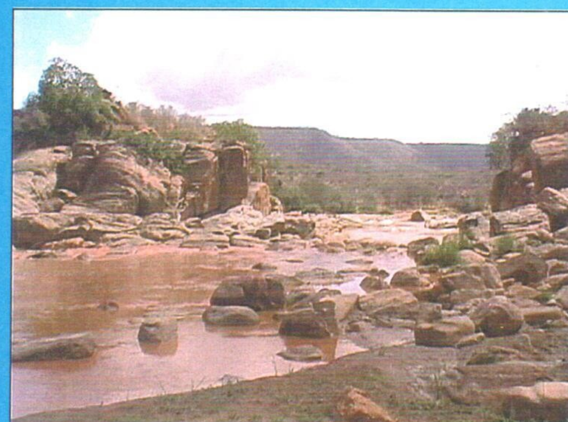
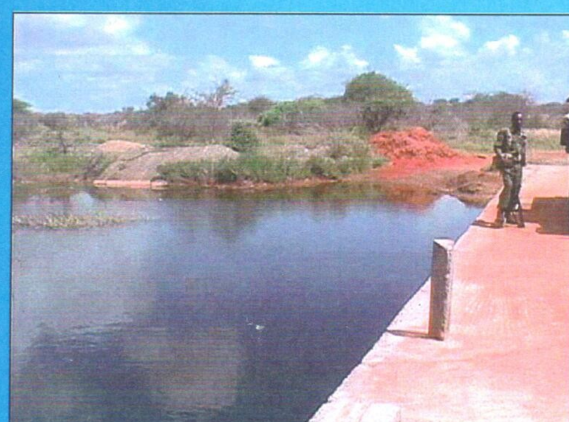
Development study on Tsavo National Park, Kenya





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PREFACE

As part of our study Civil Engineering at the Delft University of Technology we have been working and studying on a water rehabilitation project in Tsavo East, Kenya. At the same time we made a feasibility study on a river crossing that should connect the northern part of Tsavo to the southern part. We spent about two months in Kenya. These two months we have been staying for 6 weeks in Nairobi, two weeks in Tsavo National Park and a few days in Mombassa. Our group consisted of five students with different specialization. Hydrology, Watermanagement, Hydraulic Engineering and Mechanics and Construction.

This is the second year a group of students went to Kenya to work for the Westerveld Conservation Trust. The Westerveld Conservation Trust works together with the Kenya Wildlife Service to achieve an ecological rehabilitation of the national parks in Kenya. Mainly this means that more water should be hold up in the park before it streams down to the river and out of the park.

Before and during the project we had to face problems, which without the help of certain people would have been too big to handle. First of all, money to finance this trip had to be raised. We want to thank Maartje Westerop for her efforts to help us raising and her coordination role between the WCT and the Delft University of Technology. Without Maartje Westerop this project wouldn't have been possible.

For the technical and practical support from the Delft University of Technology we are very thankful to Ir. P. Ankum, Prof. Ir. R. Brouwer and Ir. W. J. Dijk. We considered your help as useful and a cling to hang on. In Nairobi Panafcon/DHV Africa provided a place to work. Charles Muyembe and Joshua were of great help with collecting the relevant data from the ministries and University. But most of all the employees of Panafcon were fantastic friends who showed us everything, told us the things we needed to now and spent there free time with us.

Francis Mwangi, we hope you recover very fast. Thanks for showing us around at the Kenya Wildlife Service. Prof. Mutiso for showing us the Sub surface dams in Kitui and standing by with advice. Ilona Eveleens for looking after us in dangerous Nairobi, we appreciated that very much.

We want to thank Peter Westerveld for giving us a great time in the park, for the discussions and the introduction to other people we wanted to meet.

Finally we would like to thank the following sponsors for their financial support:
- ANVR
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SUMMARY

PART I: SWOT- ANALYSIS

To explore the different circumstances, working in Kenya brings along, a profile of Tsavo National Park and the general variables, which you have to take into account, has been made. The profile consists of five aspects. For each aspect the relevant subjects are written down. The profile is used as a database of facts and visions. SWOT, short term and long term actions, as well as the problem analyses result from the profile.

The SWOT (strong / weak points, opportunity and threats) follows out of the profile and makes very clear where and how, what to do. It also cuts out options that should not be executed. After the inventory of the strong points, weak point, opportunities and threats, long term and short term actions are defined.

The problem definition which resulted out of the foregoing is: Tsavo East National Park is facing a lot of problems, as shown in the Profile of Tsavo and the Problem Analyses. Poaching and how to stimulate tourism are the two most urgent ones. Proper Solutions for these problems are not in stock. This means that development of the park goes slowly. For technical problems there is a lack of engineers to face them. Especially two short term actions are relevant to develop, to face poaching problems and to stimulate tourism: A descent technical fundament for the design and construction of small dams, and a feasibility study on a river crossing.

The objective for this project in general is: To find technical solutions for the existing problems Tsavo is suffering. Most urgent are: To give a technical fundament on the design and construction of small dams and the possibilities for a river crossing.
In part II and part III this objective is more specified to dams or crossing

PART II: WATER CONSERVATION

One of the problems in Tsavo East National Park is the migration of wildlife in the dry seasons, because of the lack of water. The precipitation is enough to supply the need for drinking water for wildlife. However, the precipitation occurs in only two periods and is concentrated in a few big showers. Most of the water will flow directly into the Galana river. By building small dams in the seasonal rivers water can be trapped and wildlife will be kept inside the borders of Tsavo.
The objective is the make a gereral usable design for water conservation in small seasonal drifts, in order to create drinking water reservoirs for wildlife.

Hydrology

The 10-year maximum 24 hour precipitation is 150 mm. This is the result of the analysis of the data of six rainfall stations (Airstrip, Bachuma Gate, Manyani Gate, Ndololo Campsite, Sala Gate and Tsavo Research station). The rainfall intensity for the Rational method is obtained from a intensity-duration curve. For the Curve Number method the precipitation event is systemised by a 4 hour shower with a constant intensity of 37.5 mm/hour. The 0.5-year maximum 24-hour precipitation is 50 mm. This number is used for the designing the causeway.
The results op the rainfall analysis are used in several floodmodels. In the end three floodmodels are used for the determination of the 10-year maximum discharge. For the 0.5-year dominant discharge the rational method is used. The results are:

A (km^2)	$Q_{0.5}$ (m^3/s)	Q_{10} (m^3/s)
5	10.8	35
10	12.7	50
25	24.0	85
50	39.7	135

The evaporation is calculated with the formula of Penman. Together with the design dry period from May till October this results in a total of 1163 mm of evaporation losses.
A layer of silt will be deposited in the reservoir. After ten years the depth will be:

Drainage area (km^2)	Percentage of reservoir silted up (%)	Depth deposited layer (m)
5	4	0.26
10	7	0.40
25	8	0.48
50	14	0.65

Design

Two dams are further detailed: the natural dam with a concrete centre and the causeway. The causeway is the best dam to place in a location where a road crosses a drift. The construction consists of two concrete walls, filled with natural stones and a concrete roof. The walls have a thickness of 0.5 meter and the roof has a thickness of 0.2 meter.
The natural dam is a good alternative for 'off-road' constructions. The impermeable dam consists of a concrete wall that functions as impermeable heart of the dam. Upstream of the dam the soil is covered with red soil (slope 1:2) and downstream with natural stones (slope 1:3). The width at the top of the construction is 1.0 meter. The stability of both dams is checked and are sufficient. In case the embankments do not consist of rock, anchors are needed.
The costs of the dams are shown in the table below.

Catchment Area (km^2)	Causeway	Impermeable dam
5	186,450 Ksh	393,765 Ksh
10	252,500 Ksh	487,475 Ksh
25	364,075 Ksh	773,025 Ksh
50	556,725 Ksh	1,264,875 Ksh

PART III: FEASIBILITY STUDY RIVER CROSSING

Problem analysis and objective

Tsavo East is split by the Athi-river. In the present situation the only crossing is a non-permanent Irish Bridge. This bridge can only be used in the dry periods, when the water levels are low. The absence of a permanent river crossing over the Athi, has a range of consequences for the undeveloped northern part:

- Development of tourism is not possible
- Protection and surveying of wildlife is difficult.
- Poachers are almost free to move and

Logically a permanent river crossing is a solution for these problems.

Location
Three different options have been worked out:

- A bridge
- A semi-open tunnel, preferred by the employer
- An improved Irish Bridge.

These alternatives are situated at two different locations.
The bridge and semi-open tunnel can be built in the P.W. area. This area named after the employer, lies 10 km upstream of the Athi –Tsavo river junction. The river flows through a narrow channel along one and a half-kilometer, makes a sharp turn and divides itself in three sub-channels (this is called P.W. point). 200 meter downstream the river returns to one channel bed. The bridge can be situated over the narrow channel, while the tunnel is located just downstream of the split in three channels.
The improved Irish Bridge can be build at Lugard's Falls, the location of the existing non-permanent Irish bridge. This area lies 25 km downstream of the Athi-Tsavo river junction. The river is 500 m wide at this point.

Boundary Conditions
The boundary conditions for a river crossing are can be divided in three, the most important are:

General:

- A construction should be as economical as possible
- Working in an national park means that disturbing of vegetation and wildlife should be minimal
- Advanced materials and construction methods should be avoided
- The availability of the structure should be almost 100% (Only during an El Nino the construction is allowed not to be available

Water related
The water conditions play an important role.

In general the river can be described as a permanent river with low mean discharges and high peak discharges. Key values for Athi river are depending on the location

Mean discharge	20-25 m^3/s
Maximum discharge	2500-6500 m^3/s
Gradient (average)	$3.25*10^{-3}$
Strickler coefficient	30-45

Water conditions for P.W. area and Lugard's Falls are defined with discharge and water level data, 'The national water master plan of Kenya' and drawings of cross-sections obtained through site visits.

The results are

	P.W. area	Lugards Falls
Maximum Discharge (m^3/s)	4500	6500
Mean discharge (m^3/s)	19	26
Water-level (average)* (m)	12 (upstream) 5,5 (downstream)	10.4
Water velocity (average) (m/s)	≈ 8 (upstream) ≈ 6 (downstream)	-

* Water levels and velocity depend on the location in the P.W. area. Upstream and downstream are related to the position with regard to P.W. point

Construction related
The first determination of dimensions for every structure is done with rough estimation rules. After that, these are going to be checked on shear forces bending moments and normal Forces. When all dimensions are clear, costs can be calculated. Costs are calculated to compare each alternative with each other.

Solutions
Five solutions have been considered:

- A bridge
- A semi-open tunnel
- An improved Irish bridge
- An Irish bridge with culverts
- A Pontoon bridge.

The first three solutions has been worked out, the last two are mentioned and commentated, but not in detail.

Bridge: Two concepts are possible, a river-span at once or a span in two or three sections. A single span needs narrow riverbanks. Either way it is a good solution because the deck of the bridge will never be in contact with the river. This means it is always accessible. When a heavy girder bridge is constructed it is going to cost approximately 1 million guilders.

Semi open tunnel: Because of an almost instant waterdrop of 5 meters a tunnel can be constructed that will not cause a huge reservoir upstream. A semi open tunnel will be a gigantic tourist attraction, which can be a source of income for the KWS. But, this construction is very expensive (about 2.7 million guilders) and very difficult to built.

Irish bridge: This is by far the cheapest alternative, but it's not going to be of extra value next to the Irish bridge that already exists at Lugard's Falls. To make it more than 95 % of the time accessible it has to overcome a water level of 6 meters, which is far too much for an Irish bridge.

Conclusion
In most points of view the bridge is the best option. Without making a scoring table, it can be said that the bridge satisfies to the most requirements. It is the only alternative always available, can be build relatively cheap and without to complex building methods.
Still it will be difficult to find the money for this project. Only when external investors are willing to pay a crossing can be build.

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PART I: SWOT ANALYSIS

1 PROFILE TSAVO EAST NATIONAL PARK

1.1 FUNCTIONAL-SPATIAL ASPECTS

Introduction, history and key figures
Tsavo East National Park is situated in the southeastern part of Kenya. It lies east of the Nairobi-Mombassa highway. The size is almost 12.000 km2 and together with Tsavo West N.P. it forms the biggest National Park in the world (the size of both parks together is 22.000 km2. In 1948 Tsavo West (9065 km2) and Tsavo East (11747 km2) were founded as a single wildlife park. This park was too big to control, therefore they were divided into two parks. The border between the two parks is the Mombassa Nairobi Highway. After that another two more small reserves were added to Tsavo. In the West, at the border of Tsavo West there's Chyulu National Park and there is another National Park bordering Tsavo West in the South. These four parks are forming together an almost closed eco-system. A "closed eco-system" means that the main migratory, the elephants do not leave the park throughout the year. When elephant don't leave the park, other animals will not either.

The vegetation mainly consists of an arid thorn bushland with several grassplains and some woodland. Most of the park is relatively flat. It lies at about 500 meters above main sea level. The Yatta plane, north of Athi/Galana River is 300 meters higher.
Two perennial rivers, Tsavo and Athi/Galana river, run through the park. A lot of drifts supply these two rivers, especially during the rain season.
Due to its location on the equator, Tsavo Park knows two rain seasons per year. The first season between the end of March and the beginning of Mai and the second season in November. The total amount of rainfall is about 500 mm per year.
The average day temperature in Voi is 28 degrees and 20 during the night.

Animals living in Tsavo East are the big five: Elephant, lion, leopard, rhino and buffalo. Also hippo, crocodile, zebra, giraffe, cheetah, hyena and many others are occupying the park. The density of animals is low in comparison to for example Masai Mara and this is due to poaching and drought.

KWS explores Tsavo East. A Warden is responsible for daily business. Besides the clerical staff, about 150 rangers are present to protect and survey wildlife. Their camps are located at different places inside the park.

Infrastructure in the park
The infrastructure of the park shows mainly earth roads. Drifts are generaly crossed by causeways and at some places by (very) small bridges, to connect the main points. KWS rangers and tourists can use them. There is a river crossing over Athi/Galana River near Crocodile point. This is a so-called Irish bridge, consisting of concrete slabs put on the bottom of the river slope. Water is flowing over it. The crossing can only be used during dry season when the water level is low.

Fences do not border Tsavo Park only gates lock the main entrance roads. At these gates, parkfees must be paid and drinking water is available. These gates are closed by night, when travelling inside the park is not permitted. The main gates are Voi gate, Manikin gate and Mtito Nadei gate.

Besides some very small villages, the only place of some importance is Voi. It forms the main entrance to Tsavo East and KWS headquarters for Tsavo East is situated just outside the village centre. Right now a new Tsavo HQ is under construction.

Together with the already existing HQ things are going to be monitored and controlled
These headquarters include staff buildings, an education centre and residents for the employees.

The park locates three permanent lodges for tourists: Aruba Lodge, Tsavo Safari Camp and Voi Safari Lodge. Besides these lodges several campsites are found at different places. Peter Westerveld manages one of them.
Other interference done in Tsavo East N.P. are:

- The Aruba dam.
- The Kandecha dam
- 7 small earth dams build by the Westerveld Conservation Trust. Three dams are situated at Punda Milla and four in the Rhino Release Area.
- Several airstrips
- A pipeline from Mzima Springs (Tsavo West) to KWS headquarters providing drinking water

Natural attractions are: Lugard falls, Crocodile point, Yatta Plateau and the natural wildlife

1.2 TECHNICAL ASPECTS

The technical parts of this analysis are divided in three major categories: collecting data, the quality of human built structures and the effects on the environment.

Dams in Tsavo

Since 1994 WCT and KWS build seven small dams in seasonal drifts. Most of these drifts run off in Athi/Galana River. In front of all dams water reservoirs have arisen. The main objective is to create permanent drinking-water location for animals. As mentioned before, four dams have been built in the Rhino Release Area. Starting upstream, the first dam is made of concrete, the two followers are pure earth dams and the last one is an earth dam with a concrete kernel/heart (is still under construction). Close to this area a causeway is build. At Punda Milia a series of three dams is made, consisting of one causeway and earth dams with a concrete kernel/heart (one is still under construction). The last dam has been built in Ashaka, an earth dam with a concrete kernel/heart.
The effects on the environment are not easy to measure. It has been reported that elephants that used to go to Manikin prison in their search for water now stay close to the dams. Some sources also say that the direct environment stays much greener, especially in dry season. Fact is that all reservoirs permanent store water. The behaviour of the dams during heavy rains and floods is doubtful. Peter Westerveld claims that the concrete/earth dam, build and designed by himself, sustain in all types of floods (even El Nino). The fact that the some spillways has rinsed out, during rain season, is, according to Peter, the cause of failures during the construction. Actually the time between construction and this overview is too short for a good evaluation. Remarks that have to be made are: By building dams, the choice for animals searching for water sources is limited. This makes them an easy spot for poachers as well as animal hunters. Also an optimal design (cheap, easy to make and strong) has not been made yet. And what are the actual effects on the (green) environment and the groundwater level.

In the Voi River two concrete dams are built by KWS: Aruba dam and Kandecha dam. The Aruba dam is very large and a lot of problems have occurred. Sedimentation has almost filled up the whole basin. The storage capacity has decreased significantly and this causes problems during a flood. The spillway has been damaged several times (due to a too large discharge). The spillway lost its function completely. Besides the minerals and salt in the water has caused a lot of siltation, which is a big problem for the water quality.

Data
Rainfall data of different locations are available:

Table 1.1 Rainfall data[1].

Measuring station	Daily / Monthly
Manyani Gate	Daily
Airstrip	Daily
Research station	Daily
Park HQ	Daily
Ndololo	Daily
Sala Gate	Daily
Bachuma Gate	Daily
Mudanda Rock	Monthly
Buffalo Wallows	Monthly
Lugard Falls	Monthly
Sobo	Monthly
Dika	Monthly
Rhino Camp	Monthly
Aruba dam	Monthly
Mukwaju	Monthly
Ndara borehole	Monthly
Balgude	Monthly
Inme	Monthly

Measuring has started in 1969 and appears to be quite accurate for some of these stations. Other stations collected data for only a couple of years and are now closed. All together they provide enough data to make reasonable predictions.

1.3 SOCIAL-CULTURAL ASPECTS

Tsavo East National Park is uninhabited, except for the ranger posts of the KWS, the permanent tourist lodges and the campsites, like Peter Westerveld's camp. Small-scale agriculture fields surround Tsavo East. Villages with small populations are located close to the park borders. As mentioned above, Voi is the main village with 21.000 inhabitants.

Like in the rest of Kenya the number of unemployed people is very high, in this area up to 50%. So, unemployed, and mostly unskilled workers are available in large numbers, whenever a project is going to be realised. Wages are low (120 Kshs. per person per day which is about $1,7).
A possible problem that can occur because of these unskilled workers is that they can obstruct the working process.

A better functioning park, where projects are released, will give a lot of benefits. More attractions will attract more tourists and the profits will increase. KWS will profit directly, but the locals will get swing-off activities as well.

[1] Source: KWS Research Station

Poaching
The main threats for wildlife are drought and poaching. At the start of the rain seasons most poaching takes place. Vegetation is still low and the rains easily erase tracks. Rhino and elephants are the main targets. Most poachers are retired Somalia soldiers but also tribes do their work. The ivory is sold to Japan and Southeast Asia. To avoid poaching, KWS has a 'shoot to kill'-policy. Poachers are travelling on foot. They only kill rhino's and elephants for their horns and ivory. They bury the ivory and horns under the ground and wait till the next year to collect it. The Ivory is now much lighter and because of that easier to carry.

Human / wildlife conflict
Due to shortage of water, animals cross the borders of the park in their search for water. In the park there are not enough drinking places to provide water for the wildlife population. Conflicts occur between humans and wildlife at the borders of the park. Farmers, living close by the park see their crops have been demolished by wildlife. Killing wildlife is sometimes just a matter of protecting income and food. One solution is to build fences, but this is not an option when money is not available. It is a must to create a harmony between wildlife and humans. The land outside the national reserves should be used more effectively. That will decrease the need of land for cattle and agriculture. People will not live as close to the park as they do now and problems between human and wildlife will decrease.

Dams in Kitui for agriculture
At the north side of Tsavo N.P. there is a little village called Kitui. Professor Mutiso is constructing small dams, with his organisation SASOL to create water reservoirs for agriculture purposes. Peter Westerveld is constructing dams just like Professor Mutiso, but for wildlife instead of agriculture. Within three years SASOL will approach the borders of Tsavo with its subsurface dams.

1.4 ORGANISATIONAL ASPECTS

General management
As mentioned already KWS explores Tsavo East National Park and 26 other National Reserves.
KWS headquarters is situated in Nairobi. They are responsible for general policies, financial matters and long term planning. The also have a technical department, for design and environmental assessment studies.
Every park has its own warden. He and his staff are responsible for everyday business. The warden is also responsible for rangers and other employees
The KWS intends to explore parks at three different levels of planning;
- a nation-wide system plan, which will state to national policies
- Management plans, which prescribe how individual Parks are to be managed and developed for the next five years.
- Annual plans, which are to be prepared by park wardens each year for individual areas.

Objective
The main objective of KWS for Kenya's parks can be summarised as:
The main purpose is to conserve wildlife and vegetation. Because there is so little left, conservation is a national, even global obligation. KWS conserves wildlife for the sake of people. This means that people must have the opportunity to watch these animals without disturbing them. Since there have to be roads for rangers, why don't put the roads where the animals are, so people can come close. Lodges in the park

are also allowed. Every kind of attraction is permitted as long as wildlife won't be disturbed.

Project policies
Inside the borders of the park approval by KWS headquarters is needed before a project starts. When safety of the environment and people is not in danger permission will be given easily.
When a project interferes with national interests, approval by the government is needed. In case of projects in Tsavo, the government should approve projects dealing with the Nairobi-Mombassa road and interference in the main rivers (Tsavo, Athi/Galana and Voi River)

A proposal is needed for every project been carried out (for example building a bridge). This proposal is sent to the technical department of KWS headquarters, where an E.I.A. (Environmental Impact Assessment) will be made. When money is available (for example by fundraising) the plan with the highest priority will be carried out.

Project organisation

Project supervisor	From KWS or consultant hired by KWS
Work inspector	From KWS reports monthly to HQ
Contractor	
Technical engineer	
Group leader of local workers	Partly local, sometimes foreign
Local workers	

Regulations

- Contractors are paid every month.
- When the employer agrees with the result, the contractor will never responsible for damages and failures in the future

For example: Peter Westerveld co-operates with the KWS to build the dams. Materials necessary for the projects can be hired from the government against favourable prices.

Park policies
In general every plan must fit into a bigger plan. The policy of the KWS is diverted from the ministry policy, which on his term is diverted from the national policy.
The policy of the KWS falls into a roughly hierarchical arranged components. Marketing and promotion of the National Parks is done by a ministry

Safaris are organised by extern bureau's, Parkfees per person are paid to the KWS. People are allowed to drive around freely in the park. They have to keep to the roads. They are not allowed to leave their cars. Carrying a weapon is strictly forbidden. After sunset no one except for the rangers is allowed to drive around. Animals may not be annoyed or disturbed.

1.5 FINANCIAL-ECONOMICAL ASPECTS

Tsavo East N.P. is owned by the state and managed by the KWS. They are responsible for all activities inside the park. The KWS maintains the infrastructure, buildings, gates, fences and equipment. All staff members and rangers work for KWS.

Money for conservation
KWS administers 26 National Parks. The total size of all these protected areas is 29.000 km2. For now the KWS has to rely on parkfees and donor money. There is no money coming from the government. Because the whole economy of Kenya is not working very well, the taxpayer has no money to support wildlife conservation other investments have higher priority. Maybe when the economy recovers, there will be money for this. All the parkfees of every national park in Kenya are going to the KWS Head Quarters, from there it's distributed back to the parks. The money parks receive is based on the work they are doing, not on the number of tourists that have been visiting the park. Some of the parks are making less money, still there should be money for conservation. Tsavo East has 40 % of all the national park its land. Annually the total amount of earnings is mKsh 540 ($17mln). Parkfees are $ 23-30 for foreigners and Kshs 200-500 for residents. Tsavo East receives 11 % of the total earnings. 80 percent of all income is used for salaries and overhead costs and the last 20 percent is needed for maintenance. New projects are only possible when external investors are willing to finance them.

When a project is realised (in case of financial input of an external investor) this procedure is followed.
In case of a small project (budget less than 3 M Kshs), design and construct are done by KWS. Equipment used is owned by KWS or can be hired. In practice most of this equipment is hired from the army.

Contractors
In case of a large project a contractor will do (budget more than 3 mKshs), design and construct. A contract consists of:

Constructing costs : all the costs for the actual building
Extra costs are related to the constructing costs

Unexpected costs	+15 %
Design costs	+ 5 %
Overhead costs	+ 10 %
Profit	+15-30%

The economy around Tsavo East National Park depends on tourism and agriculture. Inside the park there are some lodges and campsites for tourists and in Voi, man can find hotels and safari tour operators.

2 SWOT-ANALYSIS

2.1 FUNCTIONAL-SPATIAL ASPECTS

Strong points:
- Tsavo is the National Park most close to a closed eco-system and therefore unique. This uniqueness is the main attraction of Tsavo.
- The tourist lodges that already are situated in the park attract tourists, who stay for a longer period.

Weak points:
- Due to the wideness of the park, it is very hard to keep out all poachers.
- The earth roads can not resist the heavy rains.
- There is no permanent connection over Athi/Galana river
- The Aruba dam which was expected to make a big reservoir failed
- The number of natural drinking places for wildlife are not sufficient in periods of drought

Opportunities:
- Due to wideness and the presence of many different animals the opportunities for tourist development are high
- A river crossing over Athi/Galana river makes travelling to the northern part of the park more easy
- Small dams can also function as causeways through small riverbeds. When small dams are built like that they contribute to the road system.

Threats:
- All improvements of infrastructure make the movements of poachers more easy as well
- All infrastructure should be maintained, due to a lack of money this can become very difficult

2.2 TECHNICAL ASPECTS

Strong points:
- The small dams built by the WCT contribute to the amount of water kept in the park. As a result of that, territories of animals decrease.
- The KWS owns a lot of equipment that they can use during projects
- The KWS has at one's disposal a technical department.

Weak points:
- The only crossing of the Galana River is an 'Irish' bridge, which can't be used during the rain season.
- The design of the dams is still not detailed enough .

Opportunities:
- In the streambeds of the drifts are a lot more possible locations for dams to increase the number of small water reservoirs.
- A good, general design/recipe of a small dam that can be used in different occasions.

Threats:

- It's possible, that during construction workers make failures which makes the construction less reliable.
- It's never shore that the obtained strength of the construction is reached because of the poor equipment and construction methods

2.3 SOCIAL-CULTURAL ASPECTS

Strong points:
- People are not allowed to live in the park, which makes the impact of man on wildlife small
- The KWS is a solid organisation and is having a good reputation.
- Everyone who wants to visit the park is allowed to do that. Foreigners have to pay high parkfees. Residents only have to pay a small part of that. Visiting a national park makes man more aware of the importance of conserving wildlife

Weak points:
- Because of the high costs of building fences human-wildlife conflicts can't be prevented at the boundaries of the park.
- The unemployment rate under locals is very high. (+/- 50%)
- A lot of people have never been to school.
- Political circumstances influence the number of tourists willing to visit the country, which affects the number of tourists visiting Tsavo.
- There's a lot of poverty among the Kenyans.
- Because of the dams built by SASOL for the sake of agriculture, the land outside the border of the park is getting wetter. SASOL will approach the border in the north within three years. Before they reach the border, the land inside the park has to fulfil the need of water, otherwise the animals will cross the border in their search of water, and they will find it.

Opportunities:
- The KWS must remain a clean organisation.
- Co-operation between KWS and the WCT can help to develop the park for tourism
- Human-wildlife conflicts can be avoided by keeping the wildlife in the park. (water conservation)
- The park is that big that the quantity of wildlife can increase enormously when there is enough water.

Threats:
- The country impoverishes, which has negative affect on development.
- Business has been taken over by Libyans and Indians.
- Politics will not enable people to improve their own situation.
- Poaching.

2.4 FINANCIAL-ECONOMICAL ASPECTS

Strong points:
- The KWS is capable to design and construct small projects.
- The KWS is the only organisation who benefits from the parkfees (no taxes go to the government).
- The park provides employment for the local people in the area.

Weak points:

- There is no annual budget to develop new projects in the park.
- The KWS is completely depending on tourism.
- No financial support by the government.
- Investors are not interested to invest money in the park

Opportunities:
- A new crossing over Galana River will enable tourists to visit the north part of Tsavo East.
- It is possible to hire equipment from the government or army. Their rates are much lower than commercial rates.
- More tourism means a higher income for the KWS because of the parkfees.
- To increase the earnings of parkfees people should stay longer in the park. Walking safaris are giving people the chance to experience a safari very intense. They stay, with experienced guides of the KWS for several days in the park.

Threats:
- Trade in ivory still takes place.
- Economy of Kenya collapsing. Criminality and poverty shall decrease the number of tourists

2.5 ORGANISATIONAL ASPECTS

Strong points:
- Conservation of wildlife is being worked hard on 'ranger posts'-level.
- The KWS is an independent organisation. Government has very little influence on the KWS.

Weak points:
- The size of the organisation is too big.
- The KWS is not succeeding in generating funds for projects.
- High overhead costs of the KWS.
- Equipment is often out of order because of damage or a lack of fuel.
- Corruption of other organisations than the KWS.

Opportunities:
- To buy commercial knowledge (consultant).
- Reduce the size of the organisation.

Threats:
- The WCT quits building small dams.
- Debts of the KWS.

3 PROBLEM ANALYSES

In Kenya the gap between very rich and extremely poor is enormous. Wealth, most of the time gathered in a non-legal way, lives next to shortage of everything. It's impossible to stay out on the streets after sunset. The chances of being robbed or even killed are not imaginary. Street children and people living in the slums gather in street gangs during night times. As a result of this, crime and unsafety are part of everyday life. Especially Nairobi suffers from petty crime. Due to these facts the image of Kenya is influenced bad. Tourists are less willing to bring their dollars to Kenya because they simply feel unsafe. The Kenyan society misses a lot of income while the profit of tourism disappears. As a result money to maintain and develop tourist attractions is missing.

When focused on the national parks and especially Tsavo East, a lot of problems occurring, can be put down to the situation described above, but this doesn't mean nothing can be done.
Besides this lack of money the two main threats for Tsavo East are drought and poaching.
Droughts do not arise because the annual amount of rainfall is to small, but due to irregular rainfall. In other words, large amounts come down in short periods, followed by long times without rain. Drought forces animals to search for water in large areas. This may result in a situation where animals leave the borders of a national park. Human-wildlife conflicts will occur. For example Elephants destroy the crops of farmers in their search for water and food. The park is far too big to barrier by fences so a different solution has to be sought. The Westerveld Conservation Trust co-operates with the KWS to create water reservoirs inside Tsavo East. Up till now seven small dams has been build in seasonal river slopes. These dams create little water reservoirs, which are filled during rain season. This water can supply wildlife and vegetation around the reservoirs. The actual effects are hard to measure, but it is reliable to conclude that water reservoirs inside the borders of the park have positive effects on both wildlife and vegetation. But a quantification of the key figures, resulting in a proper technical design has not been done.

Poachers still form a big threat for especially Elephants and Rhino's. Former Somalian soldiers and tribes still try hunt for ivory. Especially in the northern part of Tsavo East, poachers can easily do their job. The only way to prevent these killings is patrolling by KWS rangers. These rangers should be able to move around easily in order to patrol large areas. This requires good infrastructure, which is not present today.

While the government does not support KWS, the only sources of income are park fees. These earnings are hardly enough to maintain the existing situation. New projects can only be done when external investors are willing to finance. An example of this is the WCT, who financed and constructed the small dams.

A better functioning park will give a lot of opportunities. More attractions will attract more tourists, so money earned by park fees will increase. The KWS will profit immediately, and eventually the people who live around the park will benefit from the swing-off activities.

	Short Term Actions	Long Term Actions
General	• Documentation about the effects of small water reservoirs. Measuring the number and species of animals using these reservoirs can do this. Also development of flora should be evaluated • Dams have to be built in the Northern part of Tsavo East, because there has to be enough water before SASOL enters the border of the park with their water for agriculture project	• Creation of more comprehension between humans and wildlife. By making an information center in or around Voi, both tourists and locals can learn about the behavior of wildlife. • Documentation about the habitats of different species of wildlife in Tsavo should be made. Both KWS rangers and tourists will profit. • Stimulation of the bio-diversity
Infrastructure: Improvement of infrastructure to make it easier for KWS rangers and tourists to move.	• To build a river crossing over Galana/Athi river • Creation of new water reservoirs in order to provide more water for wildlife • Research about these dams to come up with an optimal design • After every rain season the roads must be prepared again.	• To build fences around the villages at the borders of Tsavo. They can avoid human-wildlife conflicts. Fences around the whole area are way to expensive for the time being

Economical
 	
The commercial rate of Tsavo National Park should be improved. This can be reached by a number of actions
	• Create more attractions. Examples are:
	  • A rhino-walk area. In the rhino release area a number of water reservoirs have been constructed. This attracts both rhino's and other wildlife, and can be seen during a rhino walk. Walking Safaris are keeping tourists longer in the
	  • park, which increases the parkfees earnings.
	• To make deals with tour/safari operators (in for example Europe) about a safari in Tsavo in combination with a visit to the coast. Tsavo is situated between Nairobi and Mombassa
	• To build more lodges and safari camps inside and at the borders of Tsavo. These lodges can differ from simple
		• Create more attractions. Examples are:
		  • to build watch towers. Tsavo park mainly consists of bushes. A watchtower increases the chance to see animals (especially during sunrise and sunset) numerically.
		  • Balloon trips. A balloon trip is a unique opportunity to see wildlife from above. Animals are hardly disturbed, and a the chances of seeing wildlife are high (especially during sunrise and sunset)
		  • Improvement of Tsavo park's image. This can be reached by advertising both inside and outside of Kenya. Tourists should be made aware of the beauties of

	to luxurious. Lodges create employment, which is a positive swing off to local economics. • Yatta plateau	Kenya and not of crime and corruption. • To change the commercial / marketing philosophy. One philosophy is to decreases the park fees in general. Attractions can be charged as extra. Lower park fees will attract more tourists.
Organisation The organisation of the KWS should be very flexible and able to face the problems they are suffering.	• A masterplan for park development in the next twenty-five years has to be made in order to attract investors. • Aid organisations can support the fight against poachers and the safety of endangered	• All investment contracts should include a repayment condition. This increases the chances that money is used in a proper way and will be used to develop the park species. • The organisation of the KWS is too big. It should be reorganised. • The KWS suffers from very high debts which must be sanitised

3.1 PROBLEM DEFENITION:

- Tsavo East National Park is facing a lot of problems, as shown in the Profile of Tsavo and the Problem Analyses. Poaching and how to stimulate tourism are the two most urgent ones. Proper Solutions for these problems are not in stock. This means that development of the park goes slowly. For technical problems there is a lack of engineers to face them. Especially two short term actions are relevant to develop, to face poaching and tourism problems: A descent technical fundament for the design and construction of small dams, and a feasibility study on a river crossing.

3.2 OBJECTIVE

- To find technical solutions for the existing problems Tsavo is suffering. Most urgent are: To give a technical fundament on the design and construction of small dams and the possibilities for a river crossing

3.3 JUSTIFICATION TO WORK OUT TWO SHORT TERM ACTIONS

Out of the SWOT analyses followed a list of short-term actions. These short-term actions are in general solutions for organisational and economical problems. For example: Rhino Walks are a perfect alternative for a driving safari. To achieve that walking safaris are going to be a success a big marketing campaign is required. At the same time, to organise walking safaris is much more complicated than driving safaris, that's the other main problem. Technically speaking there's little to do for an engineer.

Other opportunities to increase the number of tourists are to make deals with tour/ safari operators about a safari in Tsavo in combination with a visit to the coast, or to construct more tourists lodges inside the park.

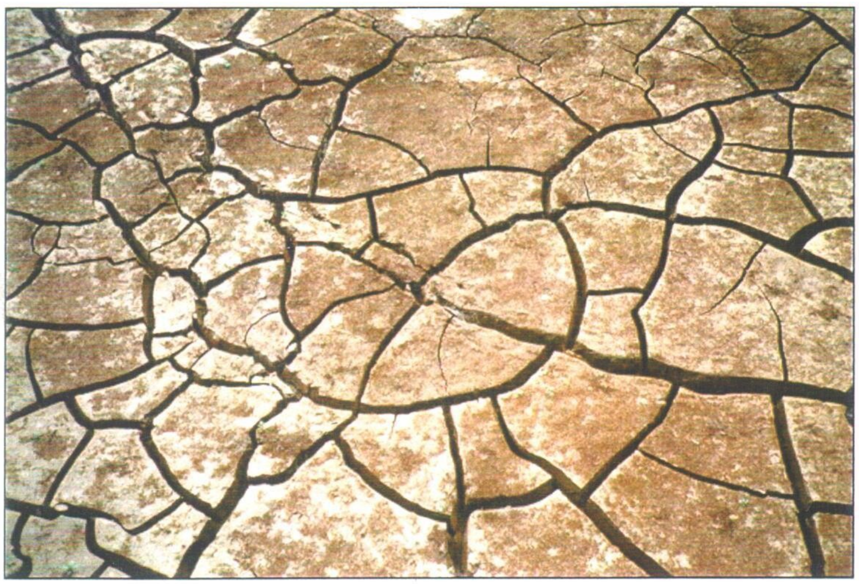
The choice to work out a river crossing and to have a really close look at the hydrology, construction and effects on the surroundings of the dams built by the WCT in Tsavo was because of their relevancy, their priority and technical background. Also because the WCT insisted that the river crossing had to be part of the project.

Research on small dams was required for two reasons. First of all, the design of the 9 dams that have been built in Tsavo East lack a solid technical foundation. What will happen when these dams have to resist big forces (for example during El Nino) is uncertain. Second, a dam building project in Kitui, the district on the northern border of Tsavo East, will enter the border within three years. That will make the land outside the park useable for agriculture, and wet. Tsavo has to be ready, by that time, with building dams in the northern part of Tsavo East, otherwise the animals will cross the border in their search for water, and they will find it in Kitui. For this reason it is very important that dams are going to be build in the northern part of Tsavo, that these dams are reliable and that their effects on the surrounding area can be predicted.

A river crossing has priority above other structures because of the threat of poaching and the opportunities for tourism. The KWS is not able to chase poachers across the Athi/ Galana River. There should be a bridge or a tunnel to make the northern part of Tsavo East easy accessible for the KWS. This would provide them a shortcut, which makes the park easier to control. Not only the KWS can make use of the river crossing, also tourism will benefit from this. When the northern part of Tsavo East is open for tourism the income thanks to parkfees will increase. There's even a possibility for toll collection.

PART II: WATER CONSERVATION

The Problem



4 INTRODUCTION

The SWOT analysis concluded that one problem to be solved is the shortage of water. A solution to this problem is to create small water reservoirs.
In this part the design of small water conservation structures will be described. The approach is as follows:

Part I: The problem
This part will describe the problem and the objective. Also the program of demands is included.

Part II: The hydrology
The second part will describe the hydrology in the area. The 'Hydrology" will start with an analysis of the rainfall data. The rainfall dates of six stations are used: Airstrip, Bachuma Gate, Manyani Gate, Ndololo Campsite, Sala Gate and Tsavo Research Station. From these stations daily rainfall data are present. These data are used to estimate the dry period, the dominant discharge ($Q_{0.5}$) and the 10-year peak discharge (Q_{10}). Furthermore the evaporation and the sedimentation and erosion of the catchment areas are described.

Part III: The design
In this part a design for the layout of water conservation structures in a seasonal drift is made. First the different alternatives are described. Together with the evaluation of the present dams in Tsavo East National Park a conclusion can be drawn.
In the end of this part a design will be made for the 'natural dam with a concrete centre' and the 'causeway'.

5 PROBLEM APPROACH

5.1 PROBLEM ANALYSIS

In the conclusion of the SWOT-analysis of Tsavo East National Park is put forward that tourism should be stimulated. At this moment most tourists visit the park during the rain season, due to the fact that in this period a big amount of wildlife is present in the Park. In the dry season, when there is a shortage of water, the wildlife migrates to places, where water is still available.
In Tsavo East National Park the total amount of precipitation per year is enough to supply the need of drinking water for wildlife. However, the precipitation occurs in only two periods and is concentrated in a few big showers. As a result the biggest part of the water almost immediately runs off into Galana River and further towards sea. By building small water conservation structures, it is possible to delay the runoff of the rainwater and to store it in small reservoirs.
Availability of water will keep the wildlife longer in the park and will extend the tourist-season. An additional effect is that the wildlife will be concentrated around the reservoirs, which makes it easy for touroperators to trace them.
The Westerveld Conservation Trust and the KWS are making series of dams in small seasonal drifts to catch water and to create drinking-water reservoirs. The dams are being built by practical experience and there is no sound technical background for the designs.
Neither is known what the best layout for the water conservation structures in a river is; which types should be built at which locations, how many should be built in a river and at which distance.

5.2 PROBLEM DEFINITION

The water available from precipitation in Tsavo East National Park almost immediately runs off into Galana River. To hold the water in the Park, series of small water conservation structures have been built by practical experience. A general usable, technical solution for water conservation and its design is missing.

5.3 OBJECTIVE

A general usable design for water conservation in small seasonal drifts, in order to create drinking-water reservoirs for wildlife.

Financial/economical

- The life expectancy of the dam should be at least 10 years.
- The reservoir should be accessible for tourists without building a new 'roadsystem'.

6.2 ASSUMPTIONS

- The infiltration to the groundwater through cracks in the bedrock is 3 mm/day.
- The open water evaporation can be calculated by using the Penman formula.
- The texture class of the soil in Tsavo East National Park is sandy loam and contains 79.8 % of sand, 6 % of silt and 14.2 % of clay (from Water for Wildlife I).
- The gradient of the seasonal drifts: 8 ‰.
- The water in the reservoirs is consumable for wildlife.
- When using the Curve Number method a shower with duration of 4 hours systemise the maximum 24-hour daily precipitation. The intensity of the shower is constant during the shower.
- The water velocity in the small seasonal drifts is estimated on 2 m/s.

6.3 STARTING-POINTS

- To avoid wildlife conflicts, at least two reservoirs should be created in each catchment area.
- The shape of the catchment areas is estimated with the most detailed maps available, provided by the Kenya Wildlife Service (scale 1:50.000).

6.4 DESIGN PARAMETERS

- The unit weight of redsoil $\gamma_{redsoil}$ = 1500 kg/m^3.
- The unit weight of concrete γ_c = 2600 kg/m^3.
- The unit weight of rockfill γ_r = 2450 kg/m^3.
- The unit weight of stones γ_s = 2000 kg/m^3.
- The unit weight of water W = 1000 kg/m^3
- The runoff coefficient in the Rational Method is 0.30.

Hydrology

7 RAINFALL ANALYSIS

To be able to determine the dimensions of the reservoir and the water conservation structures, an analysis must be made of the quantity of water that flows into the reservoir. This can be done by an analysis of the water balance. An important input factor is the runoff water from rainfall. The runoff is estimated by using flood models. Input parameters in these models are data on precipitation, like the occurring 10-year rainfall depth and the rainfall intensity. The runoff will be used to design the dimensions of the spillway.
First the available rainfall data are analysed. In the second paragraph the annual rainfall depth is determined for several stations. The analysis of paragraph 7.3 and 7.4 results in a 10 year maximum daily rainfall event and a 0.5 year maximum rainfall event. The rainfall intensity is described in paragraph 7.5. In the last paragraph a design length of the dry period is determined.

7.1 AVAILABLE DATA

Data are collected by the meteorological department for the whole of Kenya or by the Kenya Wildlife Service for the National Parks. As mentioned before in the SWOT-analysis, there are several stations in Tsavo East N.P. where rainfall data are measured.
Rainfall data from different locations are available. An overview is shown in Table 7.1.

Table 7.1 Measuring stations in Tsavo East N.P.

Measuring station	Daily / Monthly
Manyani Gate	Daily
Airstrip	Daily
Tsavo Research station	Daily
Park HQ	Daily
Ndololo Campsite	Daily
Sala Gate	Daily
Bachuma Gate	Daily
Mudanda Rock	Monthly
Buffalo Wallows	Monthly
Lugard Falls	Monthly
Sobo	Monthly
Dika	Monthly
Rhino Camp	Monthly
Aruba dam	Monthly
Mukwaju	Monthly
Ndara borehole	Monthly
Balgude	Monthly
Inme	Monthly

Rainfall is measured since 1969. Most of the stations report monthly data or incomplete series of daily rainfall. Only a small number of stations have a long period with few missing data, which can be used in a rainfall analysis.
Six stations, which are spread through the park, have been used for the rainfall analysis: Tsavo Research Station, Ndololo Campsite, Manyani Gate, Airstrip, Sala Gate and Bachuma Gate.

7.2 ANNUAL RAINFALL

The total annual rainfall gives an indication of the climate. In Appendix 1 the annual rainfall depth per station is plotted.
Table 7.2 gives the total annual rainfall of the six selected stations.

Table 7.2 Total annual rainfall depth per station (mm).

Year	Airstrip	Bachuma	Ndololo	Sala	TRS	Manyani
1973	x	317.3	x	171.7	484.2	466.9
1974	x	265.9	x	x	281.9	246.6
1975	x	x	x	x	X	252.9
1976	291.6	278.5	370.1	192.1	267.9	169.4
1977	x	356.0	x	379.7	641.6	761.3
1978	x	524.7	x	453.1	X	674.1
1979	690.2	830.8	775.9	603.2	751.7	558.2
1980	x	x	x	x	X	X
1981	x	x	x	x	X	X
1982	675.5	356.2	867.9	459.3	791.1	455.3
1983	x	240.8	x	117.5	374.6	233.9
1984	474.2	431.0	478.2	161.6	528.6	282.6
1985	435.4	484.2	360.7	208.1	651.4	226.3
1986	643.1	406.6	663.3	x	586.9	818.4
1987	312.3	x	349.8	x	305.3	X
1988	520.9	448.7	588.1	x	521.6	402.3
1989	423.8	x	563.0	x	542.0	X
1990	820.5	155.6	907.1	x	856.5	443.2
1991	x	276.2	650.4	x	580.3	472.1
1992	365.5	x	490.3	119.5	532.5	811.1
1993	555.7	235.9	579.6	261.3	570.9	529.3
1994	740.0	1012.2	808.8	452.1	760.5	419.4
1995	397.4	367.2	420.2	382.4	491.2	286.7
1996	253.2	309.7	359.5	508.0	340.5	209.5
1997	641.5	1190.5	1345.0	802.0	910.5	554.6
1998	1049.3	667.3	1256.5	733.5	1243.4	598.1
1999	400.8	307.3	430.9	x	435.2	X
Mean	538.4	450.6	645.5	375.3	584.8	429.2
Mean Annual rainfall in Tsavo East N.P.:						504

For further calculations a mean annual rainfall of 500 mm will be used.
From these totals can be concluded that most of the wet or dry years are corresponding for all stations. However, the actual year totals differ a lot. And after further examination of the daily rainfall depth, it is clear that the rainfall events are very local. The year totals show a large variation in time, with droughts and peaks succeeding each other. Droughts occur in the middle of the seventies, eighties and nineties. Very wet periods occur at the end of the seventies, in 1986, 1994 and end nineties.

7.3 10-YEAR MAXIMUM 24-HOUR RAINFALL

To estimate the peak discharge with a return period of 10 years, the occurring 10-year maximum daily rainfall must be estimated. For determining peak floods normally annual series are used. (In annual series only the maximum rainfall event for each year is extracted from the complete series.)

Description of the 'annual series'-method

- Collect the maximum daily rainfall depths for each year: N years of data give N daily rainfall depths.
- Arrange the data in descending order.
- Assign the order number m to the data (m = 1 to the largest daily rainfall depth, etc.)
- Assign the return period T to all data, the return period can be estimated by the formula: $T = \frac{N+1}{m}$ in years.
- Plot the rainfall depths and corresponding return periods on linear-Gumbel distribution paper and logarithmic-Gumbel distribution paper.
- Draw a straight line on 'best fit' through these points on both types of distribution paper.
- Select the type of distribution by testing on a straight distribution
- Read the magnitude of the ten-year daily rainfall depth on the selected distribution paper.

Conclusion:
The straight line on the log-Gumbel paper fits best and will be used for estimating the maximum 24-hour rainfall depth with a return period of 10 years. The graphs are included in Appendix 7 and Appendix 8. The results are shown in Table 7.3.

Table 7.3 10-year maximum 24-hour rainfall depth per station.

Station	P_{10} (mm/day)
Tsavo Research Station	190
Ndololo Campsite	215
Manyani Gate	135
Airstrip	150
Sala Gate	100
Bachuma Gate	135
Mean 24-hour rainfall depth	154

There is a great variety in daily rainfall depth between the different stations, due to their location in the National Park. Sala Gate has a quite low 24-hour rainfall depth, which can be the result of a lower number of rainfall data. Ndololo Campsite and Tsavo Research Station are situated quite near to each other and both have a higher 24-hour rainfall depth. The locations of these stations are closely to a hill, which can be the cause of these high figures.
The mean of the six stations is used as average for the area and for further calculations P_{10} = 150 mm/day will be used.

7.4 0.5-YEAR MAXIMUM 24-HOUR RAINFALL

In order to calculate the occurring 0.5-year discharge, the 0.5-year maximum rainfall must be known. For the determination of rainfall events with a small return period the 'partial series'-method is used.

Description of the 'partial series'-method

- Collect the daily rainfall depths above a threshold value of 40 mm: N years of data give N daily rainfall depths.
- Arrange the data in descending order.
- Assign the order number m to the data (m = 1 to the largest daily rainfall depth, etc.)
- Assign the return period T to all data, the return period can be estimated by the formula: $T = \frac{N+1}{m}$ in years.
- Plot the rainfall depths and corresponding return periods on logarithmic-logarithmic distribution paper.
- Draw a straight line on 'best fit' through these points on the log-log distribution paper.
- Select the type of distribution by testing on a straight distribution
- Read the magnitude of the twice per year daily rainfall depth on the log-log distribution paper.

<u>Conclusion:</u>
The straight line on the log-log paper will be used for estimating the maximum 24-hour rainfall depth with a return period of 0.5 years. The graphs are included in Appendix 9. The results are shown in Table 7.4.

Table 7.4 0.5-year maximum 24-hour rainfall depth per station.

Station	$P_{0.5}$ (mm/day)
Tsavo Research Station	67
Ndololo Campsite	65
Manyani Gate	48
Airstrip	55
Sala Gate	44
Bachuma Gate	48
Mean 24-hour rainfall depth	54

The mean of the six stations is used as average for the area and for further calculations $P_{0.5}$ = 50 mm/day will be used.

7.5 RAINFALL INTENSITY

Since only 24-hour rainfall data have been found and no data on rainfall intensity, analytical estimations must be used. In the literature a mathematical relationship between maximum intensity and duration of a rainfall event in East Africa has been found, the so called McCallum model:

$$I_t = K \times t^{-n}$$

where: I_t = maximum intensity (mm/h) for duration t (h)
K = one-hour maximum intensity
n = constant

Given constants:

Dar es Salaam (Tanzania)	K = 55.63	n = 0.54
Mombasa (Kenya)	K = 55.99	n = 0.62
Nairobi (Kenya)	K = 63.70	n = 0.81

Kisumu (Kenya) K = 77.72 n = 0.84

Though rainfall intensity is related to the prevailing precipitation-producing process and to the location, the constants for Mombasa (at a distance of 150 km) are used to derive an intensity-duration curve for Tsavo East N.P. This is done for both T = 10 years and T = 0.5 year, which can be seen in Figure 7.1.

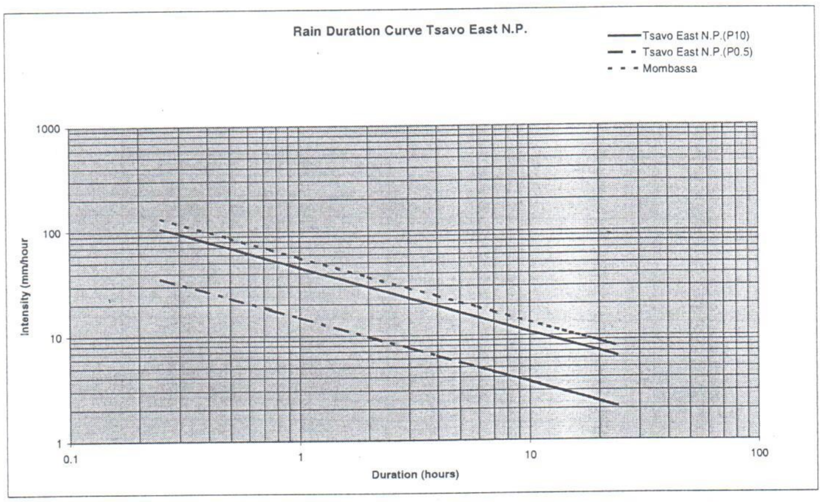


Figure 7.1 Intensity-duration curve for Tsavo East N.P. with T=0.5 year and T=10 years.

The two intensity curves for Tsavo East N.P. are determined by drawing

7.6 DESIGN DRY PERIOD

The size of the reservoir depends on the evaporation and, consequently, on the maximum duration of a dry period.
The length of a dry period is defined by the number of consecutive dry months per year. A dry month can be defined as a month in which less than 1% of the total annual precipitation occurs. With an average annual rainfall depth of 500 mm, a dry month has a maximum rainfall depth of 5 mm. Table 7.5 shows the dry months per station per year.

Table 7.5 Dry months per station.

Year	Airstrip	Bachuma	Ndololo	Sala	TRS	Manyani
1973	-	2.9	-	3.6.7.8.9.10.12	2.6.7.8.9	4.6.7.8.9.10
1974	-	1	-	-	1.2.6.7.8.9.10	1.2.3.5.6.7.8.9
1975	-	-	-	-	-	-
1976	1.2.3.6.7.8.10	1.2.7.10	1.2.3.6.7.8	1.2.3.6.7.11.12	1.2.3.6.7.8	1.2.3.5.6.7.8.10
1977	-	1.2.4.7	-	1.5.6.7.8	5.7	5.7.8
1978	-	6.7.8.9	-	6.7.8.9.10	-	-
1979	7.8.9.10	8.9.10	7.8.9	2.7.8.9.10	7.8.9	7.8.9
1980	-	-	-	-	-	-
1981	-	-	-	-	-	-
1982	1.2	1.2.12	1.2.6	2	1.2.6	1.2.6.7.8.12
1983	-	1.10	-	1.3.7.8.9.10	1.4.7.8.9.10	1.4.6.7.8.9.10
1984	1.2.5.6.8	1.2.8.9	1.2.5.6.8	1.2.3.6.8.9.10	1.2.5.6.8	1.2.3.5.6.8.9
1985	6.7.9	6	6.7	1.5.6.8.9	6.7.9	6.7.8.9
1986	2.6.7.9	2.7.9.10.11	2.6.7.9	-	2.6.7.9.10	2.3.6.7.8.9
1987	6.9.10	-	6.9	-	6.9	-
1988	2.5.6.7	1.2.8.10	2.5.7.10	-	2.5.7	1.2.5.6.7.8
1989	2.6.7.9	-	2.6.7.9	-	2.6.7.9	-
1990	5.6.7.8	4.5.6.7.8.9	6.7.9	-	5.6.7.8	4.5.6.7.8.9
1991	1.2.9	1.2.5.7.9.10	1.2.9	-	1.2.9	5.6.9.10
1992	1.2.3.6.7.8.9	-	1.2.6.7.8.9	1.2.3.6.7.8.10	1.2.6.7.8	5.6.7.8.9
1993	5.6.7.8.9	3.5.6.7.10	5.6.7.9	5.6.7.8.9	5.6.7.8.9	2.3.6.7.8.9.10
1994	7.8.9	1	8	1.2.8	8.9	1.2.4.6.7.8.9
1995	1.5.6.9.10	6.9	1.5.6.9	1.2.5.6.7.9	1.5.6.9	1.5.6.7.8.9.10
1996	1.6.7.8.9.10	1.2.6.7.8.9.10	1.6.7.8.9.10	1.2.6.8.10	1.6.7.8.9	1.6.7.8.9.10
1997	2.3.7.8.9	1.2.7.8.9	2.3.7.8.9	1.2.7.8.9	1.2.3.7.8.9	1.2.3.6.7.8.9
1998	7.8.10	6.7	6.8.10	-	6.8.10	2.3.6.7.8.10
1999	1.2.7.8.10	1.6.8.9	1.2.7.8.10	-	2.7.8.9.10	

In general the dry periods are in January and February and from June to September. The wet periods occur from March to May and from October to December. Table 7.6 gives a summary of the number of consecutive dry months per station per year.

Table 7.6 Maximum number of consecutive dry months per station.

Year	Airstrip	Bachuma	Ndololo	Sala	TRS	Manyani
1973	-	1	-	5	4	5
1974	-	1	-	-	5	5
1975	-	-	-	-	-	-
1976	3	2	3	3	3	4
1977	-	2	-	4	1	2
1978	-	4	-	5	-	-
1979	4	3	3	4	3	3
1980	-	-	-	-	-	-
1981	-	-	-	-	-	-
1982	2	2	2	1	2	3
1983	-	1	-	4	4	5
1984	2	2	2	3	2	3
1985	2	1	2	2	2	4
1986	2	3	2	-	2	4
1987	2	-	1	-	1	-
1988	3	2	1	-	1	4
1989	2	-	2	-	2	-
1990	3	6	2	-	4	6
1991	2	2	2	-	2	2
1992	4	-	4	3	3	5
1993	5	3	3	5	5	5
1994	3	1	1	2	2	4
1995	2	1	2	3	2	6
1996	5	5	5	2	4	5
1997	3	3	3	3	3	4
1998	2	2	1	-	1	3
1999	2	2	2	-	4	?
Maximum	5	6	5	5	5	6

The maximum length of a dry period is 6 months.
For further calculations will be taken a maximum dry period of 6 months, occurring from May till October.

7.7 CONCLUSIONS

Rainfall is not very predictable The design precipitations are chosen at the safe side. Also the rain intensity is a systemisation. For a better result intensities should be measured. A recommendation is to measure the intesity of rainfall events to make better calculations possible.
Compared to the results of Water for Willife the maximum 24-hour precipitation with a return period of ten years is somewhat higher. In this report the results of six stations are used compared to one station in Water for Wildlife.

8 AREA CHARACTERISTICS

In Tsavo East N.P. are several locations for possible drifts. All these locations have characteristics concerning the drainage area and the drift. The sizes, the slope, the soil, and covering of the surface vary. In this chapter a description of the area is given and a standardisation has been made.

8.1 CHARACTERISTICS OF THE DRAINAGE AREA

There are several small seasonal drifts that run out in the Galana river. In those drifts are many possible locations for small water conservation structures in order to create reservoirs. But all these locations will have different catchment areas and a different length of the riverbed. From the topographical map (1:50,000) standard sizes of drainage areas have been derived as well as a corresponding length of the riverbed. These standard characteristics, as shown in Table 8.1, will be used for further calculations.

Table 8.1 Standard sizes for drainage areas.

Drainage area (km^2)	Length of riverbed (km)
5	3
10	7
25	11
50	15

The slope of the area varies from gentle to steep. In general the area is plane with undulating and steeper areas.
A big part of the area is covered with rock, or rock lying under a layer of red soil. The red soil is a result of the weathering of the underlying rock formations.
The rock is impermeable but might contain cracks. The red soil is strongly impermeable because of the high percentage of clay in it.
The vegetation mainly consists of an arid thorn bushland with several grass plains and some woodland.

8.2 CHARACTERISTICS OF THE DRIFT

Like the characteristics of the drainage area, the characteristics of the drift vary too. An important aspect of the drift is the cross-section. The place of the cross-section in the length profile of the drift, and corresponding the size of the catchment area as well, is decisive of its sizes. For further calculations a standardisation has been made. All cross-sections can be described as an envelope curve as drawn in Figure 8.1.

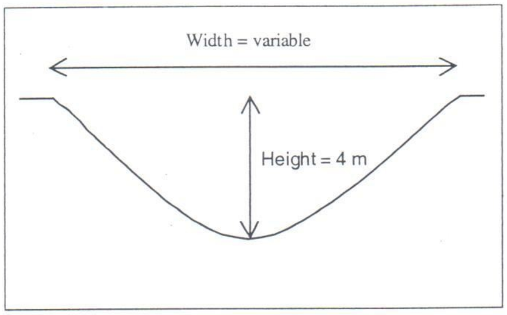


Figure 8.1 Cross-section of a drift.

The height in the middle is approximately the same for every cross-section and is fixed at 4 meters. The width, however, is variable and increases as the size of the catchment area increases. The standard sizes of a drift for different catchment areas are given in Table 8.2.

Table 8.2 Standard sizes for the cross-section of a drift.

Drainage area (km^2)	Width (m)	Height (m)
5	20	4
10	25	4
25	35	4
50	50	4

The gradient of the drifts varies, and in general a gradient of 8‰ suits well.

The riverbed consists of rock, locally covered with a layer of red soil and sandy loam of a maximum thickness of 1 meter.
A lot of rocks are present due to movement by the water flow. Due to the underlying rock layer, the riverbed is mostly impermeable, but with small permeable areas because of cracks in the rocks.
The banks of drift are covered with grass. The drift is seasonal and at several places the riverbed is covered with grass too. There are wet spots in the drifts and these are muddy by visits of animals.

9 FLOOD MODELS

An important parameter to determine the location and design of the water conservation structure, is the peak discharge.

To estimate the peak discharge, the data about the precipitation are used. In chapter 7 the precipitation occurring with a return period of 10 year and 0.5 year is determined. Also an estimation of a rainfall intensity-duration graph has been made. These data will be used to calculate the discharge that runs into the basins, created by dams.

In this chapter the discharge will be calculated with different flood models. The Curve Number method is based on a unit hydrograph and is, together with the rational method, general used. Both methods use the rainfall intensity.
The Kenya formula is a local empirical method does not use rainfall data. Neither does the Head Office method, but characteristics of the area are charged in coefficients. The Orstom method is an empirical African method that uses the rainfall depth. The Snyder method and Taylor and Schwartz method are based on synthetic unit hydrographs, multiplied with a factor dependent on the rainfall intensity.

Different discharges will be estimated.
The 10-year peak discharge is used to determine the width and height of the structure. Because the whole structure is used as spillway during a 10-year flood, this discharge is also used to control the height of the riverbanks.
The actual spillway will be designed for a dominant discharge with a return period of 0.5 year. For this estimation only the rational and Curve Number method are used. Finally the 100-year peak discharge will be determined.

9.1 RATIONAL METHOD

The rational method is widely used around the world for flood estimation on small rural drainage basins of 40 km^2 or less.
The rational equation assumes that the rainfall rate and the rate of infiltration are constant.

$$Q = C \times I \times A \times \frac{10}{3600}$$

where: Q = the peak runoff rate (m^3/s)
C = the runoff ratio (-)
I = the average rainfall intensity (mm/hours)
A = the drainage basin area (m^2)

The runoff coefficient depends on the character of the surface, here C = 0,3.
For design, the rainfall intensity is estimated from the rainfall intensity-duration-frequency data for the location (see paragraph 7.5).

The time of concentration is the length of time necessary for water to flow from the most distant part of the watershed to the point of discharge. If the period of precipitation exceeds the time of concentration, then the rational method will apply.

$$t_c = \frac{L}{v} \times \frac{1}{3600}$$

where: t_c = the concentration time (hours)
L = the length from the watershed to the point of discharge (m)
v = the water velocity in the longest river section (m/s), here v = 2 m/s.

The corresponding average rainfall intensity can be read from Figure 7.1 on page 29. The results of the rational method are shown in Table 9.1.

Table 9.1 Results of the rational method.

A (km^2)	L (m)	t_c (min)	I (mm/h)	Q_{10} (m^3/s)
5	3000	25.0	77	32.3
10	7000	58.3	46	38.2
25	11000	91.7	35	72.1
50	15000	125.0	29	119.0

9.2 CURVE NUMBER METHOD

The Curve Number Method is based on the Unit Hydrograph. By dividing a complex rainfall event in a couple of single showers different single hydrographs can be calculated. Superimposing the single hydrographs will result in the rainfall-discharge relation for the complex rainfall event.

The design rainfall event is a shower with a duration of 4 hours. The total precipitation is 150 mm and is evenly divided over the duration of the shower. The unit duration of a shower is set on 30 min, thus 8 hydrographs will be calculated.

9.2.1 Curve Number

The first step in using the Curve Number Method is determining the CN. The CN depends on Antecedent Moisture Condition (AMC), the soil group and the land use. The AMC is set on type II, average conditions. The hydrologic soil group can be determined from Table 9.2. The texture class is sandy loam, which is comparable with hydrologic soil group B.

Table 9.2 Hydrological Soil Group[2].

Hydrologic Soil Group	Description
A	Sand or loess
B	Fine Loess or sandy clay
C	Silty clay
D	Clay

With the Curve Number can be determined. As there is not much knowledge about curve numbers in National Parks in Kenya the CN for poor woods or forestland is used. The value for CN is 66.

[2] From: Prof. Ir. R. Brouwer c.s., Irrigatie en Drainage, Page number 4.5.

9.2.2 Discharge volume

The potential maximum retention can be determined from the following formula:

$$S = \frac{25400}{CN} - 254 = 131mm$$

where: S = Potential maximum retention (mm)
CN = Curve Number (-)

Now the cumulative discharge depth can be calculated with:

$$V = \frac{(P - 0.2S)^2}{P + 0.8S}$$

where: V = cumulative discharge depth (mm)
P = cumulative precipitation depth (mm)
S = potential maximum retention (mm)

In Appendix 10 calculations are given for the 4 different sizes of catchment areas and their discharge depths.

9.2.3 Unit hydrograph

The last step in the Curve Number Method is using the unit hydrographs to calculate the discharge of the 8 single showers. First the concentration time is calculated with Kirpich (1940):

$$T_c = 0.02 \times L^{0.77} \times i^{-0.38}$$

where: T_c = concentration time (min)
L = length from the watershed to the point of discharge (m)
i = gradient of drift (-)

The catchment lag T_l can be calculated by multiplying the concentration time by 0.6. The time to peak can be calculated by:

$$T_p = 0.5 \times D + T_l$$

where: T_p = time to peak (min)
D = unit duration of a shower = 30 min
T_l = catchment lag (min)

The base time of a single unit shower is determined by this formula:

$$T_b = \frac{T_p}{SR}$$

where: T_b = Base time of a single unit shower (min)
T_p = time to peak (min)
SR = Shape Ratio = 0.3

The results of the different time parameters are shown in Table 9.3. To illustrate the unit hydrograph method Figure 9.1 is made.

Table 9.3 Time parameters in the Curve Number Method.

A (km^2)	T_c (min)	T_l (min)	T_p (min)	T_b (min)
5	59.6	35.8	50.8	169.2
10	114.4	68.7	83.7	278.9
25	162.1	97.2	112.2	374.2
50	205.8	123.5	138.5	461.6

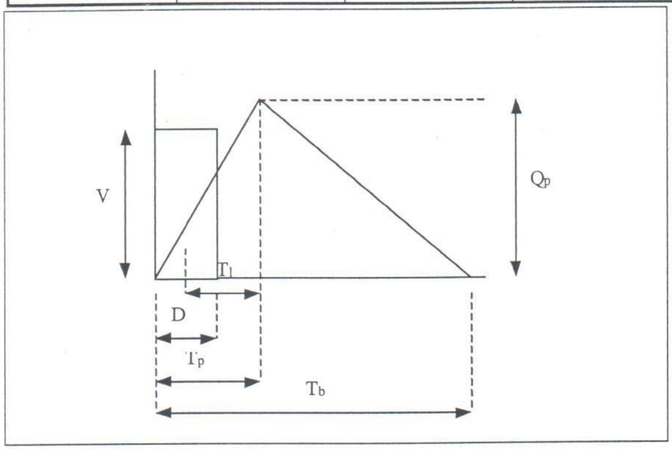


Figure 9.1 Unit Hydrograph.

9.2.4 Peak discharge

The peak discharge can be calculated by the waterbalance. Thus the total inflow must be equal to the total outflow. The inflow can be determined by the precipitation depth over a certain area. The outflow is equal to the surface under a unit hydrograph (see also). The result is this equation:

$$V \times A \times 1000 \;=\; 0.5 \times T_b \times 60 \times Q_p \quad \text{or} \quad Q_p = 2 \times \frac{V \times A \times 1000}{T_b \times 60}$$

where: V = unit discharge depth of 1 mm
A = size of catchment area (km^2)
T_b = base time of a unit hydrograph (min)
Q_p = peak discharge of a unit hydrograph with discharge depth of 1 mm (m^3/s)

By multiplying the unit peak discharge by the discharge depth all the single peak discharges can be calculated. The calculation is shown in Appendix 10 together with all the graphs. The results are shown in Table 9.4.

Table 9.4 Results of the Curve Number Method.

A (km^2)	Peak discharge (m^3/s)
5	30.4
10	49.0
25	100.7
50	171.9

9.3 KENYA FORMULA[3]

This formula does not use basin characteristics such as slope and absorption or rainfall data, but is the equation of the curve of peak discharges as a function of the catchment area. Therefor the formula is very restricted.

$Q = 15.9 \times A^{0.89}$ for $A < 23$ km^2

$Q = 67.5 \times A^{0.42}$ for $A > 23$ km^2

where: A = the drainage area (km^2)
Q = the maximum flood (m^3/s)

The results of the Kenya formula are shown in Table 9.5.

Table 9.5 Results of the Kenya formula.

A (km^2)	Q_{10} (m^3/s)
5	66.6
10	123.4
25	260.9
50	349.0

9.4 HEAD OFFICE SKETCH NO. 200 METHOD[4]

This method is only applicable for catchment areas up to 13 km^2 and therefor will not be used for the areas of 25 and 50 km^2. The method copes with basin characteristics but does not use rainfall data.

The class of the area to be drained depends on four coefficients, determined by soil, slope, projection and shape of the catchment area.

Soil	Clayey – rocky	2.0
Slope	Gentle – mixed	1.0
Projection	Scrub – grass	1.0
Shape	Long – balloon	0.5

'Class of the area to be drained' = 2.0 + 1.0 + 1.0 + 0.5 = 4.5
Given the catchment area and the class, the run off can be determined by using the given diagram. The peak discharge is calculated by multiplying the run off by the catchment area. The results are shown in Table 9.6.

Table 9.6 Results of the Head Office sketch no. 200 method.

A (km^2)	Run off (m^3/s·km^2)	Q_{10} (m^3/s)
5	5.6	28.0
10	4.5	45.0

9.5 ORSTOM METHOD[5]

The method is based on data of exceptional floods from a total of 90 representative and experimental catchments in French speaking African countries. The 'Office de la recherché scientifique et technique outremer' (Orstom) in Paris has collected these data. It is only used for the estimation of 10-year floods.

[3] From: NEDECO, – Shinyanga Water Supply Survey, a water master-plan study for the Shinyanga region.
[4] From: NEDECO, – Shinyanga Water Supply Survey, a water master-plan study for the Shinyanga region.
[5] From: Handbook Retendues D'eau in Burkina Faso.

Most of the used coefficients are derived from given diagrams, depending on the type of climate of the studied area. In our case the diagrams for a sahel to subdesert climate are used, with 150 – 800 mm rainfall per year.

$$Q_{10} = \frac{K \times K_r \times A \times P_{10} \times S}{t_b}$$

where: Q_{10} = 10-year peak discharge (m^3/s)
K = sharpness of peak discharge (-)
K_r = run-off coefficient (-)
A = area reduction coefficient (-)
P_{10} = 10-year maximum 24-hour rainfall depth (m)
S = drainage area (m^2)
t_b = base time of the flood (s)

In order to determinate the values of the coefficients, using the given diagrams, it is necessary to know the Orstom-class of the area. This class consists of two factors:
R = slope factor
Here R_3: the gradient of the drainage area varies between 0.5% and 1%, plain with undulating areas.
P = adsorption factor of the basin
Here P_2: the basin is mostly impermeable, but with small permeable areas.

Ad. K
Because there is no information on measured discharges in the drift, a table is used to determine K.

Ad. K_r
The value of K_r can be read in a given table that suits for R_3 and P_2.

Ad. A
The smaller the drainage area, the more likely it will be that a storm rainfall incident covers the whole area. When the drainage area increases, the value for A will decrease to charge this effect.
Vuillaume derived a relationship between A, the drainage area S, the return period T and the annual rainfall P_{an}:

$$A = 1 - 0.001 \times (9 \log T - 0.042 \times P_{an} + 152) \log S$$

where: P_{an} = 500 mm (see paragraph 7.2)

Ad. t_b
The base time of the flood can be read from a given diagram, depending on the drainage area and R_3. The results of the Orstom method are shown in Table 9.7.

Table 9.7 Results of the Orstom method.

A (km^2)	A	K_r	K	T_b (s)	Q_{10} (m^3/s)
5	0.90	0.70	2.6	20,880	58.8
10	0.86	0.63	2.6	30,240	69.9
25	0.80	0.53	2.5	49,680	80.0
50	0.76	0.47	3.0	72,000	111.6

9.6 SNYDER METHOD[6]

The Snyder method is based on observations made in catchment areas from 25 km^2 up to 25.000 km^2 in the Eastern United States, but the method is also widely used in other areas. This results in less useful peak discharges for the catchment areas of 5 and 10 km^2.
Snyder has derived a synthetic unit hydrograph that relates hydrograph characteristics to physical characteristics of the drainage basin and from which the peak discharge can be calculated. The peak discharge for 10 mm rainfall is defined by:

$$Q_p = \frac{(2.75 \times C_p \times A)}{t_p}$$

where: Q_p = the peak discharge for 10 mm rainfall (m^3/s)
C_p = coefficient 0.56-0.69; the value for C_p increases with a steeper basin and less impermeable terrain, here C_p = 0.6.
A = catchment area (km^2)
t_p = catchment lag (hours)

The key item of this method is the relation of the catchment lag t_p, this is the time from the centre of the rainfall excess to the peak of the hydrograph, to geometrical characteristics of the basin:

$$t_p = 0.75 \times C(L \times L_C)^{0.3}$$

where: t_p = catchment lag (hours)
C = coefficient 1.8-2.2; the steeper and less permeable the catchment, the lower the value of C, here C = 2.0
L = distance along major stream to the basin boundary (km), here L = 15 km
L_c = distance along major stream channel to approximately the centre of gravity of the basin (km), here L_c = 9 km

This results in a catchment lag of t_p = 6.5 hours.
The standard duration of a rainfall event is estimated by:

$$t_r = \frac{t_p}{5.5} = 1.2 \text{ hours}$$

Consequently the design rainfall is described by duration of 1.2 hours and the corresponding intensity of 40 mm/hour can be read from the rain duration curve (Figure 7.1 on page 29). The total depth is then 1.2 hours × 40 mm/hour = 48 mm. The peak discharge for a design storm is obtained by multiplying the unit hydrograph peak discharge Q_p with a factor 48 mm/10 mm = 4.8 The results of the Snyder method are shown in Table 9.8.

[6] From: NEDECO, – Shinyanga Water Supply Survey, a water master-plan study for the Shinyanga region.

Table 9.8 Results of the Snyder method.

A (km^2)	L (km)	Q_{10} (m^3/s)
5	3	6.1
10	7	12.2
25	11	30.5
50	15	60.9

9.7 TAYLOR AND SCHWARTZ METHOD[7]

This method is an improved synthetic hydrograph, based on observations on drainage areas in the Eastern United States. The same estimation for the standard duration of a storm as Snyder is used. The unit hydrograph peak discharge of a storm of 10 mm depth and during t_r is determined by:

$$Q_p = 7.76 \times m'' \times A \times \exp(m' \times t_r)$$

where: Q_p = the peak discharge for 10 mm rainfall (m^3/s)
A = catchment area (km^2)
t_r = standard duration of a storm, according to Snyder t_r = 1.2 hour

$$m' = \frac{0.30}{(L \times L_c)^{0.36}} = 0.051$$

where: L = distance along major stream to the basin boundary (km), here L = 15 km
L_c = distance along major stream channel to approximately the centre of gravity of the basin (km), here L_c = 9 km

$$m'' = 0.121 \times s^{0.142} - m' - 0.050 = -0.040$$

where: s = gradient (-), here s = 8‰.

Like the Snyder method, the peak discharge for a design storm is obtained by multiplying the unit hydrograph peak discharge Q_p with a factor 4.8 The results of the Taylor and Schwartz method are shown in Table 9.9.

Table 9.9 Results of the Taylor and Schwartz method.

A (km^2)	Q_p (m^3/s)	Q_{10} (m^3/s)
5	1.9	9.1
10	3.8	18.1
25	9.4	45.3
50	18.9	90.5

[7] From: NEDECO, – Shinyanga Water Supply Survey, a water master-plan study for the Shinyanga region.

9.8 CONCLUSION 10-YEAR PEAK DISCHARGE

Several methods to estimate the peak flood have been discussed. In Table 9.10 the results are collected.

Table 9.10 Comparison of flood formulae for calculation 10-year peak discharge.

Q_{10} (m^3/s) Flood formula	Drainage Area 5 km^2	10 km^2	25 km^2	50 km^2
Rational method	32.3	38.2	72.1	119.0
Curve Number method	30.4	49.0	100.7	171.9
Kenya formula	66.6	123.4	260.9	349.0
Head Office sketch no. 200 method	28.0	45.0	-	-
Orstom method	58.8	69.9	80.0	111.6
Snyder method	6.1	12.2	30.5	60.9
Taylor and Schwartz method	9.1	18.1	45.3	90.5
Average of all estimations	33.0	50.8	98.0	150.6
Average of selected estimations	37.4	50.5	84.3	135.0

In the literature and also from this table, can be concluded that the Snyder method and the Taylor and Schwartz method both give very low peak discharges. The Kenya formula however, gives very high values. The remaining methods (the Rational method, the Curve Number method, the Head Office sketch no. 200 method and the Orstom method) seem to estimate the peak discharge quite well. An average of these results is taken. For further calculations will be used and is summarised in Table 9.11.

Table 9.11 10-year peak discharge.

A (km^2)	Q_{10} (m^3/s)
5	35
10	50
25	85
50	135

Compared to the result in Water for wildlife these results are somewhat higher. This is caused by the higher design precipitation.

9.9 CONCLUSION 0.5-YEAR DISCHARGE

The 0.5-year peak discharge is needed to calculate the waterlevel that occurs twice a year above the causeway. The rational method is used. For the calculation with the rational method the rainfall intensity is estimated from Figure 7.1 on page 29. The results of the rational method are shown in Table 9.12.

Table 9.12 0.5-year discharge of the rational method.

A (km^2)	L (m)	T_c (min)	I (mm/h)	$Q_{0.5}$ (m^3/s)
5	3000	25.0	26	10.8
10	7000	58.3	15	12.7
25	11000	91.7	12	24.0
50	15000	125.0	10	39.7

For the design of the causeway the results of Table 9.12 are used.

Table 9.8 Results of the Snyder method.

A (km^2)	L (km)	Q_{10} (m^3/s)
5	3	6.1
10	7	12.2
25	11	30.5
50	15	60.9

9.7 TAYLOR AND SCHWARTZ METHOD[7]

This method is an improved synthetic hydrograph, based on observations on drainage areas in the Eastern United States. The same estimation for the standard duration of a storm as Snyder is used. The unit hydrograph peak discharge of a storm of 10 mm depth and during t_r is determined by:

$$Q_p = 7.76 \times m' \times A \times \exp(m'' \times t_r)$$

where: Q_p = the peak discharge for 10 mm rainfall (m^3/s)
A = catchment area (km^2)
t_r = standard duration of a storm, according to Snyder t_r = 1.2 hour

$$m' = \frac{0.30}{(L \times L_c)^{0.36}} = 0.051$$

where: L = distance along major stream to the basin boundary (km), here L = 15 km
L_c = distance along major stream channel to approximately the centre of gravity of the basin (km), here L_c = 9 km

$$m'' = 0.121 \times s^{0.142} - m' - 0.050 = -0.040$$

where: s = gradient (-), here s = 8‰.

Like the Snyder method, the peak discharge for a design storm is obtained by multiplying the unit hydrograph peak discharge Q_p with a factor 4.8 The results of the Taylor and Schwartz method are shown in Table 9.9.

Table 9.9 Results of the Taylor and Schwartz method.

A (km^2)	Q_p (m^3/s)	Q_{10} (m^3/s)
5	1.9	9.1
10	3.8	18.1
25	9.4	45.3
50	18.9	90.5

[7] From: NEDECO, – Shinyanga Water Supply Survey, a water master-plan study for the Shinyanga region.

10 EVAPORATION

10.1 INTRODUCTION

An important loss from reservoirs is open water evaporation. For the design of a water conservation structure the total amount of evaporation during the dry season has to be calculated.

10.2 OPEN WATER EVAPORATION

The open water evaporation can be calculated by using the Penman formula. Penman (1948) has deducted a formula based on the empirical equation of Dalton combined with the energy balance.

$$E_o = \frac{\left(\frac{s \cdot R_n}{L} + \frac{c_p \cdot \rho_a}{L} \cdot \frac{(e_s - e_a)}{r_a} \right)}{(s+\gamma)}$$

where: E_o = open water evaporation in (mm/d)
R_n = net radiation in (MJ/d·m^2)
L = latent heat of vaporisation = 2,4518 · 10^6 J/kg
S = slope of the vapour pressure curve in (kPa/ °C)
c_p = specific heath (at constant pressure) = 1004 J/kg·K
ρ_a = specific density = 1,2047 kg/m^3
e_a = actual vapour pressure in (mbar)
e_s = saturated vapour pressure at air temperature t_a in (mbar)
γ = psychrometric constant = 0,67 mbar/K
r_a = aerodynamic resistance to water vapour in (d/m)

According to a study on evapotranspiration[8] in Kenya (Table 10.1) the average temperature t_a, the net radiation R_n and the windrun u_2 has been measured at Voi. The radiation is given in Langley/day, to convert this to J/d·m^2 the given values are multiplied by a factor 41.904.
The average station barometric pressure is 952,05 mbar.

Table 10.1 Measured data in Voi

Month	t_a (°C)	R_n (Langley/day)	Rn (MJ/day·m^2)	u_2 (m/day)	u_2 (m/s)
January	25.9	486	20.37	149000	1.72
February	26.6	505	21.16	153000	1.77
March	27.1	507	21.25	161000	1.86
April	26.1	450	18.86	188000	2.18
May	25	408	17.10	241000	2.79
June	23.7	390	16.34	260000	3.01
July	22.8	364	15.25	271000	3.14
August	22.6	363	15.21	288000	3.33
September	23.4	402	16.85	252000	2.92
October	25	466	19.53	221000	2.56
November	25.8	500	20.95	166000	1.92
December	25.5	497	20.83	140000	1.62

[8] From: The Study of the National Water Master Plan.

The saturated vapour pressure depends on the temperature of the air and can be calculated by:

$$e_s = 1{,}3332 \cdot \exp\left(\frac{17{,}25 \cdot t_a}{237{,}3 + t_a} + 1{,}51977\right)$$

With: t_a = temperature of the air

When 'e_s' is known, the slope of the vapour pressure curve can be derived from:

$$s = \frac{4093{,}425 \cdot e_s}{(237{,}3 + t_a)^2}$$

The actual vapour pressure is calculated from the mean relative humidity RH in %:

$$e_a = RH \cdot e_s$$

The mean relative humidity RH can be derived from the measured minimum and maximum relative humidity in Voi (Table 10.2):

$$RH = \frac{1}{2} \cdot (RH_{min} + RH_{max})$$

Table 10.2 Mean relative humidity.

Month	RH_{min} (%)	RH_{max} (%)	RH (%)
January	45	90	67.5
February	41.7	88.3	65
March	41.7	88.3	65
April	48.3	88.3	68.3
May	51.7	86.7	69.2
June	46.7	83.3	65
July	45	82.5	63.75
August	45	85	65
September	43.3	86.7	65
October	43.3	86.7	65
November	46.7	86.7	66.7
December	50	90	70

The aerodynamic resistance r_a is a function of the windspeed:

$$r_a = \frac{245}{0{,}54 \cdot u_2 + 0{,}5} \quad \text{in [s/m]}$$

With: u_2 = 24-hour windrun at a height of 2 m above ground surface in [m/s].
In the Penman formula the r_a is needed in [day/m], therefore the value for r_a is divided by 86400.

All factors in the Penman formula can now be calculated (Table 10.3), which leads to the monthly open water evaporation:

Table 10.3 Factors in the Penman formula.

Month	e_s (Mbar)	s (kPa/ °C)	e_a (Mbar)	r_a ($\cdot 10^{-3}$ day/m)
January	33.28	1.97	22.46	1.981
February	34.68	2.04	22.54	1.947
March	35.71	2.09	23.21	1.883
April	33.67	1.99	23.00	1.693
May	31.55	1.88	21.83	1.413
June	29.19	1.75	18.97	1.334
July	27.65	1.67	17.62	1.293
August	27.31	1.66	17.75	1.233
September	28.67	1.73	18.63	1.367
October	31.55	1.88	20.51	1.507
November	33.08	1.96	22.06	1.844
December	32.50	1.93	22.75	2.062

With the monthly values of the evaporation (Table 10.4) it is possible to calculate the open water evaporation during the dry period.

Table 10.4 Monthly open water evaporation.

Month	E_o (mm/day)	E_o (mm/month)
January	7.22	217
February	7.64	229
March	7.75	232
April	6.93	208
May	6.48	194
June	6.39	192
July	6.08	182
August	6.07	182
September	6.47	194
October	7.29	219
November	7.49	225
December	7.21	216
Total for 12 months:		**2490 mm**

In paragraph 7.6 it was concluded that the maximum duration of the dry period covers the months May till October. This leads to a maximum open water evaporation of 1163 mm in the dry period.

11 SEDIMENTATION AND EROSION

In semi-arid areas sedimentation has a considerable significance in design and construction of small dams. For the design of small dams concerning sedimentation roughly two steps have to be taken. The first step is to calculate the soil loss or sediment yield. The second step is to determine the trap efficiency of a dam by using the Brune trap efficiency curve. In the last paragraph a conclusion is made.

Figure 11.1 Erosion in Kitui-district.

11.1 SOIL LOSS OR SEDIMENT YIELD

Erosion is a complex interaction between topography, geology, climate, soil, vegetation, land use and man made development. For the calculation of erosion losses the United Soil Loss Equation or USLE method is used. This method uses the product of six major factors for the calculation of the soil loss per unit area. The factors depend on rainfall, soil, topography vegetation and man made development. The USLE computed the soil loss at a given site as a product of six major factors as follows:

$$A = R \times K \times L \times S \times C \times P$$

Where: A = soil loss per unit area (tons/ha/yr)
R = rainfall erosivity factor (-)
K = soil erodibility factor in (tons/ha/yr)
L = hill slope length factor (-)
S = hill slope gradient factor (-)
C = vegetation cover factor (-)
P = soil conservation practice factor (-)

Rainfall erosivity factor (R):
This factor takes the erosive force of rainfall into account. The rainfall erosivity factor can be obtained from the kinetic energy of every rainstorm, multiplied by the maximum 30-minutes intensity of that storm. An alternative way of getting the R factor is devised by Moore (1979), who found a relation between mean annual rainfall and R-values. The values are given in Table 11.1:

Table 11.1 Rainfall erosivity factor

Mean annual rainfall (mm)	R-factor
300	56.8
400	85.8
500	118.6
600	150.9
700	181.4
800	218.5
900	249.8
1000	281.7

The mean annual rainfall in Tsavo East National Park is 504 mm and for further calculations 500 mm is token (see also paragraph 7.2 on page 26). The R-value is 118.6.

Soil erodibility factor (K):
The soil erodibility factor describes the inherent erodibility of the soil expressed in the same units as the annual soil losses in tons/ha/yr. Numerous factors control the erodibility of cohesive soils such as grain size distribution, texture, permeability and organic content. Typical values for the K-factor are shown in Table 11.2.

Table 11.2 Soil erodibility factor

Texture class	Organic matter content (%)	
	< 0.5	2
Fine sand	0.16	0.14
Very fine sand	0.42	0.36
Loamy sand	0.12	0.10
Loamy very fine sand	0.44	0.38
Sandy loam	0.27	0.24
Very fine sandy loam	0.47	0.41
Silt loam	0.48	0.42
Clay loam	0.28	0.25
Silty clay loam	0.37	0.32
Silty clay	0.25	0.23

The topsoil in Tsavo East National Park is of the texture class sandy loam and is containing less than 0.5% organic matter. Thus the K-factor is 0.27.

The slope-length-steepness factor (LS):
This is a topographic factor relating erosion losses from a field of given length and slope when compared to a standard plot 72.6 ft long inclined at 9 % slope.
An equation devised by Wischmeier and Smith (1978) is used to calculate the LS-factor:

$$LS = \left(\frac{X}{22{,}13}\right)^m \times \left(0.065 + 0.045 \times s + 0.0065 \times s^2\right)$$

where: LS = slope length-steepness factor (-)
X = slope length (m)
m = an exponent, depending on the slope gradient (-)
s = slope gradient (%)

From the map the length of slopes are measured. The average distance between two (seasonal) flows is about 1000 m. For the slope gradient 1.5% is chosen. This is also an average value for the slopes in the area perpendicular to the (seasonal) flows. The value for m is depending on the slope gradient. The values of the exponent m is given in the table

m = 0.5	slope gradient > 5%
m = 0.4	3% < slope gradient < 5%
m = 0.3	1% < slope gradient < 3%
m = 0.2	slope gradient < 1%

Thus the exponent m in this case is set on 0.3. After computing all parameters into the equation the value for the LS factor is 0.46.

Vegetation cover factor (C):
The vegetation factor is having a big effect on the soil loss. Dense vegetation intercepts most raindrops before causing splash erosion on the ground. Also the roots of the vegetation make the soil less vulnerable to erosion. For different vegetation covers Wischmeier and Smith (1978) have developed Table 11.3.

Table 11.3 Vegetation cover factor

Vegetative canopy			Percent ground cover					
Type no	Type and height	Sub-Type*	0	20	40	60	80	95+
1	No appreciable Canopy	G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
2	Tall weeds or Short brush with average drop fall height of 50 cm.	G	0.17-0.36	0.10-0.17	0.06-0.09	0.032-0.038	**0.011-0.013**	0.003
		W	0.17-0.36	0.12-0.20	0.09-0.13	0.068-0.083	0.038-0.041	0.011
3	Appreciable Brush or bushes, with average dropfall height of 2 m.	G	0.28-0.40	0.14-0.18	0.08-0.09	0.036-0.040	**0.012-0.013**	0.003
		W	0.28-0.40	0.17-0.22	0.12-0.14	0.078-0.087	0.040-0.042	0.011
4	Trees or no Appreciable low brush, with average drop fall height of 4 m.	G	0.36-0.42	0.17-0.19	0.09-0.10	0.039-0.041	0.012-0.013	0.003
		W	0.36-0.40	0.20-0.23	0.13-0.14	0.084-0.089	0.041-0.042	0.011

* G: cover at surface is grass, grasslike plants, decaying compacted duff or litter at least 5 cm deep.
W: cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

The vegetation is best described as a mix between type 2 and 3, thorn bushes with an average dropfall height between 50 cm and 2 m. The surface is covered with grass, thus the subtype is G.
About 80 % of the surface is covered with the thorn bushes. In this case is chosen for a vegetation cover factor C with a value of 0.013.

Soil conservation practise factor P:
This factor gives a reduction if any soil conservation methods are used like terracing, contour-farming or contour strips. In the National Parks of Kenya no measures are token for soil conservation. Thus the soil conservation practise factor P is 1.

With all the parameters known the USLE equation gives:

$$A = 118.6 \times 0.27 \times 0.46 \times 0.013 \times 1 = 0.191 \,{}^{tons}\!/_{ha * yr}$$

The total soil loss in m^3/ha/yr can be obtained by devising the soil loss in tons/ha/yr with the bulk density of the soil. The bulk density of the redsoil is 1.5 tons/m^3 so the soil loss is 0.128 m^3/ha/yr or 0.0128 mm/yr.

For the different sizes of catchment areas the erosion rate is calculated in Table 11.4.

Table 11.4 Total Soil loss

Drainage area (km^2)	Soil loss (m^3/ha/yr)	Soil loss (mm/yr)	Soil loss (m^3/yr)
5	0.128	0.0128	64
10	0.128	0.0128	191
25	0.128	0.0128	638
50	0.128	0.0128	1532

11.2 TRAP EFFICIENCY CURVE BY BRUNE

Not all the eroded soil will be trapped in the reservoirs. The percentage of soil loss that will be trapped in the reservoir can be obtained from the trap efficiency curve by Brune[9]. By determining the reservoir size and the annual inflow of water the trap efficiency can be determined. The larger the reservoir capacity compared with the mean annual inflow the more sediment is trapped.

From the riverbed characteristics from paragraph 8.2 the size of the reservoirs can be calculated. A pyramid shape can systemise the size of a reservoir. The length of a reservoir is about 500 meters. The depth of a reservoir is chosen at 4 m. The volumes of the reservoirs are small. This can be caused by the systemisation, but the chosen depth of 4 m is at the high side. The results are shown in Table 11.5.

Table 11.5 Reservoir volume.

Drainage area (km^2)	Dam width (m)	Reservoir volume (m^3)
5	20	6667
10	25	8333
25	35	11667
50	50	16667

The annual inflow in the reservoir is determined with the rational method. The mean annual rainfall is 500 mm/yr and the runoff coefficient is chosen on 30 %. In Table 11.6 are the mean annual inflow volumes displayed.

Table 11.6 Mean annual inflow.

Drainage area (km^2)	Mean annual inflow (m^3)
5	750,000
10	1,500,000
25	3,750,000
50	7,500,000

By dividing the reservoir volume by the mean annual inflow the specific capacity can be determined. Brune has found a relation between specific volume and trap efficiency (Figure 11.2)..

[9] From: Maidment, Handbook of Hydrology, page number 12.38.

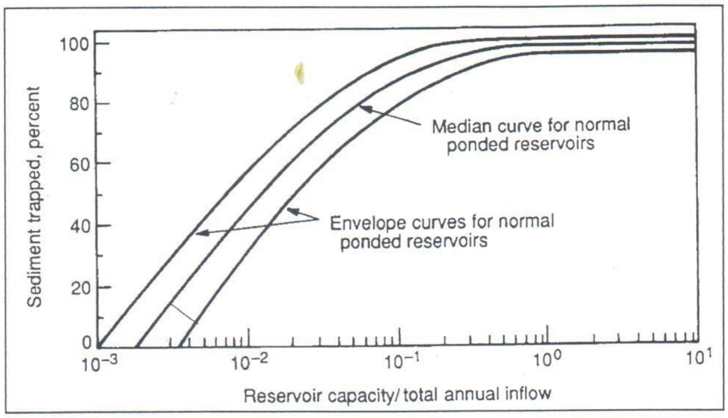


Figure 11.2 Trap Efficiency curve by Brune.

Figure 11.2 is used to determine the trap efficiency. In Table 11.7 are the different trap efficiencies shown for the different catchment areas.

Table 11.7 Trap efficiency.

Drainage area (km^2)	Reservoir volume (m^3)	Mean annual inflow (m^3)	Specific capacity (-)	Trap efficiency (%)
5	6667	750,000	0.009	40
10	8333	1,500,000	0.006	30
25	11667	3,750,000	0.003	15
50	16667	7,500,000	0.002	15

As the reservoir is designed for a life span of 10 years, the total amount of trapped sediment can be calculated. The results are shown in Table 11.8.

Table 11.8 Trapped Soil.

Drainage area (km^2)	Soil loss (m^3/yr)	Trap efficiency (%)	Trapped soil after 10 years (m^3)
5	64	40	255
10	191	30	574
25	638	15	957
50	1532	15	2298

The trapped soil is assumed to gather up in the deep at of the reservoir, close to the dam. This is not a good assumption but it can be used to get a design parameter for the height of the dam. The depth of the deposited layer is can be calculated by using the formula for the volume of a pyramid. Also the percentage of the reservoir which is silted up is determined. The results are shown in Table 11.9.

Table 11.9 Depth of deposited layer of silt after 10 years.

Drainage area (km^2)	Percentage of reservoir silted up (%)	Depth deposited layer (m)
5	4	0.26
10	7	0.40
25	8	0.48
50	14	0.65

The Design

12 SMALL WATER CONSERVATION STRUCTURES

In this chapter the general possible alternatives for a small water conservation structure will be discussed to which a distinction is made between surface water dams and groundwater dams.

12.1 SURFACE DAMS

A surface dam obstructs the flow and stores the water above the ground surface. Frequent occurring problems regarding the construction of small dams are rapid siltation of reservoirs and important water losses through evaporation. A separation can be made between an impermeable structure that obstructs the water flow and a permeable structure that delays the water flow.

12.1.1 Permeable dam

This type of dam can be used to create artificial recharge of a seasonal river or a reservoir. The dam does not contain an impervious centre and it is not necessary to construct the dam on an impervious layer or on rock, because there is no need for a

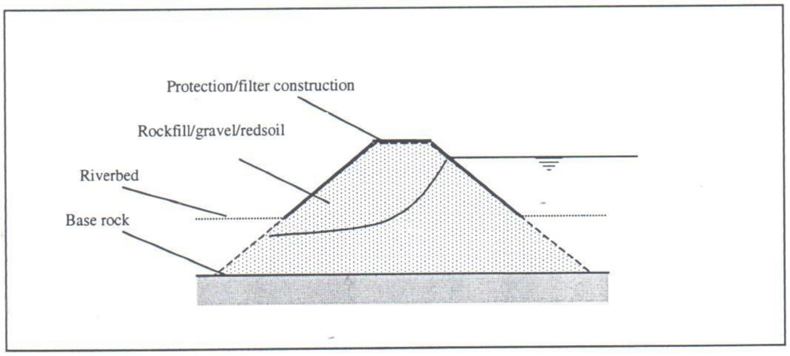


Figure 12.1 Permeable dam.

watertight connection. But for the stability of the dam it is considerable to make the foundation of the dam on the rock.
Special care should be taken of protection against piping. Making a decent filter construction could avoid this problem.

The construction mostly contains a centre of rock and gravel, covered with soil. To protect the dam from scouring vegetation is planted or other protection is provided. A spillway in the dam is constructed for small flows, but during floods the whole dam is acting like a weir. In Figure 12.1 a permeable dam is shown.

12.1.2 Impermeable dam

With an impermeable dam there should be no seepage trough and under the dam. The seepage through the dam can be solved by making an impervious wall. A construction of compacted clay, masonry, gabions, concrete or a combination of these materials can obtain this. When using a centre of clay makes sure that is well compacted to obtain the required result. To prevent underflow, the construction must be found on the rocksoil. A trench is excavated to the underlying rocksoil. A spillway in the dam is constructed for small flows, but during floods the whole dam is acting like a weir.

The general principle of an impermeable dam is shown Figure 12.2. A trench, reaching down to the bedrock/rocksoil, has been dug across a riverbed. An impervious wall has been constructed in this trench which arrests the flow. Slopes are constructed at both sides of the dam to create more stability and to give it a more natural look. The slope at the downstream side should have some protection to prevent scouring from overflowing water. It is wisely to use a filter construction of rockfill and gravel.

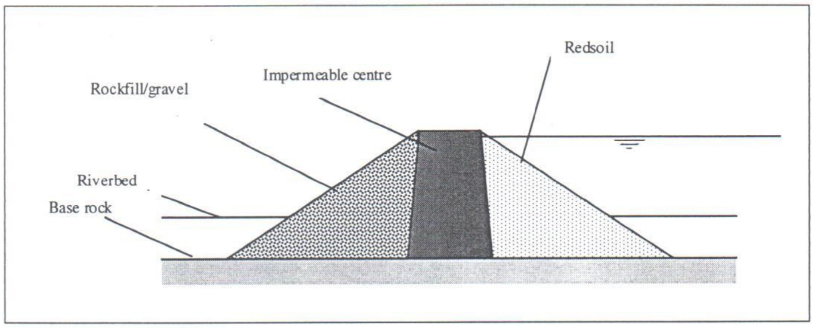


Figure 12.2 Impermeble dam.

The causeway subtype (Figure 12.3) is a dam that also functions as bridge for vehicles. The construction consists of two concrete walls, with in between rockfill and on top a concrete roof. The foundation of the causeway is also on rock. This subtype is a good opportunity to combine a dam and a bridge.

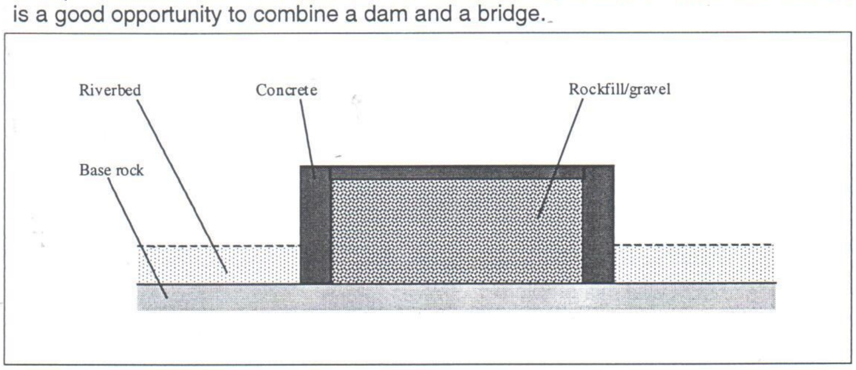


Figure 12.3 Causeway.

12.2 GROUNDWATER DAMS

A groundwater dam obstructs the flow of groundwater and stores water below the ground surface. The main advantage of using groundwater dams is a considerable reduction of water losses through evaporation. Water stored in sand storage dams is also less susceptible to pollution and other health hazards such as mosquito breeding. Groundwater dams may be of two types, sub-surface dams and sand-storage dams.

12.2.1 Sub-surface dams

A sub-surface dam is constructed below ground level and arrests the flow in a natural aquifer. The general principle of a sub-surface dam is shown in Figure 12.4. A trench, reaching down to the bedrock/rocksoil, has been dug across a valley, which contains an aquifer consisting of permeable alluvial sediments. An impervious wall has been constructed in this trench which arrests the flow in the aquifer. The impervious wall can be constructed of clay, masonry, gabions, concrete or a combination of these materials. Excess groundwater will flow above the dam crest.

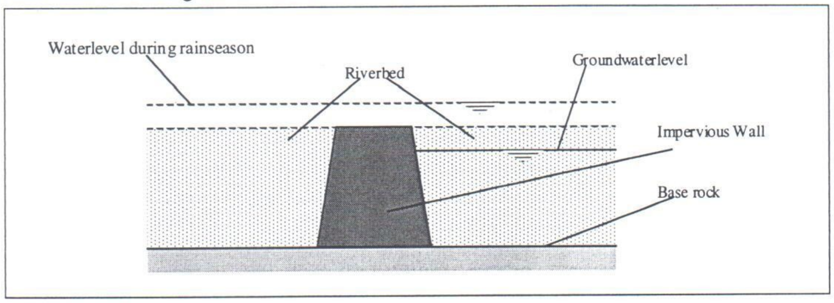


Figure 12.4 Sub surface dam.

12.2.2 Sand storage dams

A sand-storage dam impounds water in sediments caused to accumulate by the dam itself. The general principle of a sand-storage dam is illustrated in the figure below. By the construction of a weir of suitable height across a riverbed, sand carried by heavy flows during the rainy season has been deposited, and the reservoir has filled up with sand, hence constituting an artificial aquifer. This aquifer is replenished during the rains, and water for use during the dry seasons is stored.

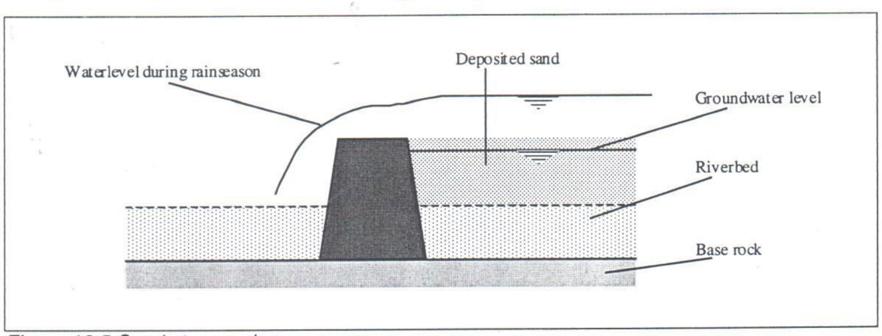


Figure 12.5 Sand storage dam.

13 DESCRIPTION AND EVALUATION OF THE PRESENT DAMS IN TSAVO EAST NATIONAL PARK

The evaluation of the present dams is separated in two main parts. In the first part contains the description of the three different series of dams in the different catchment areas. The second part will discuss the dams by construction type.

13.1 DESCRIPTION OF THE SERIES OF DAMS

In the past years several small dams have been built in three different catchment areas in the park. All dams are situated in small seasonal drifts that run into Galana River. The locations of the dams are indicated at the map in Appendix 11.

13.1.1 Rhino Release Area

Oliver's dam is the first dam built in the park, it is a concrete construction and it was finished in 1994. In 1998 downstream of Oliver's dam two earth dams have been built. The fourth dam in line, a natural dam with concrete heart, is still under construction. The distance between the dams is about 800-1000 meters. In the riverbed from some hundred meters upstream of Oliver's dam, till some hundred meters downstream the fourth dam all the time water is present, even in dry periods. The catchment area of this series of dams is about 110 km^2. Since the fist dam is six years old, the effects on the environment are already visible. Around the riverbed, a narrow green strip has evolved. The trees are obviously higher and the vegetation is greener than in the surroundings. Also in the Rhino Release Area a single causeway has been built. In Figure 13.1 a map of a part of the rhino release area is shown.

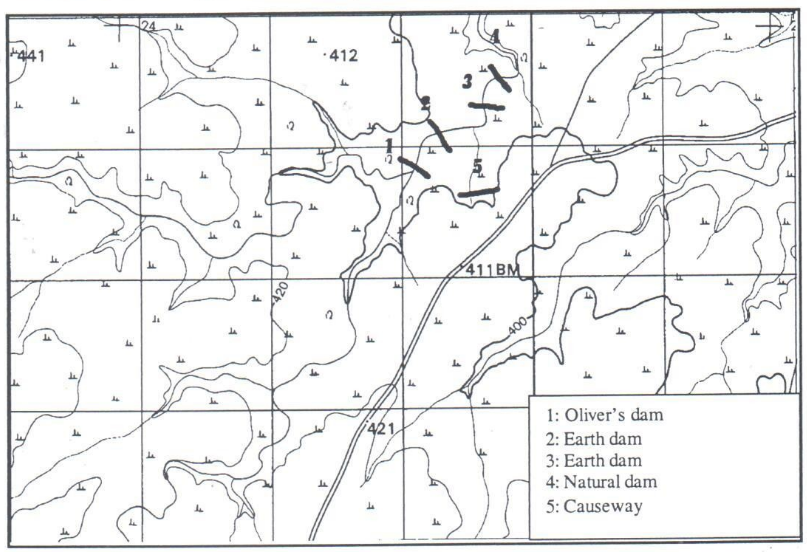


Figure 13.1 Dams in the Rhino release area.

13.1.2 Punda Milia

In Punda Milia two natural dams with a concrete heart have been built in the past year and one is still under construction. At the downstream side of these dams a causeway has been constructed. The distance between the dams is about 1000 meters. The catchment area of the series of dams is about 40 km^2. This series of dams catches water, but due to de fact that they have been built recently, nothing can be said yet about the effects on the environment. See also Figure 13.2.

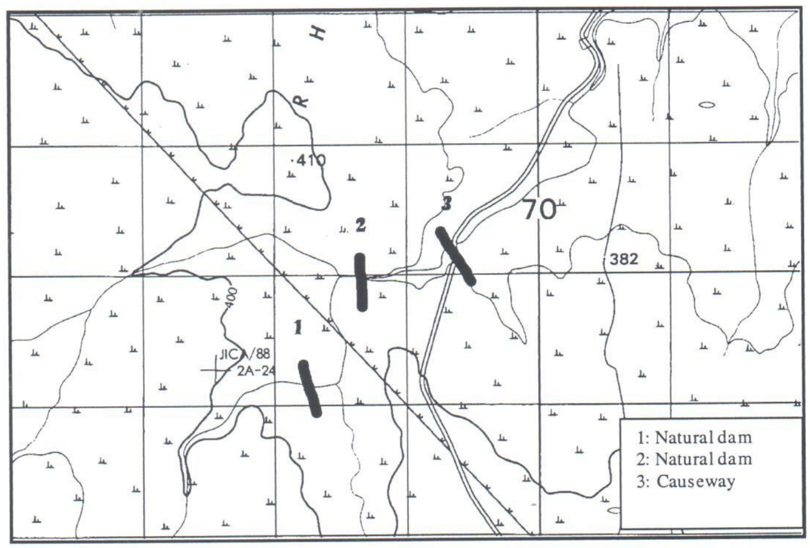


Figure 13.2 Dams in the Punda Milia area.

13.1.3 Ashaka

Last year a natural dam with a concrete heart has been constructed in Ashaka. The construction has been finished and the reservoir stores a lot of water. Downstream of the dam at great length surface water is present too. However, not much can be said about the long-term influence on the surroundings since the dam is less than a year old. The catchment area is about 15 km^2. In Figure 13.3 the location of Ashaka dam is shown.

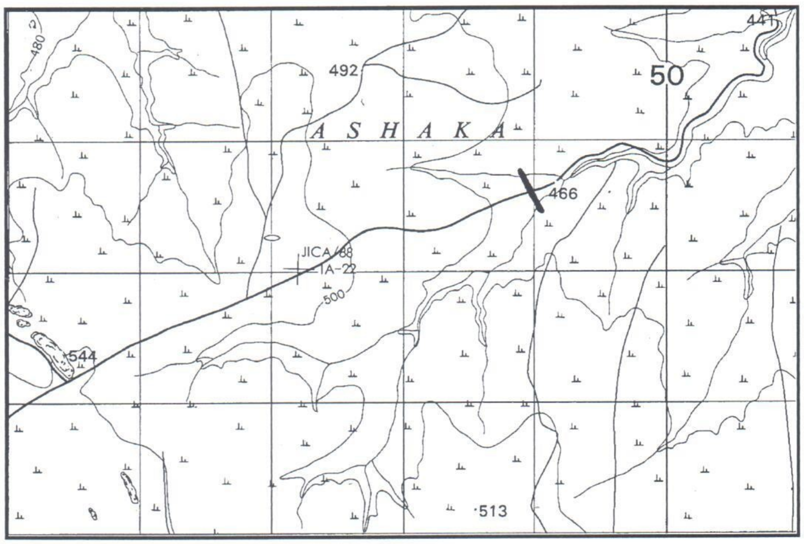


Figure 13.3 The Ashaka dam area.

13.2 DESCRIPTION OF THE CONSTRUCTIONS

In general the present dams can be divided in four types:

- Earth dam (dam 2 and 3 of the series in Rhino Release Area)
- Natural dam with concrete heart (dam 4 of the series in Rhino Release Area, dam 1 and 2 of the series in Punda Milia and Ashaka dam)
- Causeway (dam 3 of the series in Punda Milia and dam 5 in the Rhino Release Area)
- Concrete dam 1 in the rhino release area (Oliver's dam)

In the further analysis of the dams the causeway type and the concrete type will be discussed together. The differences between the two are to small to discuss them separate. The evaluated and discussed in the next paragraphs.
In the next paragraphs the three main types will be discussed.

13.2.1 Earth dam

This type of construction is used in the following locations:

- Dam 2 and 3 of the series in the Rhino Release Area.

The two earth dams have been constructed in a very short time and at low costs to bring in money to be able to build more dams. Thus the expected lifetime is short. The height of the dams is about 1,5 - 2 meters and the length varies 6 - 10 meters.

The dams have been founded on 1 meter riversand on rocksoil. The heart of the construction consists of rockfill and gravel. On top of this compacted redsoil has been placed with slopes 1:1. In the redsoil elephant grass has been planted, covered by a wire mesh to hold the construction together until the roots of the grass have developed.

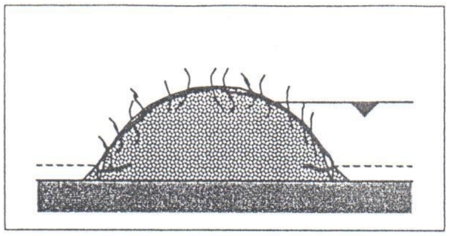


Figure 13.4 Earth dam.

As a consequence of this foundation and the used materials, there is a lot of seepage under and through the dam (Figure 13.4).

No special spillway has been constructed, but the connections with the embankments are higher than the rest of the dam to ensure that the water flows over the middle and does not scour the sides. No special protection against scouring has been made.
Neither has a stilling basin been constructed, but downstream of the dam rocksoil is at the surface.

There is a lot of scouring of the redsoil, especially in the centre that functions as spillway. At the downstream slope a big gap has occurred between the wire mesh and the remaining redsoil and rockfill. Probably the elephant grass is not strong enough to protect the dam, because the roots of the grass can not attach to the redsoil. Plans have been replace the wire mesh by sisal mats in the design for this dam.

13.2.2 Natural dam with concrete heart

This type of construction is used in the following locations:

- Dam 4 of the series in the Rhino Release Area.
- Dam 1 and 2 of the series in Punda Milia.
- Ashaka dam.

The height of the dams is about 2 - 3 meters and the length varies from 10 till 25 meters. The dam is built directly on the rocksoil. The heart consists of a concrete wall with a width of 0,5 meter, built by pouring 0,2 meter thick layers of concrete, which makes the dam non-permeable. Upstream of the dam the soil is covered with redsoil (slope 1:2) and downstream with natural stones (slope 1:3).

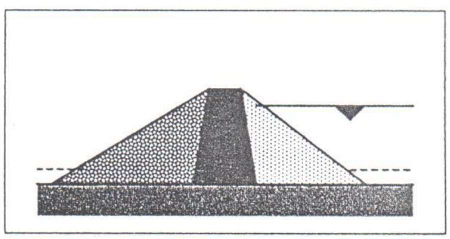


Figure 13.5 Natural dam.

The construction and connection with the embankments are covered with elephant grass to improve the consistency and to create a natural look. This dam is shown in Figure 13.5.
To prevent back-scouring the concrete wall is constructed in the embankments with an angle in upstream direction.

The spillway is covered with thick stones to protect against scouring. Under the thick stones a filter of smaller stones and gravel is constructed. The depth is 0,2 meter, the

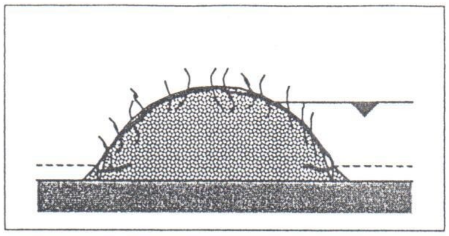


Figure 13.4 Earth dam.

The dams have been founded on 1 meter riversand on rocksoil. The heart of the construction consists of rockfill and gravel. On top of this compacted redsoil has been placed with slopes 1:1. In the redsoil elephant grass has been planted, covered by a wire mesh to hold the construction together until the roots of the grass have developed.
As a consequence of this foundation and the used materials, there is a lot of seepage under and through the dam (Figure 13.4).

No special spillway has been constructed, but the connections with the embankments are higher than the rest of the dam to ensure that the water flows over the middle and does not scour the sides. No special protection against scouring has been made.
Neither has a stilling basin been constructed, but downstream of the dam rocksoil is at the surface.

There is a lot of scouring of the redsoil, especially in the centre that functions as spillway. At the downstream slope a big gap has occurred between the wire mesh and the remaining redsoil and rockfill. Probably the elephant grass is not strong enough to protect the dam, because the roots of the grass can not attach to the redsoil. Plans have been replace the wire mesh by sisal mats in the design for this dam.

13.2.2 Natural dam with concrete heart

This type of construction is used in the following locations:

- Dam 4 of the series in the Rhino Release Area.
- Dam 1 and 2 of the series in Punda Milia.
- Ashaka dam.

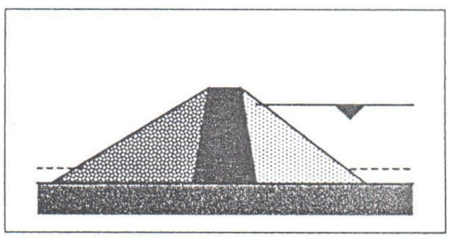


Figure 13.5 Natural dam.

The height of the dams is about 2 - 3 meters and the length varies from 10 till 25 meters. The dam is built directly on the rocksoil. The heart consists of a concrete wall with a width of 0,5 meter, built by pouring 0,2 meter thick layers of concrete, which makes the dam non-permeable. Upstream of the dam the soil is covered with redsoil (slope 1:2) and downstream with natural stones (slope 1:3). The construction and connection with the embankments are covered with elephant grass to improve the consistency and to create a natural look. This dam is shown in Figure 13.5.
To prevent back-scouring the concrete wall is constructed in the embankments with an angle in upstream direction.

The spillway is covered with thick stones to protect against scouring. Under the thick stones a filter of smaller stones and gravel is constructed. The depth is 0,2 meter, the

Figure 13.2 Dam 4 Rhino release area (under construction).

width is 0,5 meter and the length is 2 meters. This spillway is constructed only for normal discharges and during floods the whole dam functions as spillway. No special efforts are made to construct a stilling basin, it consists of the rocksoil downstream of the dam.

Of the dam in the Rhino Release Area, which is still under construction, the layers of concrete are not connected properly yet. Another failure is seepage through a crack in the underlying rock soil, which will be stopped by pouring a concrete beam.

13.2.3 Causeway or concrete dam

This type of construction is used in the following locations:

- Dam 3 of the series in Punda Milia.
- The single construction in the Rhino Release Area.
- Oliver's dam.

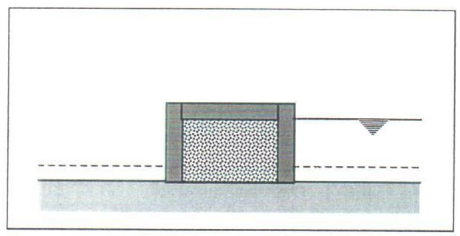


Figure 13.1 Causeway

The height of the dam is about 2-3 meters and the length varies from 10-30 meters.
This type is also used as a bridge for vehicles, so the width is at least 3 meters.
The dam is built directly on the rocksoil. The construction consists of vertical concrete walls, poured in layers of 20 cm, filled with compacted rockfill, and gravel and a poured concrete plate on top. The concrete of the dam connects with the rocks on the banks. The dam and the rock soil underneath are non-permeable. A cross-section of the causeway is shown in Figure 13.1.

No specific spillway has been constructed, but the surface of the dam is slightly curved, so the water will flow over the middle. The concrete plates don't need protection against scouring.
The stilling basin consists of rocksoil downstream of the dam, no other protection is used. In Figure 13.2 the causeway at Punda Milia is shown.

Figure 13.2 Causeway at Punda Milia.

Oliver's dam (Figure 13.3)

Oliver's dam is not used as causeway and a spillway has been constructed, with a width of 2 m and about 0,15 meter deep. The spillway is designed for the permanent flow over the dam. During extreme floods, the whole dam will function as spillway.

Figure 13.3 Oliver's dam.

Behind the whole dam there is a concrete slap of 1.5 m and 30 cm thick, but just after the spillway the concrete slap has collapsed for about 5 m, caused by the scouring of the water.
Also during El Nino the connection to the banks was damaged and the water flowed around the dam.
The connections with the banks are repaired, extended and strengthed.
There are plans to repair the concrete slap behind the spillway by dumping rockfill at the soil.

14 ADVICES FOR A DESIGNING A SEASONAL DRIFT

In this chapter advises for the design for a seasonal drift are given. First a choice between one or more dams in an area is made. In the second paragraph the choice for the dams is presented. The third paragraph will advise in the locations.

14.1 A ONE-DAM DESIGN OR MULTIPLE DAMS DESIGN

Roughly two options are possible for the design of a seasonal drift. The first option is making one big reservoir. This will attract all wildlife in the area and it is most likely that conflicts will occur between different species of wildlife. The main reasons to create only one reservoir are the costs. Also is it quite easy to find a single good location, rather then more good locations close to each other.
The second option is creating multiple small reservoirs. The main disadvantage is the costs. More dams are simply more expensive. But, more reservoirs will prevent wildlife conflicts. This is the main reason to advise more dams in an area.

14.2 TYPES OF DAMS

As described in chapter 12, there are several options for dam designs. In Tsavo East National Park it is not possible to create groundwater dams. Due to the relative low sedimentation rate is sand-storage dams are not an option. Groundwater dams are technically possible, but groundwater is not needed for wildlife.
Surface water dams are applicable. The permeable dam is a good option if you want to create a recharged reservoir. The main disadvantage of the permeable dams is piping, because of the flow of water through the dams. Impermeable dams are a solid solution. Piping is not relevant, but material for the impermeable centre must be driven in from outside the park.
The causeway and the natural dam with a concrete centre will be worked out in the next chapters. These two types are chosen because of their stability.

14.3 LOCATIONS

The best locations for dam constructions are those with the bedrock close to or on the surface. All the present dam sites are located at rocky outcrops, places where the bedrock is close to the surface. The rock is a very good foundation for the dams and it is not necessary to create a stillin basin, because the water can dissipate its energy on the rock. Piping won't occur at rocky outcrops.
Those sites where water is available even at the end of the dry period are a good place to construct a dam. Water is indication of natural recharge or a impermeable sub-surface and thus a good location for a dam.
Another good place to build a causeway at those places where a road crosses a seasonal drift. Beside creating a reservoir also the crossing with the drift will be improved.
To prevent human-wildlife conflicts, the dam sites should not be to close to the borders of the National Park. Also it is necessary to have a location with is easily accessible. Truckloads of materials should be able to reach the location.

15 DESIGN

15.1 INTRODUCTION

In this chapter the dimensions of the dam are determined. The most important parameter is the height of the dam, which follows from the condition, that there should be water in the reservoir all year round. When the height of the dam is known the dimensions of the spillway can be designed.
With the dimensions of the dam and the spillway known the stability of the structure can be checked.

15.2 DIMENSIONS OF THE DAM

The width of the dam follows from the width of the river and depends on the size of the catchment area. The following river dimensions were derived in the paragraph 'Characteristics of a drift'.

Table 15.1 Riverdimensions for different catchment areas.

Catchment area (km^2)	Width (m)	Depth (m)
5	20	4
10	25	4
25	35	4
50	50	4

The length of the dams follows from the Program of Requirements:
Causeway: L = 2.5 m.
Impermeable dam: L = 1.0 m.

The minimum height of the structure can be derived through analysing the outflow terms in the longest period without rain.
The depth-influencing parameters are:

- Evaporation
  In the chapter 'Evaporation' it was shown that the maximum dry period lasted 6 consecutive months, in the period from May until October.
  The evaporation in these months is 1163 mm, as can be seen in Table 15.2.

Table 15.2 Evaporation in the period May-October.

Month	E_o (mm/day)	E_o (mm/month)
May	6.48	194
June	6.39	192
July	6.08	182
August	6.07	182
September	6.47	194
October	7.29	219
	Total for dry period:	1163 mm

- Infiltration
  The amount of infiltration in the 'longest rainless' period follows from the assumption for the daily infiltration rate of 3 mm/day. For a period of 6 months the total amount of infiltration is 540 mm.

- Dead storage

The dead storage is the volume of the reservoir, which will be filled with sediment after 10 years. The thickness of the layer depends on the size of the catchment area as is shown in Table 15.3.

Table 15.3 Depth of deposited layer of silt after 10 years.

Catchment area (km^2)	Depth deposited layer (mm)
5	260
10	400
25	480
50	650

- Safety factor
  To take the uncertainty in the different calculated terms into account, a safety factor of 1.2 is applied.

The height of the dam can now be calculated:

$$h = \gamma \cdot (E_o + I + S)$$

Where: h = height of the dam [mm]
γ = safety factor = 1.2
E_o = open water evaporation in the 'dry period' [mm]
I = infiltration in the 'dry period' [mm]
S = dead storage [mm]

The results are shown in Table 15.4, for further calculations the design height will be used.

Table 15.4 Minimum height of the dam for different catchment areas.

Catchment area (km^2)	Minimum height (mm)	Design height (m)
5	2356	2.4
10	2524	2.6
25	2620	2.7
50	2824	2.9

15.3 DIMENSIONS OF THE SPILLWAY

For the causeway as well as for the natural dam the whole width of the dam acts as a spillway. The capacity of the spillway is dimensioned at the Q_{10}.
For the causeway an additional demand is that it should not be inaccessible for KWS-jeeps for more than two times per year. This means that during $Q_{0.5}$ the water level above the causeway may not exceed 0.5 m.

The height of the water above the causeway during $Q_{0.5}$ can be calculated by using the following depth-discharge relation:

$$Q_{0.5} = \frac{2}{3} \cdot \sqrt{(\frac{2}{3} \cdot g)} \cdot c_d \cdot b \cdot H^{3/2} \qquad \text{for } z \geq \frac{1}{3} \cdot H$$

Where: $Q_{0.5}$ = discharge with a return period of two times per year [m^3/s]
c_d = discharge coefficient
b = discharging width [m]
H = energy head above the crest [m]
y_c = critical waterdepth above the crest = $^2/_3$H [m]

The height of the water level above the dam at the maximum discharge, this is the discharge with a return period of 1:10 year, can be calculated by the following formula:

$$Q_{10} = \frac{2}{3} \cdot \sqrt{\left(\frac{2}{3} \cdot g\right) \cdot c_d \cdot b \cdot H^{3/2}} \qquad \text{for } z \geq \frac{1}{3} \cdot H$$

Where: Q_{10} = discharge with a return period of 1:10 year [m^3/s]
c_d = discharge coefficient
b = width of the dam [m]
H = energy head above the dam [m]
y_c = critical waterdepth above the crest = $^2/_3$H [m]

The discharge coefficient depends on the curve of the streamlines above the crest, and amounts c_d = 1 for a broad-crested weir and c_d =1,1 for a unrounded sharp-crested weir. A broad-crested weir occurs when the energy depth is less than ½ times the weir length: H ‹ ½L.

In the chapter on 'Flood models' $Q_{0.5}$ and Q_{10} were calculated, the values are shown in Table 15.5.

Table 15.5 $Q_{0.5}$ and Q_{10} for different catchment areas.

A (km^2)	$Q_{0.5}$ (m^3/s)	Q_{10} (m^3/s)
5	10.8	35
10	12.7	50
25	24.0	85
50	39.7	135

15.3.1 Causeway

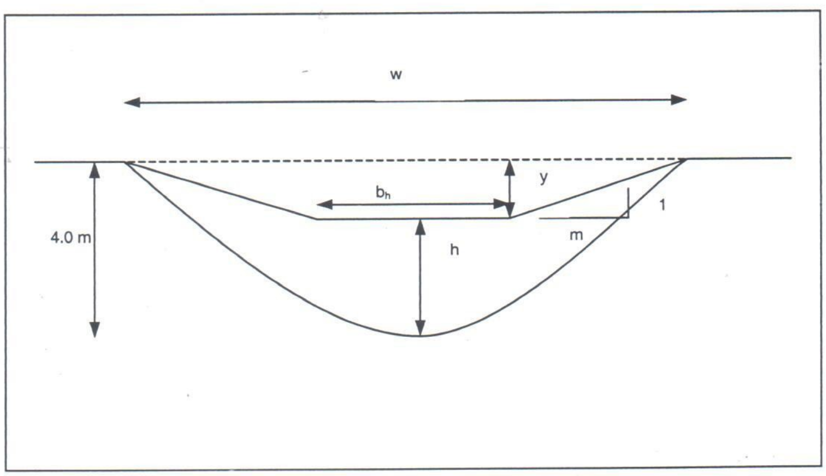


Figure 15.1 Cross-section of a causeway.

For the causeway the dimensions of the spillway are determined by the slope angle. The dimensions have to be checked for the decisive discharges:
Check for $Q_{0.5}$:
During $Q_{0.5}$ the waterlevel above the crest should not exceed 0.5 m.

1. Determination of the 'discharging width'.
   Above the triangular areas at the sides the discharge is half the amount of the discharge above a rectangular area of the same width. The discharging width can thus be calculated the following way:

   $$b = b_h + 2 \cdot \left(\frac{1}{2} \cdot m \cdot y \right) = b_h + m \cdot y \qquad \text{[m]}$$

   Where: b_h = horizontal part of the causeway = $w - 2 \cdot m \cdot (4.0 - h)$ [m]
   w = width of the river [m]
   m = side slope = m hor : 1 vert (=4)
   y = waterdepth above the causeway [m]

2. Determination of the water level.
   With $Q_{0,5}$ known and the discharging width and the energy head 'H' only dependent on the waterlevel 'y' above the causeway, this waterlevel can be calculated:

   $$Q_{0.5} = \frac{2}{3} \cdot \sqrt{(\frac{2}{3} \cdot g)} \cdot c_d \cdot b \cdot H^{3/2}$$

   $$Q_{0.5} = \frac{2}{3} \cdot \sqrt{(\frac{2}{3} \cdot 9{,}81)} \cdot 1{,}0 \cdot (b_h + 4 \cdot y) \cdot \left(\frac{3}{2} \cdot y \right)^{3/2}$$

   This can be solved through iteration.
   The calculation is made with the discharge coefficient for a broad-crested weir: c_d =1.

Table 15.6 Waterlevel above the causeway during $Q_{0.5}$

A (km^2)	$Q_{0.5}$ (m^3/s)	b_{hor} (m)	b (m)	H (m)	y (m)	Q (m^3/s)
5	10.7	7.2	9.3	0.78	0.52	10.9
10	12.7	13.8	15.5	0.63	0.42	13.2
25	24.0	24.6	26.4	0.66	0.44	24.1
50	39.7	41.2	43.0	0.68	0.45	40.7

As can be seen in Table 15.6 the values for the waterlevel are smaller than 0.5 m, except for a catchment area of 5 km^2 where the waterlevel above the causeway is 52 cms. The exceedance of the tolerable level is so small that this does not cause any problems.

In the used discharge formula two assumptions have been made, which should be checked:

- The discharge coefficient for a broad-crested weir is used. Therefor H should be smaller than ½L.
  With a causeway length of 2.5 m. 'H' should be smaller than 1.25 m. This condition is fulfilled for all different catchment area sizes.
- The formula is only valid under free flow conditions. Free flow occurs when the headloss z > $^1/_3$H. For the headloss the waterheadloss is taken. To check the free flow condition the downstream waterlevel is needed, which can be calculated using the Strickler equation:

  $$Q = k \cdot A \cdot R^{2/3} \cdot s^{1/2}$$

  Where: Q = discharge [m^3/s]
  k = roughness coefficient = 35 $m^{1/3}/s$
  A = flooded cross section [m^2]
  R = hydraulic radius [m]

s = slope of the energyhead = slope of the riverbed = 8‰

In Appendix 12 the downstream waterlevel is calculated for $Q_{0.5}$ and Q_{10}. The downstream waterlevel for $Q_{0.5}$ is 1,0 m.
The headloss can be calculated with the following formula: $z = (h+y) - Hw$.
With Hw is the downstream waterlevel. This leads to a headloss of 2.4 m.
It can be concluded that the free flow condition is fulfilled.

Check for Q_{10}:
During Q_{10} the embankments should not be overtopped. This demand leads to the maximum waterlevel y_{max} above the causeway.
The calculation is the same as for $Q_{0.5}$, which leads to the following results:

Table 15.2 Waterlevel above the causeway during Q_{10}.

A (km^2)	Q_{10} (m^3/s)	B_{hor} (m)	b (m)	y_{max} (m)	H_{max} (m)	y (m)	H (m)	Q (m^3/s)
5	35	7.2	11.4	1.60	2.40	1.04	1.56	37.7
10	50	13.8	17.6	1.40	2.10	0.95	1.43	51.0
25	85	24.6	28.5	1.30	1.95	0.97	1.46	85.2
50	135	41.2	45.1	1.10	1.65	0.98	1.47	137.1

As can be seen in Table 15.2 the values for the waterlevel are smaller than the maximum allowable values.

In the used discharge formula two assumptions have been made, which should be checked:

- The discharge coefficient for a broad-crested weir is used. Therefor H should be smaller than ½L.
  With a causeway length of 2.5 m. 'H' should be smaller than 1.25 m. All values of 'H' are larger than 1.25, thus the causeway acts as a sharp-crested weir. The discharge coefficient should be taken larger, which leads to a bigger discharge at the same energy head. Therefor it isn't necessary to make a control calculation.
- The formula is only valid under free flow conditions. The condition for free flow is: $z > {}^{1}/_{3}H$.
  The upstream waterlevel during Q_{10} is 3.9 m.
  The downstream waterlevel during Q_{10} is 1.9 m.
  This leads to an headloss of 2.0 m. Which is considerably larger than ${}^{1}/_{3}H$. Also during Q_{10} the free flow condition is met.

In appendix 13, the technical drawings of the causeway can be found.

15.3.2 Natural dam

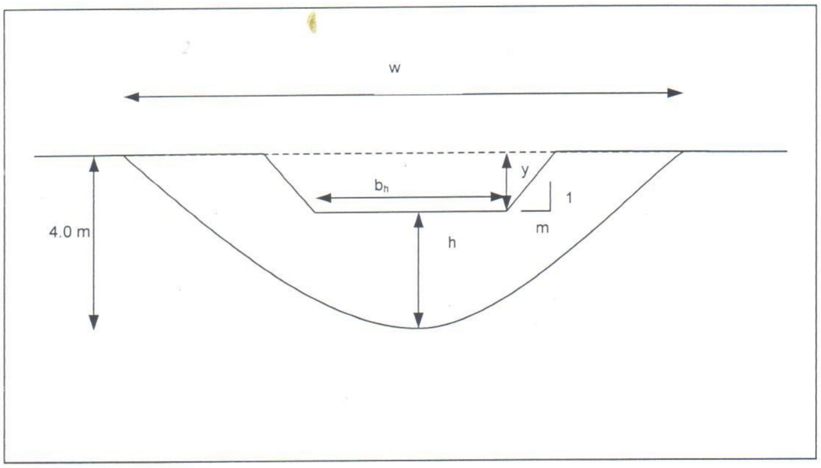


Figure 15.1 Cross-section of a natural dam.

For the natural dam the dimensions of the spillway are determined by the Q_{10}. The slopes of the spillway are 1:2. The horizontal part of the dam at both the embankments should be as large as possible under the condition that the embankments will not overtop. This leads to the condition for minimal horizontal width of the spillway.

Calculation:

1. Determination of the maximum allowable waterlevel above the spillway.
   This waterlevel can be calculated as follows:
   $y = 4{,}0 - h - f$
   Where: h = height of the dam [m]
   f = freeboard = 0.25 m.

2. Determination of the discharging width
   The discharging width can be calculated at the same way as for the causeway:
   $$b = b_h + 2 \cdot \left(\frac{1}{2} \cdot m \cdot y \right) = b_h + m \cdot y \qquad \text{[m]}$$
   Where: b_h = horizontal part of the spillway [m]
   w = width of the river [m]
   m = sideslope = m hor : 1 vert (=2)
   y = waterdepth above the spillway [m]

3. Determination of '$b_{h,max}$'
   The maximum possible horizontal width is calculated the following way:
   $b_{h,\max} = w - 2 \cdot m \cdot (4.0 - h)$

4. Determination of 'b_h'
   With Q_{10} and the waterlevel 'y' known the discharging width only depends on the width of the horizontal part of the spillway, this width can be calculated:

$$Q_{10} = \frac{2}{3} \cdot \sqrt{(\frac{2}{3} \cdot g)} \cdot c_d \cdot b \cdot H^{3/2}$$

$$Q_{10} = \frac{2}{3} \cdot \sqrt{(\frac{2}{3} \cdot 9{,}81)} \cdot 1{,}1 \cdot (b_h + 2 \cdot y) \cdot \left(\frac{3}{2} \cdot y\right)^{3/2}$$

The calculation is made with the discharge coefficient for a sharp-crested weir: c_d =1.1. The results can be found in Table 15.1

Table 15.1 Waterlevel above the natural dam during Q_{10}.

A (km^2)	Q_{10} (m^3/s)	y (m)	H (m)	$b_{h,max}$ (m)	b_h (m)	b (m)
5	37.4	1.35	2.03	13.6	4.3	7.0
10	50.5	1.15	1.73	19.4	9.6	11.9
25	84.3	1.05	1.58	29.8	20.7	22.8
50	135.0	0.85	1.28	45.6	48.3	50.0

Three checks have to be made:

- 'b_h' should not exceed '$b_{h,max}$'
  For catchment areas of 50 km^2 the needed horizontal width to be able to discharge Q_{10} with a freeboard of 0.25 m. is larger than the maximum possible width. In this case the freeboard is 0.2 m.
- The natural dam should act as a sharp-crested weir: H > ½ L
- The formula is only valid under free flow conditions. The condition for free flow is: $z > {}^1/_3 H$.

In appendix 13, the technical drawings of the dam can be found.

15.4 STILLING BASIN

The dam is foundated on bedrock, therefor there is no need for a stilling basin. The rock does not need extra protection against scouring.

15.5 CONNECTION DAM-RIVERBANK

To avoid the situation where the river flows around the dam the anchors (concrete walls) are built in the riverbanks for 5 m with an angle of about 45° with the length axis in upstream direction, this is the same for every catchment area. The walls have the same height as the embankments, 4 m. The walls have a thickness of 0.5 m.

16 STABILITY

In the chapter 'Advices for designing a seasonal drift' was concluded that the best solutions for water conservation in a National Park were:

- a 'causeway' at places where a road crosses a river, and
- a 'impermeable gravity wall' at off-road locations.

In this chapter the stability of both types of dams is checked.
In general dams must be able to resist the overturning moments, the sliding forces and the mechanical attacks due to wildlife. Furthermore scouring of the slopes should be avoided.

16.1 CAUSEWAY

The causeway is built directly on the underlying rock soil. The construction consists of two concrete walls, filled with natural stones and a concrete roof. The walls have a thickness of 0.5 meter and the roof has a thickness of 0.2 meter. The causeway is only built where a road crosses a drift. The height of the dam and the height of the water level upstream of the dam vary with the size of the catchment area, the values are calculated in the chapter 'Design' and the results are shown in Table 16.1.

Table 16.1 Height of the causeway.

A (km^2)	H (m)	y (m)
5	2.4	1.04
10	2.6	0.95
25	2.7	0.97
50	2.9	0.98

A cross-section of the causeway is drawn in Figure 16.1.

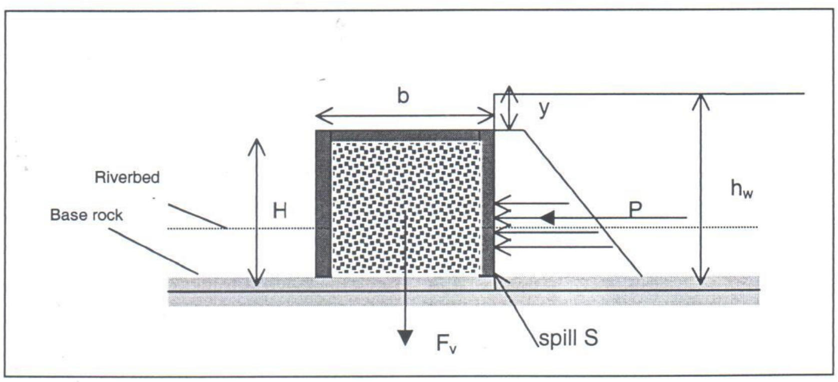


Figure 16.1 Causeway.

16.1.1 Stability of the construction

The causeway should be dimensioned to resist water pressure. Stability against sliding and overturning should be verified. The extreme situation is a 10-year peak flood, for which the water levels are calculated in the chapter 'Flood models'.

Calculation:

1. determination of the horizontal forces

Hydrostatic pressure upstream of the construction:

$$F_{hor} = P = \frac{1}{2} \cdot w \cdot \left(h_w^{\ 2} - y^2\right) \quad \text{[kN]}$$

Where: w = unit weight of water = 9.8 kN/m^3

h_w = water level upstream the construction [m]

y = water level above the crest [m]

2. determination of the vertical forces

ΣF_{vert} consists of the weight of the construction and the weight of the water above the crest. Weight of the construction and water:

$$F_{ver} = \sum \gamma \cdot V \quad \text{[kN]}$$

Where: γ = unit weight [kN/m^3]

γ_c = unit weight of concrete = 26.0 kN/m^3

γ_r = unit weight of rockfill = 24.5 kN/m^3

V = volume [m^3]

3. sliding

Sliding will not occur if:

$$\frac{\sum F_{hor}}{\sum F_{ver}} \leq 0.5$$

4. determination of the moment which is a result of the vertical and horizontal forces:

$$\sum M = \sum F \cdot a$$

Where: ΣM = total moment [kNm]

a = length between force and spill 'S' [m]

The location of spill 'S' is shown in Figure 16.1.

5. determination of R

$$R \cdot \sum F_{ver} = \sum M$$

Where: R = distance to the spill 'S' [m]

ΣF_{ver} = total vertical forces [kN]

ΣM = total moment [kNm]

6. overturning

Overturning will not occur if $R \leq 2/3 *b$

Where: b = width of the construction [m]

Since b = 2.5 m. for every catchment area, R_{max} = 2/3 *2.5 = 1.67 m.

The results of the calculation are shown in Table 16.2.

Table 16.2 Stability of the causeway.

A (km^2)	H (m)	y (m)	F_{hor} (kN/m)	F_{ver} (kN/m)	$\frac{\sum F_{hor}}{\sum F_{ver}}$	ΣM (kNm/m)	R (m)	R_{max} (m)
5	2.4	1.04	52.7	176.5	0.30	272.6	1.54	1.67
10	2.6	0.95	57.3	186.9	0.31	293.8	1.57	1.67
25	2.7	0.97	61.4	193.6	0.32	308.8	1.59	1.67
50	2.9	0.98	69.1	206.4	0.33	338.3	1.64	1.67

Points of attention are the calculated values for:

- $\frac{\sum F_{hor}}{\sum F_{ver}}$; since all calculated values are less than 0.5, no sliding will occur.
- R; since all calculated values are less than R_{max} = 1.97 m, no overturning will occur.

The crest (and the slopes) should be able to resist all mechanical attacks. In a National Park the maximum mechanical attack comes from the action of animals. The biggest animal is an elephant that weights 6.000 kg. When an elephant walks on the dam its weight is divided over two legs.
The maximum pressure on the dam is:

$$\sigma_e = \frac{F_e}{A} = \frac{m \cdot g}{\frac{1}{4} \cdot \pi \cdot D^2} = \frac{\frac{1}{2} \cdot 6{,}000 \cdot 9.81}{\frac{1}{4} \cdot \pi \cdot (15)^2} = 166.5 N / cm^2 = 1.67 N / mm^2$$

This means that concrete B25 (σ_c=25 N/mm^2) will do.

16.1.2 Foundation

The causeway should be foundated on bedrock, in order to avoid seepage under the dam and for stability reasons.

16.2 IMPERMEABLE DAM

The impermeable dam consists of a concrete wall that functions as impermeable heart of the dam. The concrete wall is vertical and is built directly on the underlying rock soil. Upstream of the dam the soil is covered with red soil (slope 1:2) and downstream with natural stones (slope 1:3). The width at the top of the construction is 1.0 meter. The height of the dam and the height of the water level upstream of the dam vary with the size of the catchment area, the values are calculated in the previous chapters and the results are shown in Table 16.3.

Table 16.3 Height of the impermeable dam.

A (km^2)	H (m)	y (m)
5	2.4	1.35
10	2.6	1.15
25	2.7	1.05
50	2.9	0.85

A cross-section of the impermeable dam is drawn in Figure 16.2.

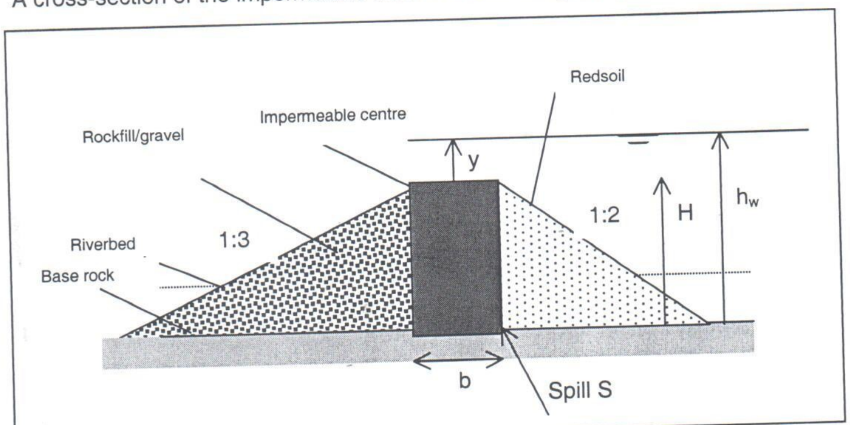


Figure 16.2 Impermeable dam.

16.2.1 Stability of the construction

Calculation:

1. determination of the horizontal forces

The total horizontal force consists of the water pressure and the ground pressure, see Figure 16.3.

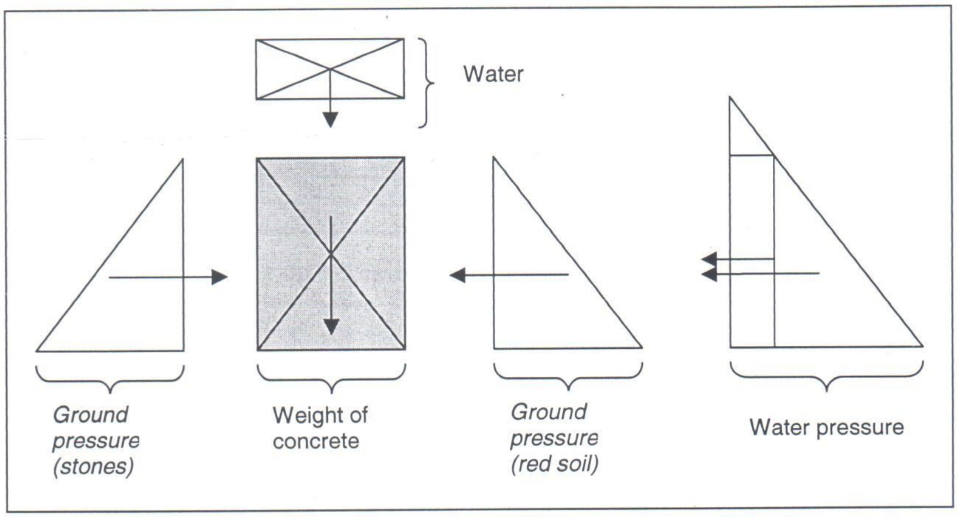


Figure 16.3 Horizontal and vertical forces on an inmpermeable dam.

Hydrostatic pressure upstream of the construction:

$$F_{hor} = P = \frac{1}{2} \cdot w \cdot \left(h_w^{\ 2} - y^2\right) \quad [kN]$$

With: w = unit weight of water = 9.8 kN/m^3
h_w = water level upstream the construction [m]
y = water level above the crest [m]

Earth pressure as a result of the natural stones:

$$F = \frac{1}{2} \cdot K \cdot \gamma \cdot H^2 \quad [kN]$$

K = coefficient of earth pressure
K_a = coefficient of active earth pressure = 1/3
K_p = coefficient of passive earth pressure = 3
K_n = coefficient of neutral earth pressure = 1
γ = unit weight of the soil [kN/m^3]
γ_s = unit weight of stones = 20 kN/m^3
γ_{rs} = unit weight of red soil = 15 kN/m^3
H = height of the dam [m]

In this case neutral ground pressure will be used. If the calculation with neutral ground pressure fits the conditions of sliding and overturning, the system of passive and active ground pressure will not be activated.

Because the slopes of the natural stones and the red soil are not horizontal, a reduction of the ground pressure occurs. An estimation of this reduction is made in Figure 16.4.

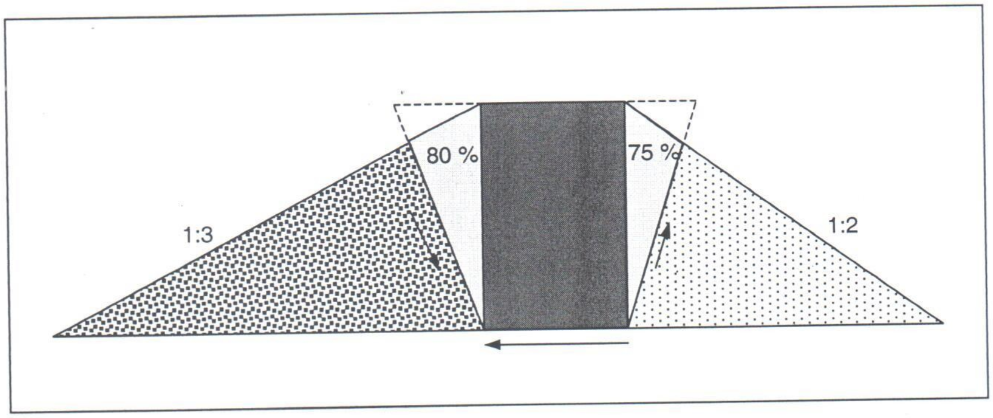


Figure 16.4 Reduction of ground pressure.

2. determination of the vertical forces

ΣF_{vert} consists of the weight of the construction and the weight of the water above the crest, see

Figure 16.3. Weight of the construction and water:

$F_{ver} = \sum \gamma \cdot V$ [kN]

Where: γ = unit weight [kN/m^3]

γ_c = unit weight of concrete = 26.0 kN/m^3

V = volume in [m^3]

3. sliding

Sliding will not occur if:

$$\frac{\sum F_{hor}}{\sum F_{ver}} \leq 0.5$$

4. determination of the moment which is a result of the vertical and horizontal forces:

$\sum M = \sum F \cdot a$

Where: ΣM = total moment [kNm]

a = length between force and spill 'S' [m]

The location of spill 'S' is shown in Figure 16.2.

5. determination of R

$R \cdot \sum F_{ver} = \sum M$

Where: R = maximum distance to spill 'S' [m]

ΣF_{ver} = total vertical forces [kN]

ΣM = total moment [kNm]

6. overturning

Overturning will not occur if R ≤ 2/3 *b

Where: b = width of the construction [m]

Since b = 1.0 m for every catchment area, R_{max} = 2/3 *1.0 = 0.67 m.

The results of the calculation are shown in Table 16.4.

Table 16.4 Stability of the impermeable dam.

A (km^2)	H (m)	y (m)	F_{hor} (kN/m)	F_{ver} (kN/m)	$\frac{\sum F_{hor}}{\sum F_{ver}}$	ΣM (kNm/m)	R (m)	R_{max} (m)
5	2.4	1.35	40.9	75.6	0.54	49.0	0.65	0.67
10	2.6	1.15	45.5	78.9	0.58	52.0	0.66	0.67
25	2.7	1.05	45.3	80.5	0.56	53.3	0.66	0.67
50	2.9	0.85	44.3	83.7	0.53	54.5	0.65	0.67

Points of attention are the calculated values for:

- $\frac{\sum F_{hor}}{\sum F_{ver}}$; all calculated values exceed 0.5 a little, but these calculations have been made with a coefficient for neutral ground pressure. In case of active and passive ground pressure $\frac{\sum F_{hor}}{\sum F_{ver}}$ will be negative, so no problem will occur.
- R; since all calculated values are less than R_{max} = 0.67 m, no overturning will occur.

The crest (and the slopes) should be able to resist all mechanical attacks. In a National Park the (maximum) mechanical attack comes from the action of animals. The biggest animal is an elephant, which weights 6.000 kg. When an elephant walks on the dam its weight is divided over two legs.
The maximum pressure on the dam is:

$$\sigma_e = \frac{F_e}{A} = \frac{m \cdot g}{\frac{1}{4} \cdot \pi \cdot D^2} = \frac{\frac{1}{2} \cdot 6{,}000 \cdot 9.81}{\frac{1}{4} \cdot \pi \cdot (15)^2} = 166.5 N/cm^2 = 1.67 N/mm^2$$

This means that concrete B25 (σ_c=25 N/mm^2) will do.

16.2.2 Foundation

The impermeable dam should be foundated on bedrock, in order to avoid seepage under the dam and for stability reasons.

17 COSTS

17.1 GENERAL

In this chapter a rough calculation of the costs has been made. The used prices of materials and labour are found in 'Water for wildlife I' and are shown in Table 17.1. The prices for the materials include costs for the labour. The price for concrete includes the price for the construction wood and in the price of the red soil, the coverage of grass is included.

Table 17.1 Costs.

Material	Unit price	
Concrete	2,750/=	Ksh/m^3
Natural stones	575/=	Ksh/m^3
Red soil	350/=	Ksh/m^3
Wire mesh	339/=	Ksh/m^2

17.2 CAUSEWAY

Construction

The causeway consists of two concrete walls and a concrete top, filled with rockfill. In the next tables a calculation of the costs is made.

Causeway: A= 5 km2 L = 20 m.

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	54 m^3	2750/=	148,500/=
Rockfill	66 m^3	575/=	37,950/=
Total:			**186,450/=**

Causeway A= 10 km^2 L = 25 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	73 m^3	2750/=	200,750/=
Rockfill	90 m^3	575/=	51,750/=
Total:			**252,500/=**

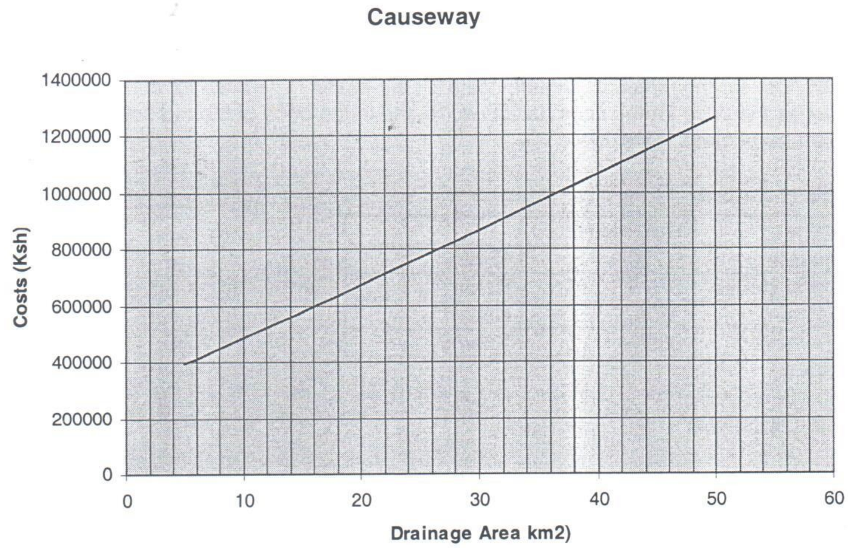
Causeway A= 25 km^2 L = 35 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	105 m^3	2750/=	288,750/=
Rockfill	131 m^3	575/=	75,325/=
Total:			**364,075/=**

Causeway A= 50 km^2 L = 50 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	160 m^3	2750/=	440,000/=
Rockfill	203 m^3	575/=	116,725/=
Total:			**556,725/=**

In the figure below the costs are displayed against the size of different catchment areas.



Connection with the embankment
The causeway has to connect with the road on the embankment. In general this embankment will consist of rock and the concrete will be built directly to the rock. No extra costs have to be made.

17.3 IMPERMEABLE DAM

Construction
The construction of the impermeable dam consists of a concrete heart, a covering of red soil upstream and a covering of natural stones downstream. On top of the red soil and natural stones, grass is planted. The dam will be covered by a wire mesh to enable the grass to connect. In the next tables a calculation of the costs is made.

Impermeable dam A= 5 km^2 L = 20 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	48 m^3	2750/=	132,000/=
Red soil	115 m^3	350/=	40,250/=
Natural stones	173 m^3	575/=	99,475/=
Wire mesh	360 m^2	339/=	122,040/=
Total:			**393,765/=**

Impermeable dam A= 10 km^2 L = 25 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	65 m^3	2750/=	178,750/=
Red soil	169 m^3	350/=	59,150/=
Natural stones	154 m^3	575/=	88,550/=
Wire mesh	475 m^2	339/=	161,025/=
Total:			**487,475/=**

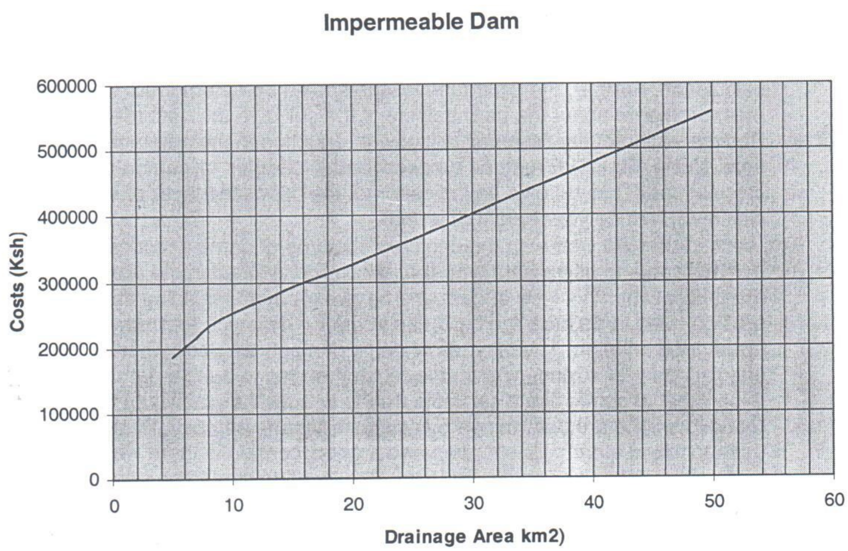
Impermeable dam A= 25 km^2 L = 35 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	95 m^3	2750/=	264,250/=
Red soil	155 m^3	350/=	54,250/=
Natural stones	383 m^3	575/=	220,225/=
Wire mesh	700 m^2	339/=	237,300/=
Total:			**773,025/=**

Impermeable dam A= 50 km^2 L = 50 m

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	145 m^3	2750/=	398,750/=
Red soil	421 m^3	350/=	147,350/=
Natural stones	631 m^3	575/=	362,825/=
Wire mesh	1050 m^2	339/=	355,950/=
Total:			**1,264,875/=**

In the figure below the costs are displayed against the size of different catchment areas.



Connection with the embankment
In case of an embankment of rock, no special efforts for the connection have to be made. In case of an embankment of (red) soil, anchors will be constructed.
The size of the anchors is the same for each catchment area.

Material	Amount	Unit price (Ksh)	Price (Ksh)
Concrete	20 m^3	2750/=	55,000/=

18 CONCLUSIONS AND RECOMMENDATIONS

18.1 CONCLUSIONS

Making dams in Tsavo East National Park is a good way to provide water for wildlife. As is concluded in chapter 14, the best way to implement a drift is to make more than one dam in a seasonal drift. Good possible locations have bedrock close under or on the surface and are easily accessible for humans, equipment and materials.

The general approach to design dams in small seasonal drift is as follows:

- First, a hydrology study must be done.
  - Rainfall data should be collected over a certain period of time. It is advisable to have at least 20 years of data to make a good estimation of the design precipitation. There is also need for rainfall intensities and the duration of showers. In arid or semi-arid areas it is also needed to determine the length of the design dry period.
  - To determine the design discharges the different floodmodels are used. The results of the floodmodels should be used carefully, because it is a systemisation. Measurements of discharges in seasonal drifts can verify the results. The Rational method, the Orstom method and the curve number methods are useable for these areas and recommended to use.
  - To determine the losses in the reservoirs a calculation on the open water evaporation is needed. The Penman formula is recommended for this calculation. Further, the infiltration losses should be determined. Local measurements about the permeability of the soil give the best results.
  - Sedimentation of the reservoir will give a reduction in the reservoir volume in time. Using the USLE method as described in chapter 11 can do the calculations. This will give an indication of the deposited layer of sediment, which will reduce your height of a dam.
- The second part will deal with the choice of the type of dams and locations.
  - It is advisable to use a dam type that is commonly used in the area. People are familiar with this type of dam and have experience building them. If erosion rates in an area are high, like in the Kitui district, groundwater dams are a good solution. These types of dams can only be used when there is no special need for surface water. It is recommended to build a dam with an impermeable centre like a concrete wall to ensure the stability of the dam.
  - Good locations are determined by accessibility and geology. Rock is a solid base to found a dam construction and a good location is there where the bedrock is close to or on the surface. The location should be easy to access to lower the costs.
- The design of dams can be done as follows.
  - First the minimum height of a dam must be determined. The most important factors are evaporation losses, the infiltration losses and the deposited layer of silt. The sum of these three factors determines the minimum design height of the dam.
  - The dimensions of the concrete centre are determined by overturning and bearing checks.
  - The dimensions of the 'spillway' can be determined from the design discharges. For small dams, like those in Tsavo East National Park, do not have a special spillway but during floods the whole dam acts as a spillway. The costs are to high to construct a separate spillway.
  - A stillin basin is needed to dissipate the energy of the water at the toe of the dam. If the dam location contains bedrock on the surface a stillin basin is not needed.

- A proper connection is needed between the dam and the riverbank. This is to make sure that backwash will not occur.

18.2 RECOMMENDATIONS

Further research of the following aspects will provide essential information for related projects in the region:

- Measuring the duration of extreme rainfall events.
- Measuring rainfall intensities during extreme rainfall events.
- Measuring the waterlevels of the reservoirs during the dry periods. This is to make a better estimation of the infiltration seepage and evaporation losses in the reservoirs.
- Make records of the constructed dams. Record quantities of used materials and equipment, number of man-hours of work and number of truckloads, unit prices of the used materials and equipment, etc. This is to make a better estimation of the cost of new dams.
- Design a set of groundwater measuring stations to determine the long-term effect of the dams on the groundwater.
- Measuring discharges in present dams during rainfall to check the calculations made in the hydrology study.

PART III: FEASIBILITY STUDY RIVER CROSSING

19 RIVER CROSSING

19.1 INTRODUCTION

This chapter "River crossing" shall deal with solutions and possibilities concerning a permanent connection between the southern part of Tsavo East and the northern part of Tsavo East. A river crossing is desirable for tourist development and wildlife safety. A bridge or tunnel will open up the northern part of Tsavo East with its unique Yatta plateau and its undamaged nature and wildlife.
There's been working on five alternatives. All these alternatives will be weight up the pros and cons, and ultimately conclusions will be drawn. Special attention to the different circumstances in comparison with the circumstances in Europe and the lack of money will get their place in the criteria analyses.
While producing these five alternatives, supplementary wishes and demands of the employer are taken into account. The employer preferred a half-open tunnel above a bridge because of its enormous tourist attraction possibilities. Most attention will be brought to the tunnel and bridge concept.

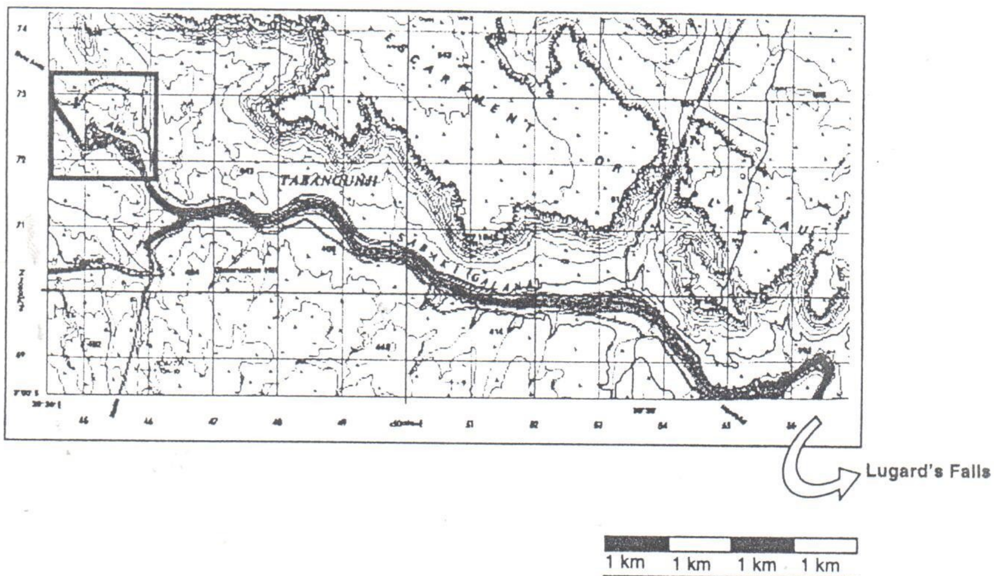


Figure 19.1 PW Area Location Ndatani(184/3) published by Survey of Kenya, Kenya Government 1991

19.2 PROBLEMANALYSIS

Tsavo park is split by the Athi River. While the southern part is developed and easy accessible, the northern part is undeveloped and hard to enter.
At this moment there is no permanent crossing over Athi River, which splits Tsavo East National Park, inside the borders of the park. Until 1997 a bridge in Kibwezi, just north of Tsavo East could be used, but it has been destroyed by El Nino Today the only way of travelling to this area is by bridges way outside Tsavo (there is one in Machakos about 300 kilometres north of Tsavo East).
The only way of crossing is an Irish Bridge, but it can only be used when the waterlevel is low and the brigade is not over flooded. In practice the crossing is inaccessible during the rain seasons.
Consequences of not being able to cross the Athi River are influencing a whole range of factors
First of all tourism development is not possible in the northern part. With its Yatta plateau, the beautiful vegetation and the swamp a big opportunity to attract tourists remains unused. As mentioned in part one tourist are very important by bringing in park fees.
Secondly not only tourists are not able to cross; also the KWS is not able to cross this river at all times. While most ranger-camps are situated in on the southern side protecting and surveying of the northern side is hard during rain season. Trucks and jeeps are not able to pass the river. As a consequence poachers are almost free move and kill.
The solutions for these problems should be a river crossing inside the borders of Tsavo East. The crossing should provide access to the northern part of Tsavo East at all times. A boundary condition is that the construction is not too expensive. An ideal concept should payback itself may even turn into a money-maker. Charging toll for tourist can do this. This is important because the KWS is gets no money from government and sporadic funds from extern investors. Another important conditionis the simplicity of the construction. Local people can work on simple constructions, complex constructions need to be tendered to a contractor, which makes it more expensive, but also more reliable.

19.3 PROBLEM DEFINITION

There is no permanent crossing over Athi/Galana River. This means that the northern part of Tsavo East N.P. is unreachable during the rain season. As a consequent of this the northern part of Tsavo east isn't attractive for tourism. Also pouching is hard to control without a descent river crossing to the North

19.4 OBJECTIVE

The objective can be divided in three parts:

- To define the physical and organisational boundary conditions of a river crossing.
- To make an identification of possible solutions and their problem areas.
- To consider each alternative and value it on its strong and weak points

Special attention will go to a bridge and a tunnel alternative

19.5 SITUATION DEFINITION

19.5.1 Location

Several site visits have been made to Athi/Galana river. During these visits two areas have been selected for a possible crossing. The selection is made with the help of maps and the actual visual view. There has been a lot of discussion with rangers and other experts on the country site. Every concept requires special circumstances. So every spot that has been selected is only suitable for one or two concepts.

Area around P.W. point
The first location is the area between point 1450 falls and P.W. point 10 km north of Athi/Tsavo junction. The river flows through a narrow channel. See figure. A waterfall upstream borders this area and PW point downstream The distance between these two points is about 1,7 km. This location is selected because the distance between the two banks belongs to the smallest along the river.

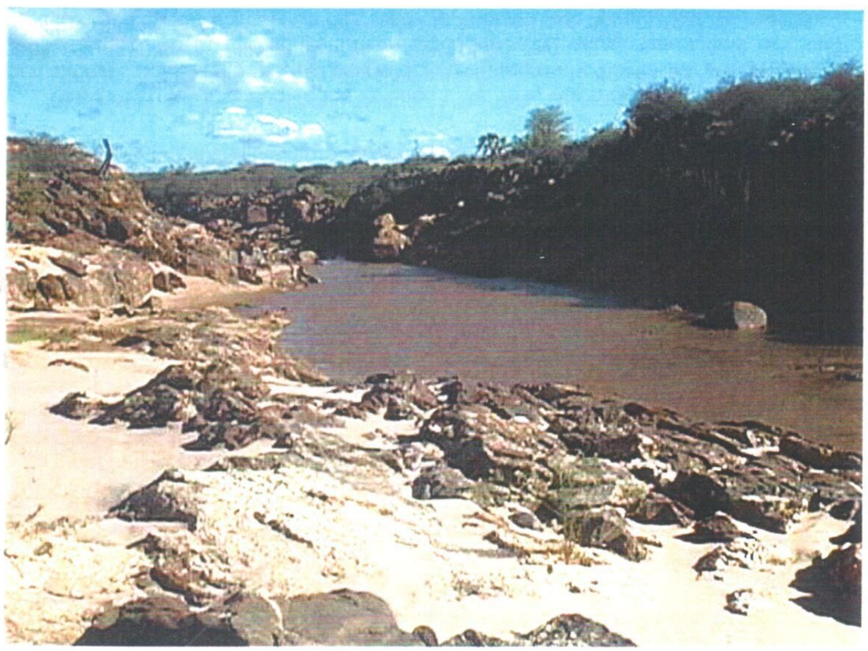


Figure 19.1 Gorge looking upstream.

At P.W. point, named after the employer, Peter Westerveld, the river divides itself into three channels. This point was selected by the employer as the location for a half open tunnel. The beautiful site and steep water drop of +/- 4 metres makes this an good location. A small tunnel at that point will not obstruct the water flow very much. Two hundred metres below PW point there's another water drop van +/- 4 metres. Hippos, crocodiles and other animals heavily inhabit the space between these two drops.

Figure 19.2 PW point looking downstream, just before waterdrop.

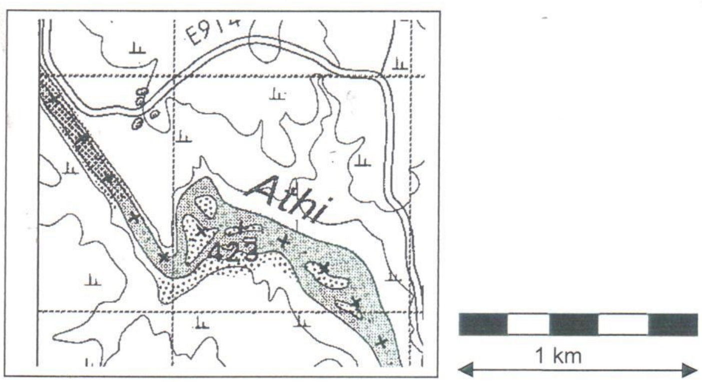


Figure 19.3 PW Area Location Ndatani(184/3) published by Survey of Kenya, Kenya Government 1991

Lugard's Falls
The second location surveyed is 35 km more downstream from the gorge, at Lugard's Falls. At this location there's an already existing crossing, an Irish bridge. The river slope is very wide at this point. The river flows from a channel into a wide landscape. As a result of that, the water level is not as high and velocities are much lower. After this it drops several meters in a waterfall downstream, which is a very spectacular view. At this location, the improved Irish Bridge and the pontoon bridge are projected.

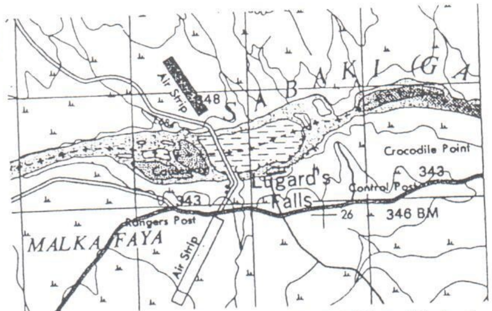


Figure 19.4 Lugard's falls, Sobo (190/2), published by Survey of Kenya,

19.5.2 Parties involved in regard to the river crossing

Westerveld Conservation Trust
Peter Westerveld, living in Holland, Nairobi and Tsavo East, is working for the Westerveld Conservation Trust on wildlife conservation. They developed the philosophy of water conservation. By building small dams in seasonal drifts, small water reservoirs are formed that supply water during the whole year for wildlife. The WCT contacted the TU Delft in the first instance to deliver a decent technical fundament for the design of these small dams. This project they also asked to make a study on a river crossing. Up till now 9 dams have been built by the WCT in co-operation with the KWS. The initiative however is still coming from the WCT.

Delft University of Technology
Five students who make a report that deals with small dams and a river crossing represent TU Delft. They are doing this as a part of their study activities. Work that they deliver should meet the standards of all party's involved with their project. At the end the report can be used as document to attract sponsors and give a better insight on the technical aspects of wildlife conservation

Kenya Wildlife Service
The KWS is the leading organisation in this project. They have to approve everything that is going to be built in the park. They also have to deliver material and know-how to execute a project this size. Eventually they have to be the employer in this project. The KWS has to tender the project if it's going to be big, and the KWS should do the biggest part of the organisation. Money problems, which is are enormous, have to be solved.

Kenyan Government
A big project like a river crossing needs permission by government, which can be a problem when the crossing interferes with the natural flow of the river. Farmers upstream and downstream will protest. To inform these farmers properly is essential for permission by government

19.5.3 Economics

The amount of money that is needed for a project this size, is can not be paid by the KWS because there is no money available. Only an extern investor can finance the whole project. Japanese and English government, the World Bank and IMF are all possible investors. A proper plan is very important to attract investors.
The crossing can make money if the KWS charges toll to tourists. This can make the crossing a interesting investment.

19.6 BOUNDARY CONDITIONS

Economical / financial

- The construction should be as cheap as possible
- The life expectancy of the crossing should be at least 50 years

Technical / physical

- No advanced materials should be used. Skilled workers are rare
- Complex materials and machines are not available
- Used materials be available
- The construction should meet the requirements of cohesion, stability, strongness and stiffness
- The construction should be El Nino proof
- Sedimentation and erosion should not change significantly after during and after the construction
- Maintenance of the construction should be minimal (because of a lack of money and skilled people

Environmental / functional

- Disturb of vegetation and wildlife should be minimised
- The construction of the crossing and access roads may not cause loss of big trees (rare in Tsavo East)
- The availability of the construction should be almost 100 % (in other words, it should always be possible to use the crossing except during El Nino

Social

- Most workers are unskilled
- During the construction accommodation (for example tents) must be arranged for the workers
- Safety guards must guarantee the safety of the workers

Organisation

- Construction planning should deal with the fact that it might be impossible to work during rain season (march-april and november)
- Availability of materials, fuel etc. should be considered in planning
- Design of the construction must fit in to national policies/regulations (interference in Athi is of national concern)

19.7 ASSUMPTIONS

Economical/financial

- Investors have to be found to finance the crossing
- Investors can only get refunded their investments by crossing fees

Technical/physical

- Missing data will be replaced by reasonable estimations

Environment/functional
- There are no other circumstances worse then El Nino

Organisation
- The number of unskilled workers is unlimited
- The project organisation is not corrupt
- The project organisation is capable to manage the project

20 PARAMETERS OF INTEREST / KEY NUMBERS FOR DESIGN: WATER RELATED

20.1 INTRODUCTION

This chapter deals with the water-related conditions of a crossing. As mentioned before, two locations has been selected, one for bridge and tunnel and one for the other solutions. Water behaviour will be described for both locations. At Lugards falls a gauging stations is situated. This station gives a lot of relevant date that will be used. Values for P.W. point are extracted with the comparison of data form three other stations, cross-sections, and longitudinal sections.
Paragraph 2.2 gives a general description of Athi river. Maps of both locations are given in 2.3. In 2.4 a discharge analysis has been done. 2.5 deals with the river parameters: the Strickler coefficient, altitude, gradient and cross-sections in the P.W. area. Finally water and energy levels are calculated in 2.6.

20.2 PURPOSE OF THIS CHAPTER

The main purpose of describing the river behaviour is to make a good estimation of discharge the water levels and velocity. Therefore discharges are defined for a maximum of once in fifty years, an average discharge and a non-exceedance of 95%. $Q_{max\,(t=50)}$ is useful while a construction should be able to resist maximum forces with a return period of 50 years. $Q_{average}$ can be used to get an idea about mean annual discharges and the amounts of water passing by a construction. $Q_{95\%}$ is calculated in order to define the conditions that has to be faced when a construction should be available for 95% of the time.
For water levels and velocity much of the same reasons count. Water levels are calculated in order make clear which circumstances occur in times of high floods.

20.3 DISCRIPTION OF RIVER

- Athi/Galana river almost lies at the equator. This means that the area two rainseasons per year. One in March-April and one in November. Two rain seasons mean two peak discharges a year
- Athi river is a rain river
  Tsavo River is mixed river: Partly consisting of ice water from Mount Kilimanjaro and partly filled with rainwater
  So Galana/Sabati river is a mixed river. However mainly consisting of rainwater
- Athi/Galana river is a meandering river

20.4 MAP OF LOCATIONS

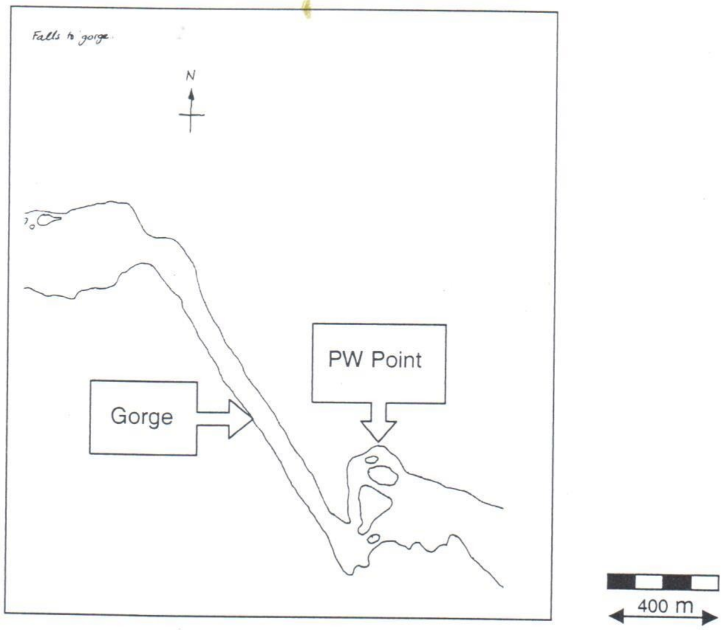


Figure 20.1 P.W. Area.

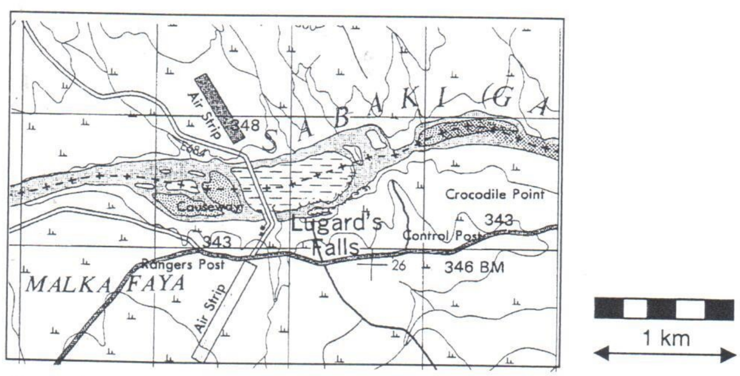


Figure 20.2 Lugard falls

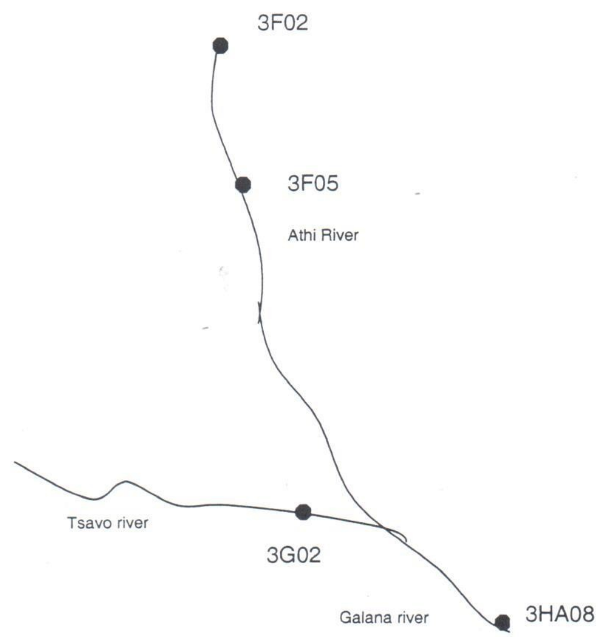
20.5 DISCHARGE

20.5.1 General

In order to calculate a design discharge data of four different gauge stations are used.
These are:

Table 20.1 gauging stations

Code	Location	Distance	Data available	
			Discharge	Water-level (daily)
3F02	Mavindini	0 km	1952-1991, 1994 and 1995	1952-1991,1994-1996
3F05	Kailenbwa	26 km	1969-1979	1967-1979
3G2	Tsavo River	-	1949-1983, 1986,1991	1949-1983,1986, 1991
3HA08	Lugard's Falls	97 km	1973-1987	1973-1986



Data are obtained from the Ministry of Water Development (Maji House, Kenya) and through the following reports: 'The study of the national water master plan", Dec 1991 and 'Main report of Malindi pipeline project' 1989.
Discharge
Normative discharges has to be found for the two locations, Lugard's Falls and P.W. point. As mentioned before a gauge-station can be found at Lugard's Falls (3HA08). To find discharges and water levels in P.W.-gorge estimations can be made with help of the three other stations.

20.5.2 Methods to find discharge

The following methods are used to find maximum, average and minimum discharge:

Maximum

Gumbel
A Gumbel-graph is used to find return periods for discharges. Data are obtained from Ministry of Water Development (Maji House, Kenya). As shown in the table above, data are available for at least 10 years for every station. Although the number of data at some stations can be called limeted, an estimation is made.

Water master-plan formula
The other way of calculating Q_{max} is done with an empirical formula, defined in the 'Study of the National water master-plan'
Q is calculated for the Mean Annual Flood (MAF). A multiplier is used to determine Q for a return period of 10 and 50 year. Q depends on the catchment area and two factors. These factors are empirical determined for each river. For Athi river this formula is:

$Q = 0.249A^{0.835}$

The multipliers for 10 and 50 years are

Q_{10}	2.62
Q_{50}	5.29

The cachtment area is given for two stations in the 'Study of the National water masterplan', for 3F02 and 3HA08. In order to define the catchment area for the other stations and P.W. point an extrapolation is done. The catchment area is assumepted to be linear with the distance. In other words, the catchment area increases per kilometer downstream.
This can be put in the following formula

$A = 10{,}272 + 153.92{*}\Delta x$
With Δx = distance from 3F02 downstream
In the table below the catchment area's are given

Station	Distance	Catchment area (A in km^2)
3F02	0	10,272
3F05	26	14,274
P.W. point	60	19,507
3HA08	97	25,203

As a result Q is determined twice for every station. These results can be compared and a estimation can be made.

Average and 95% of non-exceedance

Figure of Study of the national water master plan

The average discharge is also calculated in two ways. 'The study of the national water master plan' shows a figure of annual mean discharge along Athi river. (see appendix 16).

Q-h curve and probability of non-exceedance

The other way of calculating Q is by making a Q-h curve for all gauge stations and P.W. point. The Q-h curves for the gauge stations are made with data available about Q and h at the same time. The Q-h curve at P.W. point is made with help of the defined cross-sections. The calculations can be found in the appendices.
Beside this Q-h curve daily water-levels are available for the gauge-stations. These data are used to find the average h and a probability of non-exceedance graph has been made. Values are available for about 10.000 days per station. These values are extracted in the Q-h curve in order to find $Q_{average}$ and $Q_{95\%}$

20.5.3 Q-h curves for four stations

Q-h curve are made with the data and methods mentioned before. Maximum monthly discharges and water-levels are kwon for every month during the earlier mentioned period of time (see data appendix). These are put in a graph and this results in a Q-h curve. Notice that is has been estimated that maximum discharge and maximum water-levels occur at the same day.

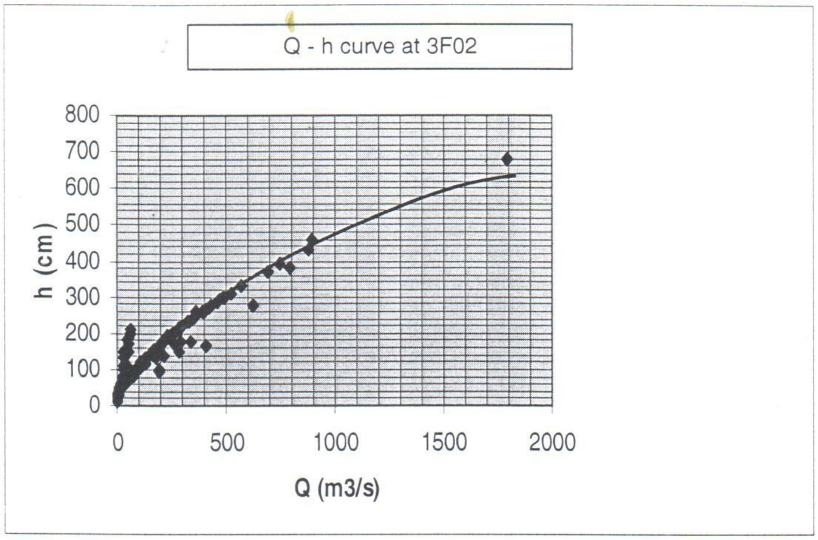


Figure 20.3 Q -h curve at 3f02

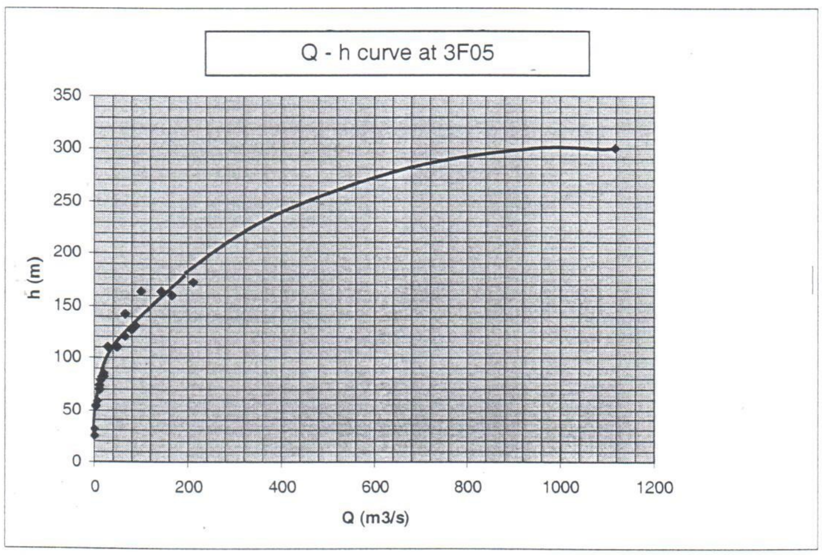


Figure 20.4 Q - h curve at 3F05

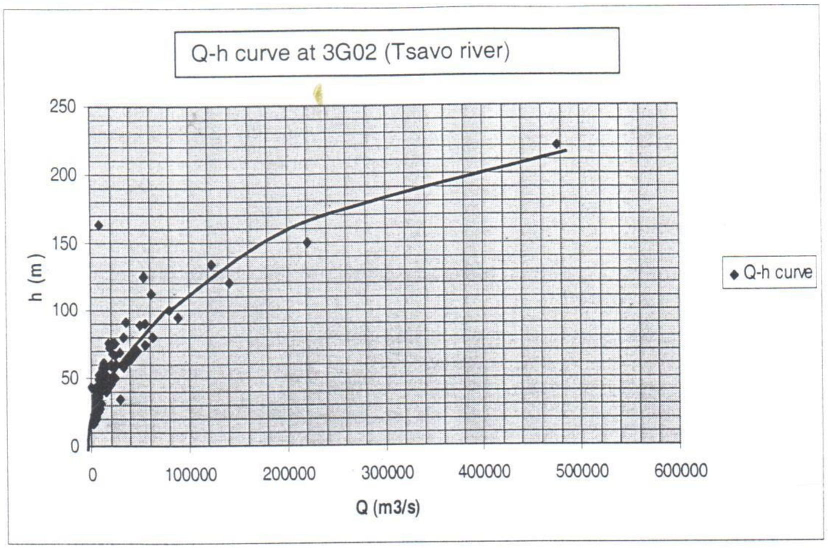


Figure 20.5 Q - h curve Tsavo river (3G02)

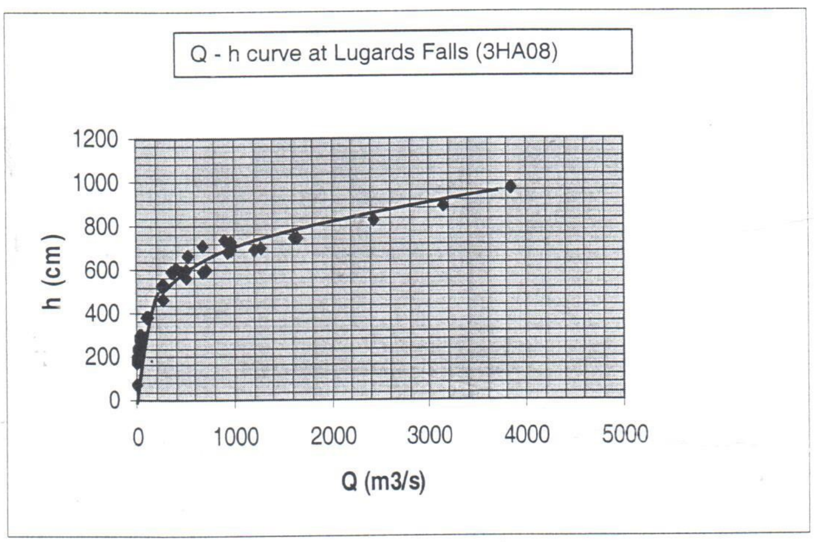


Figure 20.6 Q - h curve at Lugard's Falls (3HA08)

20.5.4 Q-h curve for P.W. point

The Q-h curves for P.W. point are calculated with the cross-sections. The cross-sections areas can be calculated for every water height. With the Strickler runoff-formula a representative discharge can be calculated. In other words:

$A = b.h$

$A = \Sigma A_{channel} + A_{forelands}$

$Q = k*R^{3/2}*i^{1/2}*A$

Q-h is calculated for the sections A-A (possible location for the bridge) and C-C (possible location for the tunnel)

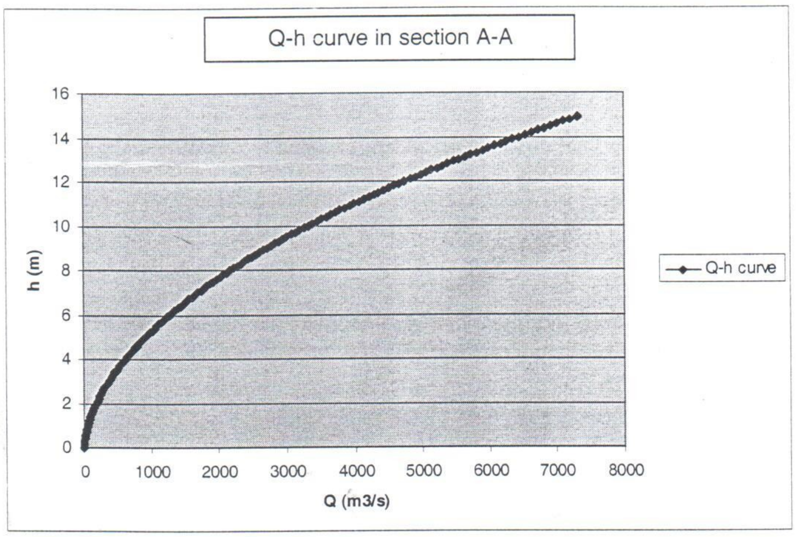


Figure 20.7 PW Point A-A

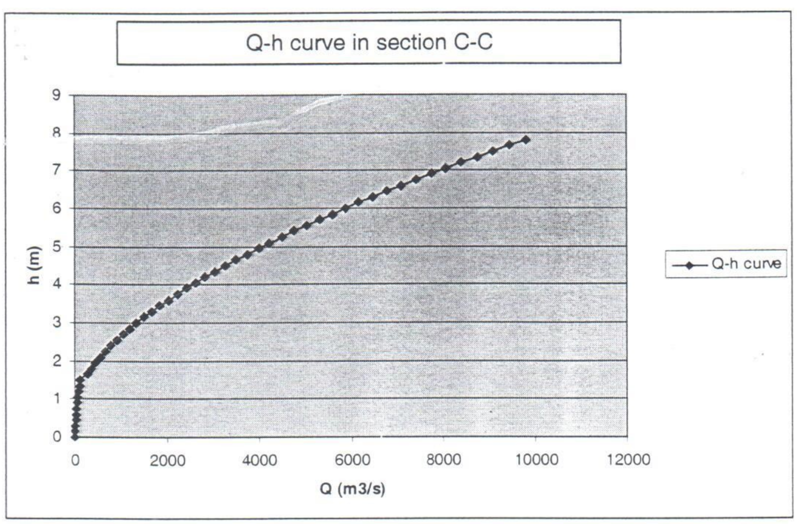


Figure 20.8 PW Point C-C

20.5.5 Probability of non-exceedance

The probability of non-exceedance graph is made with daily water-level data. These data are measured in the years mentioned in Table 20.1. In the side box relevant data for each graph are given. The line bolted line in the graph marks the 95% of non-exceedance value.

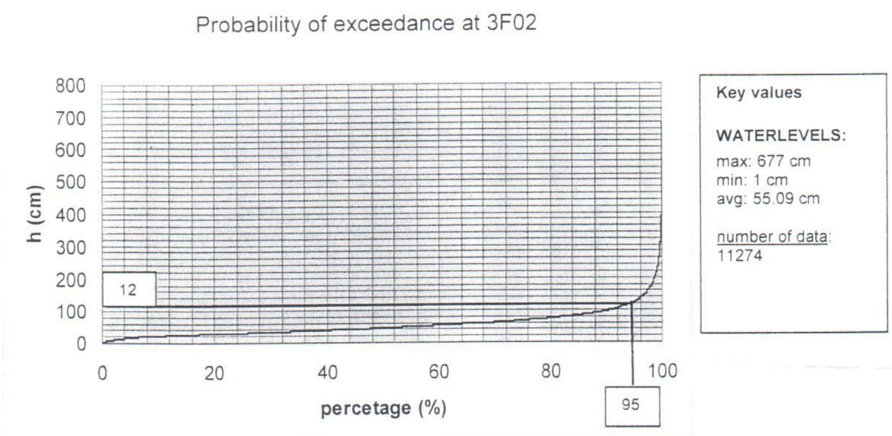


Figure 20.1 non-exeedance at 3F02

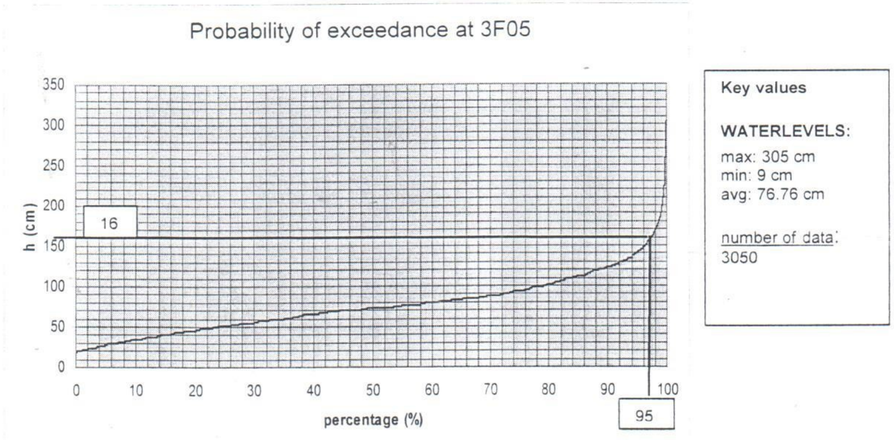


Figure 20.2 non-exeedance at 3F05

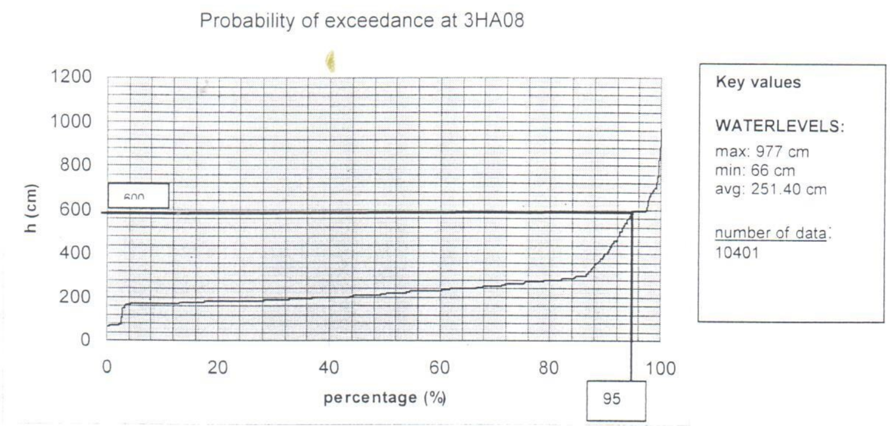


Figure 20.3 non-exeedance at 3HA08

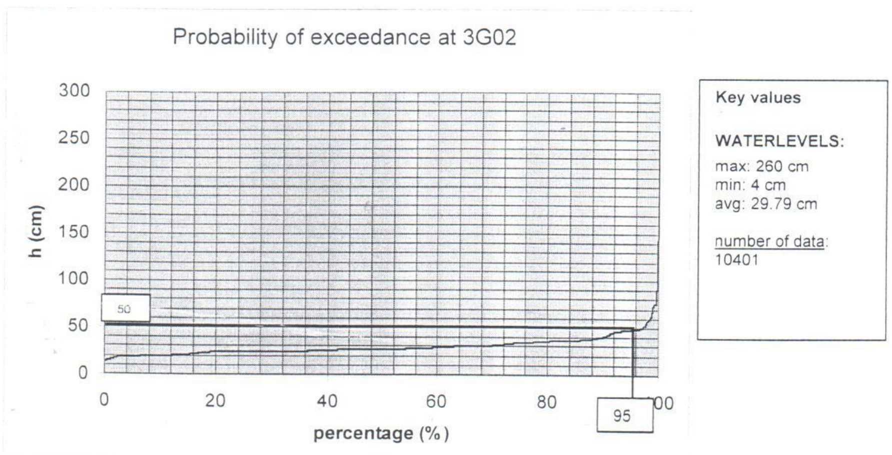


Figure 20.4 non exceedance at 3G02

20.5.6 Determination of values at P.W. point: Qavg, Q95% and

In order to design a river crossing at P.W. point key numbers for discharge has to be defined. The following considerations has been made

20.5.6.1 Maximum discharge

- While there are no discharge data available for P.W. point, a Gumbell-graph is impossible to make
- As mentioned before, the catchment area is assumed to be linear with the distance. When the results for Q_{50} from the water masterplan and the Gumbell-graph are compared, they seem to suit quite well.
- Q_{50} for P.W. point is now calculated with the 'masterplan formula' Q_{50} becomes 5034 m^3/s.
- Though Tsavo river discharges in the Athi-river just downstream of P.W.point, Q_{50} is assumed to be lower then calculated. While Q_{50} = 500 m^3/s for Athi-river, Q_{50} at P.W. is supposed to be about 4500 m^3/s (5000-500 = 4500 m^3/s)

20.5.6.2 Average discharge

- The average discharge can be defined through the 'annual-mean-discharge-figure' given in the 'water-masterplan' This gives an average of about 19 m^3/s.
- When the results values given in this figure are compared with the daily waterlevel analyses, the results for 3F02 and 3F05 are significantly higher. At Lugards Falls (3HA08) the results fit quite well.
- These facts lead to the conclusion for at P.W. point that the value of $Q_{average}$ is between 15 and 19 m^3/s. While 19 m^3/s might be to high, 15 m^3/s can be to low. For further design $Q_{average}$ = 17 m^3/s will be used.
- $Q_{95\%}$ at P.W. point is somewhere between 150 and 600 m^3/s. While P.W. is almost halfway between 3F02 and 3HA08, $Q_{95\%}$ should be about 375 m^3/s. Whit the Tsavo river fflowing in just downstream, this might be to high. For this reason $Q_{95\%}$ is said to be 350 m^3/s.

20.5.7 Table with results

The table below is a summary of the

Table 20.2 Key values for discharge in Athi river

Location	3F02	3F05	P.W. point	3G02	3HA08
Δx (km)	0	26	60	-	97
A (km^2)	10,272	14,274	19,203	-	25,203
$Q_{50,\ gumbell}$ (m^3/s)	2500	4000	-	500	6500
$Q_{50,\ master}$ (m^3/s)	2177	3878	5034	-	6200
Q (m^3/s)	2300	3900	4500	-	6300
$H_{average}$ (m)	0.5	0.77		0.29	2.51
$Q_{average}$ (m^3/s)	15	15	-	5	26
$Q_{average,\ master}$ (m^3/s)	25	24	19	-	26
$Q_{average}$ (m^3/s)	-	-	17	-	26
$h_{95\%}$ (m)	1.2	1.6	-	0.5	6
$Q_{95\%}$ (m^3/s)	110	160	350	20	650

* Data and Gumbel-graph are found in the appendices

20.6 PARAMETERS: K, I,A

20.6.1 Introduction

In order to make further calculations three parameters have to be defined. First a number cross-sections in the P.W. area are defined. Cross-section A-A models the upstream channel, B-B describes the gorge and C-C the section downstream of the gorge just upstream of the second water drop D-D models the river downstream of the section. The Strickler coefficient is defined and gradient (i)

20.6.2 Strickler coefficient (k)

The strickler coeffecient is defined in the tabel below. The higher value of k, the cleaner the slope.

Type	K
very clean slope	45-30
clean slope	35-20
little vegetation present	25-15
Moderate vegetation	20-10
Heavy vegetation	15-5

In the considered riversections, little vegetation is present. The dry season river bed, water flows through a clean slope consisting of rock and sand. k should be between 40-35.

When the water reaches his annual flood level, it meets some more friction from stones and some vegetation. But still, the profile is quite smooth. Only in the top region, trees and bushes can be found. Here k can drop till about 30.

20.6.3 Gradient (i)

The average gradient of the river slope over a length of 62,5 kilometers is $3.25*10^{-3}$. This gradient differs from section to section At Lugards falls the gradient is assumed to have this average value while gradient and altitude differences at P.W. is shown in the figures below

Global

Differences in altitude are obtained form maps Sobo (190/2), Mudanda (190/1) and Ndatani(184/3) published by Survey of Kenya, Kenya Government 1991. The scale is 1:50.000 and altitude lines are drawn every twenty meters. With this information can the gradient (I) is determinated.

Table 20.3 Altitude and gradient of Athi River in Tsavo East

Altitude	Δh (m)	Δx (m)	I ($*10^{-3}$)
420 m	20	2750	7.27
400 m			
	20	10750	1.86
380			
	20	1750	11.4
360			
	20	10750	1.86
340			
	20	2250	8.88
320			
	20	5500	3.63
300			
	20	4000	5
280			
	20	10500	1.9
260			
	20	7250	2.76
240			
	20	6000	3.33
220			
Total	**200**	**61500**	**3.25**

Local in the P.W. area

In the P.W. area a more specific profile has been made. The figure below shows gradient and water drops in this area.

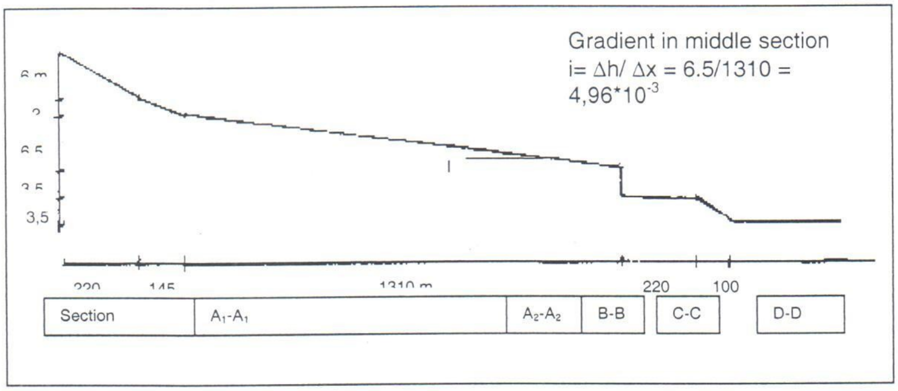


Figure 20.13 Gradient in the P.W. area

20.6.4 Schematisation of cross-sections

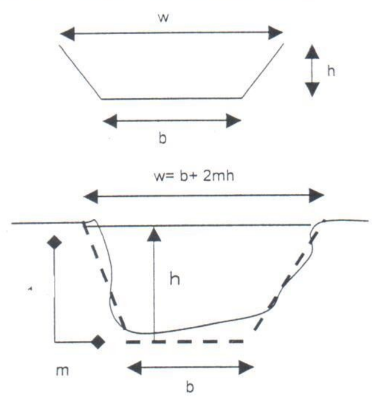
Fife cross-sections are defined in order to calculate water levels at given discharge. These cross-sections are already used in 20.5.4 Q-h curve at P.W. point. In 20.7 maximum water levels at P.W. point are calculated with this cross-sections. The formula to calculate the area and hydraulic radius are described in the theory inter-section

Formula's

Area of cross-section
$A = (b+mh)h$
With (see also figure)
b = width at the bottom
$m = (w-b)/2h$

When cross-section is not of the same form, A is build up from pieces:
$A_{tot} = A_1+A_2+A_3$ or
$A_{tot} = A_{channel} + A_{forelands}$

Hydraulic radius
$R = A/[b+2h\sqrt{(1+m^2)}$



Map with location profile's

In Figure 20.1 is shown which cross-sections are schematised.

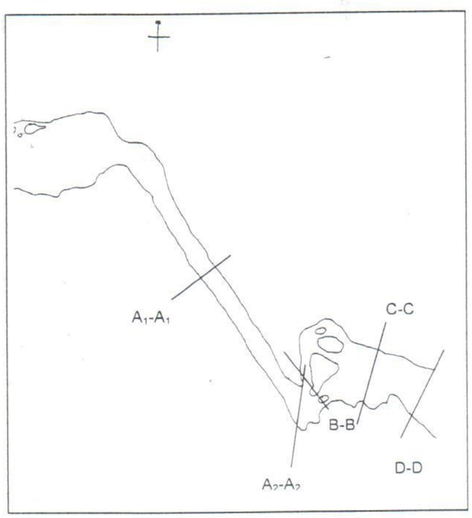
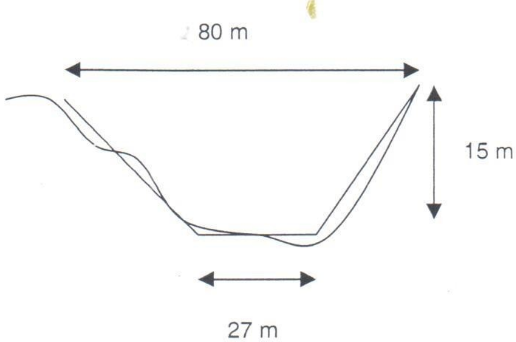


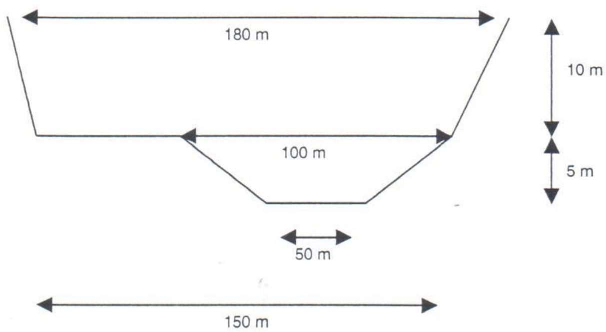
Figure 20.1 Map with locations of profiles

Schematisation of average profile in channel upstream of gorge (A_1-A_1)



$b = 27m$
$m = 80\text{-}27/2*15 = 1,76$
$A = (27+1,76*h)*h$

Schematisation of average profile in channel upstream of gorge (A_2-A_2)

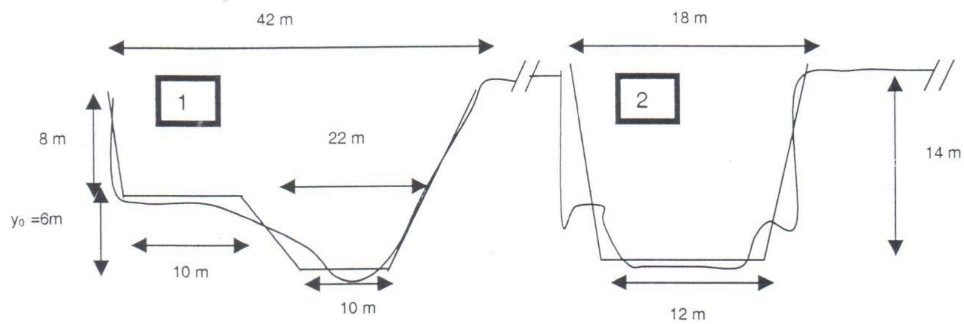


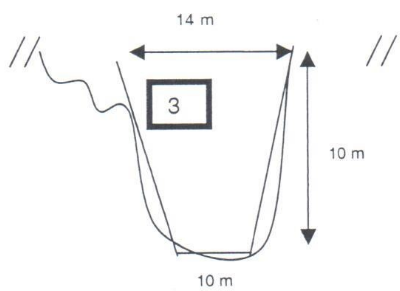
$m_{channel} = 100–50/ (2*5) = 5$
$m_{forelands} = 180\text{-}150/(2*10) = 1.5$

$A_{channel} = (b_{ch} + (m*h_0)h_0 + (b + 2mh_0)(h\text{-}h_0)$
$\quad = (50 + 5*h_0)h_0 + (50+ 2*5h_0)(h\text{-}h_0)$
$A_{forelands} = (50+1.5(h\text{-}h_0)(h\text{-}h_0)$

$$A_{tot} = 50h_0 + 5h_0^2 + 50(h\text{-}h_0)+10h_0(h\text{-}h_0) +50(h\text{-}h_0) + 1.5(h\text{-}h_0)^2$$
$$\boldsymbol{= 50h_0 + 5h_0^2 + 100(h\text{-}h_0)+10h_0(h\text{-}h_0) +1.5(h\text{-}h_0)^2}$$

Schematisation in the gorge (B-B)





1

$m_{1channel} = (22\text{-}10)/\,(2{*}6) = 1$
$m_{1forlands} = (42\text{-}32)/(2{*}8) = 0{,}652$

$$A_{1channel} = (b{+}m{*}h_0)h_0 + (b + 2mh_0)(h\text{-}h_0)$$
$$= [10{+}1{*}h_0)h_0] + [10{+}\,(2{*}1{*}h_0)(h\text{-}h_0)]$$
$$= 10h_0{+}h_0^2 + 10(h\text{-}h_0) + 2h_0(h\text{-}h_0)$$

$$A_{1forelands} = [10{+}\,0{,}652{*}(h\text{-}h_0)](h\text{-}h_0) =$$
$$= 10(h\text{-}h_0) + 0{,}652(h\text{-}h_0)^2$$

2

$m_2 = (18\text{-}12)/(2{*}14) = 0{,}214$

$$A_2 = (12{+}0.214{*}h){*}h$$
$$= 12h + 0.214h^2$$

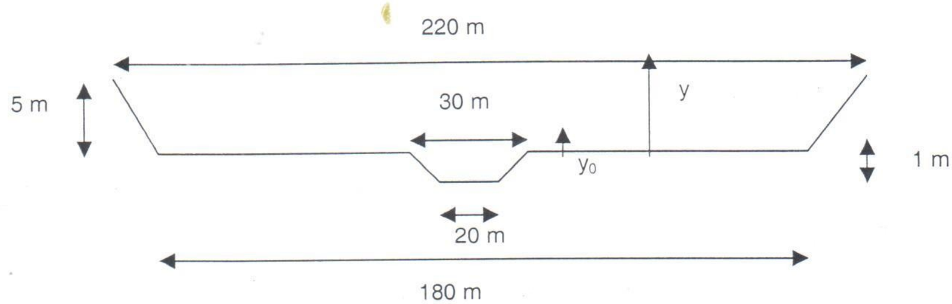
3

$m_3 = (14\text{-}10)/2{*}10 = 0{,}2$

$$A_3 = (10{+}0{,}2{*}h){*}h$$
$$= 10(h\text{-}4){+}0.2(h\text{-}4)^2 = 10h\text{-}40 + 0.2(h\text{-}4)^2$$

$$A_{tot} = (10h_0{+}\,h_0^2 + 10(h\text{-}h_0) + 2h_0(h\text{-}h_0) + 10(h\text{-}h_0) + 0{,}652(h\text{-}h_0)^2 + 12h + 0.214h^2 + 10h - 40 + 0.2(h\text{-}4)^2)$$
$$\boldsymbol{= 10h_0{+}\,h_0^2 + 20(h\text{-}h_0) + 2h_0(h\text{-}h_0) + 0{,}652(h\text{-}h_0)^2 + 22h + 0.214h^2 + 0.2(h\text{-}4)^2 - 40}$$

Schematisation of cross-section (C-C)

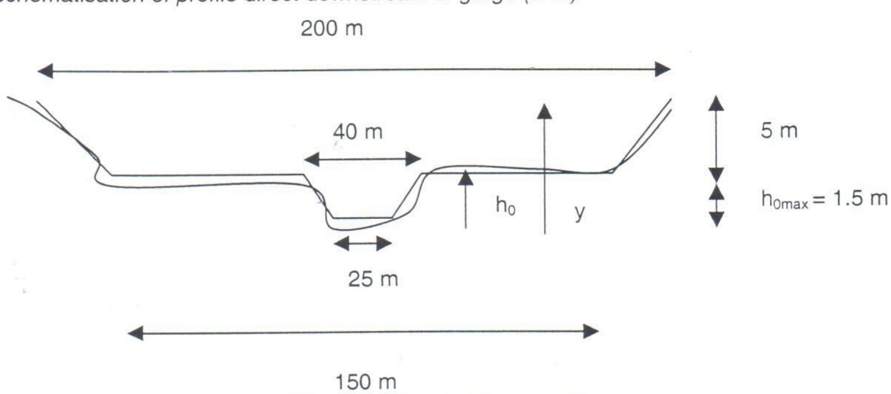


$m_{channel}$ = 30–20/ (2*1) = 5
$m_{forelands}$ = 220-180/(2*5) = 4

$A_{channel} = (b_{ch} + (m{*}h_0)h_0 + (b + 2mh_0)(h-h_0)$
$= (20 + 5{*}h_0)h_0 + (20+ 2{*}5h_0)(h-h_0)$
$A_{forelands} = (150+4(h-h_0)(h-h_0)$

$A_{tot} = 20h_0 + 5h_0^2 + 20(h-h_0)+10h_0(h-h_0) +150(h-h_0) + 4(h-h_0)^2$
$\boldsymbol{= 20h_0 + 5h_0^2 + 170(h-h_0)+10h_0(h-h_0) +4(h-h_0)^2}$

Schematisation of profile direct downstream of gorge (D-D)



$m_{channel}$ = 40 –25/ (2*1.5) = 5
$m_{forelands}$ = 200-150/(2*5) = 5

$A_{channel} = (b + m{*}h_0)h_0 + (b + 2mh_0)(h-h_0)$
$= (25 + 5{*}h_0)h_0 + (25+10h_0)(h-h_0)$

$A_{forelands} = (110 +5(h-h_0))(h-h_0)$

$A_{tot} = 25h_0 + 5h_0^2 + 25(h-h_0) + 10h_0(h-h_0) + 110(h-h_0) + 5(h-h_0)^2 =$
$\boldsymbol{= 25h_0 + 5h_0^2 + 135(h-h_0) + 10h_0(h-h_0) + 5(h-h_0)^2}$

20.7 WATERLEVELS

20.7.1 Introduction

Water levels depend on a lot of factors. The parameters defined in the previous paragraphs (roughness, cross-sections, gradient and discharge). But also water drops, narrowing, a widening play an important role. While water levels and water velocity form important boundary conditions they has to be estimated.

20.7.2 Methods to calculate water levels

In order to calculate the waterlevels in the P.W. area, as well as at Lugards Falls different methods are used. While the river flows through a wide profile at Lugards Falls, the area around P.W. point is narrow and winding. This requires other methods of calculating.
The water levels at Lugards falls are defined with the earlier mentioned Gumbel graph, Q-h curve and non-exceedance curves.
On the other hands the waterlevels in the P.W. area are estimated with the energy levels.
As shown in the appendices, the calculation is done with Excel. In this program an area calculation, energy calculation and standard step methods are combined. As a result an estimation of water levels and water velocities is done.

20.7.3 Maximum water levels at P.W. point

The calculation of the maximum waterlevels in the P.W. area is done values defined in previous paragraphs. To make the calculation clear these values are summarised.

Table 20.4 Values for P.W. area.

Q_{max}	4500 m^3/s
i (for all sections not defined different)	$3.25*10^{-3}$
$i_{channel}$	$4.96*10^{-3}$
k	30-40

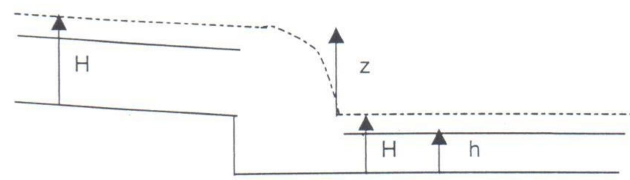
20.7.3.1 Theory

The formula's mentioned in the theory boxes below are used for calculation.

Uniform flow	
run-off formula:	$Q = k\,A\,R^{2/3}\,i^{1/2}$
Run-off formula when the cross-section is non-uniform	$Q = Q_{channel} + Q_{forelands}$
energy level	$H = u^2/2g + h$
Number of Froude	$Fr = u/\sqrt{gd}$

Non –uniform flow

run-off formula : water-level and energy-level depend on backing-up or drawing down
Downstream conditions cause backing up or drawing down (weirs, dams, waterfall etc.)
When $H_{weir} < H_{uniform} \rightarrow$ drawing down
$H_{overlaat} > H_{uniform} \rightarrow$ backing up
Depending on z, $H_{overlaat}$ can be calculated with to formula's see figure
If $z > 1/3\ H$ then weir = free flow $\rightarrow$ $Q = 1{,}8 * H^{3/2} * b$
$z < 1/3\ H$ weir = submerged $\rightarrow$ $H = u^2/2g + h + \xi u^2/2g$



Standard Step Method

In order to calculate energy and water levels, in case of backing up or drawing down the standard-step method is used (Chow 1959).
Each time verticals are considered.

Step 1
- For the whole channel section Q,i, b, m and k has to be defined.
- Starting downstream at the channel interference (in this case a weir), H can be calculated with the runoff formula, mentioned above.
- Vertical 1 is taken just upstream of the interference. Here counts that $H=H_1$
- With $H_1 = h_1 + u_1^2/2g$, $u_1 = Q/A$ and $A = (b+mh_1)h_1$
- Now u_1 and h_1 can be found by iteration
- With the numbers for u_1 and h_1 the gradient of the waterline (s) can be calculated.
- $s_1 = u^2/(k^2R^{4/3})$ with $R = [(b+mh_1)h_1]/[b+2h_1\sqrt{(1+m^2)}]$

Step 2
- Vertical to is defined on basis of the change ΔH in energy depth
- As described above, it has to be considered if backing up or drawing down takes place.
- In case of 'backing up' $H_2 = H_1 - \Delta H$
  In case of 'drawing down' $H_2 = H_1 + \Delta H$.
- $\Delta H = 0.01\ m$
- Now u_2, h_2 and s_2 can be calculated as in step 1

Step 3
- To find the distance between H_1 and H_2 the standard step equation is used:
  $\Delta x = \Delta H/[{}^1/_2(s_1 + s_2) - i]$

These steps are repeated until $H \approx H_{uniform}$

Calculation

A summary of the calculations is described below (the complete calculations can be found in the appendices):

Section D-D

Starting down stream the flow is supposed to be sub critical and uniform, so H and h are calculated with the formula for uniform flow.
The results are:

Cross-section area (A)	816 m^3
Velocity (u)	5.51 m^3/s
Number of Froude	0.8
Energy level (H)	7.50 m
Waterlevel	5.95 m

Transition D-D→ C-C

In order to see if section C-C is influenced by D-D, z is calculated.
With the free flow run-off formula $Q = 1.8\ H^{3/2}\ b$, H is calculated.
This gives for $H_{upstream}$= 6.52 meter
Now $\mathbf{z} = H_2 + \Delta h - H_1 = 6.52 + 3.5 - 7.5 =$ **2.52** meter
$z > 1/3\ H$ with $H = H_2 = 6.52 \rightarrow 1/3\ H = 2.17 < 2.52$ **so this formula is valid**

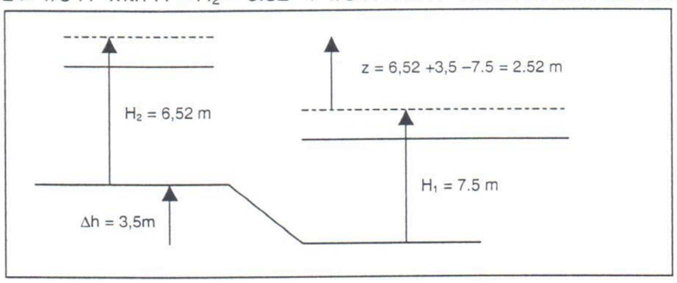


Figure 20.14 Water and Energy levels at transition D-D → C-C

Section C-C

In order to define if backing up or drawing down takes place $H_{uniform}$ for C-C is calculated.

Cross-section area (A)	863 m^3
Velocity (u)	5.21 m^3/s
Number of Froude	0.8
Energy level (H)	6.63m
Waterlevel	5.25 m

So $H_{weir} < H_{uniform} \rightarrow$ drawing down takes place
The standard step method in Excel is now used to find water and energylevels
The results are shown in Table 20.5

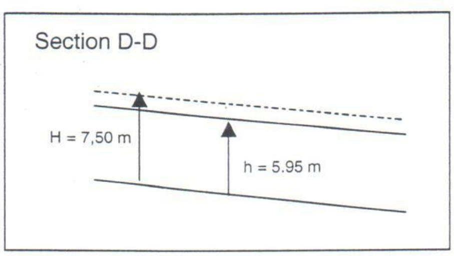


Figure 20.14 water and energy levels in D-D

Table 20.5 water and energy levels in section C-C

H	X *	U	h
6.52	0	6.3	4.497064
6.55	15.80771	5.99	4.721249
6.58	48.0066	5.5375	5.01711
6.61	179.8098	5.35	5.151157
$H_{uniform}$		$u_{uniform}$	$h_{uniform}$
6.63	220	5.21	5.254

* x = 0 at the transition of D-D and C-C and x=220 meter at the transition of C-C and B-B

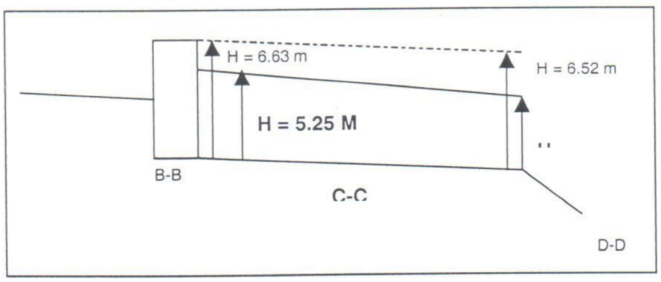


Figure 20.14 Water and Energy levels in section C-C

Transition section C-C via B-B to A_2-A_2

The free flow runoff formula with cross-section B-B is used to find the energy level. With

$Q = 1.8\ H^{3/2}\ b$ and b is estimated to be 53 meters H = 13.05 meter

This means for z

$z = H_2 + \Delta h - H_1 = 13.05 + 3{,}5 - 6.63 = 9.92$ meter

$z > 1/3\ H$ with $H = H_2 = 13.05 \rightarrow 1/3\ H = 4.35\ m < 9.92\ m$ so this formula is valid

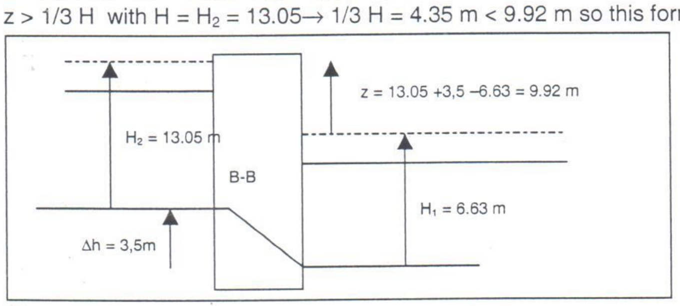


Figure 20.15 Water and energy levels at transition $C \rightarrow B \rightarrow A_2$

20.7.3.1.1 Section A_2-A_2

While water and energy levels are continuos, the levels of water and energy are supposed to be similar in B-B and just outside B-B in A_2-A_2. In other words the energy level at the downstream side of A_2-A_2 is 13.05 m

Again will be defined whether backing up or drawing down takes place.
H is supposed to be sub-critical and $H_{uniform}$ will be calculated

Cross-section area (A)	866 m^3
Velocity (u)	6.56 m^3/s
Number of Froude	0.79
Energy level (H)	9.19 m
Waterlevel	7.00 m

$H_{weir} > H_{uniform}$ so backing up takes place

This backing up is once again calculated with the standard step method.

H	X*	h
13,05	0	12,65183
13	16,94348	12,59611
12,95	33,91775	12,54266
12,9	50,92149	12,48775
12,85	67,95863	12,43281
12,8	85,02987	12,37783
12,75	102,1366	12,32254
12,7	119,2811	12,26691

* x=0 at the transition $B \rightarrow A_2$ and x=120 at the transition $A_2 \rightarrow A_1$

20.7.3.1.2 Transition A_2-$A_2 \rightarrow A_1$-A_1

A calculation (which can be found in the appendix) shows that the transition between A_2 and A_1 is sub merged.
The energy loss is now supposed to be dual.

- First energy is lost while while the river makes a sharp curve. In summary bend losses are calculated with $\Delta H = \xi * u^2/2g$ with $\xi = 0.21$ (see appendix)
- Energy loses on the weir in case of an submerged flow can be calculated with the equation of Carnot (Vloeistofmechanica page 128, Battjes) $\Delta H = (u_1 - u_2)^2/2g$ with
  u_1 = velocity of water upstream (A_2-A_2)
  u_2 = velocity of water downstream (A_1-A_1)

Calculation of the total loss gives
$\Delta H = 021 u_2^2/2g + (u_1 - u_2)^2/2g$
with
$u_1 = u_{uniform\ A\text{-}A} = 8.08$ m/s
$u_2 = 2.91$ m (calculated with standard step method
$\Delta H = 0.69 + 1.34 = 2.03$ m
this means for H upstream of the transition:
$H_{transition} = 12.7 + 2.03 = 14.73$ meter

H_{niform} in $A_1 - A_1$ is

Cross-section area (A)	557 m^3
Velocity (u)	8.08 m^3/s
Number of Froude	0.75
Energy level (H)	15.07m
Waterlevel	11.7 m

$H_{transition} < H_{uniform}$ so drawing down takes place

The water levels can now be calculated with the standard step method.

H	X	U	h
14.73	0	8 5	11.04753
14.83	133.7383	8.4	11.23367
14.93	365.1942	8.2	11.50288
15.03	1247.069	8.08	11.7

Summary
To make this calculation visual a summary is made in Figure 20.1

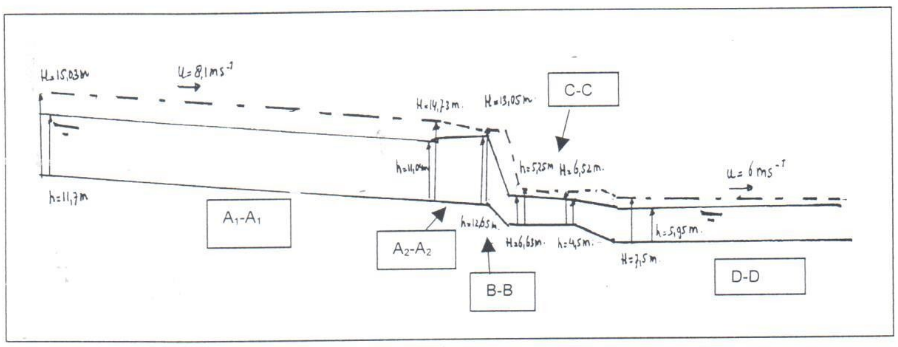


Figure 20.1 Water and Energy levels in the PW area

20.7.4 Average and Q95% discharge

Water levels for average and $Q_{95\%}$ are defined with the Q-h curves at cross-sections C-C and A_1-A_1.

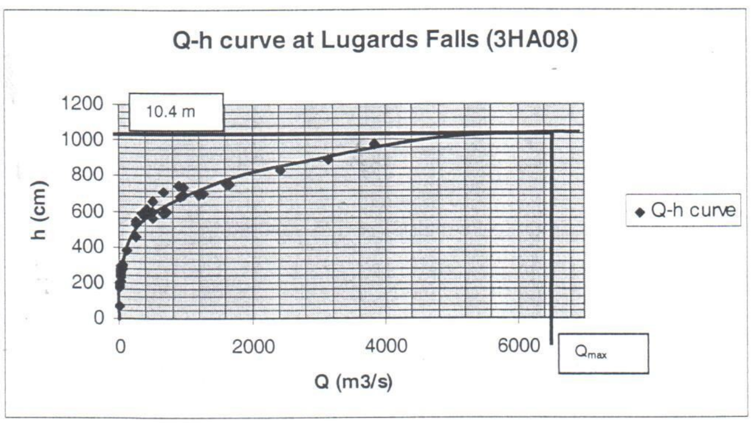
Table 20.6 avarage and 95% of non exceedance waterlevels for P.W area

Section	$Q_{avg}(m^3/s)$	$h_{avg}(m)$	$Q_{95\%}$ (m^3/s)	$h_{95\%}$ (m)
A-A	17	0.5	350	2.9
C-C	17	0.5	350	1.75

20.7.5 Waterlevels at Lugard's Falls

The water at Lugard's Falls are defined with Q-h and probability of non-exceedance curves.

Lugard's Falls	
Q_{max} (m^3/s)	6500
h_{max} (m)	10.4
$Q_{5\%}$ (m^3/s)	650
$h_{5\%}$ (m)	6
Q_{avg} (m^3/s)	2.51
h_{avg} (m)	26



21 PARAMETERS OF INTEREST / KEY NUMBERS FOR DESIGN: CONSTRUCTION RELATED

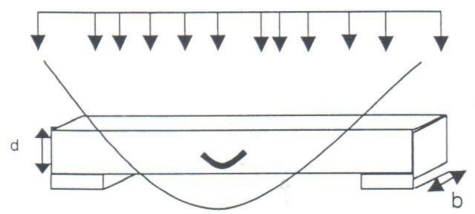
The first determination of dimensions for the construction is done with rough estimation rules. After that, these are going to be checked on shear forces bending moments and normal Forces. When all dimensions are clear, costs can be calculated. Costs are calculated to compare each alternative with each other.

Load factors:	variable load:	$\gamma_q = 1.5$
	Permanent load:	$\gamma_p = 1.2$
	Positive load:	$\gamma = 0.9$

To determine the resulting shear forces, normal forces and bending moments, "matrix frame" is used. It's very important to simplify the construction correctly so it can be imported in the program.

21.1.1 Bending Moment

The formula $M_d / (bd^2f'_b)$ is the fundament of the table. When every variable is known or estimated, the economical reinforcement percentage can be determined.



Minimal and maximal reinforcement percentage for B25 concrete

$\omega_{min} = 0.15$ %
$\omega_{max} = 1.38$ %
ω_0 = economic reinforcement percentage

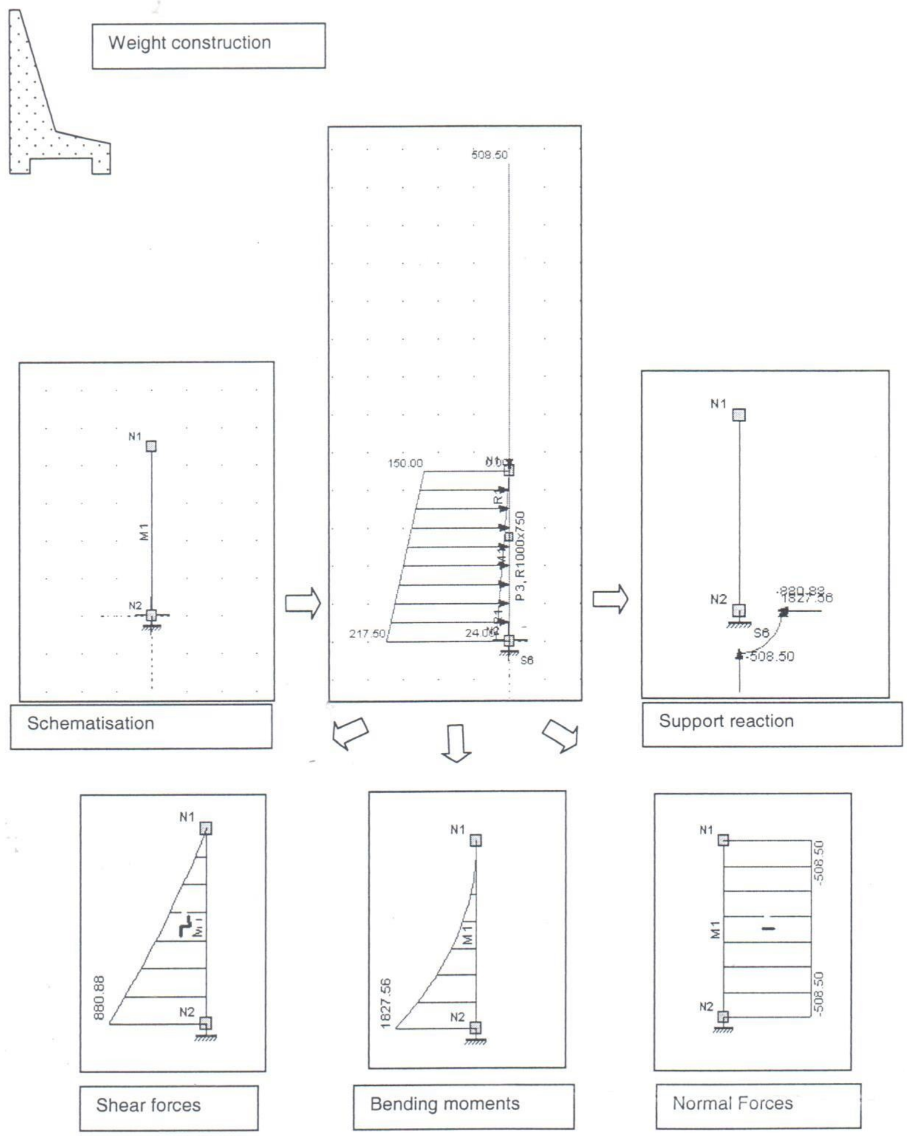
21.1.2 Shear forces

τ_d = design shear tension
$\tau_d = V_d / bd$

τ_1 = max. Shear tension for concrete
$\tau_1 = 0.4 f_b$ $\quad f_b = f'_{rep} / \gamma$ $\quad f_{rep} = 0.7 (1.05 + 0.05 * f_b') / 1.4$

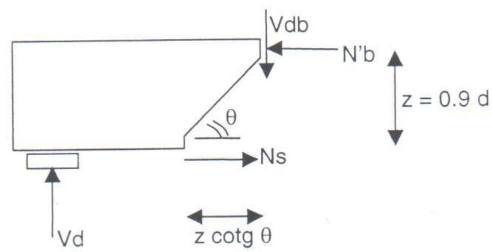
τ_2 = max total tension
$\tau_2 = 0.2 f_b'$

For Example:
To determine the bending moment in the lower left corner.



Shear force reinforcement

Shear force reinforcement is needed if $\tau_1 < \tau_d < \tau_2$ The concrete structure is not capable of absorbing all shear tension, so extra steel must be add.



21.1.3 Turn over

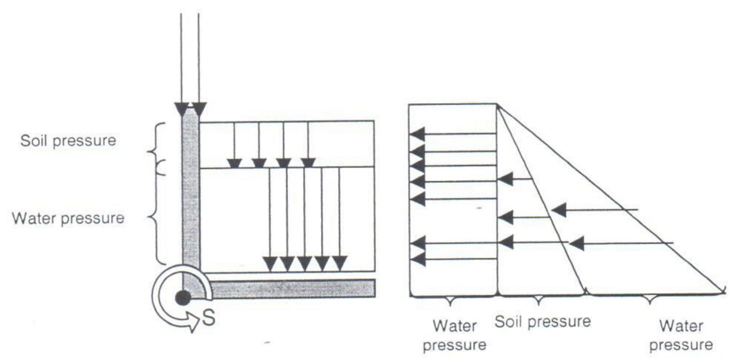


Figure 21.1 Schematisation turn over

To design a secure construction it has to be safe against "turn over". The horizontal water pressure wants to push the structure away. The structure has to be heavy enough to resist these horizontal forces, to stay on its place. To make the structure stable the concrete slab that on the ground can be enlarged. Than there's more soil and water resting on the structure, which makes it heavier, and as a result of that more stable.

In this case there are some relevant things that have to be mentioned. The structure will rest on rock. When the structure is properly made there's no water pressure under the concrete slab, which is very positive.
The structure turns over at point S. The moment at this point has to be negative. To calculate this moment the load factors are taken into account. Positive loads are multiplied with 0.9 Negative loads with 1.2 or 1.5.

21.2 PRICES

The rates inserted shall include for all expenses of supervision, agents, time keepers, plant operators, drivers, transport, tools of any kind, office expenses maintenance fuel, oil lubricants, overheads and profit

Unit prices material				
profit head contractor	%	25		
profit sub contractor	%	10		
allow for the provision and maintenance of temporary security staff accommodation	ksh	600,000		
	unit	price **[ksh]**	price **[$]**	price **[fl]**
Clear road wayleave of all vegetation, grass, debris, bushes	ha	100,000	1,333.33	3125.0
				0.0
Excavate in soft material to form roads and side-ditches	m3	100	1.33	3.1
Excavate in hard material to form roads and side-ditches	m3	500	6.67	15.6
Prepare ground as necessary and provide 200mm thick top layer	m2	1,900	25.33	59.4
Excavate for structure in soft material (0 m - 1.5 m)	m3	200	2.67	6.3
Excavate for structure in hard material (0 m - 1.5 m)	m3	1,000	13.33	31.3
Excavate for structure in soft material (1.5 m - 3 m)	m3	300	4.00	9.4
Excavate for structure in hard material (1.5 m - 3 m)	m3	1,500	20.00	46.9
Provide, fill and compact selected granual material	m3	1,250	16.67	39.1
Provide, fill and compact graded hardcore	m3	1,500	20.00	46.9
Provide, place and compact concrete class B25	m3	7,700	102.67	240.6
Provide, place and compact concrete class B15	m3	6,400	85.33	200.0
Reinforcement, high yield square twisted steel	Kg	80	1.07	2.5
Class F1 finish formwork in various inclinations	m2	220	2.93	6.9
Class F2 finish formwork in various inclinations	m2	220	2.93	6.9
Provide and fix beam guard rail	m	2,300	30.67	71.9
allow for painting	m2	250	3.33	7.8
Provide, lay and join precast concrete pipes D = 450 mm	m	5,000	66.67	156.3
Provide, lay and join precast concrete pipes D = 600 mm	m	7,000	93.33	218.8
Provide, lay and join precast concrete pipes D = 900 mm	m	9,000	120.00	281.3
Provide, lay and compact class B15 concrete pipes incl. Formwork and reinforcement	m3	7,700	102.67	240.6

	unit	price [ksh]	price [$]	price [fl]
Tractor or equivalent with dozer	Hrs	3000	40.00	93.8
Motor grader Cat. 14	Hrs	1500	20.00	46.9
Vibrating plate compactor	Hrs	1000	13.33	31.3
Concrete vibrator	Hrs	250	3.33	7.8
1.7 m3 tractor excavator	Hrs	2000	26.67	62.5
7 tonne tipper lorry	kms	50	0.67	1.6
self propelled water tanker 6000 litres	kms	50	0.67	1.6
Unit price Labour				
Unskilled labour	Hrs	25	0.33	0.8
Foreman	Hrs	75	1.00	2.3
Artisans	Hrs	75	1.00	2.3
Driver	Hrs	50	0.67	1.6
Material				
Rates to include delivery to site				
Ordinary Portland Cement	Tonne	10000	133.33	312.5
Mild steel reinforcement (any diameter)	Tonne	80000	1066.67	2500.0
High yield steel reinforcement (any diameter)	Tonne	80000	1066.67	2500.0
Fine aggregate for concrete	m3	1000	13.33	31.3

22 CONCEPT SOLUTIONS

22.1 INTRODUCTION

The problem analysis makes transparent why a river crossing is desirable. In this chapter concept solutions for the crossing are discussed. Five basic alternatives will be examined. These options are: A bridge, a semi-open tunnel, a pontoon bridge, an Irish bridge, and an Irish bridge with culverts. The following thongs are discussed for every alternative: An introduction about the concept and location; possibilities for design with demands and solutions. A rough cost calculation and a summary of possible problem areas. Finally, each alternative should meet the boundary conditions and the program of demands, which generally means it has to be as simple and cheap as possible.

22.2 ALTERNATIVE 1: BRIDGE

22.2.1 Introduction

To make a proper decision where to situate the bridge, a choice has to be made between two concepts. The first option is to span the river at once. This requires a rather narrow point, with banks close to each other. The other option is to span the river in two, three or even more sections. This method allows spanning a wider gap and makes it possible to construct with less material. Consequential columns have to be placed on the riverbed. This can cause a lot of problems. For example erosion in area around the columns, great forces on the columns in periods of high water levels and rocks and other materials dragged down by the river that cause damage. On the other hand, cost for a bridge with several spans can be much lower. Both alternative will be discussed here
A bridge will be situated about 1 kilometre upstream of P.W. point. The river here is about 75 meters wide and bordered by two steep riverbanks.

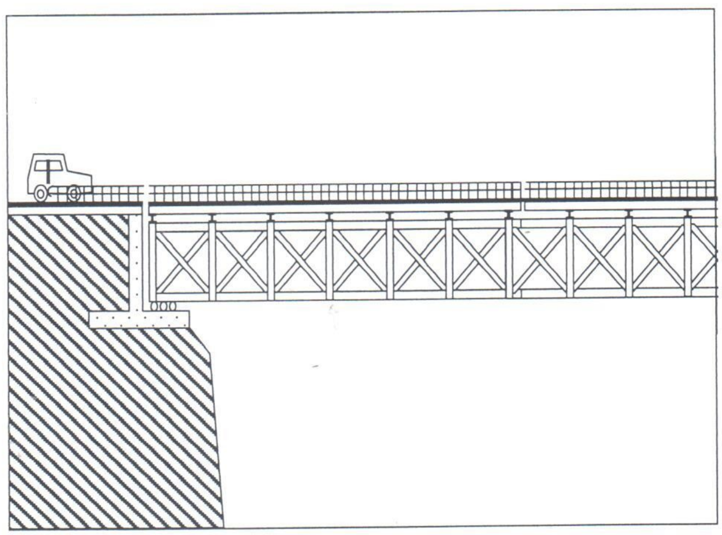


Figure 22.1 Schematization bridge

22.2.2 Justification of the location

The gorge is a good place to situate a bridge, spanning the river at once. The gorge is very small at that point, approximately 70 metres, and also high enough. A bridge on this point won't become in contact with the river even during El Nino if it spans the river at once. This is an advantage in comparison with the other locations. Safety and accessibility are maximised, the bridge is easy to reach because there are existing roads are close by. Notice that this bridge spans a gap of 70 metres. Advanced materials and techniques are required to construct a bridge like this

22.2.3 Demands

The most important demands, following the boundary conditions and parameters of interest, for bridge are:

- The span with of the bridge should be about 75 meter
- The distance between the bottom of the riverbed and the underside of the bridge should be at least 12 meter. (waterlevels can reach up till 11.7 meters
- The bridge should be able to carry a loaded truck (or three elephants)
- Access roads should include the building
- The location of the bridge should close to existing roads. The decreases the costs

Figure 22.1 Location bridge.

22.2.4 Solutions

Heavy Girder Bridge
A good solution to bridge this gap is with a so-called Heavy Girder Bridge made by for example Janson Bridging. Strength is guaranteed and measurements are very flexible. The Heavy Girder Bridge consists of small elements, which makes it possible to built it in difficult circumstances. They are designed for heavy truckloads. This means that getting equipment and materials into the northern part of Tsavo isn't a problem anymore. A big advantage is that it can be built by manpower only. A smaller version, the" Bailey Bridge" is too small to bridge a gap of 75 metres. These bridges can span distances up to 61 metres. The heavy girder bridge can go up to 90 metres

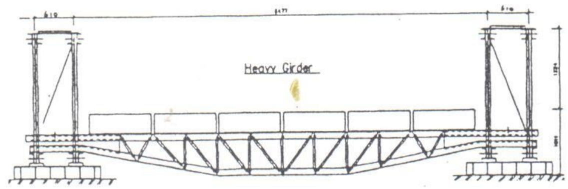


Figure 22.3 Cross section Heavy Girder bridge

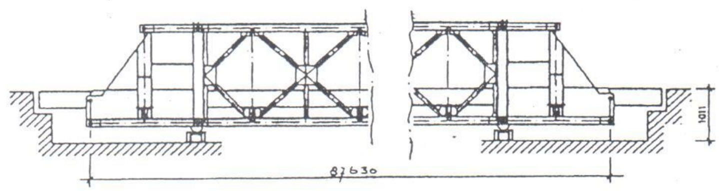


Figure 22.4 Heavy Girder Bridge

This bridge with a span of 76 meter can carry max 18 tons load. (Three elephants) It is going to be built out of 20 fields. Its total dead weight is approximately 200 tons. Or the bridge can be built out of a number of units with pilars in Figure 22.5 costs are compared with the number of units the bridge is built out.

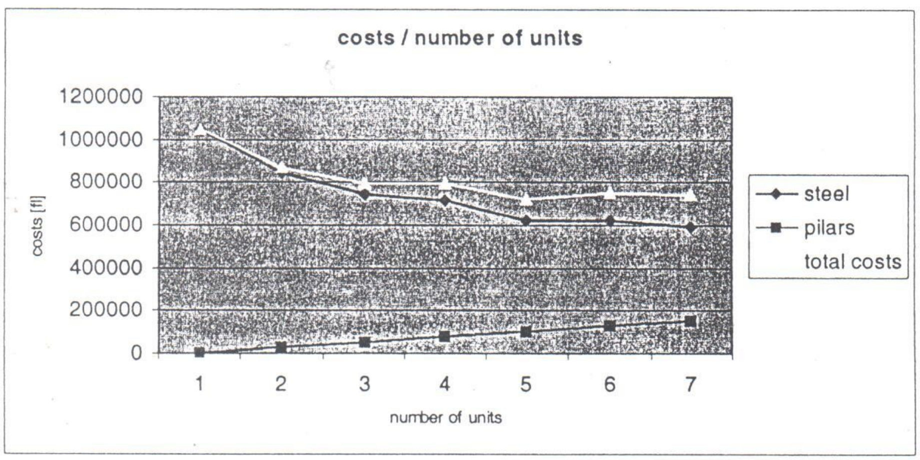


Figure 22.5 Costs of bridge compared to number of units

It seems strange that the costs are not going down smoothly. That is because these units are standard, what means that sometimes the units are not lighter but they can carry more load. In Figure 22.6 it is shown that the maximum load increases per unit. That is why in Figure 22.5 the costs stabilise.

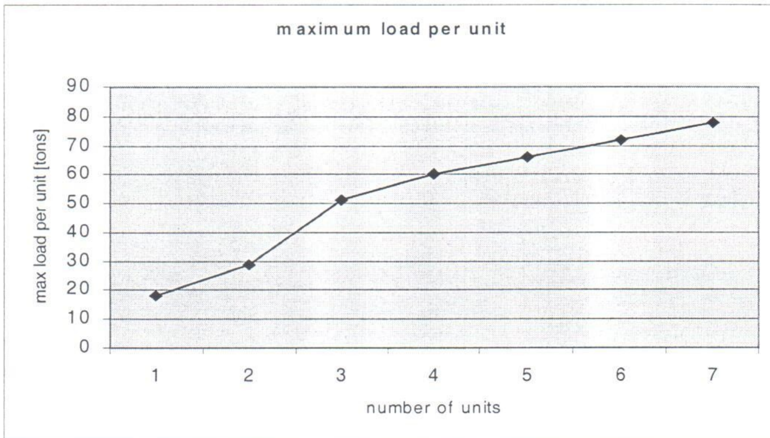


Figure 22.6 Maximum load per unit

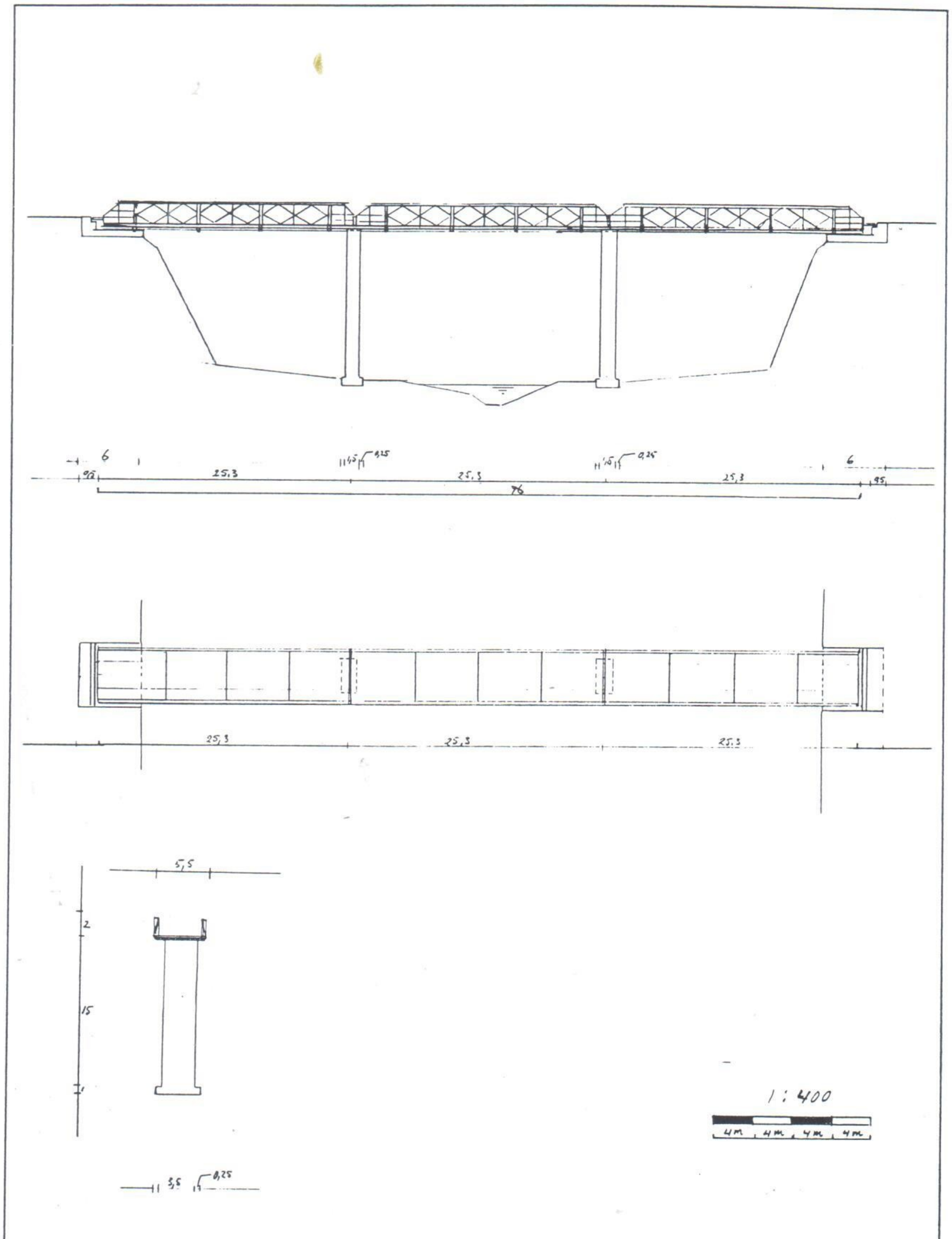


Figure 22.7 Bridge dimensions

Costs
Because the steel profiles are coming from Europe it has to be transported all over from Europe to Mombassa and from Mombassa to Tsavo. These transport costs are much higher than costs made for local transport.

The rates inserted shall include for all expenses of supervision, agents, time keepers, plant operators, drivers, transport, tools of any kind, office expenses maintenance fuel, oil lubricants, overheads and profit

		unit	quantity	price / unit	total
fundament [2x]	concrete B25	m3	7.5	fl 240.0	fl 1,800
	reinforcement steel	kg	588.8	fl 2.5	fl 1,472
	excavation hard material	m3	3.8	fl 46.9	fl 176
	formworks	m2	8.0	fl 6.9	fl 55
pillars [2x]	concrete B25	m3	56.3	fl 240.0	fl 13,500
	reinforcement steel	kg	4415.6	fl 2.5	fl 11,039
brug	steel deck	tons	162.0	fl 3,600.0	fl 583,200
	transport	tons	162.0	fl 1,000.0	fl 162,000
	montage	#			fl 157,000
Connection to shore [2x]	concrete B25	m3	17.6	fl 240.0	fl 4,224
	reinforcement steel	kg	1381.6	fl 2.5	fl 3,454
	excavation hard material	m3	34.4	fl 46.9	fl 1,612
	formworks	m2	67.8	fl 6.9	fl 468

exclusive roads towards construction

total	**fl 940,000**

22.2.5 Problem areas

Problems that can be expected and has to be considered are:

- When a pre-fabricated construction like a Heavy Girder bridge is used, the building process has to be managed very well. Although local workers might be able to do a lot of construction work, technical knowledge is required to avoid failures
- The quality of the rock, on which the construction will be founded (at each riverbank) has to be considered. Cracks in rock-layers but stiffness and strongness are yet unknown
- A maintenance program should be drawn up. For example a bridge of steel has to be painted once a few years.

22.3 ALTERNATIVE 2: TUNNEL

22.3.1 Introduction

When designing and building a tunnel a whole other range of problems has to be faced. Normally a bridge interferes very little with the river, only its columns will stand in water. A tunnel, in contrary, interferes very much with the existing flow patterns. The river will react to refine the easiest way to pass this obstacle. One of the most important things is to predict how flow patterns will change.
Unless the tunnel is sank into the riverbed, it will cause a water level rise upstream. Most of the time this is not desirable.

Figure 22.8 Location Tunnel looking upstream

22.3.2 Location

The best location for a tunnel is shown in Figure 22.8 Relevant factors which contributed to the choice for this place are its ideal geological position. But even more important is the almost abrupt drop of the water level of approximately 4 metres.

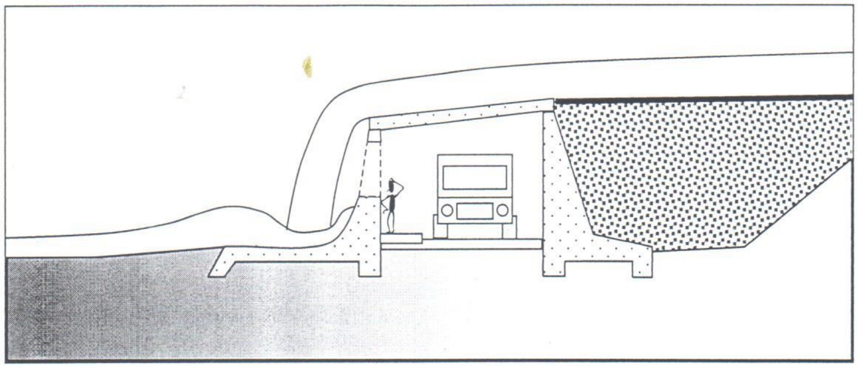


Figure 22.9 Semi open tunnel

As a result of this a possible small tunnel will not cause a big rise of the water level upstream. Interference with the natural flow of the river is minimal in comparison whit other locations. Also for tourist reasons the location is attractive. Just downstream of possible location a pool gives home to a large herd of hippo's. Crocodiles are also easy to spot and many other animals use the river to drink

22.3.3 Demands

- Traffic demands

Every tunnel alternative should be accessible for at least one truck. Costs are minimised when the construction is small. Simple road signs can organise one-way traffic. The relevant dimensions of a truck are as shown in Figure 22.10 2,40m * 2,70m

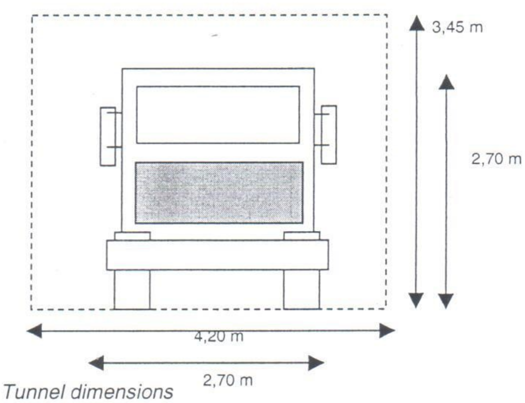


Figure 22.10 Tunnel dimensions

The free space these trucks need to drive trough for example a tunnel is 0.75m at both sides and top.

- Design demands

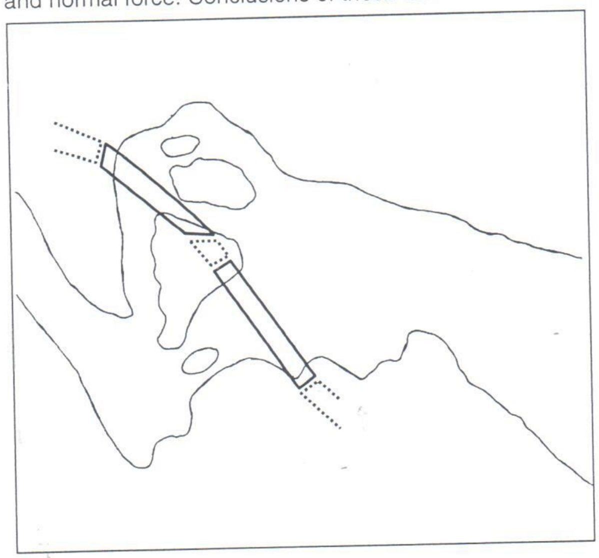
The minimum height of a tunnel should be 3,35m. When lights are required, there should be 0.5m extra. The tunnel is going to be designed with a internal **height of**

4m. The width a truck needs is at least 4,20m. With some extra space for pedestrians the **minimum width of the tunnel should be 5m**

- The tunnel should be able to resist the water forces.
- Water levels upstream of the tunnel can rise to more then 12 metres (during El Nino)
- Water levels downstream can reach a maximum of 5.25 metres
- The availability of the tunnel should be almost 100 %. Only during when El Nino discharges are at the maximum the tunnel might be closed.

22.3.4 Design solutions

The exact calculations to define the dimension of the tunnel are executed with the help of Matrix Frame. The construction is checked on shear force, bending moments and normal force. Conclusions of these calculations are translated to dimensions.



250 m

Figure 22.11 Location tunnel

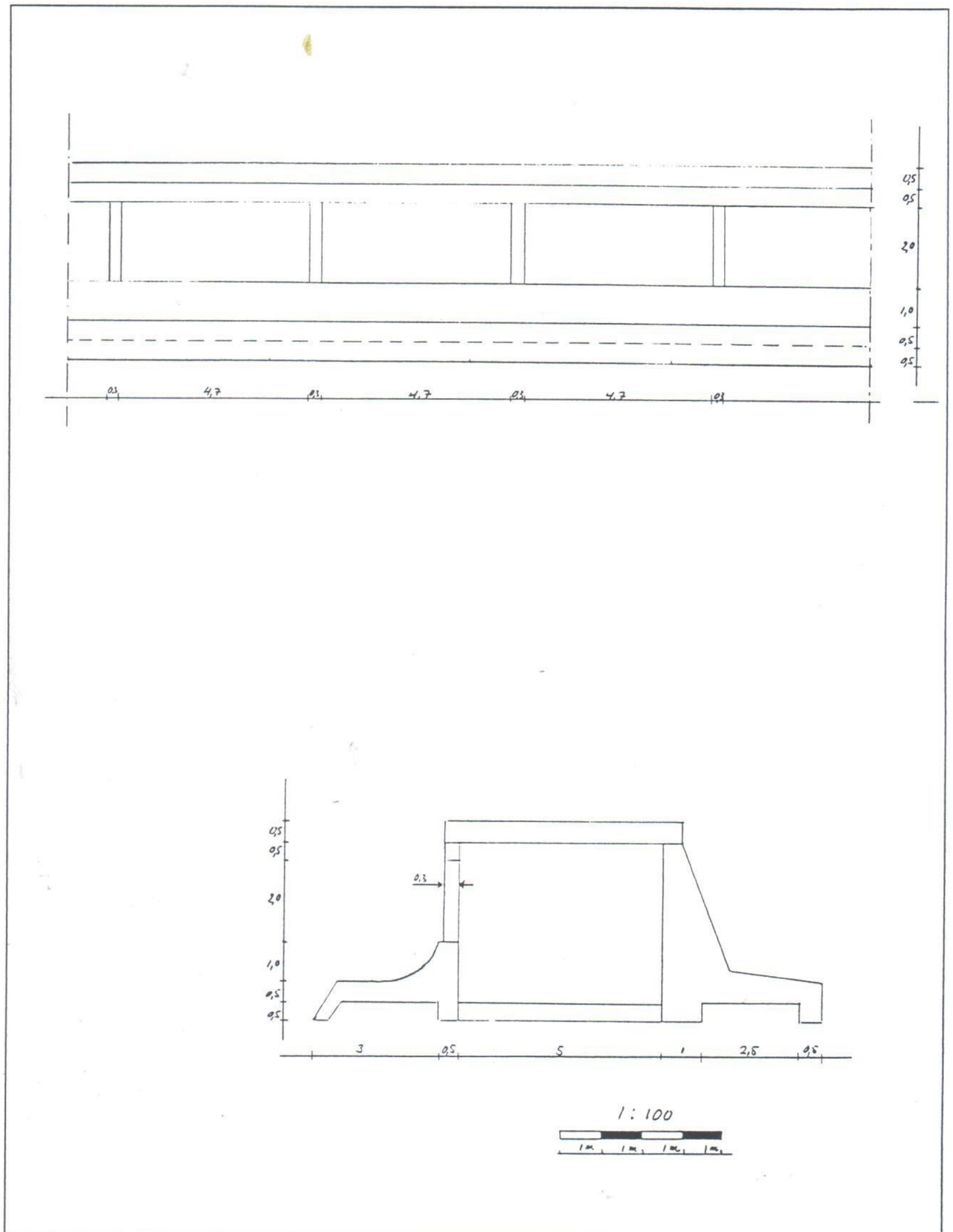


Figure 22.12 Tunnel dimensions

22.3.5 Costs

The rates inserted shall include for all expences of supervision, agents, time keepers, plant operators, drivers, transport, tools of any kind, office expences maintenance fuel, oil lubricants, overheads and profit

tunnel segment		unit	quantity	price / unit	total
Roof slab	concrete	m3	15.0	fl 240.0	fl 3,600
	formworks	m2	35.0	fl 6.9	fl 242
	reinforcement steel	kg	1,177.5	fl 2.5	fl 2,944
beam	concrete	m3	0.8	fl 240.0	fl 192
	reinforcement steel	kg	62.8	fl 2.5	fl 157
	formworks	m2	6.5	fl 6.9	fl 45
columns	concrete	m3	0.3	fl 240.0	fl 72
	reinforcement steel	kg	23.6	fl 2.5	fl 59
weight construction	concrete	m3	32.5	fl 240.0	fl 7,800
	reinforcement steel	kg	2,551.3	fl 2.5	fl 6,378
	excavation hard material	m3	13.8	fl 46.9	fl 645
	formworks	m2	50.0	fl 6.9	fl 345
stillin bassin	concrete	m3	14.0	fl 240.0	fl 3,360
	reinforcement steel	kg	1,099.0	fl 2.5	fl 2,748
	excavation hard material	m3	10.0	fl 46.9	fl 469
	formworks	m2	15.0	fl 6.9	fl 104
upstream protection	geotextile	m2	125.0		fl ?
road inside tunnel	concrete	m3	7.5	fl 240.0	fl 1,800
drainage	pump	#		fl 20,000.0	fl 20,000

segment	**total**	**fl 30,958**
tunnel	**total**	**fl 2,806,218**

22.3.6 Problem areas

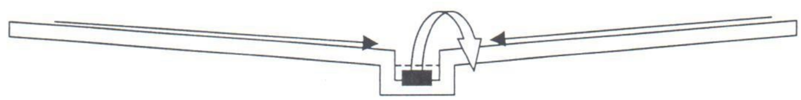
While designing and building a semi-open tunnel construction, a lot of problems has to be faced. In this section these problems will be mentioned and, if possible, solutions will be described. The main problems can be divided in three parts: water related, design related and constructing related.

Water related
Water related problems that can occur, are:

Erosion of upstream material in times of high discharge and water velocities.

Sedimentation upstream in times of low discharges
The river Athi is heavily polluted. Water samples show high concentrations of heavy metals. (See appendixes, data obtained from the National Water Master Plan). Pollution sticks to the sediment that drops upstream the tunnel. This will produce polluted sand storage, just in front of the tunnel. Flora and fauna will suffer from this.

Leakage occurs in two ways. River water that flows over the tunnel but also through the roof and other components into the tunnel, Groundwater that finds its way through the rocky out bolder, what causes waterpressure underneath the structure Thanks to the continuous water flow on top of the tunnel everything is going to be wet, inside and outside the tunnel. Problems can be expected with maintenance of the tunnel. Because of the moistness of the interior everything gets slippery. Little plants will start to grow, together with little animals that will find their new habitat there. Another question that rises is where to drain superfluous water. The drain must be simple. A solution could be to install a pump at the lowest part of the tunnel pump the water out. The tunnel has to have a little gradient towards the pump otherwise the water won't flow in the direction of the pump. Since there is no electricity, it is best to use a diesel aggregate.



22.3.6.1 Design related

One of the biggest problems is the enormous differentiation of forces this tunnel has to withstand. First of all, when El Nino takes place, ten metres of water will flow above the tunnel. Big moments and shear forces will make the use of a huge amount of concrete and steel necessary, which is very expensive.
During El Nino the water level upstream rises, the water level below the tunnel will also rise. Eventually water will stream into the tunnel, and it can happen that the whole tunnel is will be under water. When this happens, forces on the construction will change completely
Actual constructing the tunnel will also cause a lot of problems. The river has to be diverted before the elements can be placed in position. And the construction must be completed between two rain seasons. Put that together with simple building methods and little access to money, and a big challenge is born

A big reservoir upstream the river caused by this tunnel is not desirable. It's possible that because of the reservoir water level elevation plus the El Nino water level elevation, the river goes over the riverbanks, which can cause a shifting of the river.

22.4 ALTERNATIVE 3: IRISH BRIDGE

22.4.1 Introduction

The Irish Bridge is the simplest alternative for a river crossing. It consists of concrete slabs that are placed on the riverbed. Water is supposed to go over it without making it inaccessible. There's a lot in favour of building an Irish Bridge because of its simplicity and low costs. There is only one big problem, during rain season the water level will be too high. When the water level becomes higher then +/- 35 cm, the river will be impossible to cross. It makes the bridge useless during rain season. Most of poaching is done at the beginning of rain season, because vegetation is low and traces are washed away. It's doubtful the KWS can chase poachers to the northern part of Tsavo East when there is too much water on the crossing. Further research is needed to determine if this alternative can be successful. At Luggard Falls, an Irish bridge has already been build. An evaluation of this crossing can be a source of information.

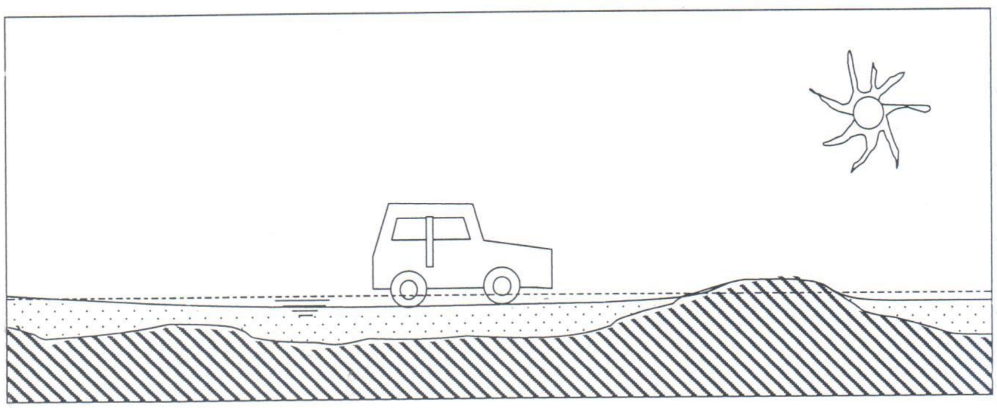


Figure 22.13 Irish Bridge

22.4.2 Location

The best location for an Irish Bridge is a place where the water level is low and the river very wide. When the river discharge increases, the water level doesn't rise very much. Most of the time it will be accessible. The best location in Tsavo is Lugard's Falls. An Irish bridge on this location should be 500 m long, 5 m width, 1 m high.

22.4.3 Demands

- The Irish bridge should be accessible during 95% of the time
- A truck should be able to cross the bridge A truck needs is at least 4,20m. With some extra space for pedestrians the minimum width of the bridge should be 5m
- The Irish bridge should not obstruct the river flow. In other words water should be able to flow (almost) free along the construction.

22.4.4 Design

It is advisable to make use of Cyclope concrete. This is a mixture between rocks and concrete. The rocks are already in the park, so they don't have to be bought, just collected. 1 m3 costs fl150,- / m3

22.4.5 Costs

Length 500 m
Width 5 m
Height 1 m

500m*5m*1m = 2500 m^3 concrete

	Costs per unit	Units	
Concrete	Fl 150,-	2500	Fl 375000,-
Formworks	Fl 6.9,-	300	Fl 2070,-
		total .	**Fl 377070,-**

Problem areas

Problems that can occur are:
To achieve a non exeedance rate of 95% the height of the Irish bridge should be at least 6 m
An extra Irish bridge does not give extra benefits compared with the one at Lugard's Falls, which is already there.

22.5 ALTERNATIVE 4: PONTOON BRIDGE

The Pontoon bridge differs from all other concepts. A normal bridge spans the water, only its columns make contact with water. A tunnel is placed underneath the water level, so the water will go over it. A pontoon bridge finds itself on the verge of water and the open air. It floats on the water stream. During dry season most of the riverbank will be dry, which means most of the pontoons will rest on the rocky riverbed. When all the pontoons are floating the admission of all forces have to be at the borders of the first and the last pontoon. Another possibility is to build constructions that control the pontoons lifting when the water level rises. Now the admission of all forces to the ground can be done by the constructions that are keeping the pontoons in place. A positive point is that because the pontoons are floating on water, the pontoons and the weights on it won't cause big moments. This makes it possible to design a very thin construction. Extra attention should be paid to, how to connect these pontoons, how to connect them to land and how to construct the constructions keeping the pontoons in place.

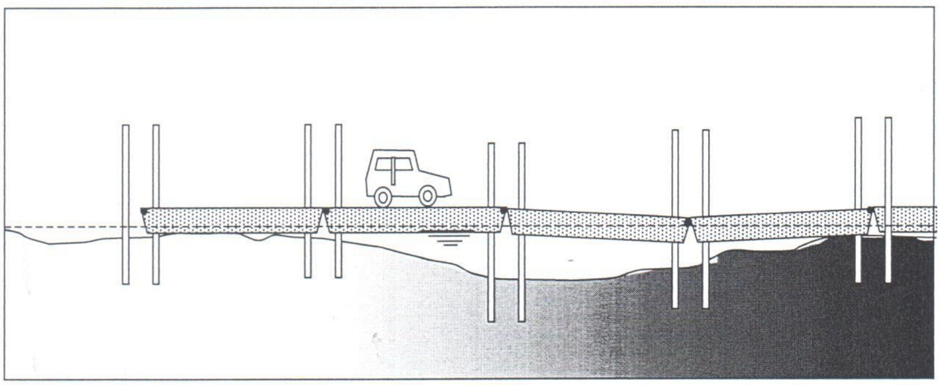


Figure 22.14 Pontoon Bridge

Costs estimation with al these unknown factors isn't relevant. First there has to be done an extra feasibility study on this alternative. Nevertheless this is a very complicated construction. To build this will require a lot of technical skills and money to spend. To protect the construction against the fierce conditions during El Nino it has to be very large, and because of that expensive.

22.6 ALTERNATIVE 5: DAM WITH CULVERTS

This alternative is an intermediate form between a bridge and a dam. When the water level is not to high, the discharge capacity of the culverts will be sufficient. Its possible that during high water levels the upper layer of the dam will be under water and by that unable to cross.
It's not very hard to imagine that when the dam with culverts is low, it will be inaccessible during rain season. When a higher construction is build the chances decrease it will become inaccessible. A consideration has to be made between costs and accessibility.

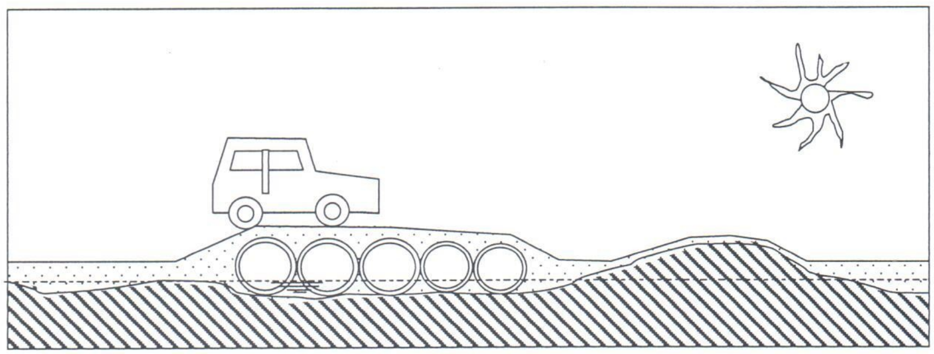


Figure 22.15 Irish Bridge with Culverts

23 CONCLUSIONS

23.1 GENERAL

The environmental circumstances in a national park like Tsavo are difficult. To build a construction under these circumstances is very complex, because there is a lot of risk. To get building materials and tools into the park can cost a lot of problems. Very old Trucks have to drive on roads that often are not in good shape. Clean water to make concrete is hard to find. B25 is the strongest concrete to work with "in situ".

23.2 DISCHARGE

The average discharge and $Q_{95\%}$ are low. Most of the time not more than 1/10 of Q_{max}. However once in about seven years an El Nino flood passes. Constructions should be capable to sustain these floods. Nevertheless it's very hard to keep these structures accessible during El Nino. Floods normally last only a few days.

23.3 WATERLEVELS

The water levels can reach up till 12 meters in the P.W. area. Water velocity can grow up till 8 meters per second. The flow is almost supercritical at some places. Constructions should contact the water as less as possible, because high velocities and water levels are making it very hard to prevent damages.

23.4 SOLUTIONS

Bridge:

- A very good solution for a river crossing, because there's as little as possible physical contact with the river. It's important that the bridge should stay above El Nino level, which is 12 meter (h_{50}). This is not a problem because the gorge is much deeper than 12 meter, approximately 20 meter.
- The bridge will always be accessible because of its height.
- It's an expensive solution. The Heavy Girder Bridge will cost around 1 million guilders. Roads towards the bridge are not included.
- A heavy Girder Bridge is relatively easy to build. It comes in packages of 7.6 meters and can be assembled with only manpower. Failure risks are low, it's a very safe alternative
- It's not a very subtle concept. A bridge will interfere very much with the beautiful environment.

Semi-open tunnel:

- From a tourist point of view, this is a wonderful alternative. This tunnel will be an attraction just by standing there.
- However it's very expensive. 1.6 million guilders is too much money. There might be a possibility to collect toll for people crossing the river.
- A construction like this is very difficult to build. Failures during construction are hard to exclude.
- Big forces force the tunnel to be massive. During El Nino 8 meters of water will flow over its roof with the danger of the tunnel getting damaged by big rocks that are dragged down by the water.
- During El Nino and other floods the tunnel will be full with water, because the downstream water level will rise. It's hard to predict what happens with the structure when this occurs.

- It always will be very moistures in the tunnel

Irish Bridge:

- This is by far the cheapest alternative
- But it's not going to be of extra value next to the Irish bridge that already exists at Lugard's Falls.
- To make it more than 95 % of the time accessible it has to overcome a water level of 6 meters, which is far too much for an Irish bridge.
- If accessibility throughout the year wasn't the main criteria, an Irish bridge would be a very good alternative.

23.5 SUMMARY

The bridge in the gorge is in most points of view the best alternative. It's the only alternative that is always accessible. Still it's going to be a big problem to collect the money. An Irish bridge is the cheapest solution but it won't be of extra help because it's too low. The tunnel is a prestigious project, which can attract a lot of tourists. But it's too expensive and very complicated to build.

APPENDIX 1 INTERVIEWS

INTERVIEW WITH FRANCIS MWANGI ON MARCH 22. 2000

General

Has there ever been made a masterplan on Tsavo East N.P. ?
A masterplan has been made in the last years, but it's not public, because it's a proposal that has not been approved.

What kind of people do poach and where
Tsavo, Samburu and Masai Mara has suffered most from poaching. Somalians and som tribes are the main poachers. The ivory of elefants and rhino's is their main purpose

What is the purpose of the KWS in general?
To protect wildlife and conservation of the present situation. The most important thing is not to increase but to maintain the current level of wildlife and bio-diversity.

What is being done to reach this purpose?
KWS-rangers are patrolling inside the borders of the park to let there presence know. They have the right to shoot on everyone carrying a weapon (shoot to kill). Roads are being held in good condition to move around easily.

Peter Westerveld has a special status in Tsavo. To run a camp like he does, is against the KWS rules.

The biggest problem of the KWS is a lackage of money. The only source of income are the parkfees. In total KWS administrates 9(?) parks and the income is yearly 540 million shilling (17 million guilders) Parkfees differ between $23 and $30 for foreigners and Ksh 200 -500 for locals
80 % of all income is used for salary of Head Quarters and rangers, the rest is used to maintain the park.
For example the budget of roads is 4 million Khs a year (= 125.000 guilders). This means that there is 1000 Khs available per km road.
The amount of money left is hardly sufficient to keep up with the damages that appear.
New projects can only be realised with money gathered by fund raising.

Who are the users of Tsavo N.P.?
Inside the borders of the park you will find only animals, rangers and a few human beings in lodges and camps.
At the border and outside the borders you can find villages and farms. Although the Tsavo area is less populated, accidents occur between wildlife and man. To avoid these accidents fences are build at places where these conflicts can occur. A fence costs about Ksh 150.000, that's why it doesn't surround the complete park

Tourists
About 800.000 tourists visit the N.P.'s every year. This number fluctuates and is dependent of the stability of Kenya. When people are being killed in for example Nairobi less tourists will come. When the government should succeed in reducing crime and increasing safety, more tourists may be willing to visit Kenya and the parks.

Political situation
As mentioned above Kenya and especially Nairobi suffers from a lot of crime. The police is corrupt. Most politicians are corrupt and a big amount of money disappears to 'friends'of the government. This is the reason why foreign investors move out and new investors are less willing to invest in Kenya.

Income
Tourists are the main source of income to Kenya, bringing in about 6 billion Ksh each year. This is more than the export of coffee and tea produces (about 2 billion Ksh yearly)

About Tsavo East N.P.

Facts at a glance

size:	22.000 km2 for Tsavo East and West (about 10.000 km2 Tsavo West and about 12.000 km2 Tsavo East)
vegetation	most plains are filled with bushes and some trees. The amount of grass plains are few.
animals	Tsavo park is not well developed, so there is little wildlife. Tsavo has the big five. After poachers had reduced the number of rhino's to only one, KWS started a Rhino release project in 1997. 37 rhino's has been released and except for one (that was killed by lions) they are all alive and kicking.
infrastructure	There are a few roads through the park connecting the main. Roads are being used by rangers and tourists.
politicians	People are allowed to drive around freely in the park. They have to keep to the roads. They are not allowed to leave their cars. Carrying a weapon is strictly forbidden. After sunset no one except for the rangers is allowed to drive around. Animals may not be annoyed or disturbed.
KWS	about 150 rangers are working in Tsavo East.

Main problems
The main problem of Tsavo East is still poaching. Especially in the northern part (north of Athi and Galana river) of the park, poachers are still active. This is mainly because there is no river crossing over Galana and Athi.. In dry season there is a ford where cars can pass but this is not usable during the rainseason (in which most poaching takes place).
An other problem which has already been mentioned is a lackage of money. This means for Tsavo that the number of rangers is far to small to monitor the whole area (in other words, poaching is still far to easy)

Projects done by KWS

permission
Inside the borders of the park approval by KWS head quarters is needed. When safety of the environment and people is not in danger permission will be given easily. When a project interferes with national interests approval by the government is needed. Projected on Tsavo projects dealing with the Nairobi -Mombasa road and interference's in the main rivers (Tsavo, Athi/Galana and Voi river) should be approved by the government

procedure

When man wants to carry out a plan (for example building a brigde) a proposal has to be made and send into the technical department of KWS headquarters. A M.E.R. (milieu effect rapportage) will be made. When money is available (by fundraising f.e.) the plans will be pulled out and man seeks for the highest priority.

Technical

small project (< 3 mln Ksh)
A project costing less then 3 million Ksh will mostly done by the KWS itself. KWS will use his own equipment. Local workers will be used. Design and construct are done by KWS and no contractors are involved. Weekly report has to be made for KWS headquarters, to check and control. This gains the efficiency and little money will be spoiled.

large project (>3 mln Ksh)
The technical department of KWS makes a design and the project is advertised. Contractors can bit on the design. The contractor with the best plan (not always the cheapest) will get the project. In practice most KWS projects are done by a group of five contractors. Most of them have Asian roots

project organisation

project supervisor (KWS or consultant hired by KWS)
work inspector (KWS) reports monthly to HQ
Contractor
technical engineer
grouphead of local workers (partly local, sometimes foreign)
local workers (casuals)

regulations

- contractors are paid every month.
- a contract consist of:
  constructing costs
  unexpected costs +15 %
  design costs + 5 %
  overhead costs + 2,5%
  profit +15-30%
- When the principal (opdrachtgever) agrees with the result, the contractor will never responsible for damages and failure in the future

prices

materials
- *concrete (B25)+ labour + transport (used in situ) 7000 Ksh/ m3*

it is impossible to make B35 concretein situ
- *form work 300 Ksh/m2*

1 m3 steel = 7850 kg
- *galvanized steel 90 Ksh per kg 706500 Ksh/m3*
- *reinforcemnt steel 32 Ksh per kg 251200 Ksh/m3*
- *wire mesh 139 Ksh/m2*
- *nails,hammers etc. 4-5 % of construct costs*

equipment
1 gasoline = 40 Ksh
1/3 *costs of gasoline = costs voor maintainance
Cost of equipment per day (owned by KWS)

• bulldozer	180 l gasoline * 40 = 7200 * 4/3	= 9600 Ksh
• shovel	150 l gasoline * 40 = 6000* 4/3	= 8000 Ksh
• grader	150 l gasoline * 40 = 6000* 4/3	= 8000 Ksh
• truck	+/- 80 l * 40 = 3200* 4/3 = 4300 Ksh	
•	(hired)	= 10.000 Ksh + 100 Ksh/km
• excavator	180 l gasoline =	= 9600 Ksh
• crane	(hired) 8000 Ksh per hour*8 = 64000 Ksh	
	(borrowd from gouverment or army) = 20000 Ksh per day	

labour
costs per day

• engineer	2000 Ksh
• skilled worker	300-600 Ksh
• casuals (local workers unskilled)	160 Ksh (Nairobi)
	120 Ksh (Tsavo/Voi)

At last

what kind of river crossing do you prefer?
I prefer a bridge to cross the river. Kenya contractors are more experienced in making bridges. Also the idea of making a combination between a river crossing and tourist attraction is not preferred. A design of a bridge has been made on a more downstream location.

what kind of failures occur in Kenyan building projects ?
Most failures occur because of the poor quality of concrete, steel and form work. Also contractors misuse the money paid for work that has not been done

What do you think we forgot to ask?
You didn't ask about the the duration of projects. A project has to be prepared suitable and planned well ahead because of the rainseasons
You also didn't ask about the security. Bandits sometimes attack the building site to steel the wege from the workers. (weges are paid cash)

INTERVIEW WITH KIO, ASSISTANT DIRECTOR TSAVO EAST OF KWS

What are the park's main attractions:

To answer this question the history of the park is relevant. In 1948 Tsavo West (9065 km2) and Tsavo East (11747 km2) were founded as a single wildlife park. This park was too big to control and that's why they were divided into two parks. The border between these two parks is the Athi River. After that two more small reserves were added to Tsavo. In the West, at the border of Tsavo West there's Chyulu National Park and there's another National Park bordering Tsavo West in the South. These four parks are forming together an almost closed eco-system. A "closed eco-system" means that the main migratory, the elephants don't leave the park throughout the year. When elephant don't leave the park, other animals won't either.
Tsavo is the National Park most close to a closed eco-system and therefore unique. This uniqueness is the main attraction of Tsavo.
Another big attraction is the Yatta Plateau (270 * 60 km2). This is the second biggest plateau in the world and the only one where there has been no human interference. Because of that it's also interesting for research.

At the border of National Parks human -wildlife conflicts occur. What do you think is the best approach to face this problem:

The land outside the national reserves should be used more effectively. That will decrease the need of land for cattle and agriculture. People won't live as close to the park as they do now and problems between human and wildlife will decrease.

What's the main purpose of the KWS in relation to Tsavo:

The main purpose is to conserve wildlife and vegetation. Because there is so little left, conservation is a national, even global obligation. But first you have to ask yourself for whom are we conserving. The answer on the question is: for people. This means that people must have the opportunity to watch these animals without disturbing them. Since there have to be roads for rangers, why don't put the roads where the animals are, so people can come close by. Lodges in the park are also allowed. Every kind of attraction is permitted as long as wildlife won't be disturbed. For example balloon flights seem to be a peaceful and attractive way to watch the animals, but the animals are frightened for the shadows these balloons make. There is no point in building a fence around the park and closing it for people. It would be like a store full of supplies without it being able to sell them.

Where does the money for conservation come from and on what is it spend:

Money comes from parkfees and donors. There is no money coming from the government. Because the whole economy of Kenya is not working very well the taxpayer has no money to support wildlife conservation. Maybe when the economy recovers, there will be money for this. For now the KWS has to rely on parkfees and donor money. All the parkfees of every national park in Kenya are going to the KWS Head Quarters, from there it's distributed back to the parks. The money parks get is based on the work they are doing, not on the number of tourists that have been visiting the park. Some of the parks are making less money still there should be conservation. All the parks are involved with other parks. Every park has it's own uniqueness
To increase the income of the parkfees tourists should stay longer in the park. Tsavo is big enough to show them every day a different site. People must take their time, to

enjoy the park best. Walking safari's with experienced guides is a perfect alternative to achieve this.
The money Tsavo East gets from HQ is enough to pay salary and to maintain the roads. Occasionally there is money for special projects.
Funds from donors unless they are big enough must be declared by government. Big donors are The Worldbank and IMF. When they insist on what the money should be spend, plans have to chance. In a way, donors do distort the plans that are there.

How are new projects organized.:

Everything that is going to be built is valued on three criteria:

- Is it worth building.
- Is the infra structure ready for extra pressure
- People of KWS Research do an Environmental Impact Assessment for everything that is going to be built

How develops the KWS its policy:

In general every plan must fit into a bigger plan. The policy of the KWS is diverted from the ministry policy, which on his term is diverted from the national policy.
The policy of the KWS falls into a roughly hierarchical arranged components:

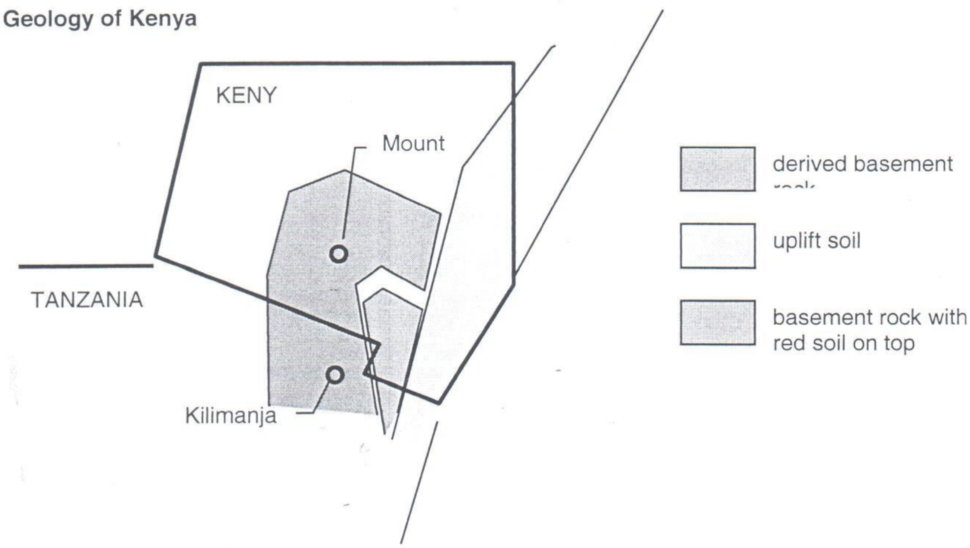
- Policy
- Legislation
- System Plan
- Five year management plans
- Annual development plans (including budgets and work programs)
- Reports
- Monitoring and Evaluation

Marketing and promotion of the National Parks is done by a ministry

INTERVIEW WITH PROF. MUTISO

The water of the Galana river contains a lot of sediment during the rain season. A lot of erosion occurs in the catchment area of Tsavo river. Tsavo river carries the biggest amount of sediment compared to the Athi river. Studies in the past on National Parks were only focussed on grasses, forest and wildlife. There were no studies on water behavior or erosion.
Tsavo East National Park converted from forest to grassland because of the large population of elephants. The 44000 elephants in the sixties destroyed most of the trees. Because of this, bushland changed into grassland. Since grassland erodes more than forests (because of the length of their roots) erosion in Tsavo East only increased. Most of the erosion takes place during the first rains of the season.

Geology of Kenya



In the early days the Kilimanjaro and the Mount Kenya were active Vulcan's. The erupted lava created a region of what is called "derived basement rock" Characteristics of derived basement rock are: The rock formations are fragmented. On top of the rock formations lays a thin layer of red soil. The surface of this region is rather hilly. The coastal area consists of uplift soil. The thick layer of red soil on basement rock in the coastal area is the main difference with derived basement rock. Kitui is situated on the derived base rock, Tsavo on the uplift land.

Kitui

The dams
In the Kitui district SASOL (Sahelian Solution Foundation), professor Mutiso and Maji na Ufanisi have built about 240 subsurface dams in intensive series (500 m between two dams). The dams have a height up to ten meters, but most of the dams are about 1,5 m high. The bottom of these dams is constructed on rock, so that no water can flow underneath the dam and cause piping. During floods there's a lot of sediment in the water, but that is not a problem. The purpose of these dams is to catch sand and water. The trapped water is stored in the sand in the upstream side of the dam. Because there's no surface water, evaporation is minimalized. Wells are constructed to have access to the artificial created groundwater. The sand has a filtering effect that makes the water bacteriological save. Each well has one bucket to

get the water out. It's important people only use that bucket because other buckets brought by the farmers can carry diseases.
The construction of the dam is made from rock, held together with iron bars and wire netting. The outside of the dam is covered with flat stones. Cement is used to connect the rocks together and is making the dam non-permeable. The iron bars are connected in the rock foundation to anchor the dam. After construction most dams are already covered with sand. The other dams are covered within a few rains, because if the amount of suspended load.
The best spot for a sub-surface dam is a place where rock is close to the surface and where a spring is located. It's very hard to locate these springs, sometimes springs are found. This is more out of luck than out of reason.
The soils in Kitui are fertile brown soils with a small fraction of clay. In the higher parts of Kitui there are volcanic soils. They are very fertile but also very dry. The problem is getting water to those places to irrigate the land.
The dams have lot effects on the surrounding area. The groundwater flow is lateral and also longitudinal since the dams are not completely non-permeable. The problem with measuring the groundwater flow is the cracks in the rock formation. During El Nino the heavy floods destructed one dam, the rest of the dams survived.
Other ways of getting water is drilling boreholes into the fragmented rock. The problem is to locate the wells and research is expensive. There are a few people specialised in this area.

Organisation
The local people who will benefit of the dam construct the sub-surface dams. SASOL and 'Maji na Ufanisi' help them by providing building materials and technical support, to build the dams. The local people invest time and effort. The main advantage of this method is making the local people feel responsible for their dam. They have to make arrangements about the water. The local women do the management concerning the actual building activities. At the time there is not much interest for constructing sub-surface dams. The technology is not spectacular enough. Once a forum was organised, but the main departments didn't show up.

Juridical
The local people own the land around the dams. The government owns normally 30 metres besides a river. The dams turn a seasonal river into a perennial river. This can cause a conflict between government and the local farmers. The government can claim the land besides the river now that it has become perennial. Nevertheless the river was created by local people and financed by local organisations, they also claim the land. Up till this moment no conflicts have occurred and it's to be expected it never will, only when the land is of national interest it might be claimed

Economics
There are big economical effects because of the dams. Immediate after building a dam, there is water available for growing crops. In some areas where dams are constructed even export of vegetables is possible.
One dam costs about 150,000 to 250,000 Kshs ($ 2000 - $ 3300). The proceeds of the surrounding land will increase firmly. The perspectives for these areas are good. The conditions for growing mango's (enough water dry weather and a lot of sun) are now achieved. Mangos bring in a whole lot more money than traditional crops. Fact is that you can only grow mango's when there's an over-production, otherwise the farmers have to grow crops that only supply their own food, which is not mango. The problem is getting the money to start investing in dams. Local people don't have the mentality of borrowing money to invest and pay back later. This water conservation

project can exist by aid of foreign organisations. It might be possible to use private investors to finance the sub-surface dams.

Difference between Kitui and Tsavo

The main difference between the Tsavo East National Park and Kitui is how the available amount of water is used. In Tsavo East National Park the water is used for wildlife and in Kitui it is used for agriculture and drinking water supply for humans. Another big difference is the way of storing the water. The Tsavo reservoirs are surface water reservoirs and in Kitui the water is stored as groundwater. So, the losses of water by evaporation, in the Kitui district are not relevant in contrast with Tsavo where the losses are big. The construction of the dams is also different. Peter Westerveld hires local people to build the dams. Al the costs are covered by the Westerveld Conservation Trust. In Kitui local people who are not paid for their work build the dams. In the end the dam becomes theirs. Now they can make money out of it. The target of the dams in Kitui is using the water as good as possible.
In Tsavo it is a must to create more than one dam in an area. Otherwise the reservoir will be used by all animals, which will cause conflicts. For example lions can guard the dam and just wait for prey. Multiple dams can prevent this problem.

Research

The design of the sub-surface dams and the effects on the surrounding land lack a descent technical foundation. Research on the influence of the dams on the direct surroundings is needed. Eventually every variable in the water balance has to become quantified. Where does the water go? How much evaporates how much water goes through the dam? Is it true that water that percolates to the groundwater can't be used anymore, or does it come back to the surface thanks to natural springs? Still It's not known where the water will flow. There are some assumptions where the water goes, but that is after the dams are built.

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APPENDIX 3 CONTACT REFERENCES

- Mr. Oliver
  KWS, Head of the Rhino Patrol Unit and constructor of the first dam

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- Mr. Kio
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  p.o. box 14333 Nairobi, Kenya
  Telephone 02-860772

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- Sillah Owiti Oriato
  Database Maji House, Department of Water resources
  Report: Fiddles, D ; Forsgate, J.A. ; Grigg, A.O. – The prediction of storm rainfall in East Africa
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APPENDIX 4 SUPERVISION

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Ir. P. Ankum, Delft University of Technology, Faculty of Civel Engineering and Geosciences.
Ir. W.J. Dijk, Delft University of Technology, Faculty of Civel Engineering and Geosciences.

Process manager:

Ir. M.E. Ertsen, Delft University of Technology, Faculty of Civel Engineering and Geosciences.

Advisors:

Ir. G. Pichel, DHV Consultants B.V.
Prof. Ir. H. Savenije, Delft University of Technology, Faculty of Civel Engineering and Geosciences.
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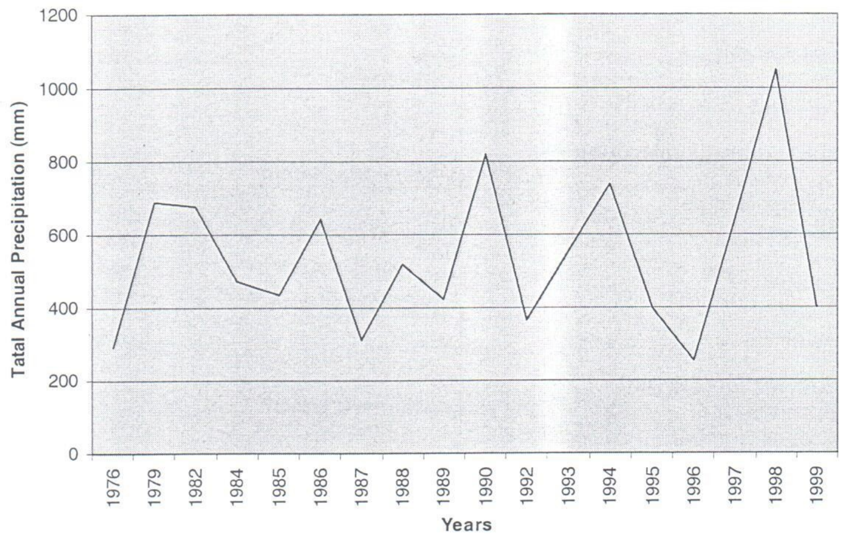
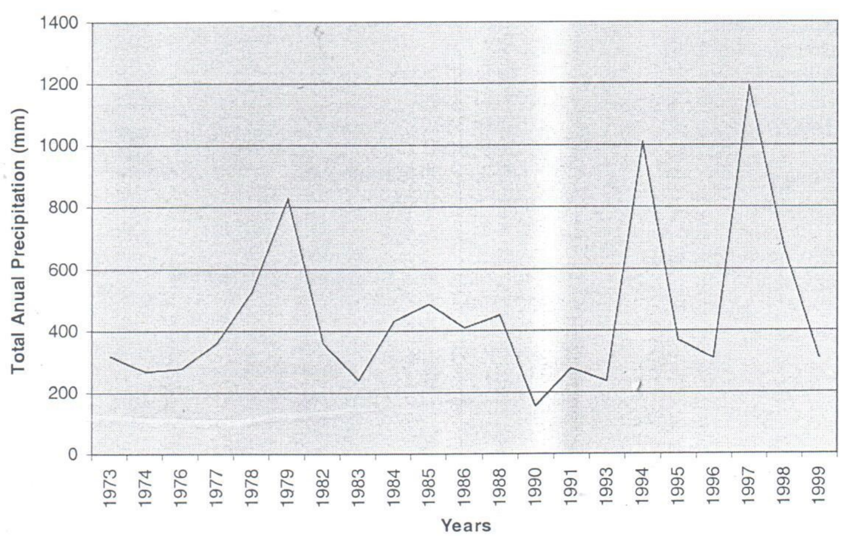
APPENDIX 6 TOTAL ANNUAL RAINFALL DEPTH PER STATION

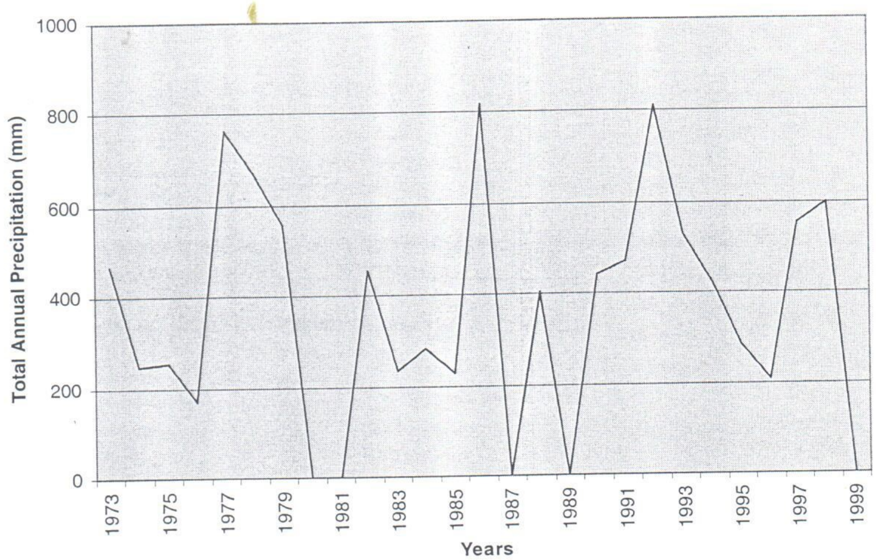
- Airstrip



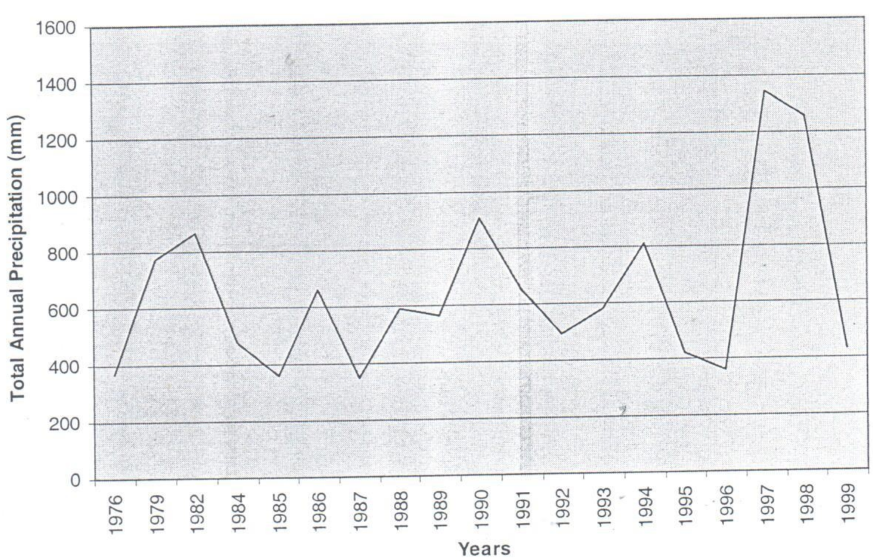
- Bachuma Gate



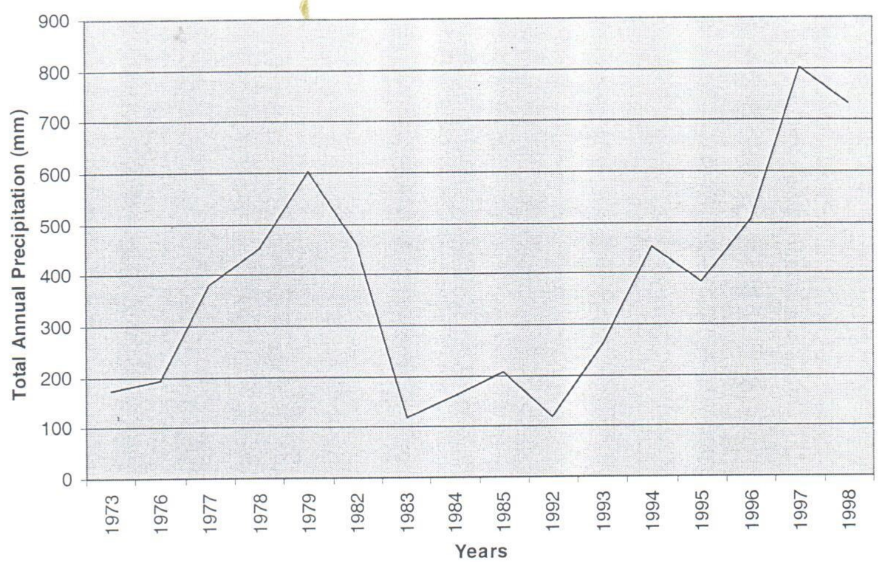
- Manyani Gate



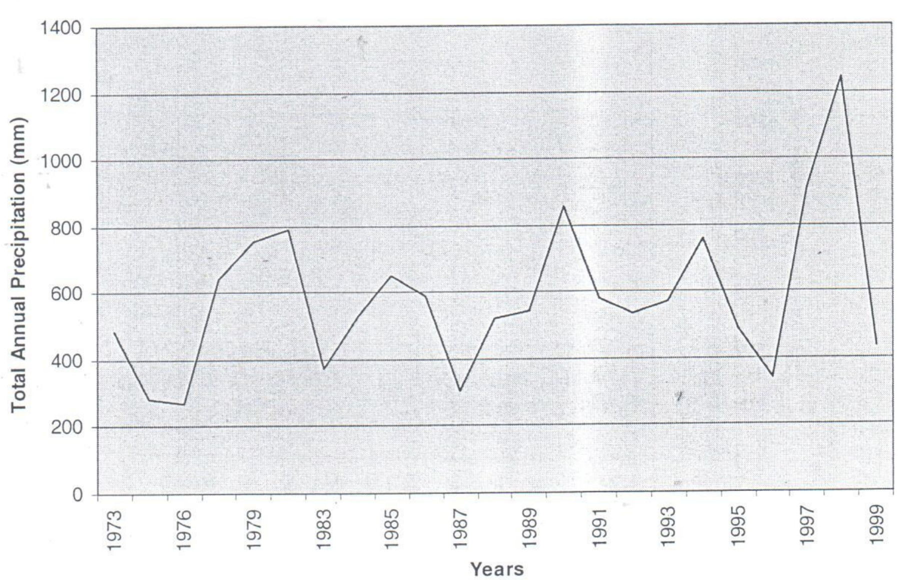
- Ndololo Campsite



- Sala Gate

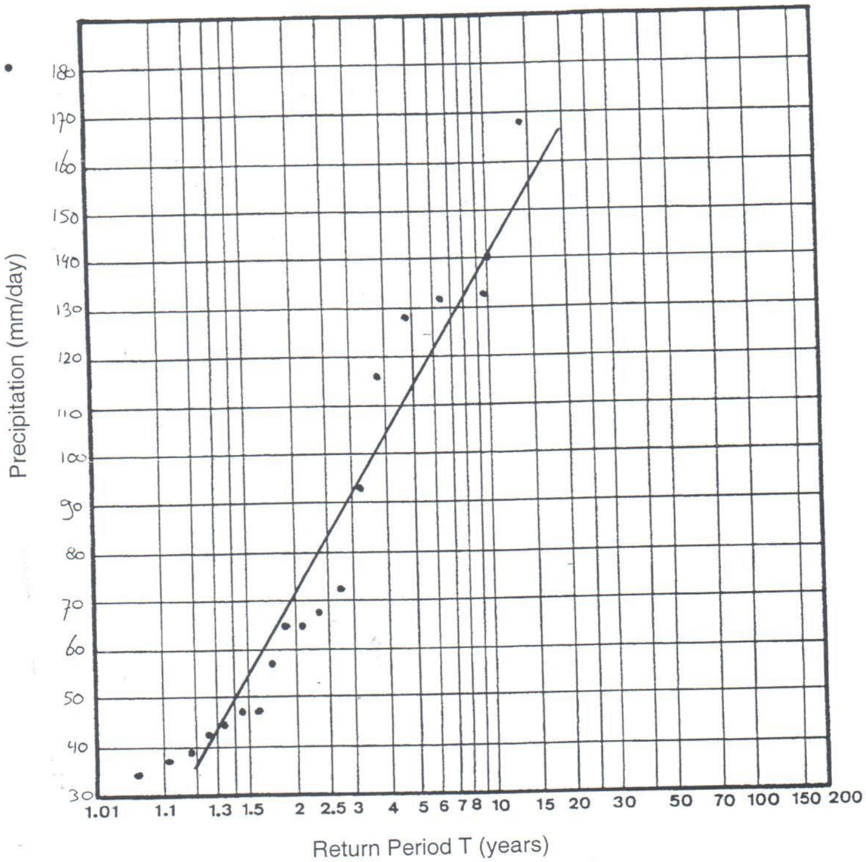


- Tsavo Research Station



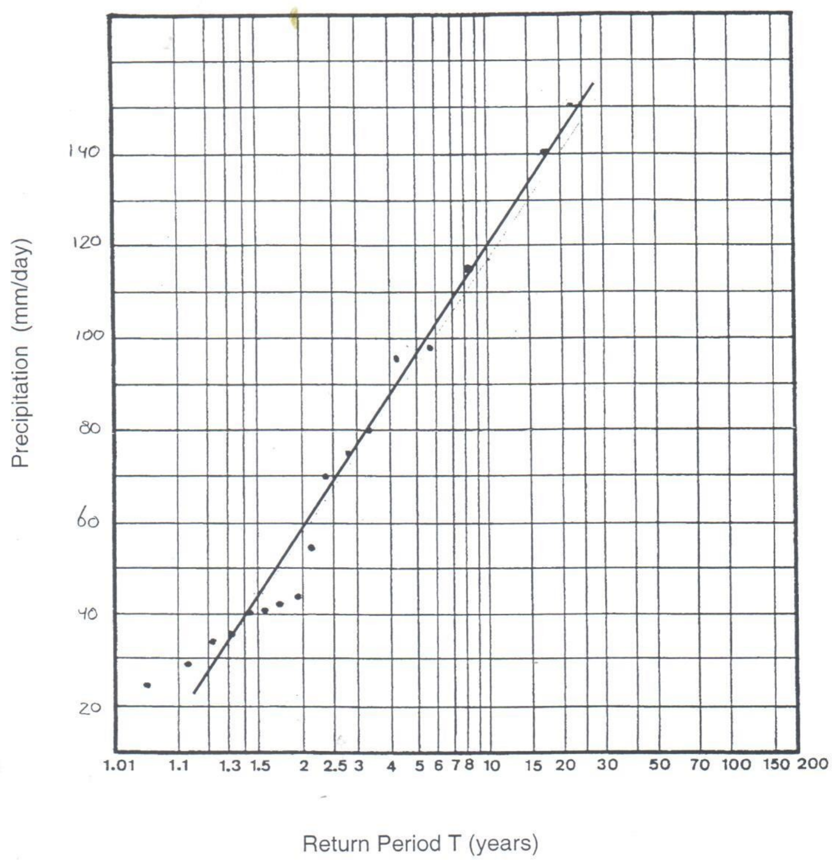
APPENDIX 7 MAXIMUM 24 HOUR RAINFALL DATA: ANNUAL SERIES ON LINEAIR-GUMBEL DISTRIBUTION-PAPER

- Airstrip



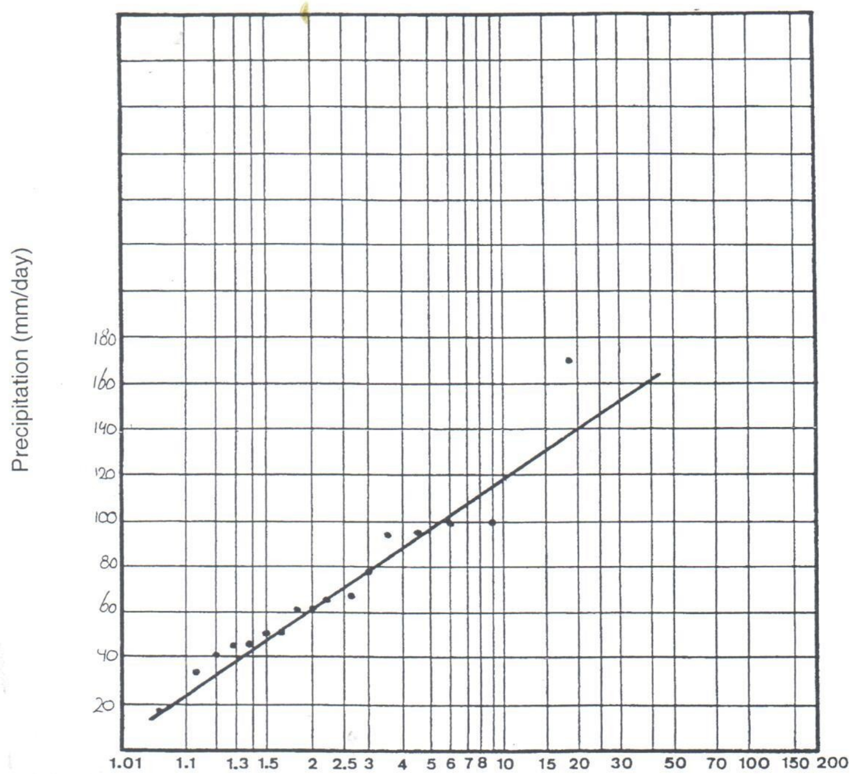
P_{10} = 140 mm/day

- Bachuma Gate



P_{10} = 118 mm/day

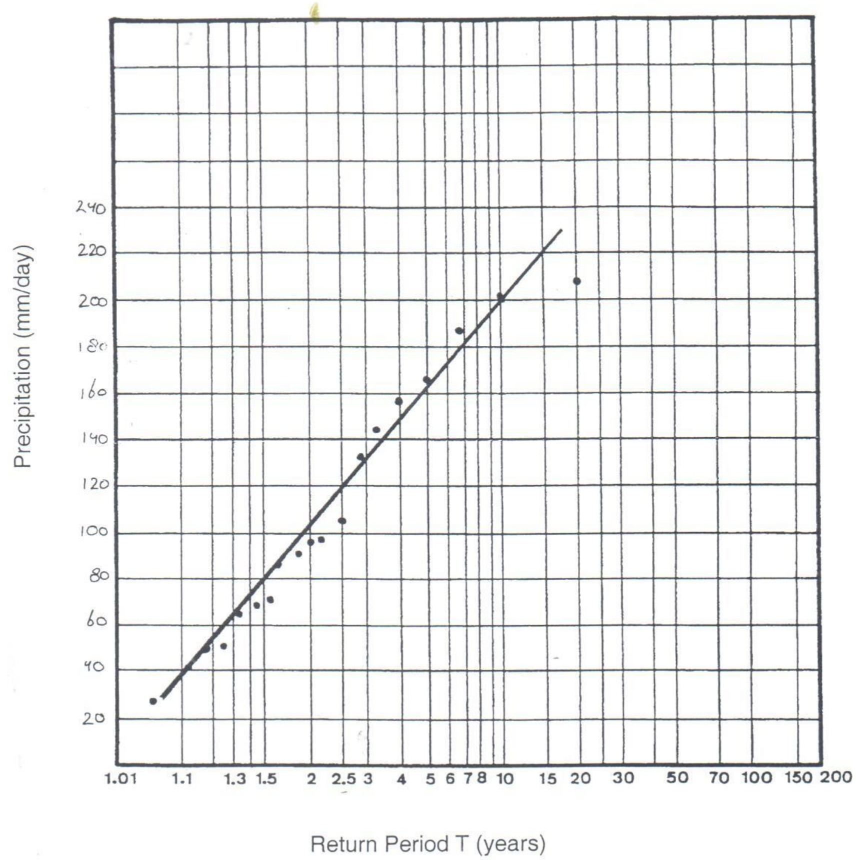
- Manyani Gate



Return Period T (years)

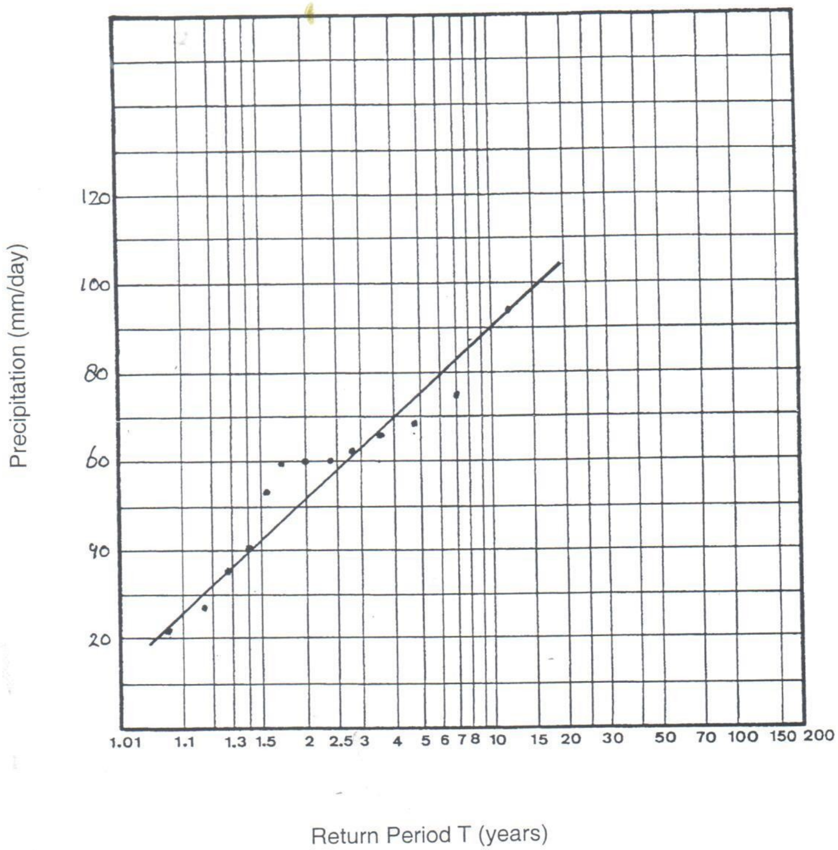
P_{10} = 118 mm/day

- Ndololo Campsite



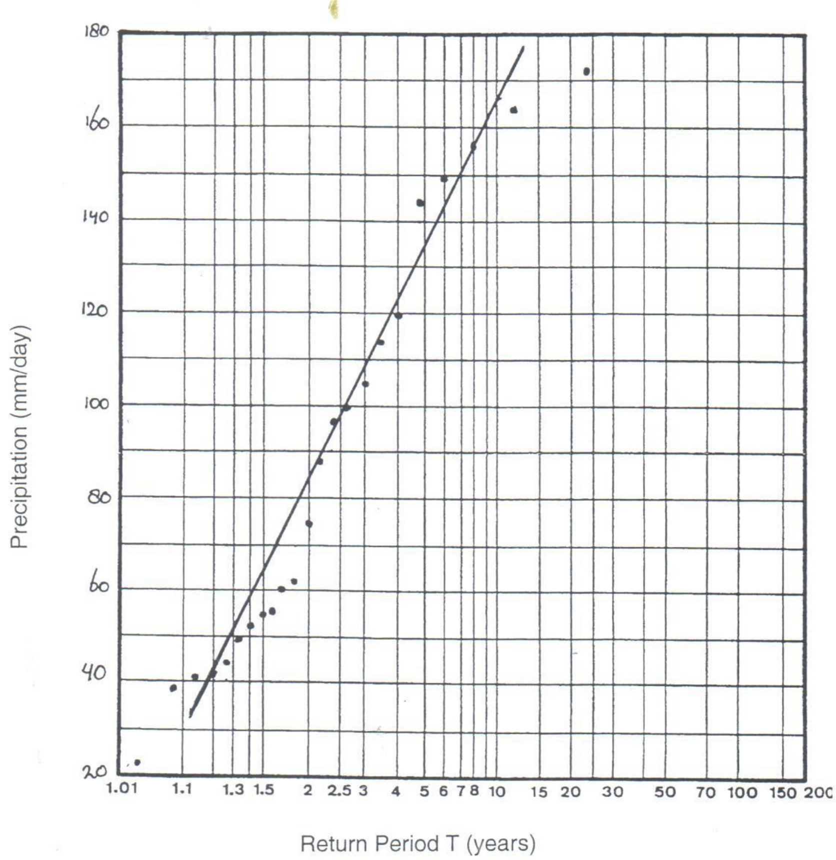
P_{10} = 200 mm/day

- Sala Gate



P_{10} = 90 mm/day

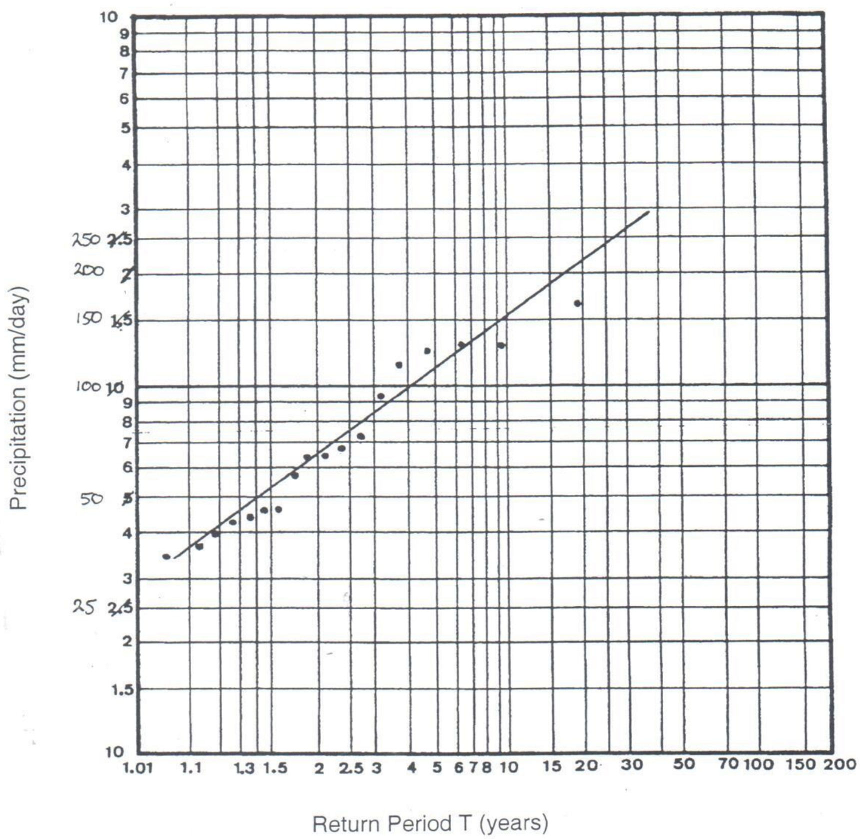
- Tsavo Research Station



P_{10} = 167 mm/day

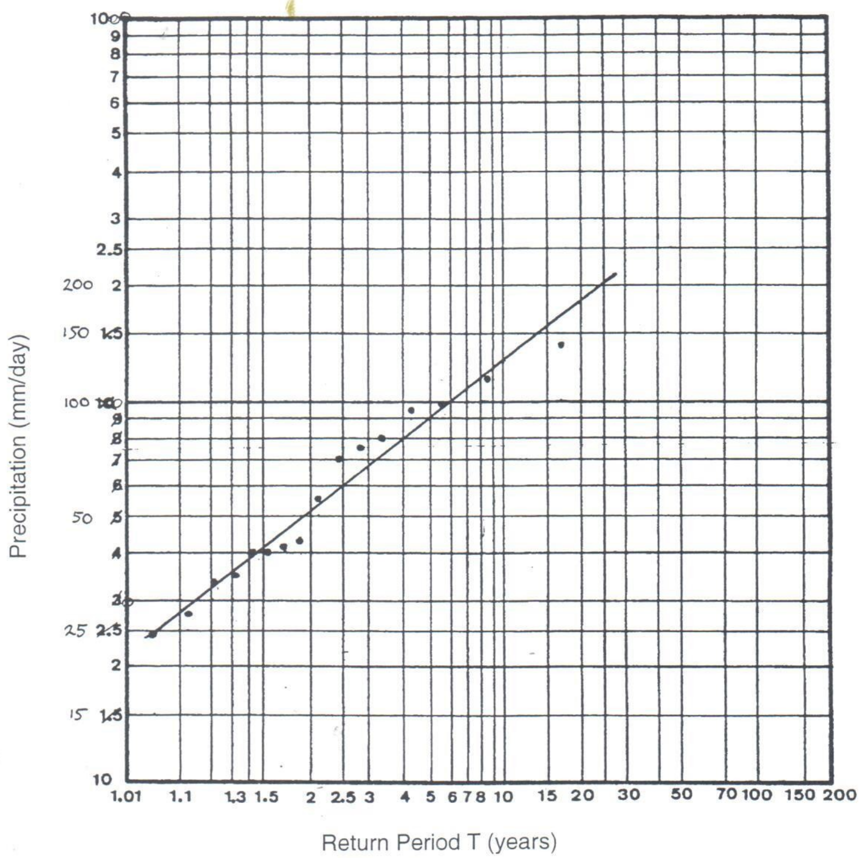
APPENDIX 8 MAXIMUM 24 HOUR RAINFALL DATA: ANNUAL SERIES ON LOGARITHMIC-GUMBEL DISTRIBUTION-PAPER

- Airstrip



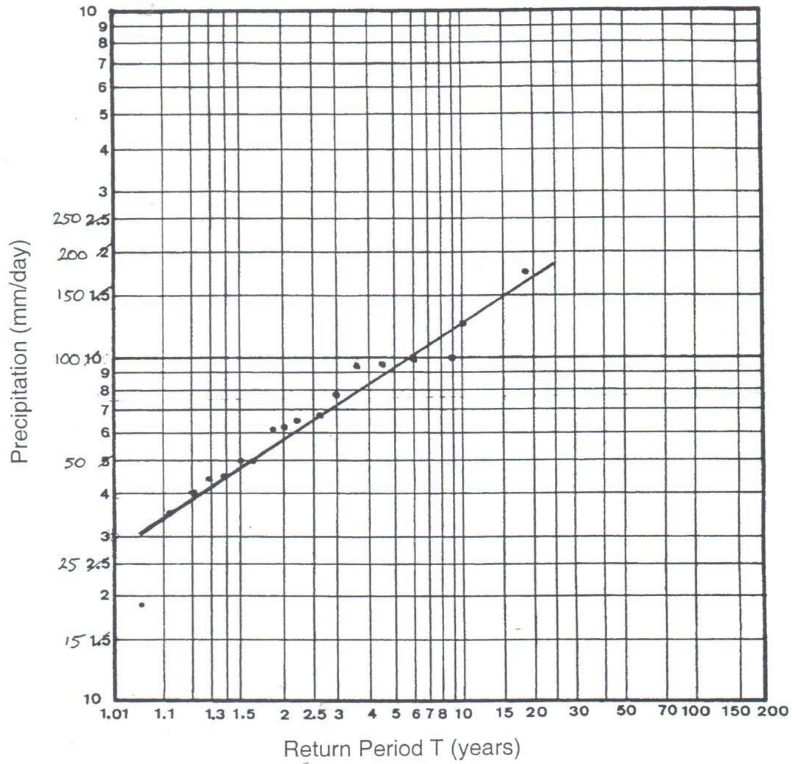
P_{10} = 150 mm/day

- Bachuma Gate



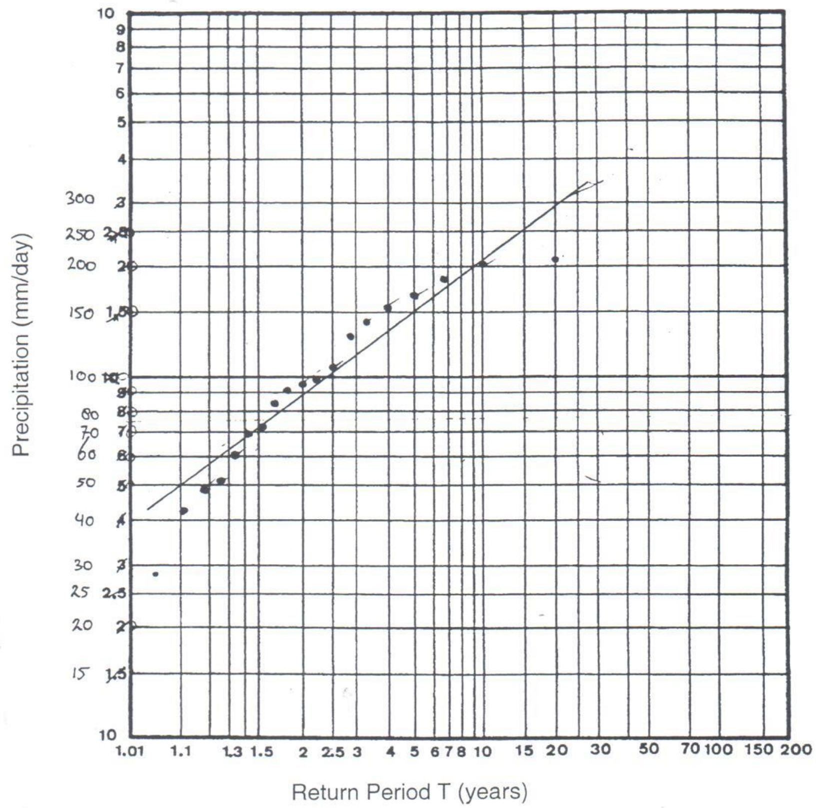
P_{10} = 135 mm/day

- Manyani Gate



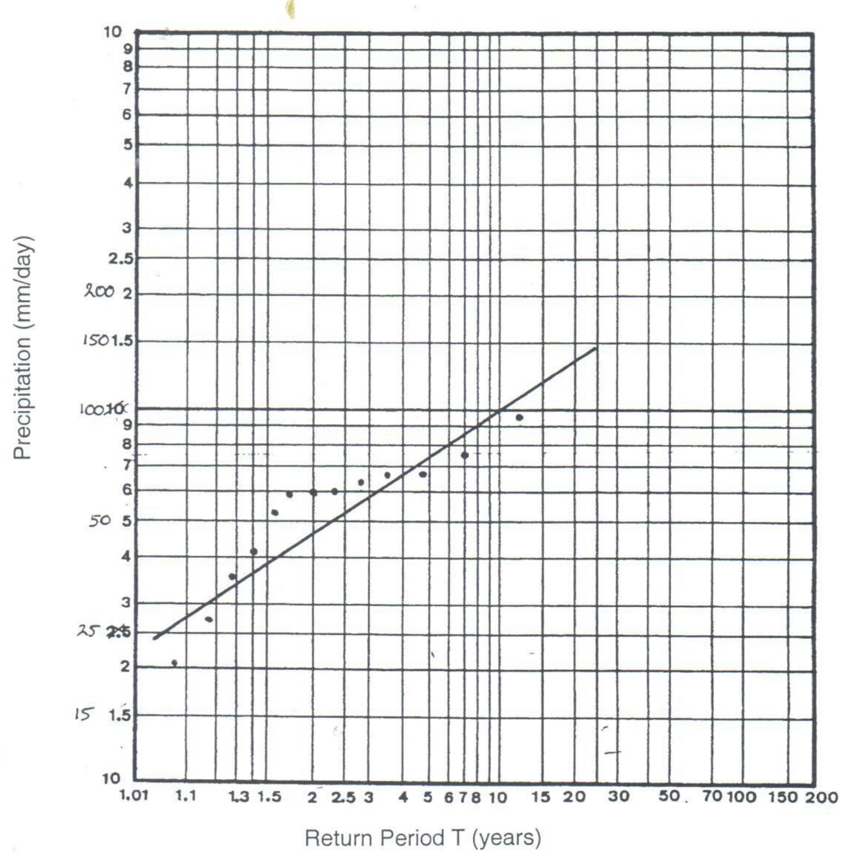
P_{10} = 135 mm/day

- Annual series Ndololo Campsite



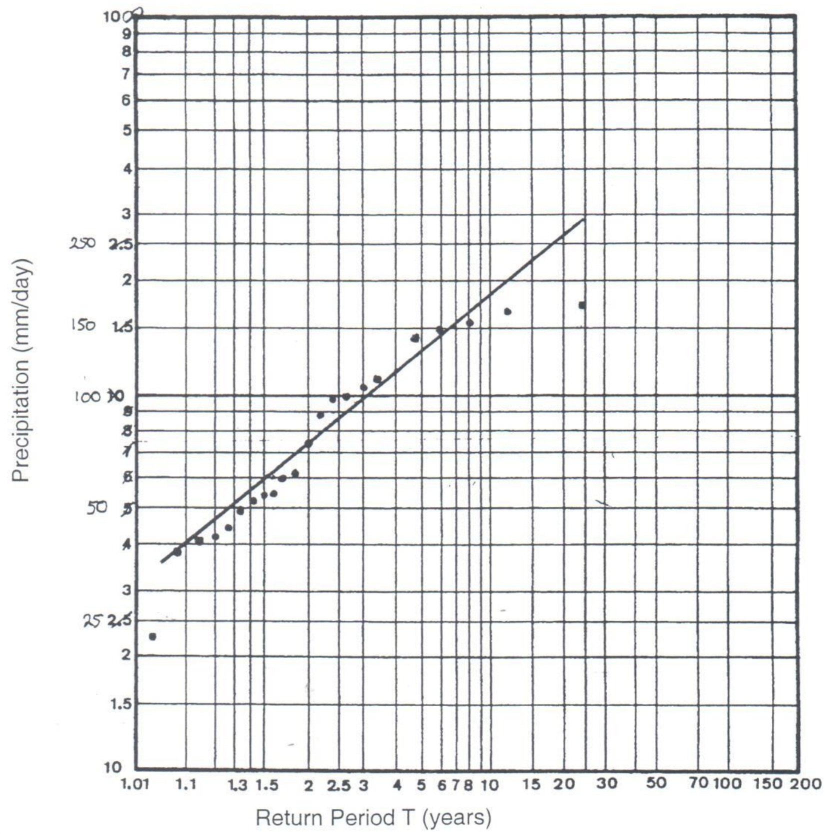
P_{10} = 215 mm/day

- Sala Gate



P_{10} = 100 mm/day

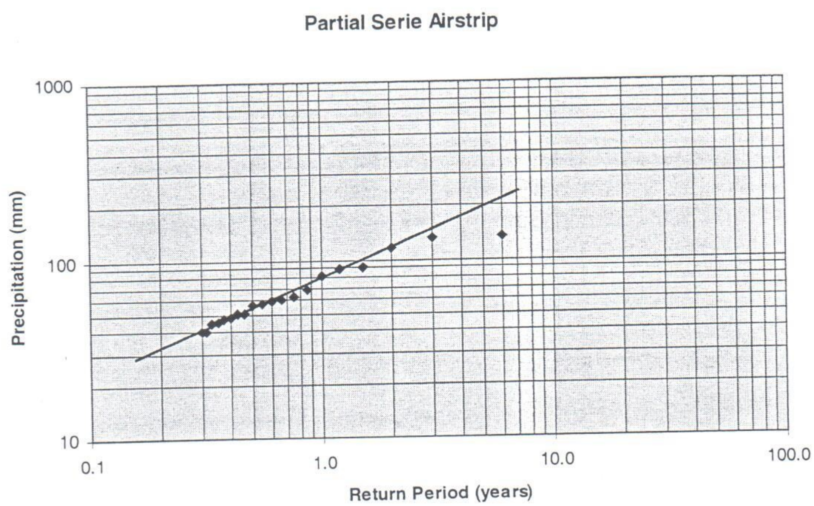
- Tsavo Research Station



P_{10} = 190 mm/day

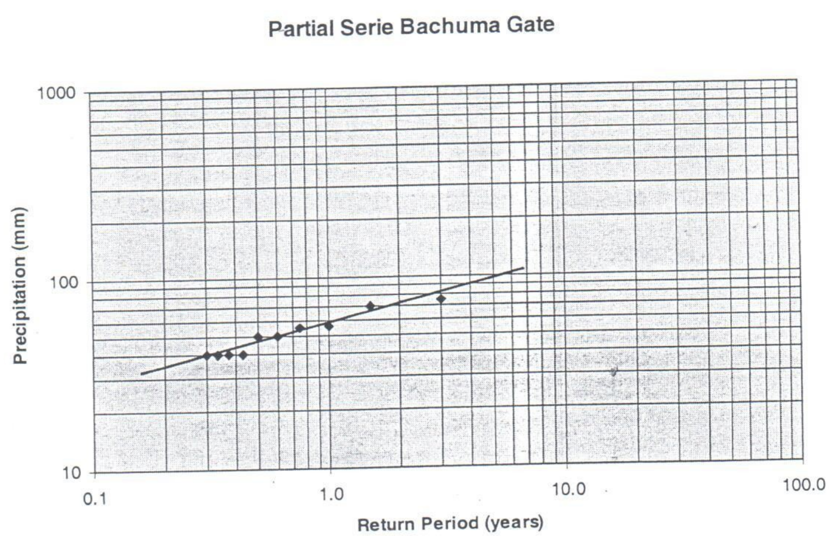
APPENDIX 9 MAXIMUM 24 HOUR RAINFALL DATA: PARTIAL SERIES ON LOG-LOG DISTRIBUTION-PAPER

- Airstrip



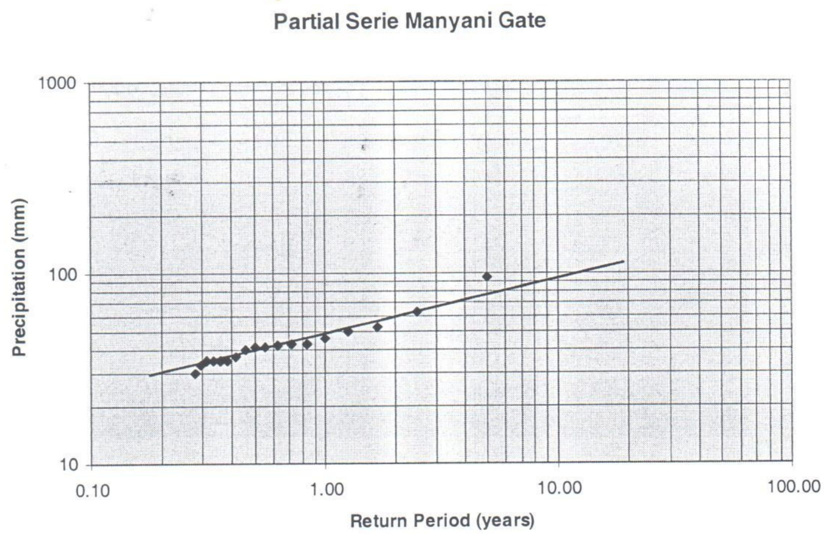
$P_{0.5}$ = 55 mm/day

- Bachuma Gate



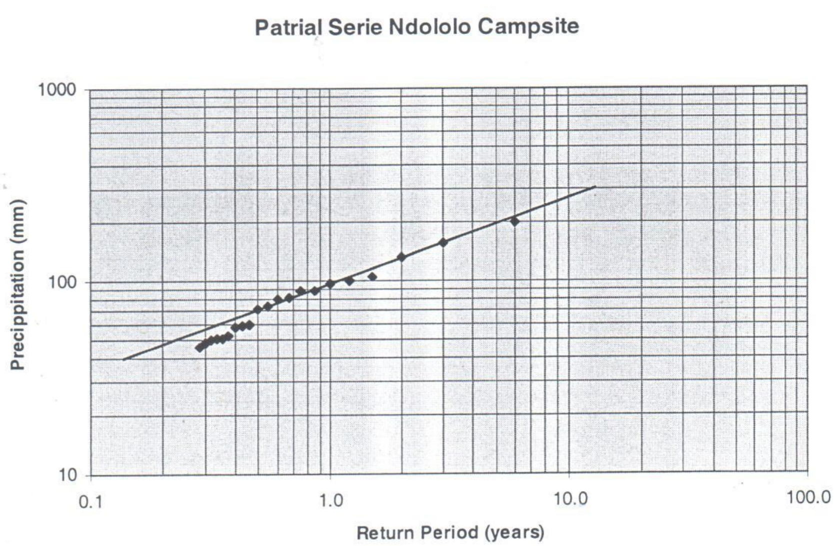
$P_{0.5}$ = 48 mm/day

- Manyani Gate



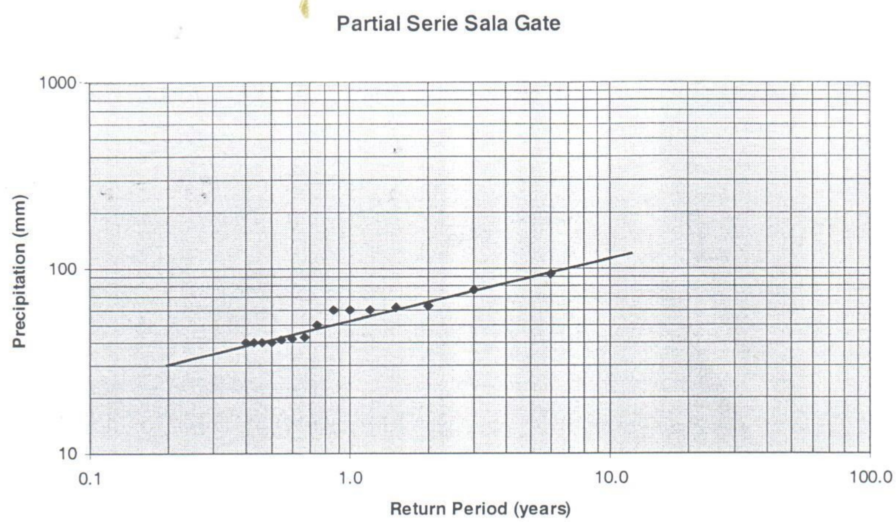
$P_{0.5}$ = 48 mm/day

- Ndololo Campsite



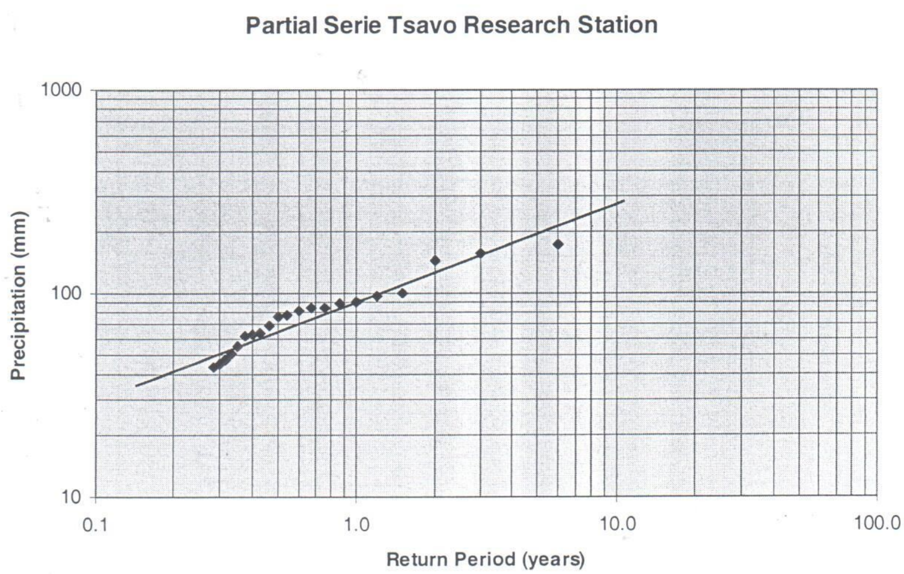
$P_{0.5}$ = 65 mm/day

- Sala Gate

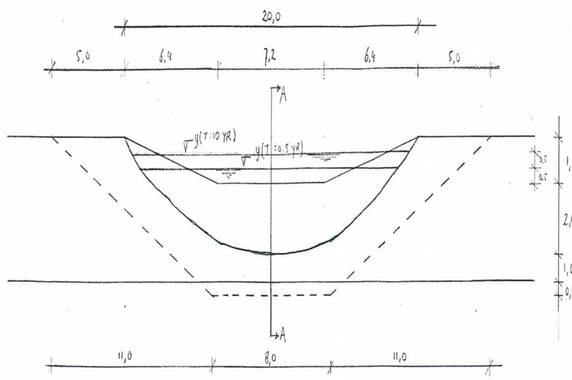


$P_{0.5}$ = 44 mm/day

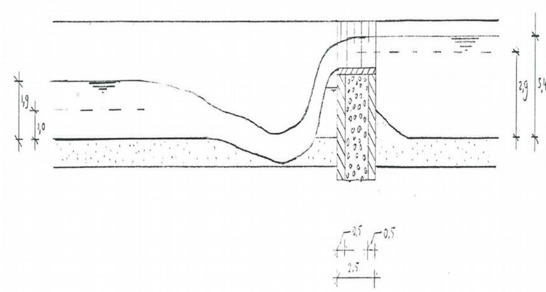
- Tsavo Research Station



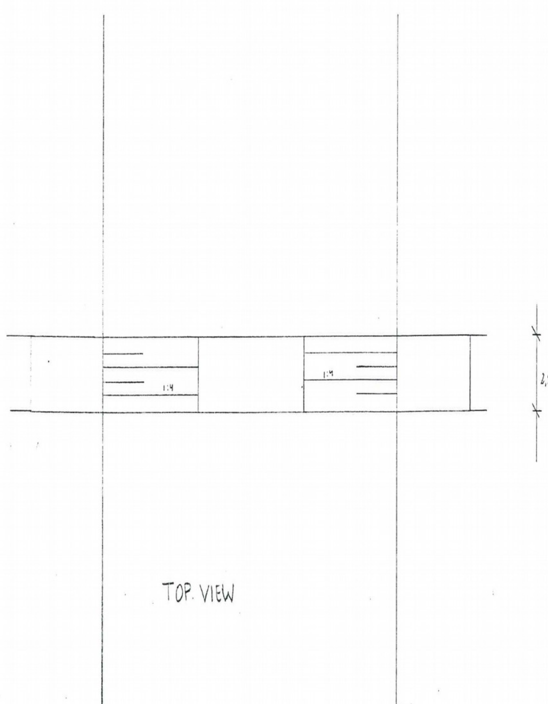
$P_{0.5}$ = 67 mm/day



FRONT VIEW

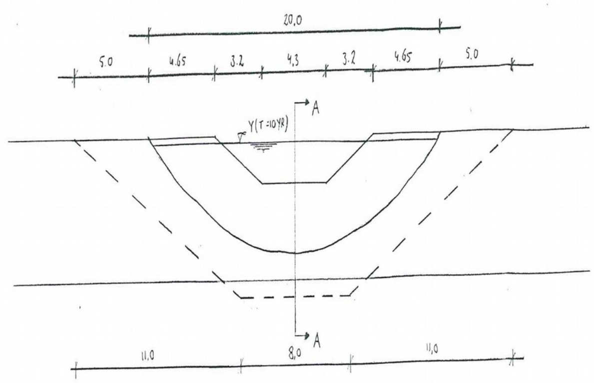


CROSS-SECTION A-A

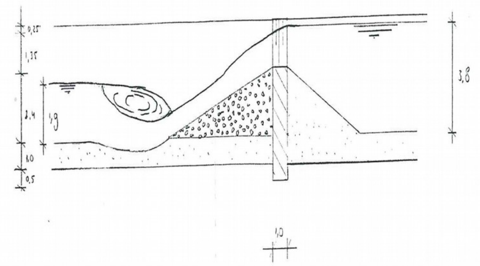


TOP VIEW

SCALE 1:200	DATE 01-06-2000	SIZE A3
Arjen Keuthof	UNIT M	DRAWING NO. 2
"CAUSEWAY" $A = 5 km^2$		
DELFT UNIVERSITY OF TECHNOLOGY	FACULTY OF CIVIL ENGINEERING	



FRONT VIEW



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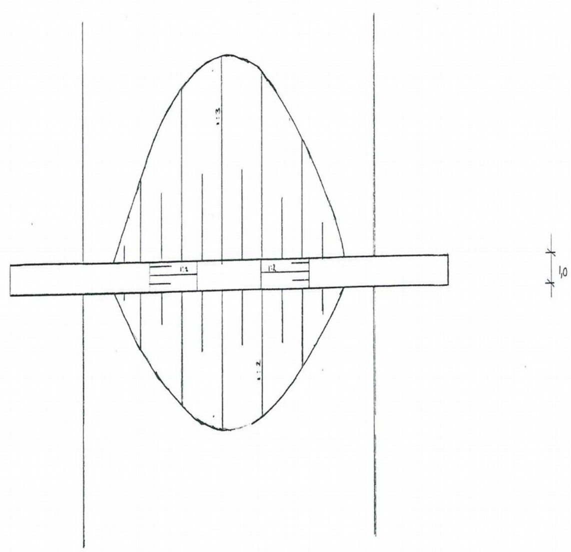

TOP VIEW

SCALE	DATE	SIZE
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Arjen Kruithof	UNIT M	DRAWING NO. 7
"NATURAL DAM" A=5km²		
DELFT UNIVERSITY OF TECHNOLOGY		FACULTY OF CIVIL ENGINEERING

APPENDIX 10 CALCULATIONS AND GRAPHS CURVE NUMBER METHOD: Q_{10}.

- Catchment area is 5 km^2.

I (mm/h)	Sigma P (mm)	Sigma V (mm)	V (mm)	Q_p (m^3/s)
37.5	18.8	0.6	0.6	0.6
37.5	37.5	0.7	0.1	0.1
37.5	56.3	5.0	4.3	4.3
37.5	75.0	12.3	7.3	7.2
37.5	93.8	21.7	9.4	9.2
37.5	112.5	32.7	11.0	10.8
37.5	131.3	44.9	12.2	12.0
37.5	150.0	58.0	13.1	12.9

- Catchment area is 10 km^2.

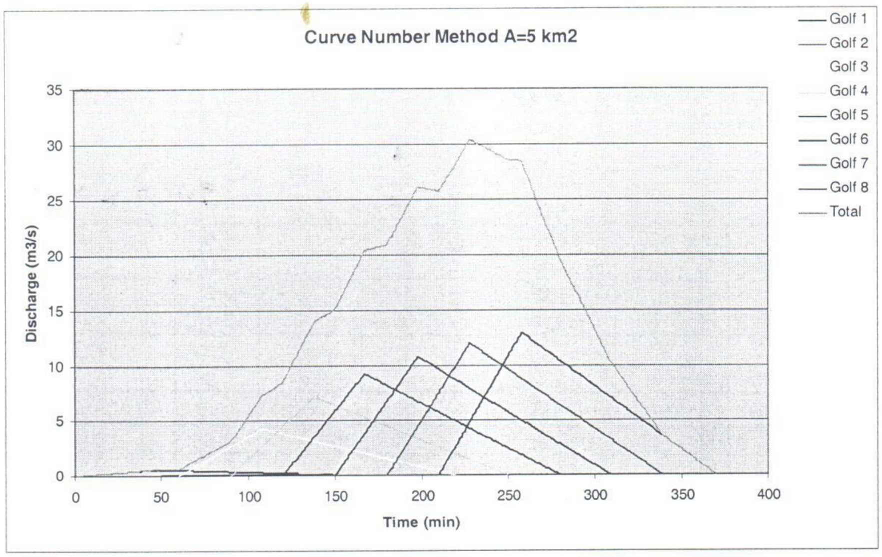
I (mm/h)	Sigma P (mm)	Sigma V (mm)	V (mm)	Q_p (m^3/s)
37.5	18.8	0.6	0.6	0.7
37.5	37.5	0.7	0.1	0.1
37.5	56.3	5.0	4.3	5.2
37.5	75.0	12.3	7.3	8.7
37.5	93.8	21.7	9.4	11.2
37.5	112.5	32.7	11.0	13.1
37.5	131.3	44.9	12.2	14.6
37.5	150.0	58.0	13.1	15.7

- Catchment area is 25 km^2.

I (mm/h)	Sigma P (mm)	Sigma V (mm)	V (mm)	Q_p (m^3/s)
37.5	18.8	0.6	0.6	1.3
37.5	37.5	0.7	0.1	0.3
37.5	56.3	5.0	4.3	9.7
37.5	75.0	12.3	7.3	16.2
37.5	93.8	21.7	9.4	20.9
37.5	112.5	32.7	11.0	24.4
37.5	131.3	44.9	12.2	27.1
37.5	150.0	58.0	13.1	29.2

- Catchment area is 50 km^2.

I (mm/h)	Sigma P (mm)	Sigma V (mm)	V (mm)	Q_p (m^3/s)
37.5	18.8	0.6	0.6	2.1
37.5	37.5	0.7	0.1	0.4
37.5	56.3	5.0	4.3	15.7
37.5	75.0	12.3	7.3	26.2
37.5	93.8	21.7	9.4	33.9
37.5	112.5	32.7	11.0	39.6
37.5	131.3	44.9	12.2	44.0
37.5	150.0	58.0	13.1	47.4


Curve Number Method A=5 km2
Discharge (m3/s)
Time (min)
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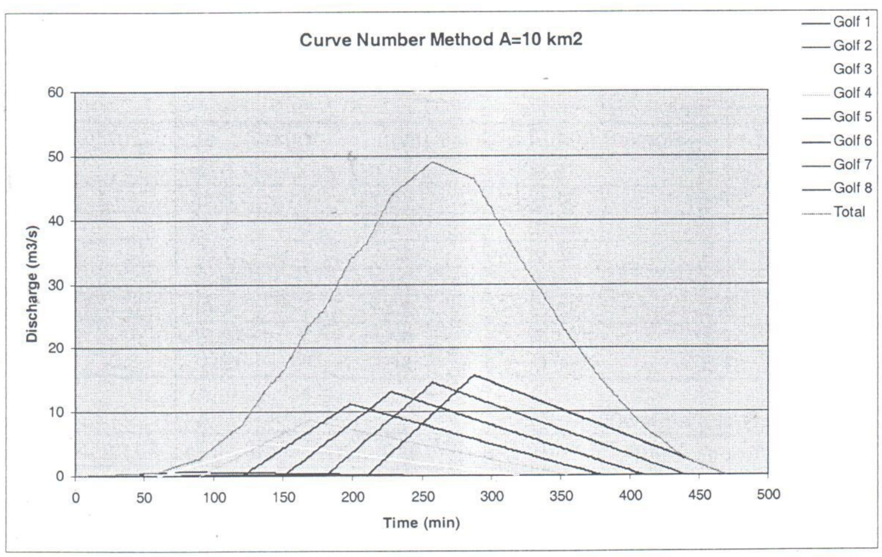
5
10
15
20
25
30
35
50
100
150
200
250
300
350
400
Golf 1
Golf 2
Golf 3
Golf 4
Golf 5
Golf 6
Golf 7
Golf 8
Total


Curve Number Method A=10 km2
Discharge (m3/s)
Time (min)
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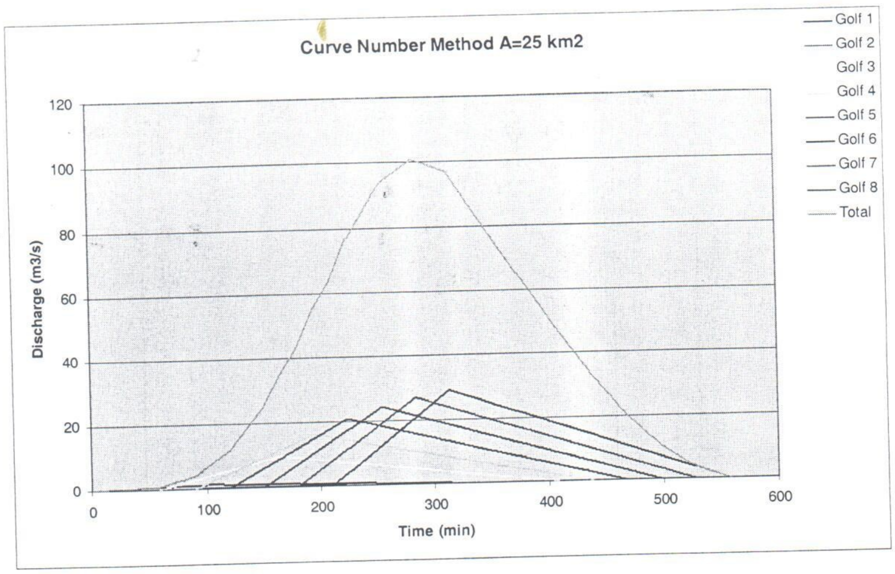
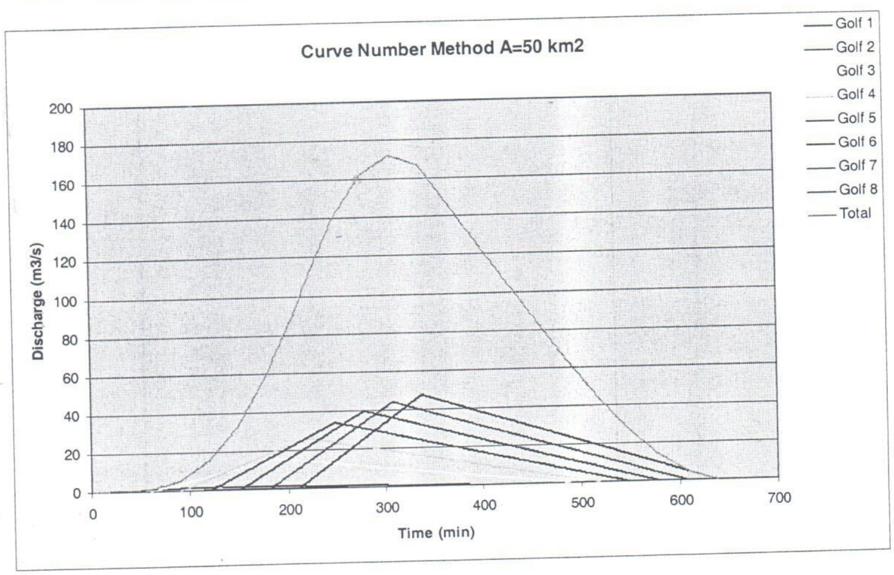
10
20
30
40
50
60
100
150
200
250
300
350
400
450
500
Golf 1
Golf 2
Golf 3
Golf 4
Golf 5
Golf 6
Golf 7
Golf 8
Total


Curve Number Method A=25 km2
Discharge (m3/s)
Time (min)
0
20
40
60
80
100
120
0
100
200
300
400
500
600
Golf 1
Golf 2
Golf 3
Golf 4
Golf 5
Golf 6
Golf 7
Golf 8
Total


Curve Number Method A=50 km2
Discharge (m3/s)
Time (min)
0
20
40
60
80
100
120
140
160
180
200
0
100
200
300
400
500
600
700
Golf 1
Golf 2
Golf 3
Golf 4
Golf 5
Golf 6
Golf 7
Golf 8
Total

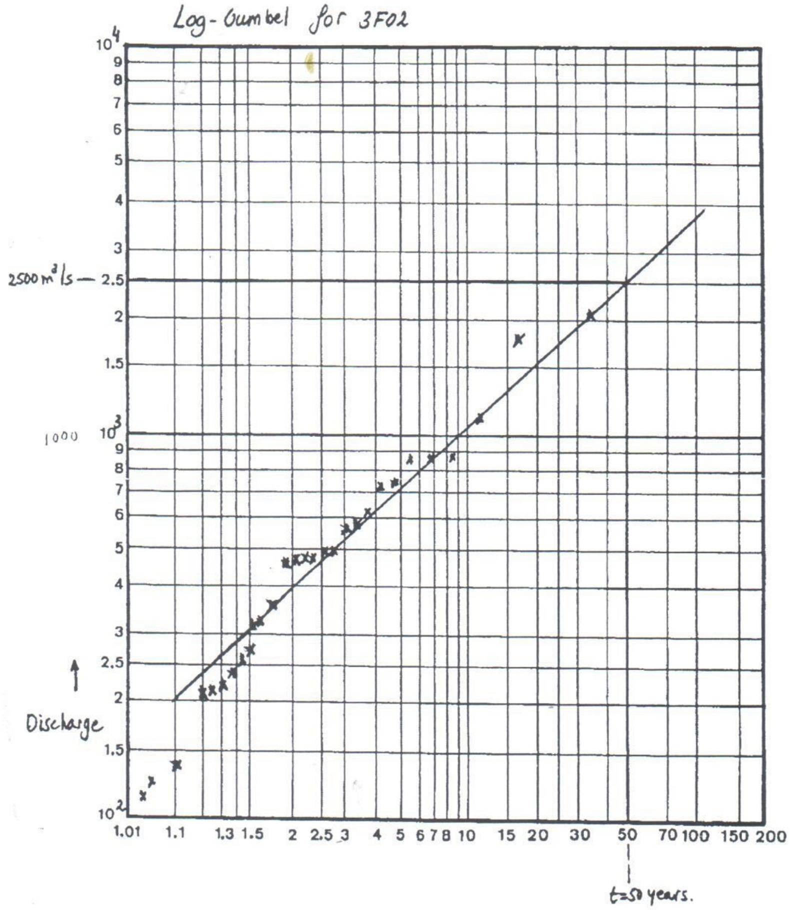
APPENDIX 11 MAP 'MUDANDA' OF THE SURVEY OF KENYA

APPENDIX 14 LOG-GUMBEL GRAPH FOR FOUR GAUGE STATIONS

Maximum discharge values for 3F02

year	discharge	ordenumber	Return period
1978	2110	1	33
1961	1800	2	16.5
1986	1080	3	11
1975	877	4	8.25
1977	877	5	6.6
1982	877	6	5.5
1971	751	7	4.714286
1981	721	8	4.125
1988	619	9	3.666667
1962	590	10	3.3
1967	574	11	3
1979	496	12	2.75
1985	496	13	2.538462
1963	493	14	2.357143
1989	491	15	2.2
1958	486	16	2.0625
1968	470	17	1.941176
1974	465	18	1.833333
1970	364	19	1.736842
1956	323	20	1.65
1976	282	21	1.571429
1965	269	22	1.5
1957	257	23	1.434783
1959	249	24	1.375
1964	217	25	1.32
1983	211	26	1.269231
1980	202	27	1.222222
1972	187	28	1.178571
1966	148	29	1.137931
1969	143	30	1.1
1984	132	31	1.064516
1960	122	32	1.03125

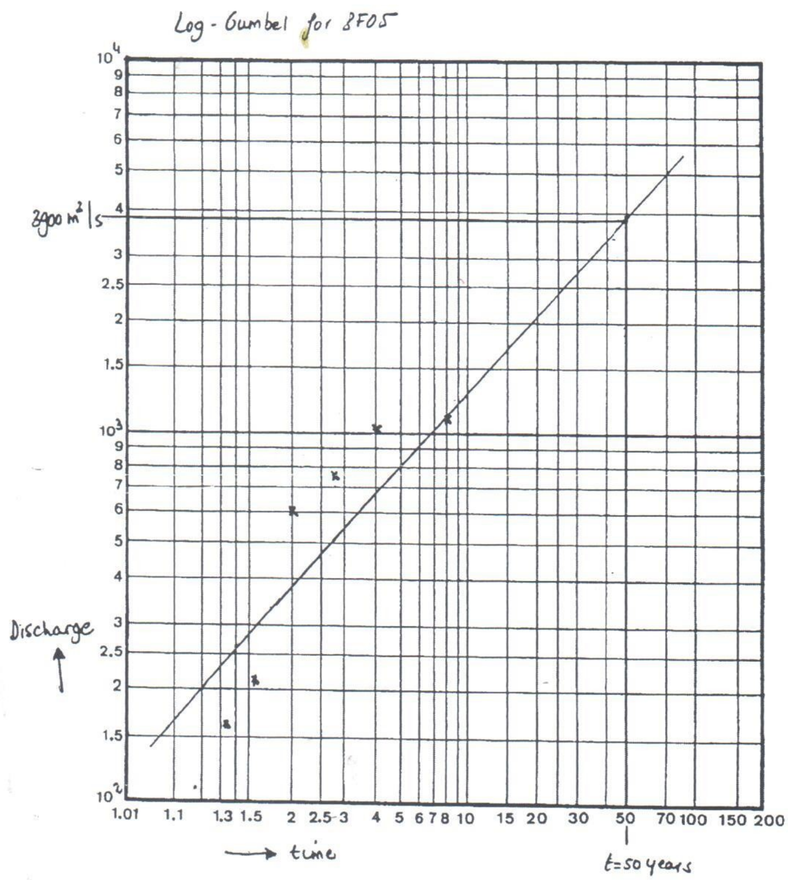
Data source : Ministery of Water development (Maji House, Nairobi, Kenya)


Log-Gumbel for 3F02
2500 m³/s
1000
Discharge
t=50 years.

Maximum discharge values + returnperiod for 3F05

year	discharge	ordernumber	return period
1978	1116.5	1	8
1971	1040.9	2	4
1972	771	3	2.666667
1970	602.198	4	2
1973	210	5	1.6
1979	165	6	1.333333
1969	64.113	7	1.142857

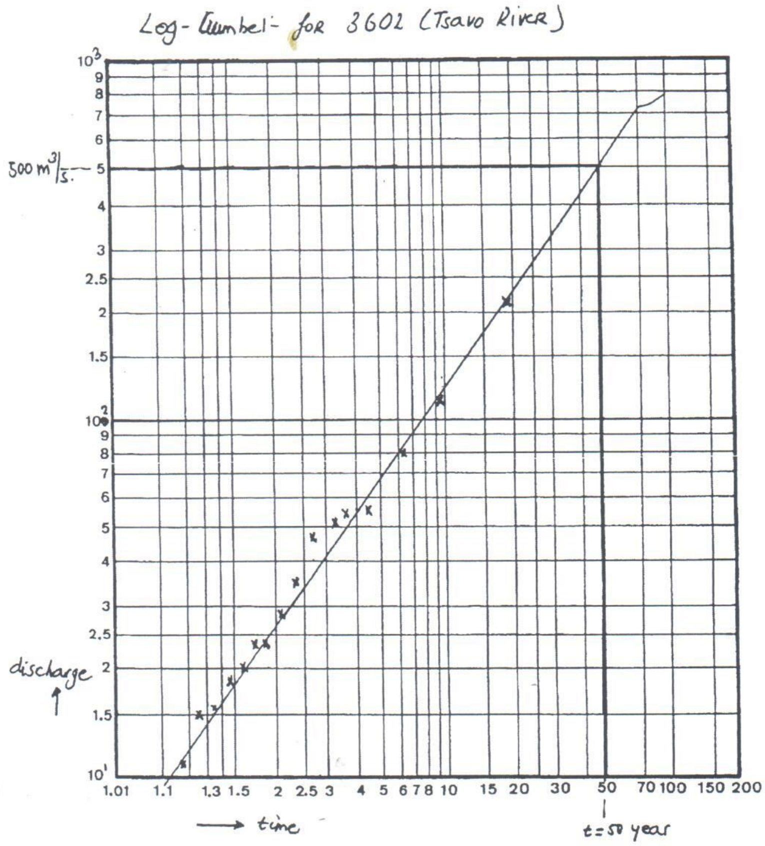
Data source : Ministery of Water development (Maji House, Nairobi, Kenya)


Log - Gumbel for 3F05
3900 m³/s
Discharge
time
t=50 years
1.01 1.1 1.3 1.5 2 2.5 3 4 5 6 7 8 10 15 20 30 50 70 100 150 200

Maximum discharge values + returnperiod for 3G02

	discharg e	ordernumber	return period (years)
1973	220.9	1	19
1967	123.5	2	9.5
1982	80	3	6.333333
1972	54.9	4	4.75
1951	54	5	3.8
1968	50.7	6	3.166667
1983	47	7	2.714286
1959	36.4	8	2.375
1963	29.3	9	2.111111
1954	23.8	10	1.9
1955	23.8	11	1.727273
1965	20	12	1.583333
1950	18	13	1.461538
1969	15.3	14	1.357143
1966	15	15	1.266667
1964	11	16	1.1875
1953	8.2	17	1.117647
1952	8	18	1.055556

Data source : Ministery of Water development (Maji House, Nairobi, Kenya)



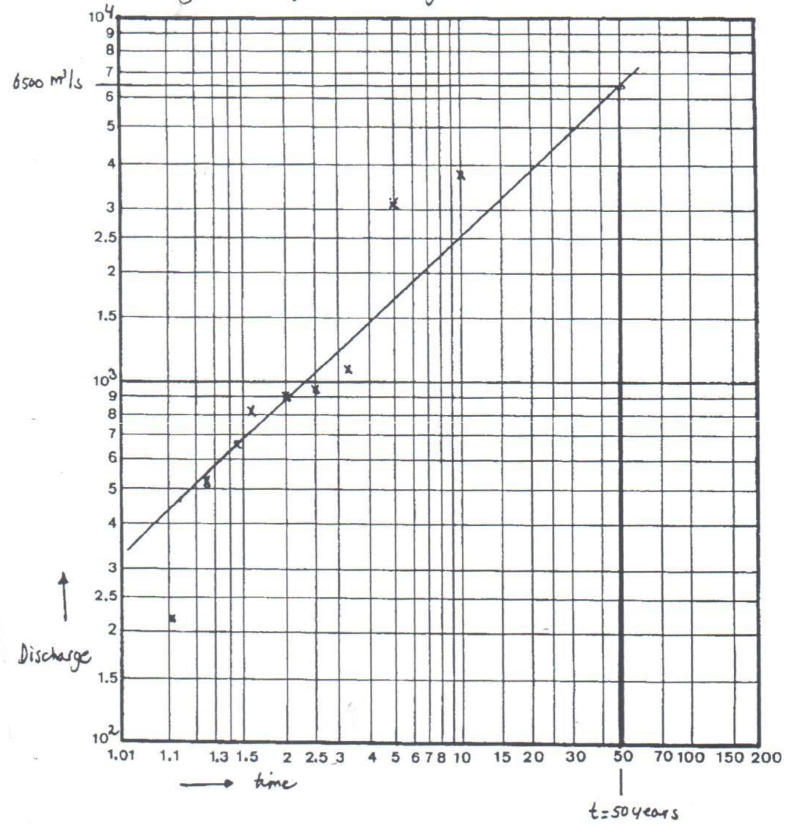
Log-Gumbel- for 3602 (Tsavo River)
500 m³/s.
discharge
time
t=50 year

Maximum discharge values + returnperiod for 3HA08

	year	discharge	ordernumber	return period
3841	1979	3841	1	10
3149	1978	3149	2	5
1057.9	1977	1057.9	3	3.333333
975	1976	975	4	2.5
899	1975	899	5	2
822	1982	822	6	1.666667
674	1980	674	7	1.428571
519.7	1984	519.7	8	1.25
221	1981	221	9	1.111111

Data source : Ministery of Water development (Maji House, Nairobi, Kenya)

Log-Gumbel for 3HA08 (Lugards Falls).



APPENDIX 15 ESTIMATION OF PROBABLE FLOOD DISCHARGES ON REGIONAL FLOOD FREQUENCY BASES

Table B7.2 Estimation of Probable Flood Discharges on Regional Flood Frequency Curve Basis

Drainage Area	Major River	Equation Deriving M.A.F.	Multiplier to M.A.F. for Estimation of Discharges of Each Return Period					
			5	10	20	25	50	100 year
1	Nzoia	$Q = 0.080\ A^{0.898}$	1.57	2.09	2.68	2.92	3.78	4.82
	Yala	$Q = 0.780\ A^{0.644}$	1.60	2.08	2.61	2.82	3.59	4.48
	Nyando	$Q = 0.093\ A^{0.960}$	1.70	2.35	2.99	3.28	4.33	5.58
	Sondu	$Q = 0.345\ A^{0.756}$	1.77	2.32	2.94	3.13	4.06	5.39
	Kuja/Migori	$Q = 0.205\ A^{0.849}$	1.48	1.84	2.23	2.37	2.83	3.33
	Others	$Q = 3.005\ A^{0.363}$	1.47	1.84	2.17	2.30	2.63	3.13
2	—	$Q = 0.030\ A^{1.069}$	1.71	2.40	3.15	3.49	4.58	6.00
3	Athi	$Q = 0.249\ A^{0.835}$	1.87	2.62	3.43	3.79	5.29	7.22
4	Tana	$Q = 0.480\ A^{0.794}$	1.52	1.93	2.36	2.52	3.08	3.71
5	Ewaso Ngiro (North)	$Q = 0.812\ A^{0.564}$	1.92	2.63	3.39	3.79	5.35	7.45

Note: M.A.F.: Mean Annual Flood
Q : Flood Discharge (m^3/s)
A : Catchment Area (sq km)

Table B7.3 Selected Floods and Stations

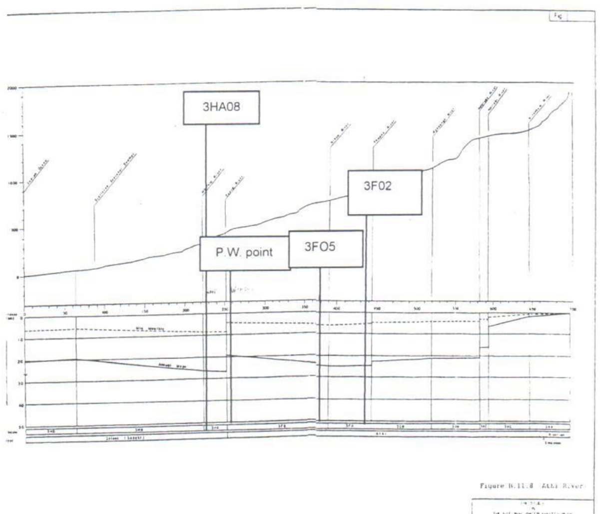
Drainage Area	River	Flood	Gauging Stations	
			Water Level	Rainfall
1	Nzoia	Nov. 1977	1EE1	8935133 (Eldoret)
	Yala	Aug. 1979	1FG1	8934087 (Kobujoi)
	Nyando	Dec. 1982	1GD3	9035244 (Kericho)
	Sondu	Dec. 1982	1JG1	9035244 (Kericho)
	Kuja/Migori	Apr. 1985	1KB5	9134025 (Migori)
2	Turasha	Apr. 1977	2GC4	9036002 (Naivasha)
3	Athi	Dec. 1986	3F2, 3DA2	9137098 (Machakos)
4	Tana	Dec. 1968	4G1, 4F13, 4EA7, 4DD2, 4AA5	8937051 (Meru) 9036017 (Nyeri) 9037050 (Embu)
5	Ewaso Ngiro	Apr. 1968	5E3, 5D5, 5AC8, 5BE20, 5BE4	9036135 (01 Joro Orok) 9036260 (Lamuria)

BT-8

Source : The Study on the National Water Master Plan, page BT-8, Japan International Cooperation Agency, Kenya 1992

APPENDIX 16 AVERAGE DISCHARGES IN ATHI RIVER

Figure of Average Discharge along Athi River



Source : The Study on the National Water Master Plan, page B.11, Japan International Cooperation Agency, Kenya 1992

APPENDIX 17 WATER LEVELS IN P.W. AREA

Starting downstream the energy and waterline are constructed.

Cross-sections D-D
The section most downstream is assumed to be uniform. This means that that Strickler equation can be used.

$$Q = k\,A\,R^{2/3}\,i^{1/2}$$
$$Q = Q_{channel} + Q_{forelands}$$

with
$Q = Q_{max} = 4500\ m^3/s$
$k_{channel} = 40$
$k_{forelands} = 35$
$i = i_{average}$ for both channel and forelands
$A_{channel} = (25 + 5{*}h_0)h_0 + (25{+}10h_0)(h{-}h_0)$
$A_{forelands} = (110 + 5(h{-}h_0))(h{-}h_0)$
$R_{channel}\ (25 + 5{*}h_0)h_0 + (25{+}10h_0)(h{-}h_0)/\ (25{+}2h_0\sqrt{26})$
$R_{forelands} = (110 + 5(h{-}h_0))(h{-}h_0)/(110{+}2(h{-}h_0)\sqrt{26})$

When $Q = Q_{max} \rightarrow h > h_0 \rightarrow h_0 = h_{0max} = 1{,}5$ m
$A_{channel} = 48.75 + 40(h-1.5) = 40h - 11.25$
$A_{forelands} = 5h^2 + 95h - 153.75$
$R_{channel} = (40h - 11.25)/25 + 3\sqrt{26}$
$R_{forelands} = 5h^2 + 95h - 153.75/110 + (2h - 3)\sqrt{26}$

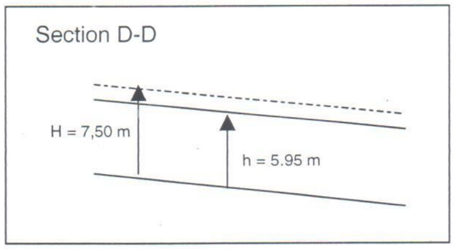
To calculate h, a small excel-programme is constructed, and by trail and error h is found.

When h = 5.957 meter $Q = 4500\ m^3/s$ and the currents in channel as well as forelands are sub-critical

A_{tot}	816.6242
u_{avg}	5.510825
$u_{channel}$	7.220073
$u_{forelands}$	4.852659
Froude	$Fr = u/\sqrt{gd}$
$Fr_{channel}$	0.944481
$Fr_{forelands}$	0.733878

Energy-level is now calculated with

$\boldsymbol{H = u^2/2g + h}$ $= 5{,}51^2/2g + 5.957 = 7{,}50$ meter

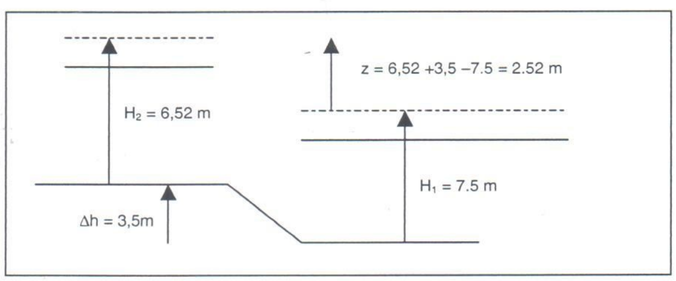


Transition from section D to Section C

As mentioned above the run-off formula depends on z.
First H is calculated with $Q = 1.8\ H^{3/2}\ b \rightarrow H^{2/3} = Q/(1.8*b)$
with
$Q = 4500\ m^3/s$
$b = 150$ m (see table above)

$H^{2/3} = 4500/1.8*150 \rightarrow H = 6.52$ meter

To see if this formula is valid the value of z is defined
$z = H_2 + \Delta h - H_1 = 6.52 + 3.5 - 7.5 = 2.52$ meter
$z > 1/3\ H$ with $H = H_2 = 6.52 \rightarrow 1/3\ H = 2.17 < 2.52$ so this formula is valid



h can be found with $H = u^2/2g + h$ and $Q = u*A$
with
$H = 6.52$ m
First $H_{uniform}$ is calculated:

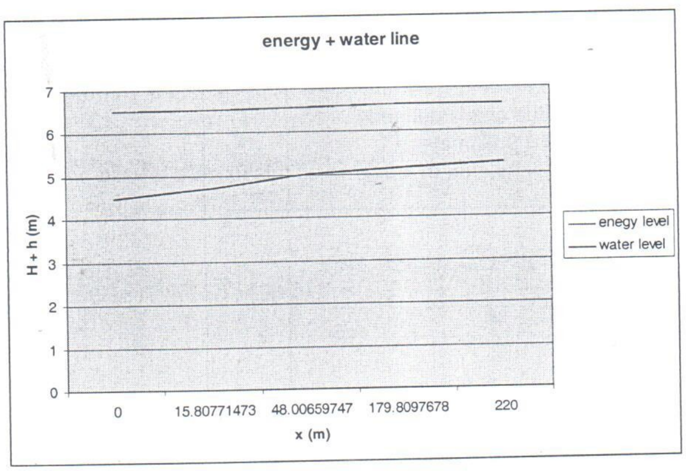
H	5.254
A_{tot}	863.1061
u_{avg}	5.214367
$u_{channel}$	6.715682
$u_{forelands}$	4.891868
$Fr_{channel}$	0.935429

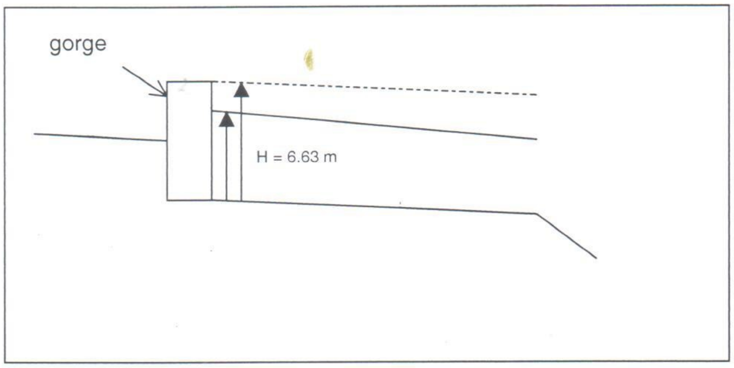
$Fr_{forelands}$	0.757254
H = h + u2/2g	
H	6.639811

So $H_{overlaat} < H_{uniform} \rightarrow$ drawing down takes place

Now the standard step methods is used to find water and energylevels
The calculation is done in excel. The following results are obtained
Start data
$H_1 = 6.52$ m
$u_1 = 6.3$ m/s
$h = H - u^2/2g = 6.52 - (6.3^2/2g) = 4{,}49$ meter
$\Delta H = 0.03$

H	X	U	h
6.52	0	6.3	4.497064
6.55	15.80771	5.99	4.721249
6.58	48.0066	5.5375	5.01711
6.61	179.8098	5.35	5.151157
$H_{uniform}$		$u_{uniform}$	$h_{uniform}$
6.63	220	5.21	5.254



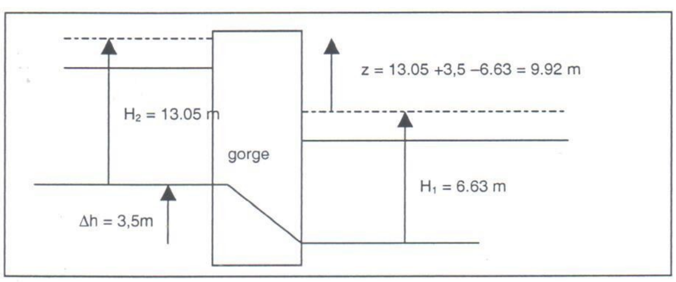


Situation in gorge:

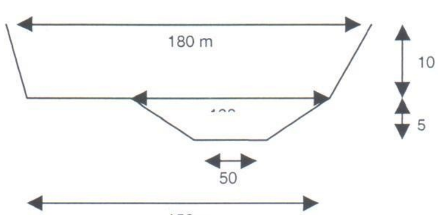
As mentioned above the run-off formula depends on z.
First H is calculated with $Q = 1.8\ H^{3/2}\ b \rightarrow H^{2/3} = Q/(1.8 * b)$
with
$Q = 4500\ m^3/s$
$b = 150$ m (see table above)

$H^{2/3} = 4500/1.8 * 150 \rightarrow H = 6.52$ meter

To see if this formula is valid the value of z is defined
$z = H_2 + \Delta h - H_1 = 13.05 + 3{,}5 - 6.63 = 9.92$ meter
$z > 1/3\ H$ with $H = H_2 = 13.05 \rightarrow 1/3\ H = 4.35\ m < 9.92\ m$ so this formula is valid



Just upstream of the gorge the river makes a sharp curve. The cross-section is schematized as A_2-A_2



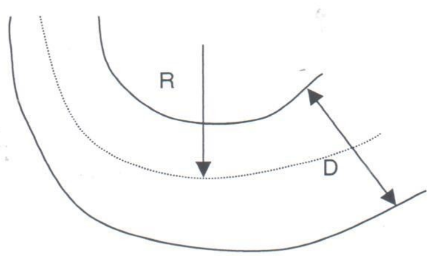
H is supposed to be sub-critical and $H_{uniform}$ will be calculated

h	7 m
A	686m^2
u	6.56 m/s
H	***= h +u2/2g***
H	9.19m
Fr	***= u/√(gd)***
Fr	0.797389

$H_{overlaat} > H_{uniform}$ = backing up takes place
With the standard step method, H and h can be calculated. in order to find friction losses.

H	X	u	h
13,05	0	2,795	12,65183
13	16,94348	2,815	12,59611
12,95	33,91775	2,827	12,54266
12,9	50,92149	2,844	12,48775
12,85	67,95863	2,861	12,43281
12,8	85,02987	2,878	12,37783
12,75	102,1366	2,896	12,32254
12,7	119,2811	2,915	12,26691

Beside friction losses, energy is lost while the river makes a sharp curve. Bend losses depend on the water velocity, radius, diameter and angle. The figure below shows the angle and radius.



θ	90°
R/D	1

In 'Vloeistomechanica' Battjes, 1997 the energy loss is estimated to be

$\Delta H = \xi * u^2/2g$

with
u = water velocity upstream
ξ = factor depending on angle and radius an diameter

R/D θ	1
90°	0.21

So $\xi = 0.15 \rightarrow \Delta H = 0.15u^2/2g$

To find u an iteration process has to be done. First u is assumed to be $u_{uniform}$ (upstream).

Upstream of the bend the river flows through a narrow channel. At the transition between these two sections the also gives energy losses.
In order to find this loss it has to be considered whether the flow in the widening is free or merged.

Free Flow

As mentioned before free flow occurs when
Fr =1 and z > 1/3 H
Then the run-off formula:

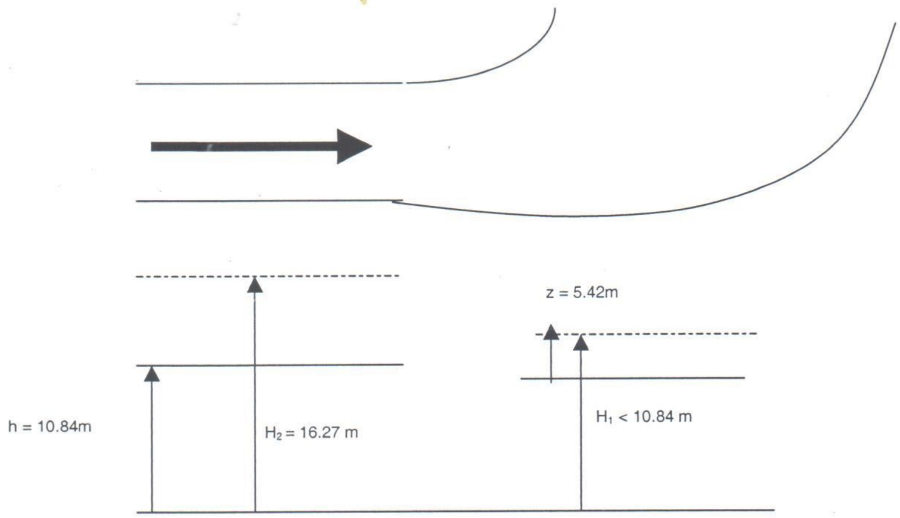
$Q = 1.8(B+{}^2/_3mH)H^{3/2}$ (Polders, Drainage and Flood Control, Ankum) is valid.

$Q = 1.8(B+{}^2/_3mH)H^{3/2}$
with
$Q = 4500\ m^3/s$
B = 20 m
m= 1.667

This gives for H:
H = 16.272 m

Since Fr = 1 → h = 2/3 H = 10.84 m

z = 1/3 H = 5.424 m
This means for H_1 (downstream of widening)< 16.27 – 5.42 < 10.84 m



As calculated before H_1 will always be larger then 12.7m (this is even without bend losses). In other words the widening is submerged and this calculation is not valid.

Sub merged Flow

Energy losses in case of an submerged flow can be calculated with the equation of Carnot, while the widening is quite sudden (Vloeistofmechanica page 128, Battjes)

$$\Delta H = (u_1 - u_2)^2 / 2g$$

with
u_1 = velocity of water upstream (A_2-A_2)
u_2 = velocity of water downstream (A_1-A_1)

To make a first calculation: $u_1 = u_{uniform\ A\text{-}A}$ = 8.08 m/s
u_2 = 2.91 m/s (calculated above)

while

23.5.1.1.1.1.1 $Q = k^{*} A^{*}R^{2/3*}i^{1/2}$

K		30
I		0.00496
A		557.739
Q		4500
H		11.7
U		8.085361
H	**h + u2/2g**	
H		15.03196
Fr		0.754696

so

$\Delta H = 021u_2^2/2g + (u_1\text{-}u_2)^2/2g$

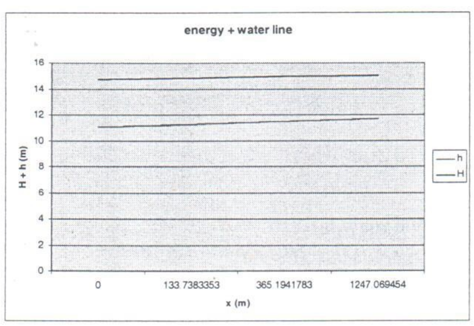
$\Delta H = 0.69 + 1.34 = 2.03$ m

this means for H upstream of the transition:

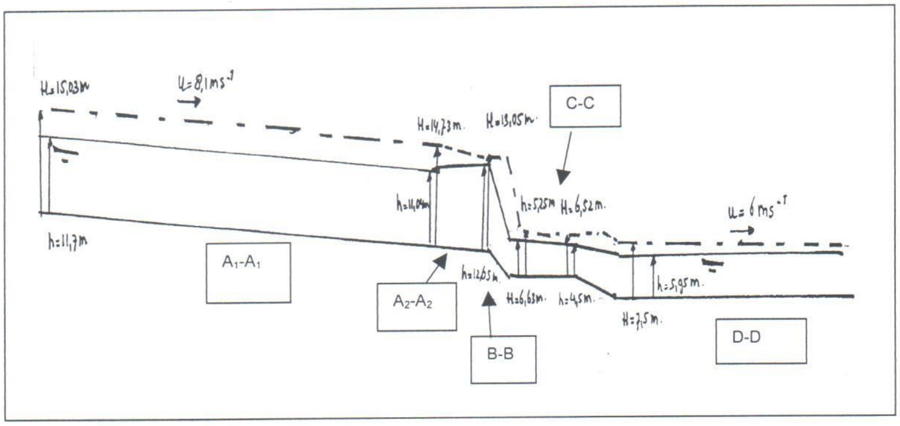
$H = 12.7 + 2.03 = 14.73$ meter $\rightarrow$
$H_{transition} < H_{uniform}$ so drawing down takes place.

The water levels can now be calculated with the standard step method.
And will reach $H_{uniform}$

H	X	u	h
14.73	0	8.5	11.04753
14.83	133.7383	8.4	11.23367
14.93	365.1942	8.2	11.50288
15.03	1247.069	8.08	11.7



Shows a summary of the calculations in one figure.



Water and Energy levels at PW point

APPENDIX 18 WATER QUALITY AT LUGARDS FALLS

Sampling point : 3HA12 at Lugard's Fall of Athi River

No.	Item	Unit	Sampling Date 20/02/91
1	pH		7.8
2	Electrical Conductivity	m.mhos/cm	410.0
3	Total Alkalinity (as CaCO3)	mg/l	190
4	Phenophthalein Alkalinity		13.0
5	B.O.D.	mg/l	28.5
6	C.O.D.	mg/l	4.8
7	Lead	mg/l	0.2
8	Copper	mg/l	N.D.
9	Zinc	mg/l	0.05
10	T.D.S.	mg/l	45.3
11	Potassium	mg/l	42.5
12	Sodium	mg/l	78.0
13	Cadmium	mg/l	N.D.
14	Chromium	mg/l	N.D.
15	Magnesium	mg/l	20.0
16	Chloride	mg/l	398.1
17	Fluoride	mg/l	1.12
18	Calcium	mg/l	76.00
19	Manganese	mg/l	N.D.
20	Mercury	mg/l	N.D.
21	Arsenic	ppb	0.0043
22	Iron	mg/l	0.20
23	Sulphate	mg/l	111.40
24	Nitrite	mg/l	0.018
25	20 min. P.V.		28

Note : N.D. means "not detected".

Source : The Study on the National Water Master Plan, page B.19-8, Japan International Cooperation Agency, Kenya 1992