3	75	7	175	1	25	9	225
Total		**830**		**890**		**420**		**450**		**640**		**575**		**460**
		2		1		7		6		3		4		5

Table A1.5: Sensitivity analysis for sand-storage dam 5

Criteria	Weight factor	Option
Costs	10	8. Concrete sand-storage dam
Maintenance	5	9. Stone masonry sand-storage dam
Durability	15	10. Gabion sand-storage dam with clay cover
Vulnerability	15	11. Gabion sand-storage dam with clay core
Suitability Kitui	15	12. Stone-fill concrete sand-storage dam
Construction	5	13. Stone sand-storage dam
Extendability	15	14. Concrete arch dam
Water tightness	20	
Total	100	

Option / *Criteria*	8		9		10		11		12		13		14	
	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	2	20	4	40	4	40	4	40	4	40	10	100	1	10
Maintenance	10	50	9	45	1	5	3	15	10	50	5	25	10	50
Durability	10	150	10	150	5	75	6	90	8	120	1	15	10	150
Vulnerability	10	150	10	150	1	15	3	45	10	150	4	60	8	120
Suitability Kitui	5	75	8	120	6	90	6	90	8	120	5	75	1	15
Construction	4	20	8	40	10	50	10	50	4	20	10	50	1	5
Extendability	10	150	10	150	1	15	1	15	5	75	10	150	1	15
Water tightness	10	200	8	160	3	60	3	60	7	140	1	20	9	180
Total		**815**		**855**		**350**		**405**		**715**		**495**		**545**
		2		1		7		6		3		5		4

Table A1.6: Sensitivity analysis for sand-storage dam 6

Criteria	Weight factor	Option
Costs	60	8. Concrete sand-storage dam
Maintenance	20	9. Stone masonry sand-storage dam
Durability	5	10. Gabion sand-storage dam with clay cover
Vulnerability	5	11. Gabion sand-storage dam with clay core
Suitability Kitui	5	12. Stone-fill concrete sand-storage dam
Construction	5	13. Stone sand-storage dam
Extendability	0	14. Concrete arch dam
Water tightness	0	
Total	100	

Option / *Criteria*	8		9		10		11		12		13		14	
	a	b	a	b	a	b	a	b	a	b	a	b	a	b
Costs	2	120	4	240	4	240	4	240	4	240	10	600	1	60
Maintenance	10	200	9	180	1	20	3	60	10	200	5	100	10	200
Durability	10	50	10	50	5	25	6	30	8	40	1	5	10	50
Vulnerability	10	50	10	50	1	5	3	15	10	50	4	20	8	40
Suitability Kitui	5	25	8	40	6	30	6	30	8	40	5	25	1	5
Construction	4	20	8	40	10	50	10	50	4	20	10	50	1	5
Extendability	10	0	10	0	1	0	1	0	5	0	10	0	1	0
Water tightness	10	0	8	0	3	0	3	0	7	0	1	0	9	0
Total		**465**		**600**		**370**		**425**		**590**		**800**		**360**
		5		2		6		4		3		1		7

APPENDIX 2: EVALUATION FORM

EVALUATION FORM DRINKING WATER DAMS KITUI DISTRICT, KENYA

Dam number:	20 A	Supervisor:	British
Dam name:	KKIA - KILANGO	Written design:	YES / ~~NO~~
Name of stream/river:	MWEWE	Written instructions:	~~YES~~ / NO
Deistrict:	KITUI		
Location:	NZAMBANI	Inspector:	CJ599
Grid reference:		Inspection date:	16-04-01
Designer/Engineer:	BRITISH		
Start construction date:	1998	signature	
Completed date:	1998		

River characteristics

Kind of river
- stream [ ]
- river [ ]
- sesional [X]
- permanent [ ]

River slope
- 0.001 [ ]
- 0.005 [ ]
- 0.010 [X]
- otherwise

Dam dimensions

total length L	crest length Lc	spillway length Ls	wing dam length Lw
19,1 m	– m	16,9 / 19,1 m	5,5 / 6,0 m

reference level is the lowest foundation level

embankment crest H (crest level)	spillway crest Hs (FSL)	gross freeboard GF
5,2 / 5,3 m	5,0 m	0,20 / 0,30 m

previous bottom level
2,0 m

crest width W	spillway width Ws	foundation width Wf
– m	0,62 m	0,62 m

wing dam width Ww	wing dam angle
0,62 m	0° degrees

total fill volume V
m3

Upstream slope
- vertical 1:0 [X]
- 1:1 [ ]
- 1:1.5 [ ]
- 1:2 [ ]
- 1:2.5 [ ]
- 1:3 [ ]
- otherwise:

Downstream slope
- vertical 1:0 [X]
- 1:1 [ ]
- 1:1.5 [ ]
- 1:2 [ ]
- 1:2.5 [ ]
- 1:3 [ ]
- otherwise:

Embankment characteristics

Kind of dam
- surface storage dam [ ]
- Groundwater dam [X]
  - Sand storage [X]
  - Sub-surface [ ]
  - Combination [ ]
- Water storage pan [ ]

Plunge basis
- yes [ ]
- no [X]

- Permeable [ ]
- Impermeable [X]

Spillway characteristics

design capacity spillway
m3

Kind of spillway
- common spillway [X]
- spilway canal [ ]
- otherwise: [ ]

Upstream slope
- vertical 1:0 [X]
- 1:1 [ ]
- 1:1.5 [ ]
- 1:2 [ ]
- 1:2.5 [ ]
- 1:3 [ ]
- otherwise:

Downstream slope
- vertical 1:0 [X]
- 1:1 [ ]
- 1:1.5 [ ]
- 1:2 [ ]
- 1:2.5 [ ]
- 1:3 [ ]
- otherwise:

Banks characteristics

Slope
- 1:1 [ ]
- 1:1,5 [ ]
- 1:2 [X]
- 1:2,5 [ ]
- 1:3 [X]
- otherwise:

Well characteristics

diameter D	depth d	distance to the centre of the dam x	y
m	m	m	m

water level Hw	lowest water level Hwl	highest water level Hwh
m	m	m

Water abstraction

handpump	[ ]
electric pump	[ ]
pulley + bucket	[ ]
gravity	[ ]
otherwise	[X] Open water.

Draw-off system

Discription

diameter of pipe	length of pipe	ground level at intake	outlet level at valve
mm	m	m	m

Materials

Outside dam		volumes	*Core dam*		volumes	*Spillway*		volumes
concrete	[ ]	m3	concrete	[X]	m3	concrete	[ ]	m3
masonry	[ ]	m3	reinforcement	[X]	m3	masonry	[ ]	m3
stones	[ ]	m3	Rockfill	[ ]	m3	stones	[ ]	m3
sand-cement blocks	[ ]	m3	Rockfill + cement	[X]	m3	soil	[ ]	m3
soil	[ ]	m3	Gravel	[ ]	m3	kind of soil:		
kind of soil:			soil	[ ]	m3	gabions	[ ]	m3
gabions	[ ]	m3	kind of soil:			otherwise:	[X]	m3
otherwise:	[ ]	m3	sheets	[ ]	m3	kind of material:	Same as core dam	
kind of material:			kind of material:					
			injection screen	[ ]	m3			
			plastic membrane	[ ]	m3			
			otherwise:	[ ]	m3			
			kind of material:					

Protection

vegetation	[ ]	kind of vegetation
riprap	[ ]	
filter material	[ ]	kind of filter material
otherwise:	[ ]	

Conditions

Condition		*Weak points*		
very bad	[ ]	erosion	[X]	place: abutments / wings
poor	[ ]	impermeable	[ ]	place:
acceptable	[X]	stability	[ ]	place:
good	[ ]	cracks	[ ]	place:
excellent	[ ]	settlements	[ ]	place:
		pollution	[ ]	place:
		banks	[ ]	place:
		materials	[ ]	place:
		protection	[X]	place: No protection on banks / erosion by wings
		landslides	[ ]	place:
		abutment	[ ]	place:
		excessive seepage	[ ]	place:
		downstream toe	[ ]	place:
		draw-off sys./well	[ ]	place:
		fences	[ ]	place:
		rats/other rodents	[ ]	place:
		vegetation	[ ]	place:
		road	[X]	place: ends of dam
			[ ]	place:
			[ ]	place:

Area characteristics

Catchment area

suface Ac	mean width Wc	length Lc	longest path length of river
km2	km	km	km

Effects		*Fences*		*Vegetation*	
increasing farming activities	[x]	YES	[ ]	banks:	[illegible]
increasing crops	[x]	NO	[x]	area close by dam:	
healtier grass upstream	[x]			surroundings:	Oranges / citrus / bananas
dry out of land downstream	[ ]				

Topographical and geological conditions encountered at the site and in the catchment area

Soil characteristics

River bottom		*Banks*		*Foundation*	
base rock	[ ]	base rock	[ ]	base rock	[x]
red soil	[ ]	red soil	[ ]	red soil	[ ]
clay	[ ]	clay	[ ]	clay	[ ]
sand	[x]	sand	[ ]	sand	[ ]
otherwise:	[ ]	otherwise:	[x] sandy loam.	otherwise:	[ ]

Groundwater characteristics

groundwater level
surface m

Reservoir characteristics

Dimensions

impounded area	storage capacity	dead storage volume	netto storage volume
m2	m3	m3	m3

Throwback	width	maximum depth
1,5 m	30 m	2,0 m

Flood characteristics

Current velocity

fast	[x]
normal	[ ]
slowly	[ ]

Flood flow

Q50 flood flow	Q100 flood flow
m3	m3

Rainfall characteristics

Mean annual rainfall	Mean annual run-off
mm	Mm3

Dry periods				*Intensity heavy rains*			
january	[ ]	july	[ ]	< 1 hour	[ ]	< 40 mm	[ ]
february	[x]	august	[x]	1 - 2 hours	[x]	40 - 60 mm	[x]
march	[x]	september	[x]	2 - 3 hours	[ ]	60 - 80 mm	[ ]
april	[ ]	october	[x]	3 - 4 hours	[ ]	80 - 100 mm	[ ]
may	[ ]	november	[ ]	4 - 5 hours	[ ]	100 - 120 mm	[ ]
june	[ ]	december	[ ]	otherwise:	[ ]	otherwise:	[ ]

Evaporation characteristics

open water surface	mean annual evaporation
m	mm

Sedimentation characteristics

Kind of sediment		*Percentage*	*Reservoir filled rate*	
clay	[ ]		empty	[ ]
sand	[x]		1/4	[ ]
gravel	[ ]		1/2	[x]
stones	[ ]		3/4	[x]
otherwise	[ ]		1/1	[ ]

Erosion characteristics

Place of erosion		*Position*	*Erosion depth*
banks upstream	[ ]		m
bottom downstream	[ ]		m
banks downstream	[ ]		m

Users characteristics

accessibility dam		*Community involvement*			*Uses stored water*		*Use per person/day*
very bad	[ ]	Organisation:	[X]	yes	domestic water supply	[X]	____ l
poor	[ ]		[ ]	no	livestock supply	[ ]	____ l
acceptable	[ ]	Tasks: poor.	[X]	maintenance	irrigation	[X]	____ l
good	[X]		[ ]	instructions for use	wildlife supply	[ ]	
excellent	[ ]		[X]	inspections	otherwise:	[ ]	____

Number of users	*Distance village-well*
800 p	0,5 km
	7-8 min

Not in service

january	days	july	days
february	days	august	days
march	days	september	days
april	days	october	days
may	days	november	days
june	days	december	days

Total days:

General opinion water available quantity		*General opinion users drinkwaterdam*	
shortage	[ ]	very bad	[ ]
insufficient	[ ]	poor	[ ]
even	[ ]	acceptable	[ ]
enough	[ ]	good	[X]
plenty of water	[X]	excellent	[ ]

Comment users

They used it as a bridge.
Happy about water.

Construction cost

Total cost: 150.000 Ksh

Recommended maintenance and improvements

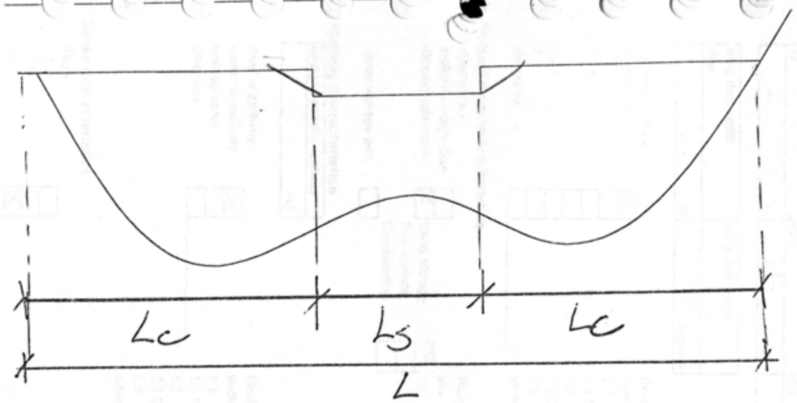
		comments
protection banks upstream	[X]	Abutments/wings specially.
protection banks downstream	[X]	
protection river bottom downstream	[ ]	
repair cracks	[ ]	
improving stability	[ ]	
repair cracks	[ ]	
place (new) fences	[ ]	
repair weak materials	[ ]	
improve protection	[X]	
landslides: recover and protection	[ ]	
adapt abutment	[ ]	
decrease seepage	[ ]	
protect downstream toe	[ ]	
repair draw-off system/well	[ ]	
repair fences	[ ]	
remove vegetation	[ ]	
other things:	[ ]	
fill up top dam	[ ]	
____	[ ]	
____	[ ]	
____	[ ]	

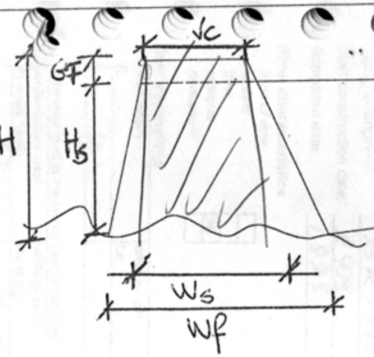
Photos of the features of the dam (weak points etc.)

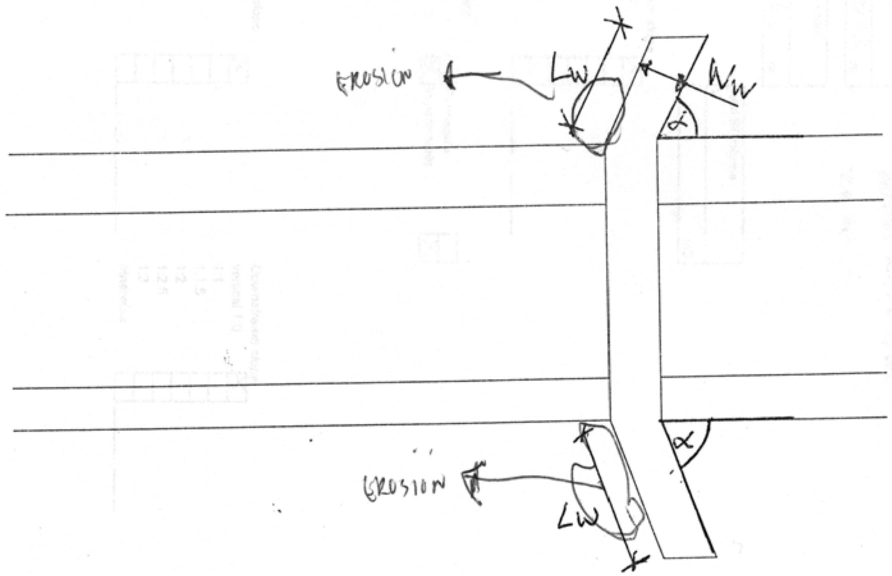
Description photo 1: ____ Description photo 3: ____
Description photo 2: ____ Description photo 4: ____

Other interesting things

There was a little bit seepage. The community has repaired it by there own
own initiativ.



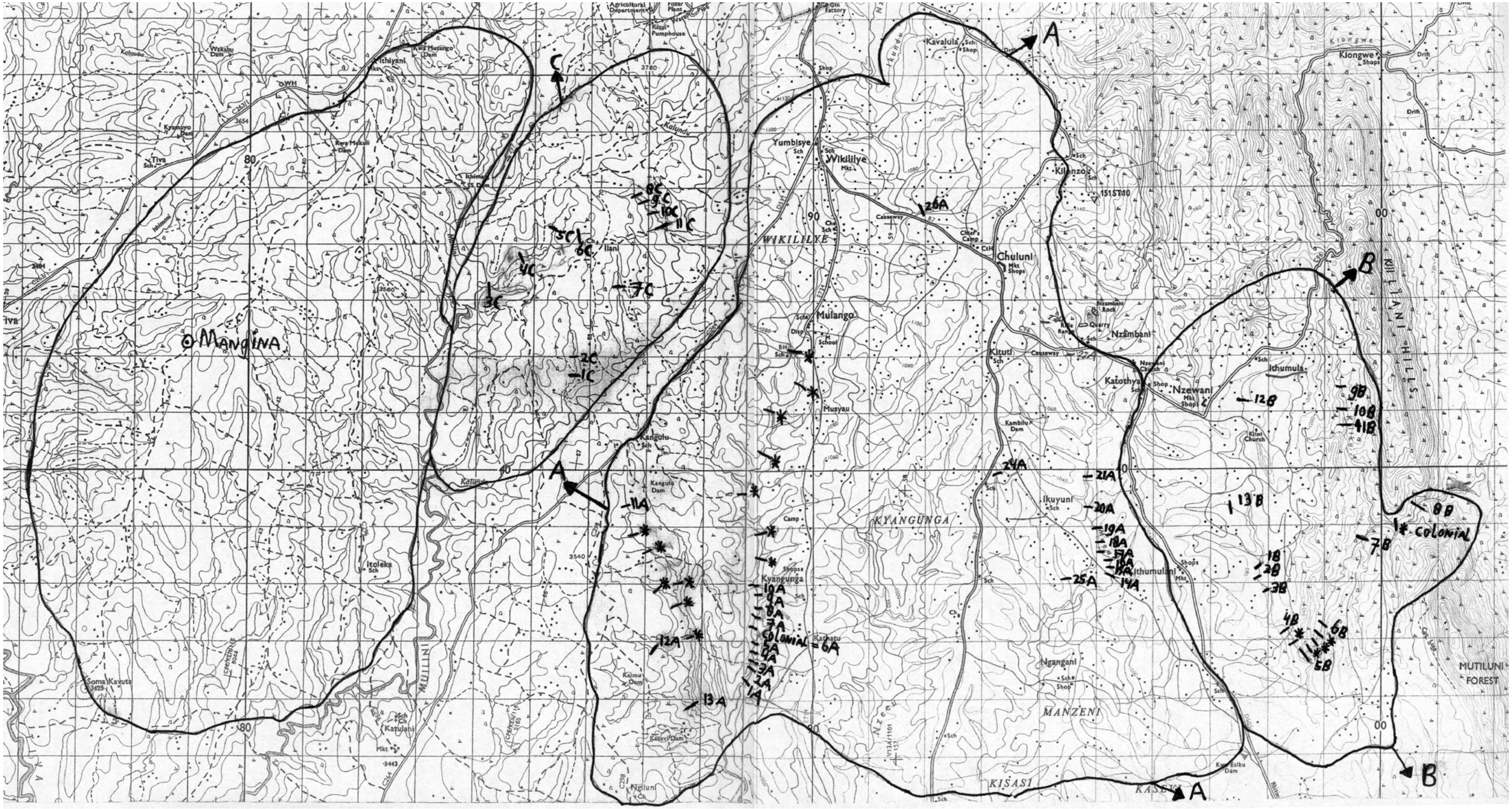




Dam Number :

Dam Name :

APPENDIX 3: INVESTIGATED CATCHMENT AREAS



A
B
C
MANGINA
WIKILILYE
KYANGUNGA
MANZENI
KISASI
KILILIANI HILLS
MUTILUNI FOREST
MUILUNI
Yumbisye
Wikililye
Mulango
Musyau
Chuluni
Kituti
Kalulula
Kilonzo
Nzambani
Nzewani
Katothya
Ithumula
Ithumulani
Ikuyuni
Kangulu
Kangulu Dam
Kyangunga
Kathatu
Nganganl
Ilani
Ithiyani
Kwa Musungo Dam
Kwa Mukuli Dam
Kyamuyo Dam
Itoleka
Katulani
Kaimu Dam
Kalundu
Kiongwe
Kiongwe Shops
Agricultural Department
Filter Plant
Kitui Pumphouse
Soma Kavuta
Kwa Esiku Dam
Kambilu Dam
Kilimi Church
COLONIAL
1A
2A
3A
4A
5A
6A
7A
8A
9A
10A
11A
12A
13A
14A
15A
16A
17A
18A
19A
20A
21A
24A
25A
26A
1B
2B
3B
4B
5B
6B
7B
8B
9B
10B
11B
12B
13B
1C
2C
3C
4C
5C
6C
7C
8C
9C
10C
11C

APPENDIX 4: INVENTORY 50 SAND-STORAGE DAMS

Evaluation Sand-Storage Dams, Kitui District

DATA AND PRIMARY CALCULATIONS

Date: June 2001
Inspection Sand-Storage Dams: april 2001
by Projectgroup CF599, TU Delft

Explaination

*		#3		#5		#5S	
	(1) yes		(1) fast		(1) very bad		(1) shortage
	( ) no		(2) normal		(2) poor		(2) insufficient
			(3) slowly		(3) acceptable		(3) even
					(4) good		(4) enough
					(5) exellent		(5) plenty of water

no.	code	name dam	area	name river	execution					materials	length						width			heigth				angle	volume dam	embankment		banks	water abstraction								protection				condition																		
												wings		spillway			crest													well							materials					weak points																	
					duration	competion date	age	rebuild of repair date	total human workdays	cement	overall	left	right	base	left	right	base	top	wing	gross freeboard	spillway	embankment	average height	wing		plunge basin	impermeability	slope	draw-off system	handpump	electric pump	bucket (+ pulley)	gravity	others	scoop holes	others	vegetation	riprap	filter material	others	condition	erosion	stability	landslide	settlement	cracks	seepage	impermeable	polution	rats/other rodents	materials	bank	abutment	downstream toe	protection	draw-off system/well	fence	vegetation	others
					days		years		days	bag (50 kg)	m	m	m	m	m	m	m	m	m	m	m	m	m	degrees	m3	*	*	1: x	*	*	*	*	*	*	*	*	*	*	*	*	#5	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
1	1A	Kamumbuni	A	Kiindu river	27	maart 1995	6		830	115	28	0	0	28	0	0	1	1	0	0	3	3	0,93		26		1	5							1		1				5						1													
2	2A	Kwa Kavoo	A	Kiindu river	16	juni 1995	6		286	68	30	0	0	30	0	0	1	1	0	0	3	3	1,90		57		1	5							1		1				3	1	1		1	1						1	1							
3	3A	Kwa Mutinga	A	Kiindu river	30	juni 1995	6		857	120	26	0	0	26	0	0	1	1	0	0	5	5	1,29		33		1	3							1		1				4					1	1													
4	4A	Kwa Mukumbe 1	A	Kiindu river	21	juni 1995	6		291	64	19	3	0	16	0	0	1	1	0,6	0	3	3	0,61	35	12		1	1				1			1						4	1											1		1	1				
5	5A	Kwa Mukumbe 2	A	Kiindu river	24	juni 1995	6		502	83	16	0	4	12	0	0	1	1	0,87	0	1	1	1,06	30	17		1	1,5							1						4	1											1							
6	6A	Uvalti (colonial dam	A	Kiindu river	N/A	1959	6		N/A	N/A	17	0	0	17	0	0	0,9	0,6	0	0	N/A	N/A	2,50		32		1	0							1					1	5												1							
7	7A	Nzemeini	A	Kiindu river	26	maart 1996	5		378	95	22	0	0	22	0	0	1	1	0	0	5	5	1,16		26		1	3				1			1		1				4																			
8	8A	Kwa Langwa	A	Kiindu river	45	juli 1996	5	1997	1465	150	46	0	11	35	0	0	1,2	1	0,6	0	4	4	2,57	45	130		1	1,5				1			1		1				2		1				1						1		1	1				
9	9A	Kwa Mangya	A	Kiindu river	17	juli 1996	5		915	72	16	0	0	16	0	0	1	1	0	0	3	3	1,44		23		1	1				1			1		1				5															1				
10	10A	Kwa Ndunda	A	Kiindu river	32	juli 1996	5	november 1997	467	80	23	0	0	23	0	0	1	1	0	0	1	1,7	1,31		30		1	1				1			1		1				3	1											1							
11	11A	Kwa Tito	A	Katili	42	september 2000	1		250	161	17,5	4	3	1,5	4,5	4,5	1	0,75	0,75	0,3	7	7,3	1,28	0	20	1	1	0,5				1					1				4	1												1						
12	12A	Kwa Mutia	A	Katili	24	september 2000	1		789	100	17	3	7	5,5	0,5	1	0,75	0,75	0,75	0,3	3	3,3	2,28	0	29		1	1				1					1				3	1																		
13	13A	Kwa Wambua	A	Kwa Ngoo	28	september 2000	1		464	155	30	8	8	10	2	2	1	0,75	0,63	0,6	3	3,6	0,76	0	20	1	1	1						1			1				5																			
14	14A	Kathini	A	Mwewe	91	1997	4	2000	1880	212	47	3	2,3	13,6	20	8,1	1	1	1	1,9	5,1	7	4,00	0	188		1	1							1						4	1													1					1
15	15A	Ndia Aimu	A	Mwewe	162	juli 1997	4		1620	371	43	8,2	12	13,6	4,6	4,6	1,5	0,8	0,85	0,8	3	3,8	2,07	0	102		1	0							1		1			1	4	1											1							
16	16A	Kwa Milu	A	Mwewe	127	juli 1997	4		2014	325	43	14	12	6	6	5	2	1	0,94	0,7	3,3	4	1,55	0	100		1	1							1		1				5	1										1								
17	17A	Tendelya	A	Mwewe	86	juli 1997	4		613	126	31	7	6	10	4	4	0,8	1	0,8	0,6	2	2,6	0,66	0	18		1	1							1		1				4	1																		
18	18A	Kyanguni	A	Mwewe	162	juli 1997	4		1836	374	52	8,3	24,8	10,5	4,2	4,2	1,45	1,15	0,95	0,4	2	2,4	1,91	0	129		1	2							1		1				1	1	1			1	1					1								1
19	19A	Imooni	A	Mwewe	66	juli 1997	4		671	221	36	8	4,1	16,5	3,7	3,7	1	1,5	0,85	0,6	3	3,6	1,48	0	67		1	1							1		1			1	4	1					1								1					1
20	20A	Kwa Kilango	A	Mwewe	82	1998	3		1035	162	30,6	5,5	6	16,9	1,1	1,1	0,75	0,65	0,62	0,3	5	5,3	3,50	0	75		1	2							1						3	1										1			1					1
21	21A	Mitauni	A	Mwewe	55	juni 1999	2		665	142	25	6,3	5,1	5,4	5,5	2,7	1	0,75	0,8	1,2	3	4,2	1,76	0	38		1	1				1			1		1			1	3	1											1		1					
22	22A	Musalani	A	Mwewe	23	juni 1997	4		367	95	24	5,6	6,4	5,4	3,3	3,3	1	0,75	0,7	0,9	3	3,9	1,41	0	30		1	1,5				1			1		1				5																			
23	23A	Miuni	A	Mwewe	58	oktober 1999	2		331	150	37,4	7	12,7	7,2	5,3	5,2	1	0,75	0,71	0,6	4	4,6	2,40	0	79		1	3				1			1		1				5	1																		
24	24A	Kithumulani	A	Kiliva	52	november 2000	1		652	126	24,9	12	6,2	3,25	2	1,45	1	0,75	0,55	1,3	3	4	1,81	25	39		1	3				1			1		1				5											1								
25	25A	Kamukui	A	Kiliva	42	oktober 2000	1		567	125	14	3	2,8	2,6	2,8	2,8	1	0,75	0,66	1,5	2	3,2	4,00	0	49		1	1				1			1		1				5	1													1					
26	26A	Kakunike A	A	Kakunike	48	september 1998	3		766	130	41	11	20	6	2	2	1	0,75	0,75	0,4	2	2,4	1,82	50	65		1	3				1			1					1	5																			
27	1B	Munyuni A	B	Kyamukaa	18	februari 2000	1		369	107	28	9	9,5	4,7	2,4	2,4	1	0,7	0,65	0,5	4	4,5	2,27	0	54		1	1							1						2	1										1	1	1	1					1
28	2B	Munyuni B	B	Kyamukaa	12	februari 2000	1		367	78	22,5	6,3	8,8	3,4	2	2	1	0,6	0,67	0,6	1	1,6	1,75	0	31		1	1							1		1				4	1												1	1					
29	3B	Kianguni	B	Kyamukaa	19	februari 2000	1		556	89	32	8	10	7	3,5	3,5	1	0,75	0,77	0,6	0,5	1,1	1,15	0	32		1	7							1						4	1											1		1					
30	4B	Kavithe B	B	Kyamukaa	85	februari 2000	1		565	157	27	9	7	9	1	1	1	0,75	0,7	1	3	4	4,64	0	110		1	1							1						4	1											1		1					
31	5B	Kyangala B	B	Kyamukaa	53	april 2001	0		230	100	24	4,6	13,2	6,2	0	0	1	0,75	0,73	0,7	4	4,7	1,72	0	36		1	1							1						4	1										1	1		1					
32	6B	Kwa Ngungu A	B	Kyamukaa	8	juli 2000	1		240	25	10	0	0	10	0	0	1	1	0	0	1	1	0,46		5		1	2							1		1				5																			
33	7B	Nguni	B	Kyamukaa	26	februari 2000	1		402	90	31	5	7	9	6	4	1	0,75	0,7	0,8	1	1,8	1,43	0	39		1	1							1		1				5					1														
34	8B	Kyangombe	B	Kyamukaa	52	juli 2000	1		649	89	47	8	4	28	3	4	1	0,75	0,75	0,9	1	0,5	0,69	0	28		1	1							1		1				5																			
35	9B	Musaala	B	Kiliku	19	juli 2000	1		541	106	25,9	8	8,1	4,9	2,9	2	1	0,75	0,65	0,8	1	1,8	2,11	0	48		1	2							1		1				2	1										1	1		1					1
36	10B	Metika Mbuu A	B	Kiliku	36	juli 2000	1		504	126	26	6	3	7	5	5	1	0,75	0,65	1,3	2	1,3	2,02	0	46		1	1							1		1				4	1																		
37	11B	Metika Mbuu B	B	Kiliku	31	september 2000	1		527	107	25	6	3	6	5	5	1	0,7	0,65	2	1	3	2,04	0	43		1	2							1		1				4	1																		1
38	12B	Nganzani	B	Nganzani	33	augustus 2000	1		410	95	29	19	2	6	1	1	1	0,75	0,7	0,3	1	1,3	1,27	0	32		1	1							1		1				5																			
39	13B	Kwa Kavula	B	Nganzani	27	augustus 2000	1		408	115	52	0	0	52	0	0	1	0,7	0,8	0	0,8	0,8	0,79		35		1	2							1		1				5																			
40	1C	Kwa Ndifu	C	Vili	55	augustus 1999	2		806	178	27	9,5	9,5	4	2	2	1	0,75	0,7	0,6	3	3,6	2,55	0	60		1	1				1			1		1				3	1													1					1
41	2C	Kwa Masila	C	Vili	44	oktober 1999	2		470	133	25,7	13,6	4,2	3,5	2,2	2,2	1	0,75	0,75	0,7	3	3,7	2,25	0	51		1	2							1		1				4	1													1					
42	3C	Kwa Nziani	C	Kilamba	62	oktober 1999	2		849	195	39,1	7	21,8	4,3	3,5	2,5	1	0,75	0,6	0,7	2	2,7	2,15	0	73	1	1	1,5							1		1				4	1												1	1					
43	4C	Kwa David 1	C	Kilamba	36	oktober 1999	2		656	138	28	7	11	4	3	3	1	0,75	0,62	1	3	3,7	1,92	0	47		1	1							1						4	1																		
44	5C	Kwa David 2	C	Kilamba	33	oktober 1999	2		418	108	28	10	7,4	4	3,3	3,3	1	0,75	0,65	1,1	4	4,7	2,21	0	54		1	1				1			1						4	1					1													1
45	6C	Kwa Robert	C	Kilamba	25	oktober 1999	2		381	81	18	4,5	8,5	2	1,5	1,5	1	0,75	0,7	0,2	1	1,2	1,55	0	24		1	1				1			1		1				4	1													1					
46	7C	Kwa Katiwa	C	Maungu	49	februari 2000	1		358	159	22,6	3	13	3	1,8	1,8	2	0,8	0,6	0,6	8	8,6	4,55	0	144		1	1							1						4	1					1								1					1
47	8C	Kwa Mukola	C	Mutungini	36	september 1999	2		466	111	27	4	14,5	4,1	2,2	2,2	1	0,8	0,65	0,5	4	4,5	2,80	0	68		1	1				1			1		1				5																			
48	9C	Kwa Nguthu	C	Mutungini	27	oktober 1999	2		177	121	16,9	4,3	5,3	4,3	1,5	1,5	1	0,75	0,67	0,4	2	2,4	2,13	0	31		1	1							1		1				4	1																		1
49	10C	Mutungini	C	Mutungini	25	september 1999	2		385	124	20,9	7	3,4	4,3	3,4	2,8	1	0,75	0,75	0,8	3	3,8	4,21	0	77		1	1							1		1				4	1																		
50	11C	Munandani	C	Mutungini	49	september 2000	1		463	149	27	6,9	7,2	5,45	3,15	4,3	1	0,75	0,63	0,6	3	3,6	2,06	0	49		1	1				1			1		1				5																			

Statistics

Average					46				647	134,8	28,3	5,7	6,7	11,1	2,6	2,3	1,05	0,83	1	0,58	3	3	1,96		53,1			1,61													4,0																			
Median					36				504	121	27,0	6,0	6,1	6,6	2,1	2,0	1,00	0,75	1	0,56	3	4	1,86		41			1													4																			
Minimum value					8				177	25	10,0	0,0	0,0	1,5	0,0	0,0	0,75	0,60	0	0,00	1	1	0,46		5			0													1																			
Maximum value					162				2014	374	52,0	19,0	24,8	52,0	20,0	8,1	2,00	1,50	1	2,02	8	9	4,64		188			7													5																			
Variance					1128				185490	4743	99,1	18,1	33,4	####	9,7	3,6	0,05	0,03	0	0,24	3	3	1,00		1370																																			
Standard Deviation					34				431	69	10,0	4,3	5,8	10,1	3,1	1,9	0,23	0,16	0	0,49	2	2	1,00		37																																			
Standard Deviation/Average					73%				67%	51%	35%	75%	86%	91%	####	84%	22%	20%	47%	###	###	###	51%		70%																																			

Counting

				total dams																																																							
Hits in Area A			A	26				3																		2	26		0	0	0	13	0	1	23	0	20	0	0	5		16	3	0	1	3	5	0	0	0	0	5	8	1	7	3	0	0	4
Hits in Area B			B	13				0																		0	13		0	0	0	0	0	0	13	0	9	0	0	0		8	0	0	0	1	0	0	0	0	0	3	5	2	6	0	0	0	3
Hits in Area C			C	11				0																		1	11		0	0	0	5	0	0	11	0	8	0	0	0		9	0	0	0	0	2	0	0	0	0	0	0	1	5	0	0	0	4
Hits in Total Area			Total	50				3																		3	50		0	0	0	18	0	1	47	0	37	0	0	5		33	3	0	1	4	7	0	0	0	0	8	13	4	18	3	0	0	11

General parameters

average porocity 35%

Evaluation Sand-Storage Dams, Kitui District

DATA AND PRIMARY CALCULATIONS

Date: June 2001
Inspection Sand-Storage Dams: April 2001
by Projectgroup CF599, TU Delft

Explaination

*		#3		#5		#5S	
	(1) yes		(1) fast		(1) very bad		(1) shortage
	( ) no		(2) normal		(2) poor		(2) insufficient
			(3) slowly		(3) acceptable		(3) even
					(4) good		(4) enough
					(5) exellent		(5) plenty of water

soil, reservoir, flood, sedimentation

no.	code	name dam	area	soil: river bottom: base rock *	river bottom: red soil *	river bottom: clay *	river bottom: sand *	river bottom: others *	banks: base rock *	banks: red soil *	banks: clay *	banks: sand *	banks: others *	foundation: base rock *	foundation: red soil *	foundation: clay *	foundation: sand *	foundation: others *	reservoir: throwback (m)	width (m)	depth (m)	volume (m3)	flood: velocity (#3)	sedimentation kind: clay *	sand *	gravel *	stones *	others *	filled rate
1	1A	Kamumbuni	A	1							1	1	1	1					2000	35	3,0	12250	3		1				1
2	2A	Kwa Kavoo	A				1				1	1					1		2000	40	5,0	23333	3		1				1
3	3A	Kwa Mutinga	A				1				1		1			1			400	40	5,0	4667	3		1				1
4	4A	Kwa Mukumbe 1	A				1		1		1					1			400	40	2,6	2427	3		1				1
5	5A	Kwa Mukumbe 2	A				1				1		1	1					1000	30	2,0	3500	3		1		1		1
6	6A	Uvalti (colonial dam)	A	1					1					1					500	20	2,5	1458	2		1		1		1
7	7A	Nzemeini	A				1				1			1				1	400	20	2,0	933	2		1				1
8	8A	Kwa Langwa	A					1	1		1							1	500	15	2,0	875	2		1				1
9	9A	Kwa Mangya	A				1		1			1		1				1	1000	10	3,0	1750	2		1				1
10	10A	Kwa Ndunda	A				1				1		1	1				1	1000	40	3,0	7000	2		1				1
11	11A	Kwa Tito	A					1					1					1	400	10	1,6	373	3		1			1	0
12	12A	Kwa Mutia	A					1					1					1	300	8	3,0	420	3		1				1/4
13	13A	Kwa Wambua	A	1									1	1					500	10	1,0	292	3		1				3/4
14	14A	Kathini	A	1					1				1	1					2000	30	7,0	24500	1		1				1
15	15A	Ndia Aimu	A	1			1		1				1	1					300	30	3,0	1575	1		1				2/4
16	16A	Kwa Milu	A	1			1		1				1	1					2000	20	3,0	7000	1		1				2/4
17	17A	Tendelya	A	1					1				1	1					500	15	2,0	875	1		1				2/4
18	18A	Kyanguni	A	1									1	1					2000	17	2,0	3967	1		1			1	1/4
19	19A	Imooni	A	1					1				1	1					200	20	1,5	350	1		1				2/4
20	20A	Kwa Kilango	A				1						1	1					1500	30	2,0	5250	1		1				2/4
21	21A	Mitauni	A	1					1				1	1					200	15	4,0	700	1		1				2/4
22	22A	Musalani	A				1						1	1					100	15	2,0	175	1		1				1
23	23A	Miuni	A				1					1		1					3000	10	4,0	7000	1		1				3/4
24	24A	Kithumulani	A	1									1	1					300	5	2,0	175	1		1				2/4
25	25A	Kamukui	A			1	1				1			1					400	4	2,5	233	3		1				2/4
26	26A	Kakunike A	A	1								1		1					400	10	2,0	467	3		1				1/4
27	1B	Munyuni A	B				1						1	1					2000	10	1,0	1167	1		1				1/4
28	2B	Munyuni B	B			1							1	1					300	11	1,0	193	1		1				1/4
29	3B	Kianguni	B	1									1	1					1000	10	1,0	583	1		1				1/4
30	4B	Kavithe B	B				1					1	1				1		500	10	4,0	1167	1		1				3/4
31	5B	Kyangala B	B	1									1	1					300	10	3,0	525	1		1				2/4
32	6B	Kwa Ngungu A	B	1					1			1	1	1					1000	8	1,0	467	1		1				1/4
33	7B	Nguni	B	1					1				1	1					1000	25	1,5	2188	1		1				3/4
34	8B	Kyangombe	B	1					1				1	1					1500	60	1,0	5250	1		1				2/4
35	9B	Musaala	B	1									1	1					1000	6	1,5	525	1		1				3/4
36	10B	Metika Mbuu A	B	1					1				1	1				1	500	10	2,0	583	1		1				1
37	11B	Metika Mbuu B	B	1									1	1					500	5	1,0	146	1		1				1
38	12B	Nganzani	B	1									1	1					3000	5	1,0	875	3		1				3/4
39	13B	Kwa Kavula	B	1					1				1	1					4000	6	2,0	2800	2		1				2/4
40	1C	Kwa Ndifu	C				1						1	1					200	5	3,3	193	1		1			1	3/4
41	2C	Kwa Masila	C				1						1	1					400	4	3,0	280	1		1			1	2/4
42	3C	Kwa Nziani	C				1						1	1				1	300	15	2,0	525	1		1			1	3/4
43	4C	Kwa David 1	C					1					1	1					200	7	4,0	327	1		1				3/4
44	5C	Kwa David 2	C				1						1	1				1	100	5	4,0	117	1		1			1	3/4
45	6C	Kwa Robert	C				1				1			1					500	6	1,0	175	1		1			1	2/4
46	7C	Kwa Katiwa	C	1					1				1	1					1000	5	8,0	2333	1		1			1	2/4
47	8C	Kwa Mukola	C				1						1	1					200	5	4,0	233	2		1				1/4
48	9C	Kwa Nguthu	C				1						1	1					200	5	1,0	58	2					1	0
49	10C	Mutungini	C	1									1	1					400	5	1,0	117	2					1	0
50	11C	Munandani	C				1				1			1					500	6	3,0	525	2		1				2/4

erosion, user characteristics, recommendations maintenance and improvements, other intresting points

no.	code	erosion place: banks upstream *	river bottom downstream *	banks downstream *	water abstraction: accessibility (#5)	distance village-dam (m)	available water quantity (#5S)	number of users	community organistation *	organisation tasks: maintenance *	instructions for use *	inspections *	protection: banks upstream *	banks downstream *	river bottom downstream *	downstream toe *	reparation: cracks *	fences *	draw-off system/well *	decrease seepage *	protection *	landslide: recover/protect *	weak materials *	improvement: fill up top dam *	remove vegetation *	place (new) fences *	improve stability *	adapt abutment *	others *	other intresting points
1	1A	1			5	2000	5	2400	1	1																				
2	2A	1			4	2000	5	800	1				1																	no function anymore
3	3A	1			5	N/A	5	N/A	1				1																	
4	4A	1			4	N/A	5	N/A	1				1																1	dirty well, cut roots tree nearby dam
5	5A	1		1	3	N/A	5	N/A	1				1	1																big stones in river
6	6A				4	N/A	5	N/A																					1	abutment damaged. repairation needed
7	7A				3	2000	5	2400	1																					
8	8A	1		1	4	1000	5	3200	1	1		1	1	1															1	platform and lid well is missing
9	9A				5	500	5	3040	1	1	1	1																		terraces built, good protection
10	10A				5	N/A	5	N/A	1	1	1	1																	1	cut tree nearby dam
11	11A	1	1		4	N/A	N/A	N/A	1	1	1	1	1	1	1						1									plunge basin too short, square well
12	12A				4	N/A	N/A	N/A	1	1	1	1				1													1	
13	13A		1		4	N/A	5	N/A	1	1		1		1																well not finished, top wing width smaller
14	14A	1		1	4	1000	5	24000	1	1		1	1	1							1									drift
15	15A			1	2	500	5	800	1	1	1	1		1																repaired by a new aided wall
16	16A	1		1	4	500	5	3200	1	1	1	1	1	1							1									
17	17A				4	500	5	560	1	1	1	1		1																protect wing, general protection is good
18	18A	1			4	500	4	560	1	1	1	1	1				1										1			piping, repair dam immidiately by building new wall
19	19A		1		4	500	5	960	1	1	1	1		1						1										
20	20A				4	500	5	800	1	1		1	1	1							1									seepage repaired
21	21A				2	1000	5	1600	1	1	1	1		1																
22	22A				4	500	5	1600	1	1	1	1																		dam is protecting a spring
23	23A				3	500	4	800	1	1	1	1																		wing is also blocking other channel
24	24A				4	500	4	640	1	1	1	1	1																	
25	25A				2	500	4	800	1	1	1	1	1	1							1									
26	26A				4	500	5	160	1																					
27	1B	1	1	1	4	1000	5	2400	1					1	1	1					1							1	1	no protection at all, no maintenance
28	2B	1			3	500	5	2400	1	1			1	1																
29	3B				5	250	5	3600	1	1		1																		
30	4B	1		1	5	500	5	4000	1	1			1	1							1									
31	5B	1			2	1000	5	1600	1	1	1	1	1	1							1									
32	6B				2	1000	5	800	1												1									
33	7B			1	1	N/A	5	400	1	1			1																	
34	8B				4	1000	5	3000	1	1		1		1							1									
35	9B			1	4	2000	5	2400	1					1							1	1								
36	10B			1	5	3000	5	2400	1					1							1									intention to extend high dam til 7 m
37	11B			1	5	3500	5	1600	1					1							1									
38	12B				4	500	5	1600	1	1	1	1		1																
39	13B				4	2500	5	1600	1	1	1	1																		
40	1C		1		4	500	4	264	1	1	1	1		1							1									
41	2C	1		1	4	500	4	320	1	1	1	1	1	1							1									
42	3C				4	500	3	200	1			1		1							1									community don't use this water
43	4C		1		2	1000	3	70	1					1							1									community don't use this water
44	5C				3	1000	4	70	1					1							1									
45	6C		1		4	1000	4	70	1					1							1									natural plunge basin by rocks
46	7C		1		2	500	2	160	1											1	1									consider to extend high dam
47	8C				4	500	4	120	1	1	1	1																		
48	9C				4	500	4	576	1	1	1	1				1													1	
49	10C				4	400	4	160	1	1	1	1																		
50	11C				5	N/A	5	N/A	1	1	1	1																		

Statistics

	throwback	width	depth	volume	velocity	filled rate	accessibility	distance village-dam	available water quantity	number of users
Average	878	16	2,6	2658	1,64	0,62	3,7	953,8	4,6	1906
Median	500	10	2,0	642	1	0,5	4	500	5	800
Minimum value	100	4	1,0	58	1	0	1	250	2	70
Maximum value	4000	60	8,0	24500	3	1	5	3500	5	24000
Variance	740527	161	2,3	25197648						
Standard Deviation	861	13	1,5	5020						
Standard Deviation/Average	98%	81%	59%	189%						

Counting

	area	rb: base rock	rb: red soil	rb: clay	rb: sand	rb: others	banks: base rock	banks: red soil	banks: clay	banks: sand	banks: others	found.: base rock	found.: red soil	found.: clay	found.: sand	found.: others	sed.: clay	sed.: sand	sed.: gravel	sed.: stones	sed.: others	erosion: banks upstream	river bottom downstream	banks downstream	community organistation	maintenance	instructions for use	inspections	prot.: banks upstream	prot.: banks downstream	prot.: river bottom downstream	prot.: downstream toe	cracks	fences	draw-off system/well	decrease seepage	protection	landslide: recover/protect	weak materials	fill up top dam	remove vegetation	place (new) fences	improve stability	adapt abutment	others
Hits in Area A	A	12	0	1	13	3	10	0	9	5	17	20	0	2	1	6	0	26	0	2	2	10	3	5	25	19	14	18	12	12	1	1	1	0	0	1	5	0	0	0	0	0	1	0	5
Hits in Area B	B	10	0	1	2	0	5	0	0	2	13	12	0	0	1	1	0	13	0	0	0	4	1	6	13	8	3	5	4	9	1	1	0	0	0	0	8	1	0	0	0	0	0	1	1
Hits in Area C	C	2	0	0	8	1	1	0	2	0	9	11	0	0	0	2	0	9	0	0	8	1	4	1	11	6	6	7	1	6	0	1	0	0	0	1	7	0	0	0	0	0	0	0	1
Hits in Total Area	Total	24	0	2	23	4	16	0	11	7	39	43	0	2	2	9	0	48	0	2	10	15	8	12	49	33	23	30	17	27	2	3	1	0	0	2	20	1	0	0	0	0	1	1	7

APPENDIX 5: DISTANCE, COVERED BY A FREE FALLING JET

The jet has a horizontal velocity of u_c, the critical velocity on top of the spillway. The time needed for the free fall of the jet is calculated with:

$$T = (2 \cdot \frac{h}{g})^{\frac{1}{2}}$$

g = Gravitational acceleration [m/s^2] = 10 m/s^2
h = Vertical fall distance [m]

The horizontal distance, X [m]:

$$X = u_c \cdot T$$

The bottom side of the jet falls from a height of h_s, the topside falls from a height of $h_s + h_c$. This results in:

$$X_{min} = u_c \cdot \left(2 \cdot \frac{h_s}{g}\right)^{\frac{1}{2}} \quad \text{and} \quad X_{max} = u_c \cdot 2 \cdot \left(\frac{(h_s + h_c)}{g}\right)^{\frac{1}{2}}$$

X_{min} = Minimum distance covered by the jet [m]
X_{max} = Maximum distance covered by the jet [m]
u_c = Critical velocity on top of the spillway [m/s]
h_s = Height of the spillway above the original river bottom [m]
h_c = Critical water depth on top of the spillway [m]

These formulas are reworked until X_{min} and X_{max} depend on the discharge per unit width ($q = Q / L_s$) and h_s. Q is known from the Slope-Area Method and the height of the spillway (h_s) and the length (L_s) of the spillway are also known.

$$h_c = \frac{2}{3} \cdot H$$

$$\frac{u_c^2}{2 \cdot g} = \frac{1}{3} \cdot H \quad \Rightarrow \quad u_c = \left(\frac{2 \cdot g \cdot H}{3}\right)^{\frac{1}{2}} \quad \text{and} \quad H = 3 \cdot \frac{u_c^2}{2 \cdot g}$$

$$q = u_c \cdot h_c$$

$$q = \left(\frac{2 \cdot g \cdot H}{3}\right)^{\frac{1}{2}} \cdot \frac{2 \cdot H}{3}$$

$$q = 1.721 \cdot H^{\frac{3}{2}}$$

$$q = 1.721 \cdot \left(\frac{3 \cdot u_c^2}{2 \cdot g}\right)^{\frac{3}{2}} \quad \text{and} \quad q = 1.721 \cdot \left(\frac{3 \cdot h_c}{2}\right)^{\frac{3}{2}}$$

$$q = 0.100 \cdot u_c^3 \qquad q = 3.16 \cdot h_c^{\frac{3}{2}}$$

$$u_c = 2.154 \cdot q^{\frac{1}{3}} \qquad h_c = 0.464 \cdot q^{\frac{2}{3}}$$

$$X_{min} = u_c \cdot \left(\frac{2 \cdot h_s}{g}\right)^{\frac{1}{2}} = 2.154 \cdot q^{\frac{1}{3}} \cdot \left(\frac{2 \cdot h_s}{g}\right)^{\frac{1}{2}} = \mathbf{0.96 \times q^{\frac{1}{3}} \times h_s^{\frac{1}{2}}}$$

$$X_{max} = u_c \cdot \left(2 \cdot \frac{h_s + h_c}{g}\right)^{\frac{1}{2}} = 2.154 \cdot q^{\frac{1}{3}} \cdot \left(2 \cdot \frac{h_s + 0.464 \cdot q^{\frac{2}{3}}}{g}\right)^{\frac{1}{2}} = \mathbf{0.96 \times q^{\frac{1}{3}} \times \left(h_s + 0.464 \times q^{\frac{2}{3}}\right)^{\frac{1}{2}}}$$